Technical Report 3

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

Kingstowne Section 36A (KT36A) is a 200,000 SF mixed use building currently being constructed in Fairfax County Virginia. When completed, the lower half of the building will serve as a parking garage serving the office tenants of the upper half of the building. The parking garage levels utilize flat slab concrete construction while the office levels use a composite steel construction. A more thorough description of the existing structure can be found in the first half of this report.

The purpose of Technical Report 3 is to analyze the lateral force resisting system found in KT36A based on ASCE 7-10 provisions for strength and serviceability requirements. For the purpose of this report, all lateral loads were viewed as being resisted by steel moment frames, steel braced frames, or concrete shear walls with no contribution from the gravity system in the structure. This is a conservative way for obtaining the loads on the lateral systems since in reality, the gravity system will see some small percentage of the lateral load, although usually not enough to impact the design of the gravity system. In order to efficiently achieve this for the entire building, a three-dimensional structural model was created using ETABS, a modeling and analysis software commonly used in the structural engineering profession for obtaining an accurate and realistic response of the structure. The uses of the model included:

- Finding the story shear resisted by each lateral element with a unit load applied and dividing this value by the story drift of the lateral element to obtain the relative stiffness of each element
- Obtaining the natural period of the building for different modes
- Determining the center of rigidity, center of mass, and center of pressure
- Finding the maximum design forces in members for different load combinations
- Determining maximum floor displacements and story drifts for the different load combinations

After conducting the analysis of the lateral system, different loading combinations were found to control in different directions of the building. All of the loading combinations analyzed also considered the lateral earth pressures acting on the first two and a half stories of the North face of the building. Considering story drifts, seismic loads control in the E-W direction while wind loads control in the N-S direction of the building. This makes sense considering the larger surface area for the wind to act on at the North and South faces of the building. Other controlling load combinations were also found to control the overturning moment of the building and the forces imparted upon certain lateral load resisting elements, all of which were found to be adequately resisted by the lateral system in the building.

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.

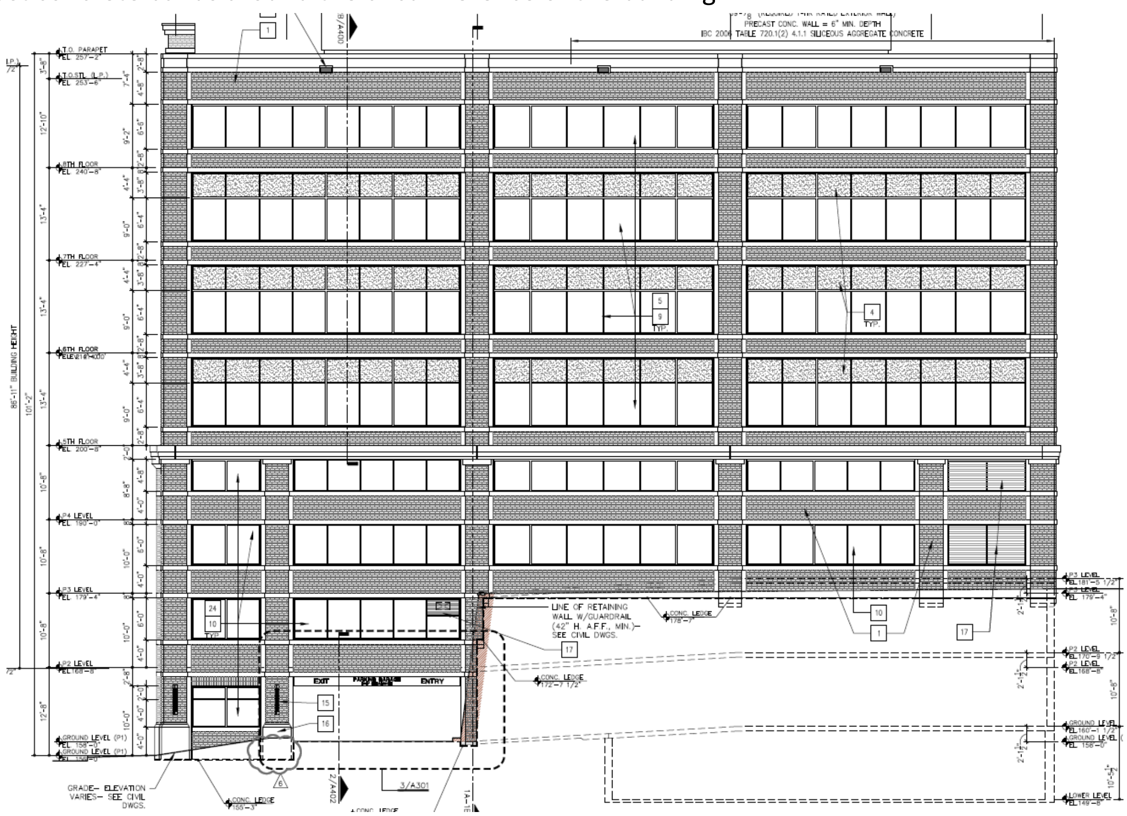


Figure 1: Elevation Looking East Showing Grade Differences (Source: DCS Design Drawing A-301)

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 the building 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.

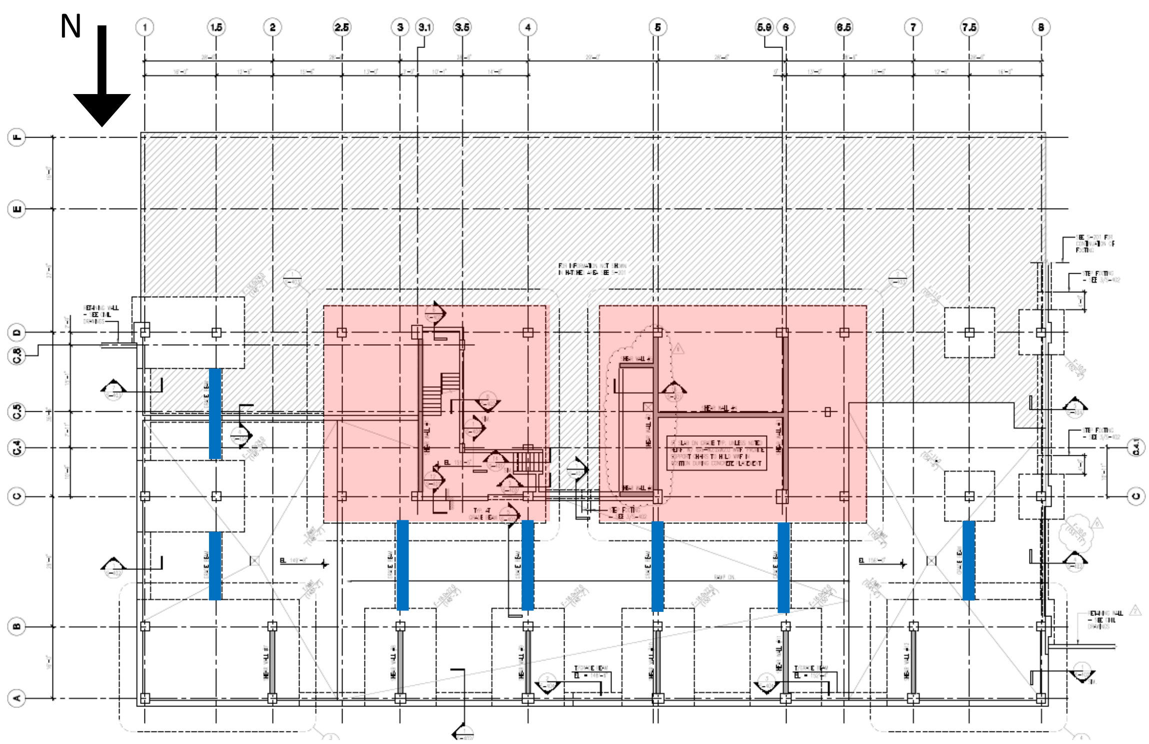


Figure 2: Foundation Plan (Level P0) Showing 48" Thick Mat Foundations Shaded in Red (Source: Cagley & Assoc. Drawing S-200)

Each of the borings found 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 bearing on Geopiers 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 slab 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 is subjected to 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 in certain middle strips and continuous drop panels 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 in the column strips 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".

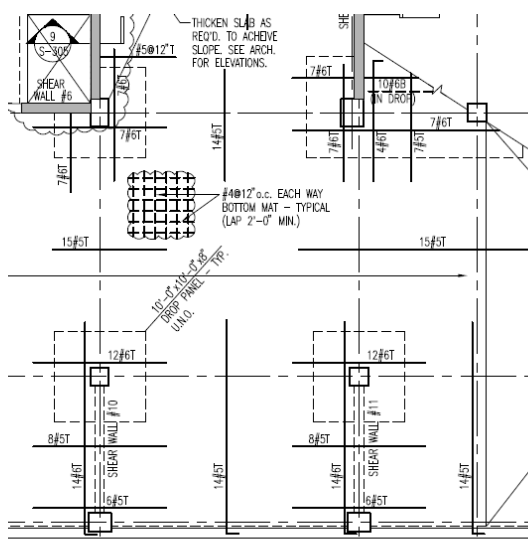


Figure 3: Partial Plan Level P1 (Source: Cagley & Assoc. Drawing S-201)

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 ties. 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 the N-S direction while the other three walls are oriented in the E-W 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.

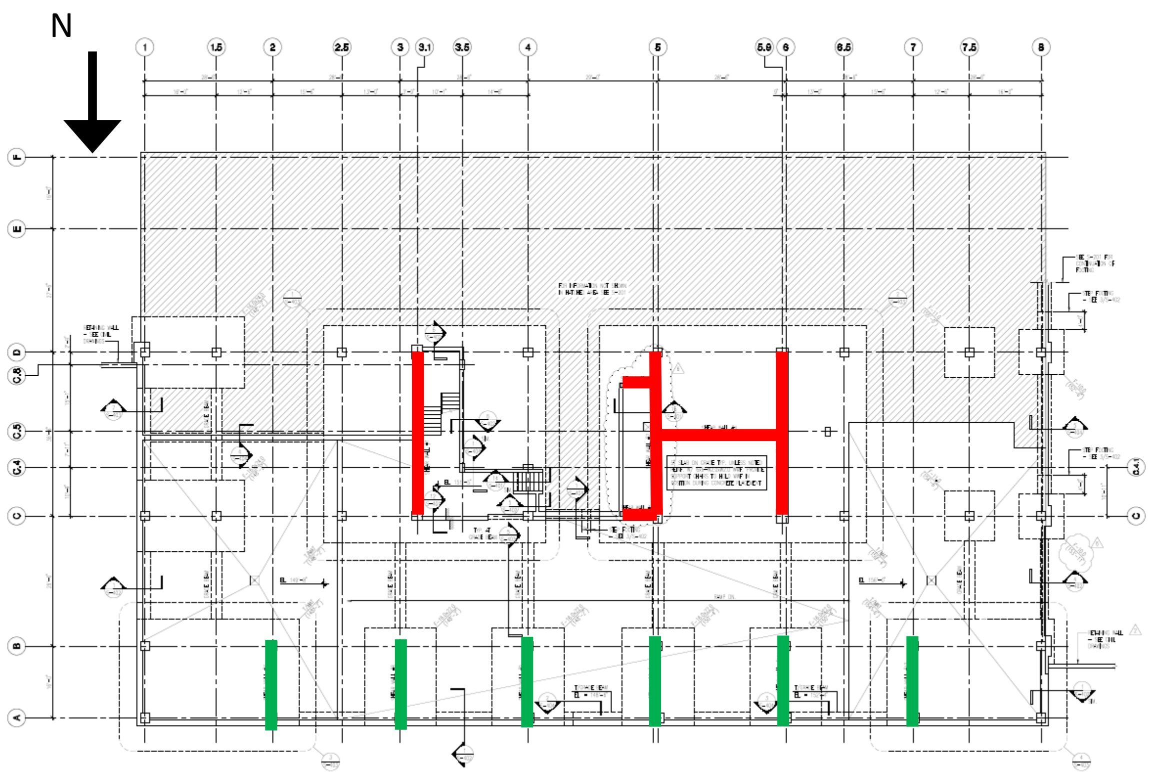


Figure 4: Foundation Plan (Level P0) Showing Shear Walls (Source: Cagley & Assoc. Drawing S-200)

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 lightweight 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. The four W30x99 girders each span 44'-0" which warrants the use of the camber to satisfy the total deflection criteria. 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.

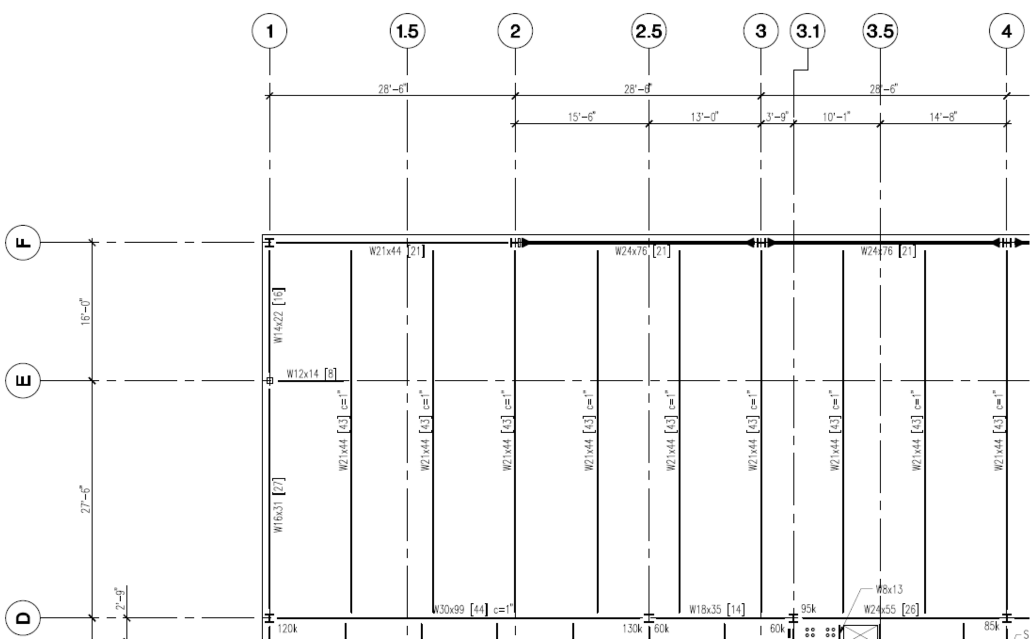


Figure 5: Typical Composite Slab Partial Plan (Level OL3) (Source: Cagley & Assoc. Drawing S-207)

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 and lower lateral loads acting on the building in this direction.

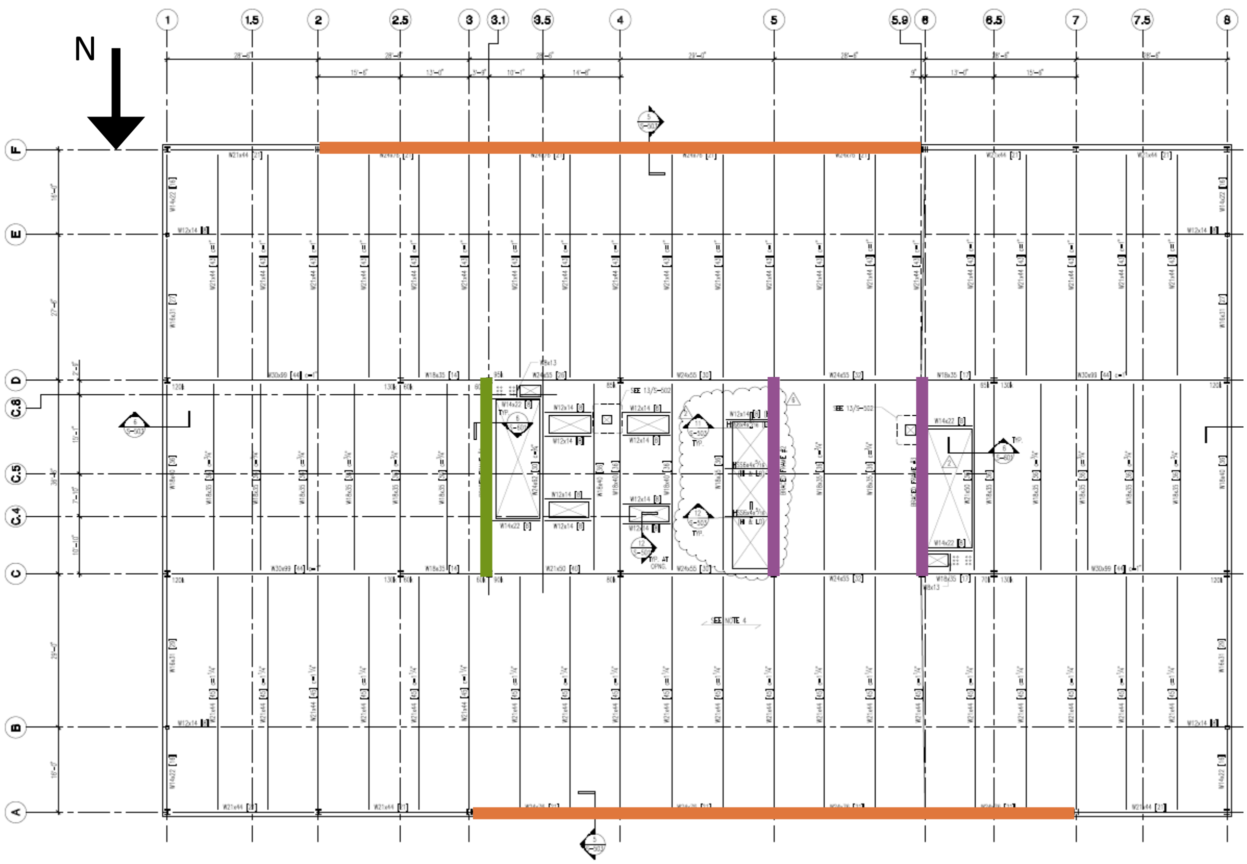
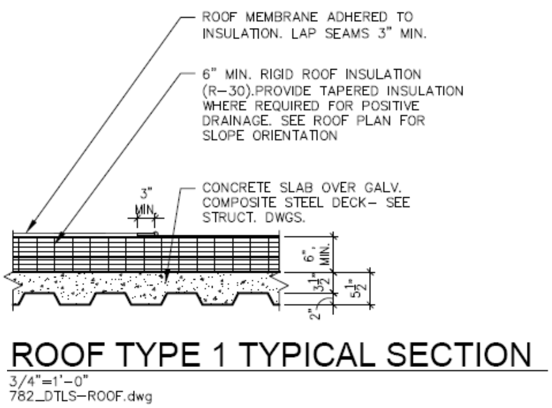
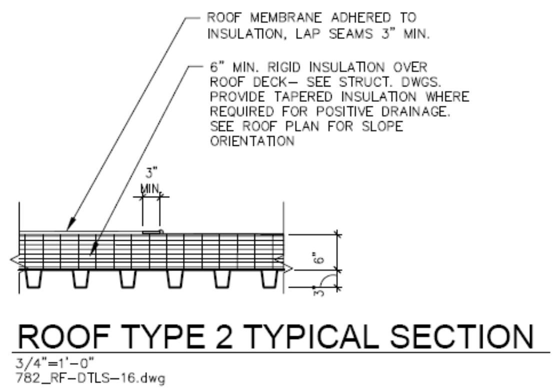


Figure 6: Typical Composite Slab Plan (Level OL3) (Source: Cagley & Assoc. Drawing S-207)

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)

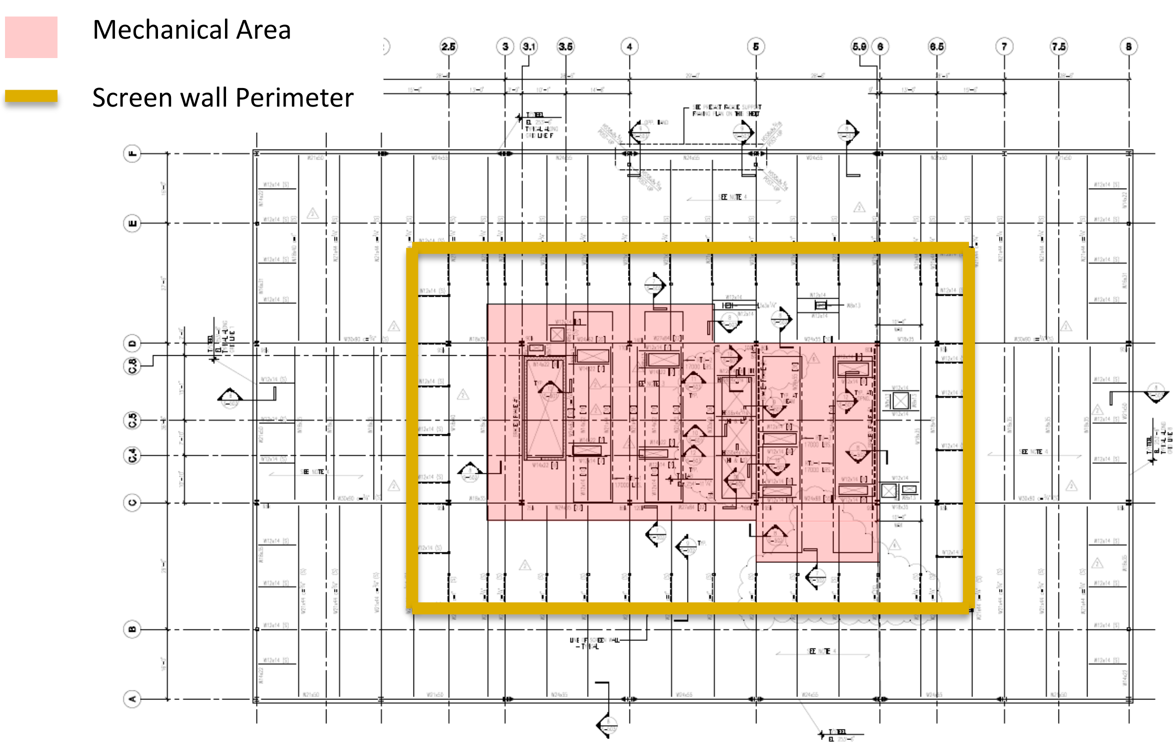


Figure 9: Structural Roof Plan (Source: Cagley & Assoc. Drawing S-209)

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, 13th 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, 14th 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.

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

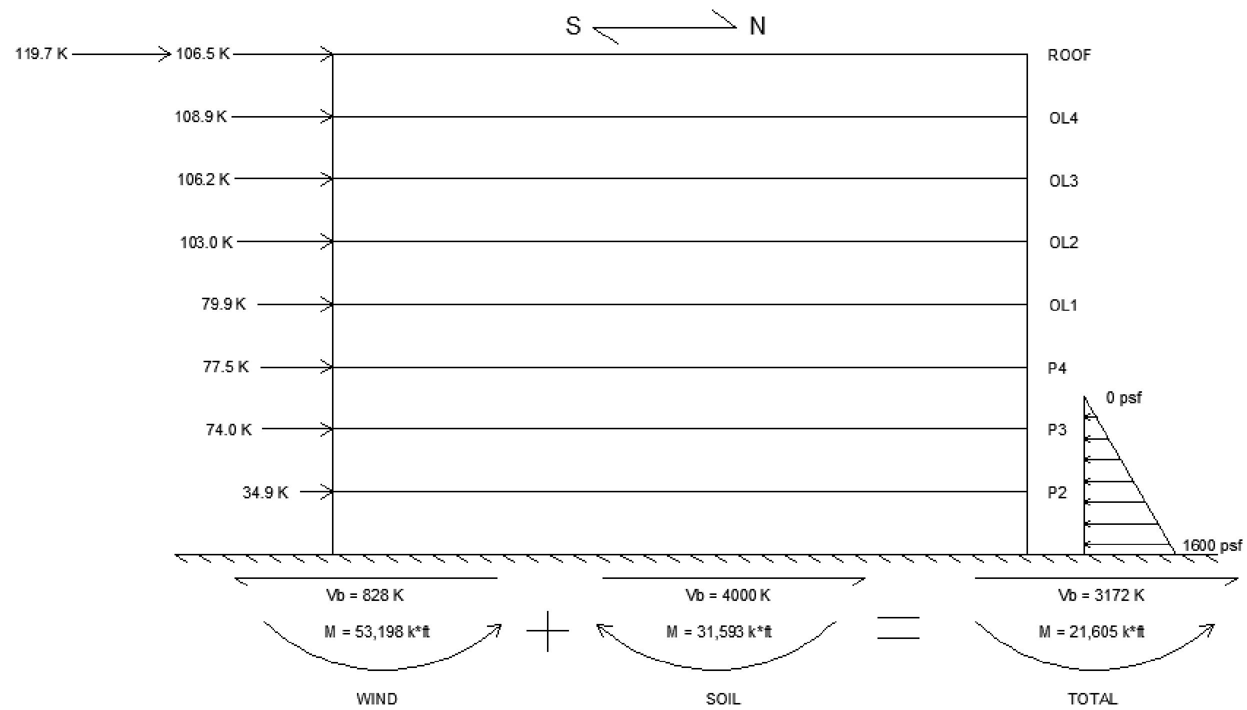
Snow loads for KT36A were calculated using ASCE 7-10 for comparison to the snow loads used in the design of the building. According to Figure 7-1 in this code, Kingstowne Virginia is located in a 25 psf ground snow load area. After applying equation 7.3-1 in ASCE 7-10, this equates to a 17.5 psf flat 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.

WIND LOADS

Wind loads for KT36A were calculated using the directional procedure outlined in Chapter 27 of 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. Considering the difference in grade elevation from the South side to the North side of the building, wind pressures had to be calculated for a North or South wind in addition to the East-West wind. Wind loads on the screen walls shown in Figure 9 were also taken into consideration. Since the main wind force resisting elements of the building do not extend above the roof line, the loads from the screen walls are transferred to the resisting elements through the roof slab and its' supporting members. To represent this in the analysis of the building, two resultant point loads are applied at the roof level in the direction of the prevailing wind. Figures 11, 12, and 13 on the following pages show the results of the wind load calculations and the corresponding lateral force diagram for the given wind direction. Figures 11 and 12 regarding the South and North winds, respectively, also show the effects of the soil load on the North side of the building. The combined effects of the wind and soil loads are further detailed in the Analysis section of this report. Figure 10 gives a summary of the parameters used in finding the wind loads on KT36A. See Appendix A for soil load calculations and Appendix B for wind load calculations.

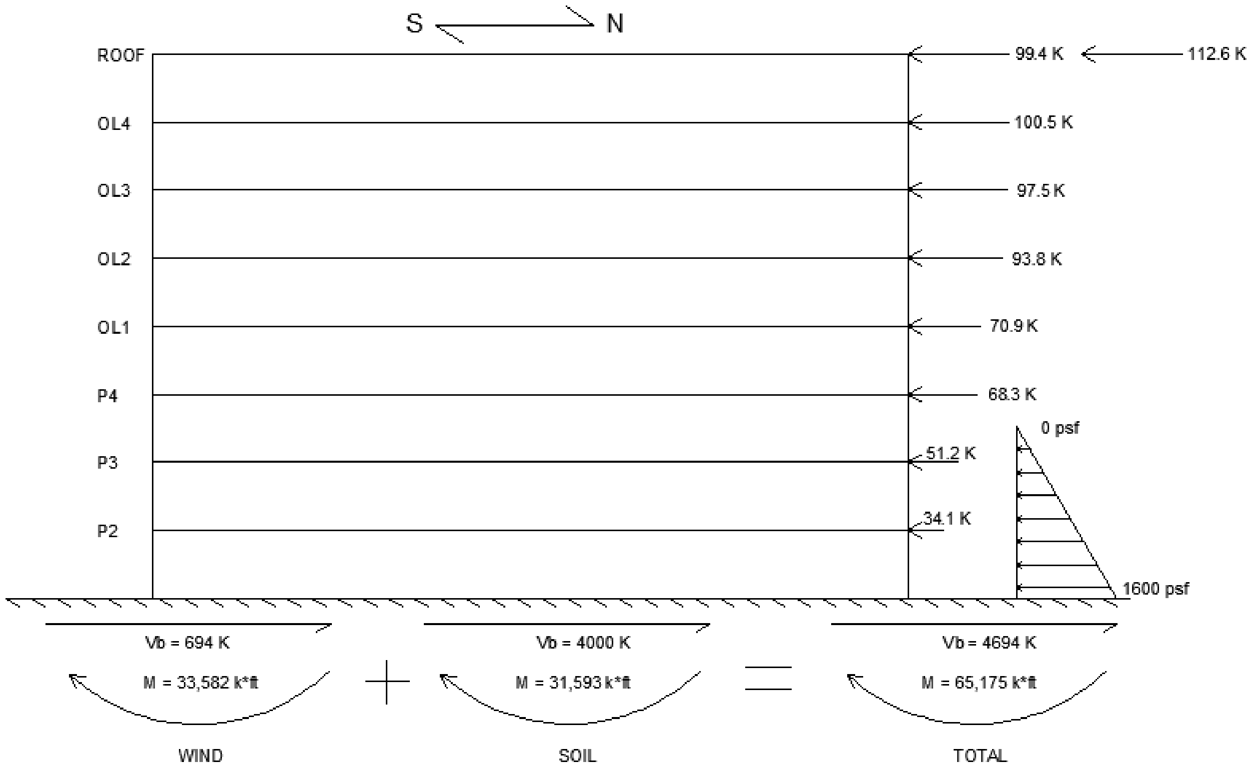
Wind Parameter Summary	
Velocity	115 MPH
Exposure	B
Kd	0.85
Kzt	1.00
Gust Factor G	0.85
GCpi	+/- 0.18
Flexible or Rigid?	Rigid

Figure 10: Wind Parameters (Source: Chavanic)



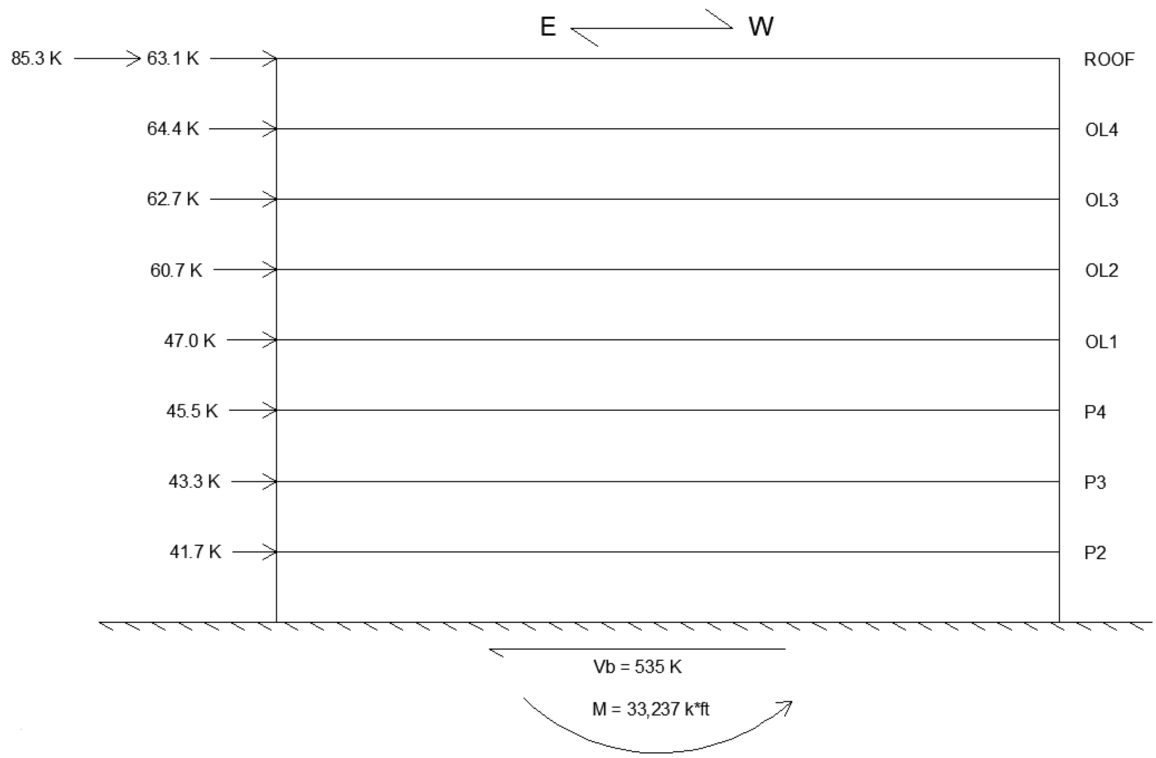
North - South (MWFRS) - South Wind										
Floor	Elevation	z	kz	qz	qh	Windward (psf)	Leeward (psf)	Tributary Area (ft2)	Force (k)	
Ground (P1)	158	0	0.57	16.40	28.06	16.2		1078	17.5	
P2	168.67	10.67	0.57	16.40	28.06	16.2		2155	34.9	
P3	179.33	21.33	0.63	18.13	28.06	17.4	-17.0	2153	74.0	
P4	190	32	0.712	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

Figure 11: South Wind Force Diagram and Calculation (Source: Chavanic)



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

Figure 12: North Wind Force Diagram and Calculation (Source: Chavanic)



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

Figure 13: East-West Wind Force Diagram and Calculation (Source: Chavanic)

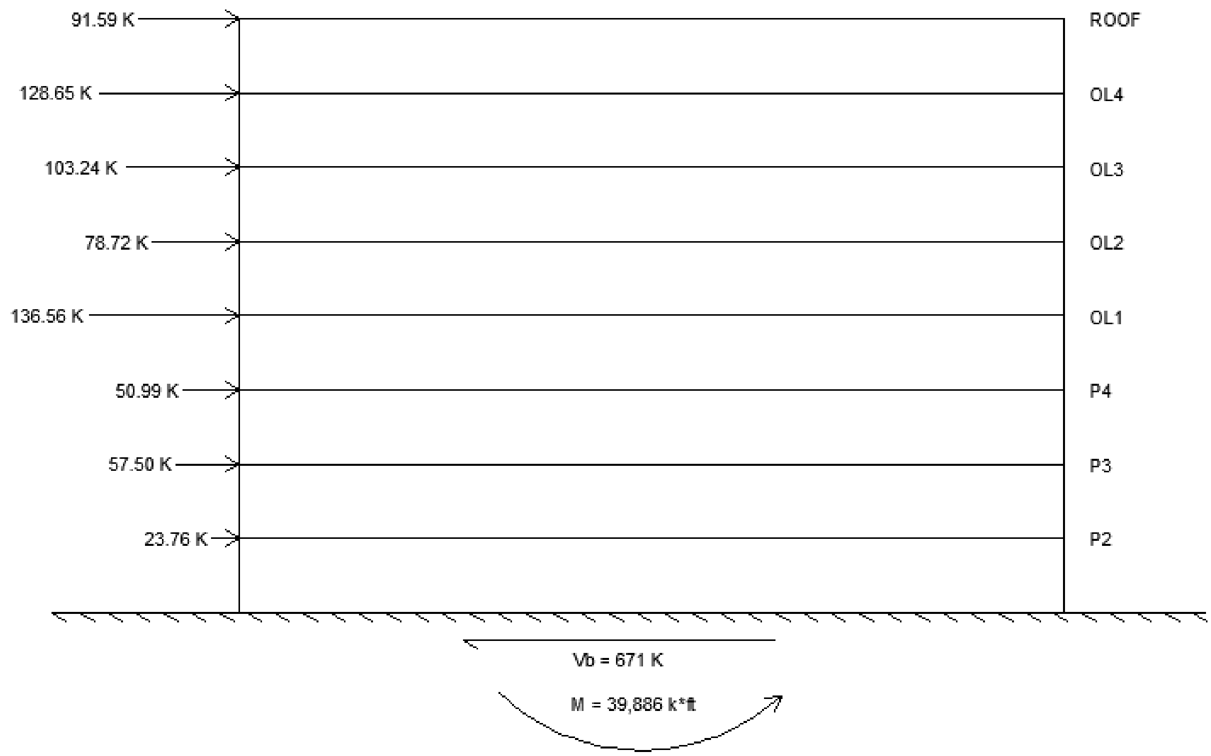
SEISMIC LOADS

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 if S_{ds} and S_{d1} used in the design of the building were obtained from the more accurate USGS online Seismic Design Maps application. Since the building weight was calculated to be nearly the same as the one used in design and the total base shear calculated was higher than the design one, seismic forces calculated in Tech I were also used for this report. See Figure 14 for a summary of the parameters used in determining the seismic loads on KT36A. A summary of the calculated loads and how they were determined can be seen in Figure 15. Appendix C details the seismic load calculations.

Referencing ASCE 7-10 Section 12.8.4.2, accidental torsion due to seismic loading should be considered when loading the building. Accidental torsion is applied to account for any possible differences in the center of mass or center of rigidity of the building from their anticipated locations. When applied, this torsion causes additional shear load in some of the lateral resisting elements. The inherent eccentricity of the building was used to determine which direction to apply the accidental torsion so as to cause the maximum effect on the building. Since KT36A was classified as a seismic design category "B" building in determining the seismic loads, applying an accidental torsional moment amplification factor was not required per ASCE 7-10 Section 12.8.4.3. Calculations of the accidental torsion at each floor of the building can be seen in Figure 16.

Seismic Parameter Summary	
Site Class	D
Risk Category	II => I=1.0
S_l	0.052
S_s	0.13
S_{Dl}	0.0832
S_{Ds}	0.1387
Seismic Design Category	B
Structural steel not specifically detailed for seismic resistance	R = 3
	$\Omega_O = 3$
	$C_d = 3$
C_s	0.0267
Building Weight	25,132 k

Figure 14: Seismic Parameters (Source: Chavanic)



			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

Figure 15: Seismic Force Diagram and Calculation [E-W and N-S] (Source: Chavanic)

Seismic Loading Torsion E-W Direction (X)							
Floor	**Story Force (k)**	**COR Location**	**COM Location**	**e (ft)**	$M_{inherent}$ (k-ft)	M_{acc} (k-ft)	M_{total} (k-ft)
RF	91.59	62.089	62.5	0.411	37.643	-572.4	-534.8
OL4	128.65	62.749	62.5	-0.249	-32.034	-804.1	-836.1
OL3	103.24	63.112	62.5	-0.612	-63.183	-645.3	-708.4
OL2	78.72	63.315	62.5	-0.815	-64.157	-492.0	-556.2
OL1	136.56	62.755	62.5	-0.255	-34.823	-853.5	-888.3
P4	50.99	62.607	62.5	-0.107	-5.456	-318.7	-324.1
P3	57.5	62.554	62.5	-0.054	-3.105	-359.4	-362.5
P2	23.67	62.524	62.5	-0.024	-0.568	-147.9	-148.5

Seismic Loading Torsion N-S Direction (Y)							
Floor	**Story Force (k)**	**COR Location**	**COM Location**	**e (ft)**	$M_{inherent}$ (k-ft)	M_{acc} (k-ft)	M_{total} (k-ft)
RF	91.59	105.558	100	-5.558	-509.057	-915.9	-1425.0
OL4	128.65	104.016	100	-4.016	-516.658	-1286.5	-1803.2
OL3	103.24	102.046	100	-2.046	-211.229	-1032.4	-1243.6
OL2	78.72	102.469	100	-2.469	-194.360	-787.2	-981.6
OL1	136.56	98.528	100	1.472	201.016	-1365.6	-1164.6
P4	50.99	98.622	100	1.378	70.264	-509.9	-439.6
P3	57.5	100.934	100	-0.934	-53.705	-575.0	-628.7
P2	23.67	102.245	100	-2.245	-53.139	-236.7	-289.8

Figure 16: Accidental Torsion Calculation (Source: Chavanic)

Computer Model

To efficiently analyze the effects of the lateral loads on the building as a whole, a three-dimensional structural model was created using ETABS. ETABS is a modeling and analysis program commonly used by the structural engineering industry to obtain an accurate and comprehensive analysis of the building lateral systems. After applying the appropriate property modifiers and structural considerations to the building, member forces and story displacements/drifts can be easily obtained for the controlling load case(s). For this analysis, only members participating in the lateral system of the structure were modeled since only lateral forces were considered in this analysis. See Figure 17 on the following page for a three-dimensional view of the lateral system model in ETABS.

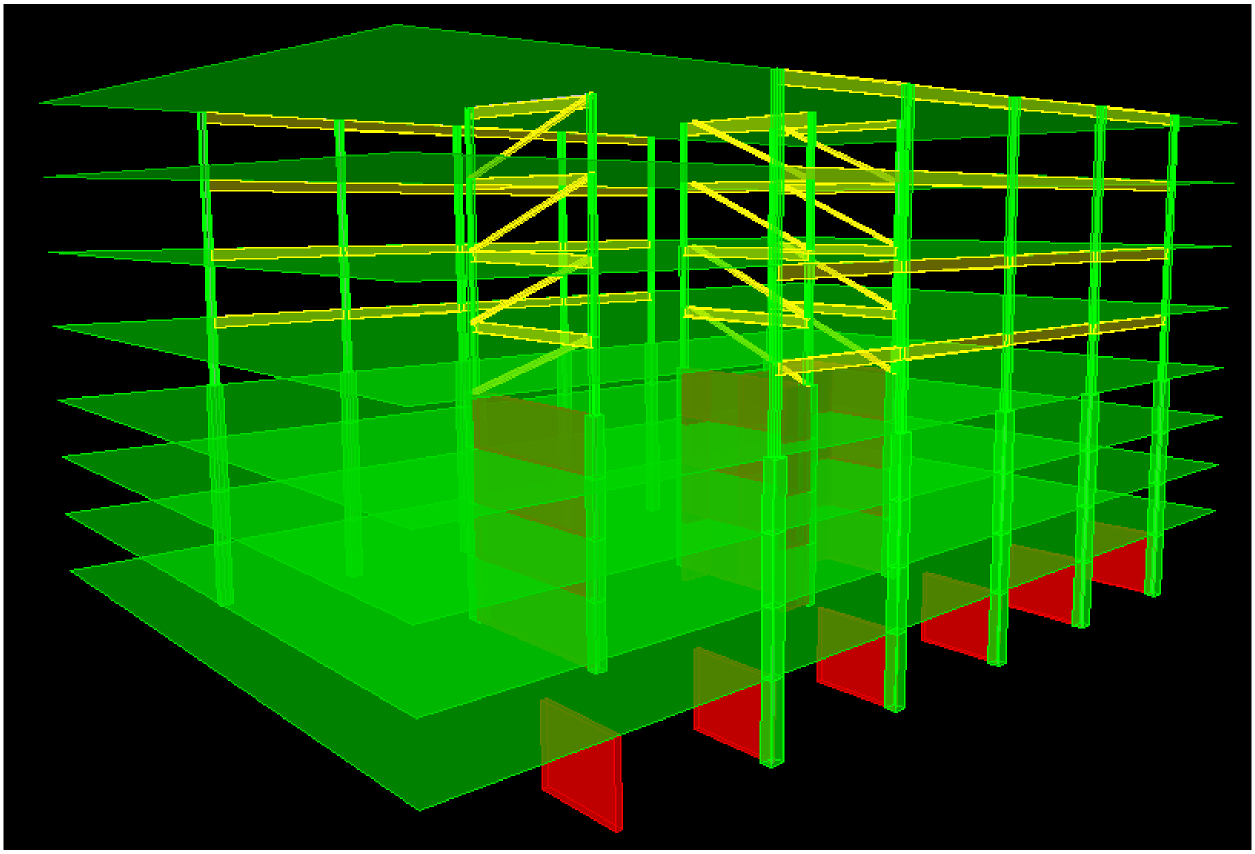


Figure 17: View from North-East corner of ETABS Model (Source: Chavanic)

In order to accurately predict the realistic behavior of the structure, the following assumptions and considerations were made when defining the model:

- Per ASCE 7-10 Section 12.7.3
    - Effects of cracked concrete considered in accordance with ACI 318-11 8.8.2
        - Column moment of inertia modified by $0.7*I_g$
        - Shear wall moment of inertia modified by $0.7*I_g$ (The uncracked modifier was used here since wind controls the lateral design of the structure)
    - Panel Zones accounted for in steel moment frames
- Each floor level was modeled as a rigid diaphragm so that all points at each level would displace together
- Steel column splices were modeled at the OL2 floor level instead of just above the floor level
- All shear walls were modeled as membrane elements so as not to resist out of plane forces
- All concrete column and shear wall base restraints were modeled as fixed connections
- Diagonal bracing in the braced frames were modeled with end moment releases
- Steel columns were modeled with fixed base restraints at the tops of the concrete columns
- Shear walls were “meshed” with a maximum size of 18” x 18” to properly account for shear deformations in both axes of the plane of the wall

Analysis

Relative Stiffness

Working on the basis of equilibrium, the story shear at each level of a building must be resisted by the lateral system elements at that level. Each of the load resisting elements carries some percentage of the story shear. This percentage is directly proportional to the stiffness of the element, which stems from the principle that load follows stiffness. The relative stiffness of each of the lateral load resisting elements in KT36A at each floor level was calculated based on this principle. To accomplish this, two 1000 kip loads were applied in orthogonal directions at the roof level of the building. Analyzing each direction independently, the shear force and drift of each element were recorded for each level. Using the equation $k=p/\delta$, the stiffness of each element at the level of concern was found by dividing the shear force by the drift of the element. For each element, this value was then divided by the sum of the element stiffness's for that level to obtain the relative stiffness. Obtaining the relative stiffness's of the elements at a particular level is key for determining the distribution of direct shear and torsion induced shear in the elements of concern. See Figure 19 for representative calculations of relative stiffness.

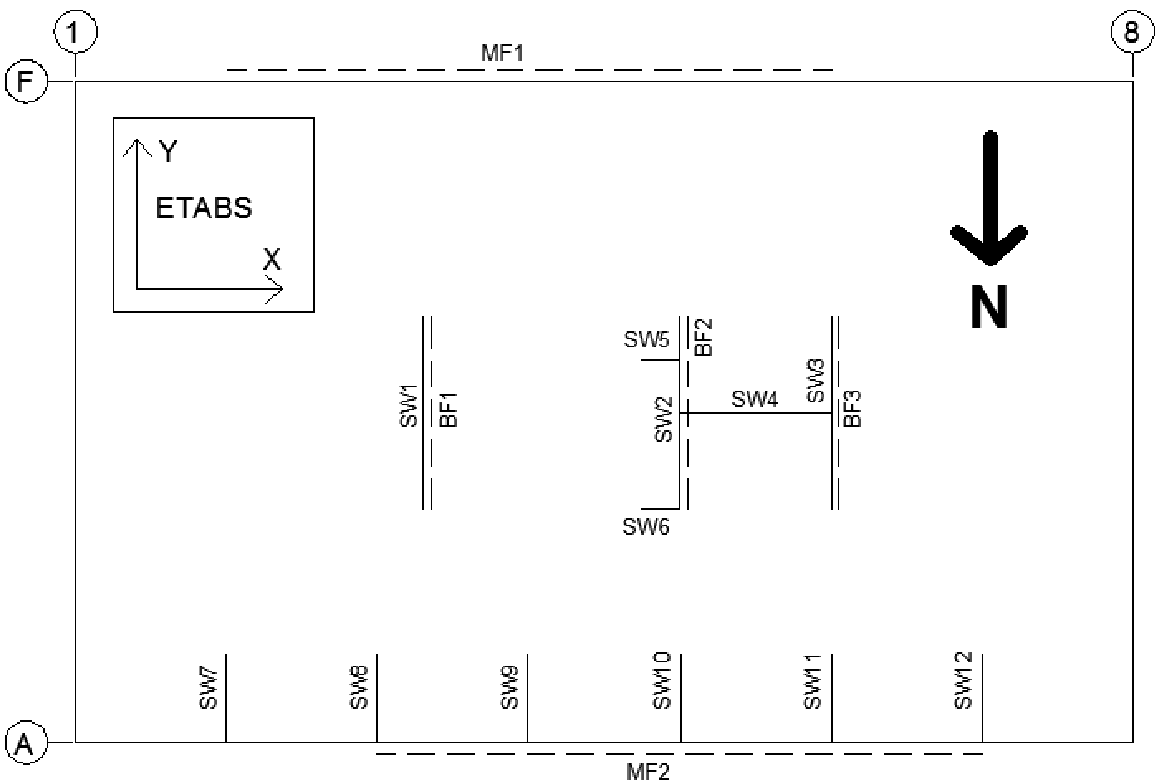


Figure 18: Labeling of Lateral System Elements (Source: Chavanic)

Forces at Story P1

	Member	ETABS Individual Load P (k)	ETABS Drift Ratio (in/in)	Stiffness K (k/in)	Relative K (%)
X-Direction Loading	MF1	17.95	0.000156	757	1.71%
	MF2	14.21	0.000143	654	1.48%
	SW4	841.85	0.000149	37171	84.14%
	SW5	55.155	0.00015	2419	5.48%
	SW6	55.42	0.000147	2480	5.61%
	Cols @ D	7.402	0.000151	322	0.73%
	Cols @ C	8.31	0.000147	372	0.84%
	Sum	1000.30		44176	

	Member	ETABS Individual Load P (k)	ETABS Drift Ratio (in/in)	Stiffness K (k/in)	Relative K (%)
Y-Direction Loading	SW1	170.84	0.000071	15830	14.32%
	SW2	179.57	0.000058	20369	18.42%
	SW3	147.34	0.00005	19387	17.53%
	Cols @ 3.1	2.168	0.000071	201	0.18%
	Cols @ 5	0.6755	0.000058	77	0.07%
	Cols @ 5.9	0.571	0.000051	74	0.07%
	SW7	79.82	0.000079	6647	6.01%
	SW8	105.07	0.000072	9601	8.68%
	SW9	94.12	0.000065	9526	8.62%
	SW10	83.92	0.000058	9519	8.61%
	SW11	73.84	0.00005	9716	8.79%
	SW12	62.91	0.000043	9625	8.70%
	Sum	1000.8445		110571	

Forces at Story OL4

	Member	ETABS Individual Load P (k)	ETABS Drift Ratio (in/in)	Stiffness K (k/in)	Relative K (%)
X-Direction Loading	MF1	501.26	0.015972	204	49.99%
	MF2	501.33	0.015969	204	50.01%
	Sum	1002.59		408	

	Member	ETABS Individual Load P (k)	ETABS Drift Ratio (in/in)	Stiffness K (k/in)	Relative K (%)
Y-Direction Loading	BF1	366.39	0.005395	441	33.87%
	BF2	355.9	0.004875	474	36.41%
	BF3	274.61	0.004607	387	29.73%
	Sum	996.9		1302	

Figure 19: Relative Stiffness Calcs for Story P1 and OL4 (Source: Chavanic)

One of the most important concepts of a lateral analysis is understanding the relationship of the center of rigidity (COR) to the center of mass (COM) and center of pressure (COP) of the building. Earthquake forces are typically viewed as being applied to the COM while wind pressure induced forces can be viewed as being applied to the COP. Any difference in the COR from the COM or COP causes an eccentricity between where the load is applied and where it is resisted. Figure 20 shows the execution of finding the COR to compare to the ETABS output for COR while Figure 21 tabulates the ETABS output for COR. COM, and COP including the eccentricities between the COR and COM of the ETABS output.

X COR Story P2		
Element	Stiffness (% K_{story})	Dist. From Origin (ft)
SW1	31.49	60.75
SW2	37.44	114.5
SW3	30.67	143
	X COR =	106.28

Y COR Story P2		
Element	Stiffness (% K_{story})	Dist. From Origin (ft)
MF1	1.38	125
MF2	1.22	0
SW4	95.28	62.75
SW5	0.58	74
SW6	1.02	45
	Y COR =	62.73

Figure 20: Sample Calculation for Obtaining COR at Story P2 (Source: Chavanic)

		Hand Calc	Obtained from ETABS				
Story	Direction	COR (ft)	COR (ft)	COM (ft)	COP (ft)	e_x (ft)	e_y (ft)
P1	X	106.59	105.558	100	100	-5.56	
	Y	62.5	62.089	62.5	62.5		0.41
P2	X	106.28	104.016	100	100	-4.02	
	Y	62.73	62.749	62.5	62.5		-0.25
P3	X	107.42	102.046	100	100	-2.05	
	Y	62.1	63.112	62.5	62.5		-0.61
P4	X	111.96	102.469	100	100	-2.47	
	Y	62.28	63.315	62.5	62.5		-0.81
OL1	X	95.81	98.528	100	100	1.47	
	Y	62.49	62.755	62.5	62.5		-0.26
OL2	X	98.78	98.622	100	100	1.38	
	Y	62.5	62.607	62.5	62.5		-0.11
OL3	X	105.45	100.934	100	100	-0.93	
	Y	62.49	62.554	62.5	62.5		-0.05
OL4	X	104.77	102.245	100	100	-2.25	
	Y	62.49	62.524	62.5	62.5		-0.02

Figure 21: Summarized COR, COM, COP e_x and e_y (Source: Chavanic)

Load Cases And Combinations

Section 2.3.2 of ASCE 7-10 lists seven different load combinations for LRFD strength design. The combinations are used to determine the factored ultimate loads on the building for combined gravity and lateral loading. Since only lateral forces were considered in this analysis, the combinations considering the highest wind or seismic factor were viewed as the controlling load combinations. Also considering dead load, live load, and snow load, combination 4 controlled for wind and combination 5 for seismic. Below are the ASCE 7-10 combinations:

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

Using the controlling load combinations, 12 combination load cases were created in ETABS to observe the effects of combined lateral loading on the building. The first two combinations considered the earthquake loading (considering accidental torsion) acting simultaneously with the lateral soil load, in the respective orthogonal directions. An orthogonal combination of the seismic loads acting in the "X" and "Y" directions together is typically considered; however, according to ASCE 7-10 12.5.2, this was not required for this analysis since KT36A is located in seismic design category B. Even though the seismic loads are dynamic in nature, they were treated as a constant static load. Although the levels below the differential grade line do not deflect much at all (fractions of an inch), their deflection could cause some amount of soil settlement behind the basement walls causing a complex loading situation on the foundation walls when the building returns to rest. This loading was not taken into consideration due to the time constraints of this report.

ETABS Case Name	Description
EQXTSOIL	E-W Seismic load + Accidental Torsion + Soil
EQYTSOIL	N-S Seismic load + Accidental Torsion + Soil

Figure 22: Seismic Load Case Descriptions (Source: Chavanic)

The remaining ten combinations considered wind loading and were derived from Figure 23. In order to limit the number of load cases that needed to be input to ETABS, the controlling eccentricity directions were determined by hand and can be found in Appendix D. See Figure 24 below for a summary of the wind load case descriptions applied in ETABS.

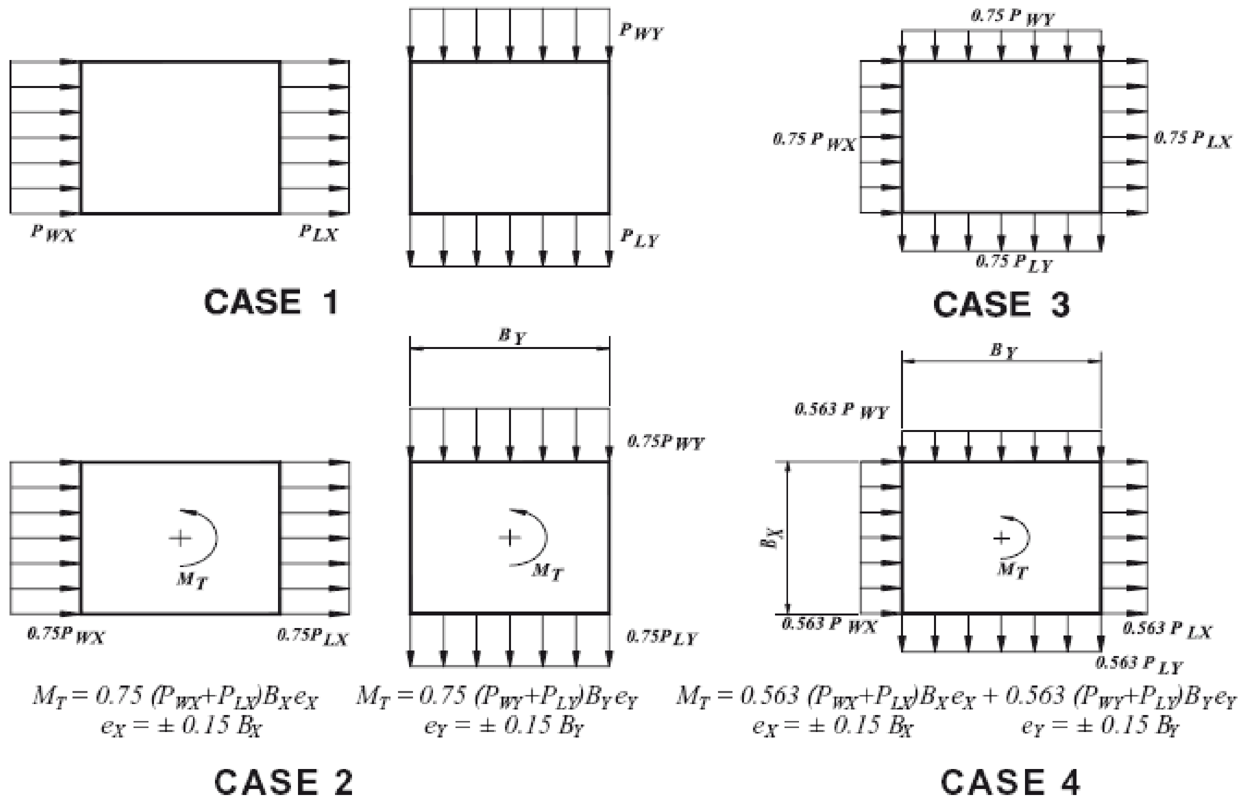


Figure 23: Design Wind Load Cases (Source: ASCE 7-10 Figure 27.4-8)

ETABS Case Name	Description
CASE1NW	Case 1 North Wind + Soil
CASE1SW	Case 1 South Wind + Soil
CASE1EWW	Case 1 East-West Wind + Soil
CASE2NW	Case 2 North Wind + Soil
CASE2SW	Case 2 South Wind + Soil
CASE2EWW	Case 2 East-West Wind + Soil
CASE3NW	Case 3 North Wind + East-West Wind + Soil
CASE3SW	Case 3 South Wind + East-West Wind + Soil
CASE4NW	Case 4 North Wind + East-West Wind + Soil
CASE4SW	Case 4 South Wind + East-West Wind + Soil

Figure 24: Wind Load Case Descriptions (Source: Chavanic)

STORY DRIFTS AND DISPLACEMENTS

Story drifts were calculated for KT36A based on the floor deflections obtained from the ETABS model. Each of the seismic loading combinations controlled for its' respective direction since only two cases were considered in the model. As mentioned earlier, it was not necessary to examine other seismic loading combinations. The controlling wind loading combinations varied in the N-S direction of the building while remaining the same for the E-W direction. This was due to the North wind acting with the soil load at reduced pressures and the South wind acting against the soil load at increased pressures. At stories OL4 and OL3, CASE1SW controls the amount of story deflection from wind loads in the N-S direction. The remaining stories are controlled by CASE1NW in the N-S direction. All story wind drifts are controlled by CASE1EWW in the E-W direction.

In the seismic loading drift calculations, the story drifts were checked against a limit of 0.020 h_{sx} for a risk category II in accordance with ASCE 7-10 12.12.1. It is also important to note that the seismic displacement values obtained from ETABS were amplified by a factor of (C_d/I) as specified in section 12.8.6 of ASCE 7-10. Referencing the ASCE 7-10 commentary, wind load story drifts were checked against a limit of H/400 with H being the height of the story being analyzed.

The following figures display the drift values for the controlling load cases and their corresponding directions.

Seismic Displacement and Drift E-W						
Story	**Story Ht. (ft)**	**X Disp. (in)**	**X Disp. Amped (in)**	**Amped X Story Drift (in)**	**Allow. Drift (in)**	**Acceptable?**
OL4	12.8333	3.583236	10.7497	1.3771	3.080	YES
OL3	13.3333	3.124191	9.3726	2.6347	3.200	YES
OL2	13.3333	2.245953	6.7379	3.5420	3.200	NO
OL1	13.3333	1.065288	3.1959	2.8537	3.200	YES
P4	10.6667	0.114041	0.3421	0.1729	2.560	YES
P3	10.6667	0.056395	0.1692	0.0657	2.560	YES
P2	10.6667	0.03448	0.1034	0.0587	2.560	YES
P1	12.6667	0.01491	0.0447	0.0447	3.040	YES

Seismic Displacement and Drift N-S						
Story	**Story Ht. (ft)**	**Y Disp. (in)**	**Y Disp. Amped (in)**	**Amped Y Story Drift (in)**	**Allow. Drift (in)**	**Acceptable?**
OL4	12.8333	0.704309	2.1129	0.3174	3.080	YES
OL3	13.3333	0.598514	1.7955	0.4966	3.200	YES
OL2	13.3333	0.432984	1.2990	0.5641	3.200	YES
OL1	13.3333	0.244967	0.7349	0.5276	3.200	YES
P4	10.6667	0.069116	0.2073	0.0701	2.560	YES
P3	10.6667	0.045764	0.1373	0.0430	2.560	YES
P2	10.6667	0.031432	0.0943	0.0398	2.560	YES
P1	12.6667	0.018173	0.0545	0.0545	3.040	YES

Figure 25: Seismic Drift Summary (Source: Chavanic)

Wind Displacement and Drift CASE1NW					
Story	Story Ht. (ft)	Y Displacement (in)	Y Story Drift (in)	Allowable Drift (in)	Acceptable?
OL4	12.8333	0.9603	0.1909	0.385	YES
OL3	13.3333	0.7694	0.2284	0.400	YES
OL2	13.3333	0.541	0.2422	0.400	YES
OL1	13.3333	0.2988	0.2243	0.400	YES
P4	10.6667	0.0745	0.0253	0.320	YES
P3	10.6667	0.0492	0.0161	0.320	YES
P2	10.6667	0.0331	0.0144	0.320	YES
P1	12.6667	0.0187	0.0187	0.380	YES

Wind Displacement and Drift CASE1SW					
Story	Story Ht. (ft)	Y Displacement (in)	Y Story Drift (in)	Allowable Drift (in)	Acceptable?
OL4	12.8333	0.9759	0.2002	0.385	YES
OL3	13.3333	0.7757	0.2408	0.400	YES
OL2	13.3333	0.5349	0.2565	0.400	YES
OL1	13.3333	0.2784	0.2378	0.400	YES
P4	10.6667	0.0406	0.0224	0.320	YES
P3	10.6667	0.0182	0.0143	0.320	YES
P2	10.6667	0.0039	0.0106	0.320	YES
P1	12.6667	-0.0067	-0.0067	0.380	YES

Wind Displacement and Drift CASE1EWW					
Story	Story Ht. (ft)	X Displacement (in)	X Story Drift (in)	Allowable Drift (in)	Acceptable?
OL4	12.8333	3.3613	0.5843	0.385	NO
OL3	13.3333	2.777	0.8606	0.400	NO
OL2	13.3333	1.9164	1.0299	0.400	NO
OL1	13.3333	0.8865	0.8011	0.400	NO
P4	10.6667	0.0854	0.0441	0.320	YES
P3	10.6667	0.0413	0.0157	0.320	YES
P2	10.6667	0.0256	0.0142	0.320	YES
P1	12.6667	0.0114	0.0114	0.380	YES

Figure 26: Wind Drift Summary (Source: Chavanic)

Overturning Moment And Impact on Foundations

When exposed to lateral loads, a building can impart unique loading situations on its' foundations. This can range anywhere from a significantly increased compression load over the gravity compression load to uplift on the foundation, which cannot be resisted by most foundations. Lateral loads place what is called an overturning moment on the building which must be transferred to the foundations of the building. The resistance to this overturning moment comes from the self-weight of the building. Commonly, the building has more than plenty of self-weight to counter the overturning moment; however, a very lightly framed building can see foundation complications if overturning moment is not taken into consideration.

Using the value found in the seismic load calculations, the self-weight of KT36A is 25,132 k. The shorter side of the building, approximately 125 ft, will allow for the smaller resisting moment arm. The smallest moment arm is desired in this calculation since it presents the highest chance for the overturning moment to be greater. From loading combinations, combining earthquake loading with the soil loading produces the largest overturning moment at 71,479 k*ft. The resisting moment was found to be 1,047,167 k*ft. This is clearly significantly larger than the overturning moment and results in a factor of safety against overturning of 14.65 which indicates that the building is more than capable of resisting the overturning moment. Calculations for determining the resistance to overturning moment can be found in Appendix E of this report.

STRENGTH SPOT CHECKS

Two members were chosen for a spot check to verify the validity of the model and the ability of the members to resist the applied loads. The first member checked was the HSS 10x10x5/8 diagonal brace in BF1 at office level 1 (OL1). Based on the direction of the applied loading, this member can see either compression or tension forces. Obtaining results from the ETABS model showed that the brace sees 247k in tension from wind CASE1SW and 228.1k in compression from wind CASE1NW. Considering the unbraced length of 38.4 ft, the brace was found to be adequate for both loading conditions. The 247k force obtained from the model was confirmed as accurate when compared with the 250k force used in the design of the connections for the braced frame. This value was obtained from the elevation of shear wall/braced frame #1 found on sheet S-303. The shear wall at level P1 was also checked for this wall/brace combo and was determined to more than sufficiently carry the 665k from controlling load case EQYTSOIL. Calculations can be found in Appendix F.

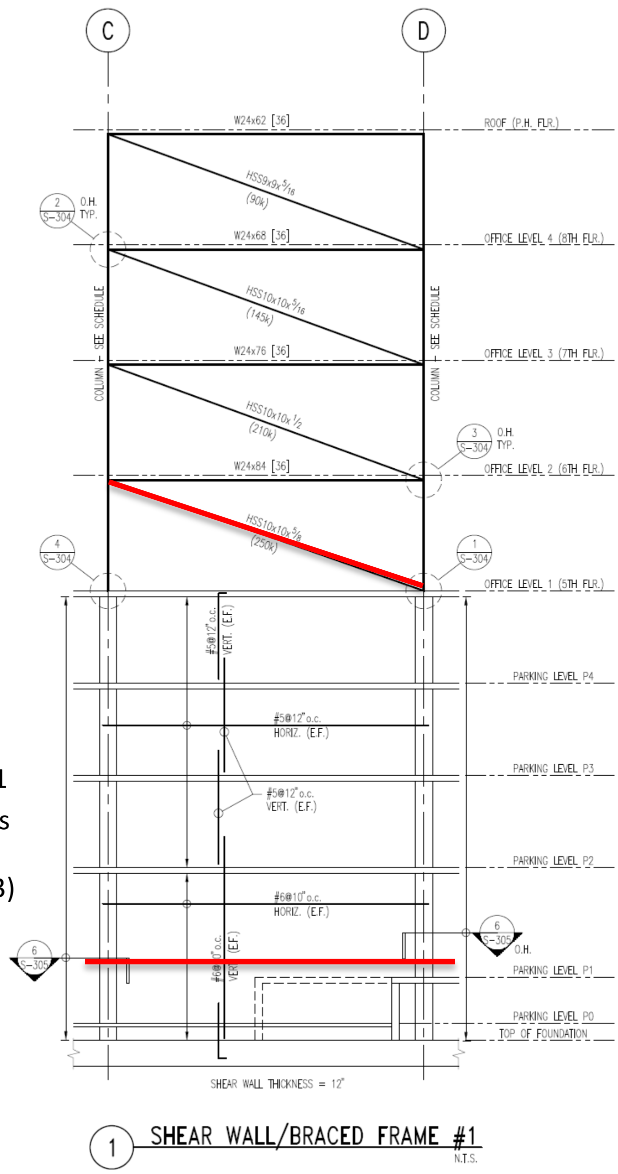


Figure 27: Shear Wall/Braced Frame #1 Showing Highlighted Checked Elements

(Source: Cagley & Assoc. Drawing S-303)

Conclusion

This report was to analyze the lateral force resisting system of Kingstowne 36A by considering both strength and serviceability requirements. KT36A uses a dual system consisting of steel moment frames and braced frames at the office levels and concrete shear walls at the parking levels to resist lateral loads. The ASCE 7-10 Code was used heavily throughout the analysis to both guide the lateral analysis and ensure that the structure meets the provisions of the code.

Lateral loads calculated in Technical Report 1 were deemed to be accurate so they were used for the analysis conducted in this report. Lateral earth pressures were also calculated and included in this analysis as they are a significant contribution to the total base shear of the building. The calculated loads were then applied to a three-dimensional structural model created in ETABS to observe how the building reacts to different lateral load conditions.

Due to the unique relationship of the structure to the site it is on, multiple loading combinations were found to control the building. Each of the seismic cases controlled the building for seismic considerations in their respective directions. Case 1 of the ASCE 7-10 wind load cases controlled wind considerations for all directions of the building based on story displacements with the E-W winds creating the largest overall deflections.

One earthquake loading story drift was found to be unacceptable by the provisions of the ASCE 7-10 code. This is most like attributable to the calculated seismic loads being higher than the ones used for design of the building. It should be noted that this unacceptable drift was found in the direction of the building resisted by the steel moment frames. For the controlling wind case in the E-W direction, the story drift was found to be unacceptable in the floors also resisted by the steel moment frames. This is most likely a result of using 50-year wind pressures which are required for strength design when only 10-year wind pressures are suggested for serviceability limits.

Overturning moment was controlled by earthquake loading in the N-S direction combined with the soil load while total base shear was controlled by a North wind combined with the soil load.

When considering an overall comparison, the building is controlled by seismic loading in the E-W direction and wind loading in the N-S direction, both based on story displacements. Maximum forces for all members in order to know which loading controlled the building in strength requirements were not obtained for this report due to time constraints of the assignment. Instead, the strength requirements were taken to be adequate based on two spot checks, one performed on a steel bracing member and one on a concrete shear wall at its' base. The ability of the building to resist overturning moment was also considered and found to be more than adequate with a factor of safety of about 14.

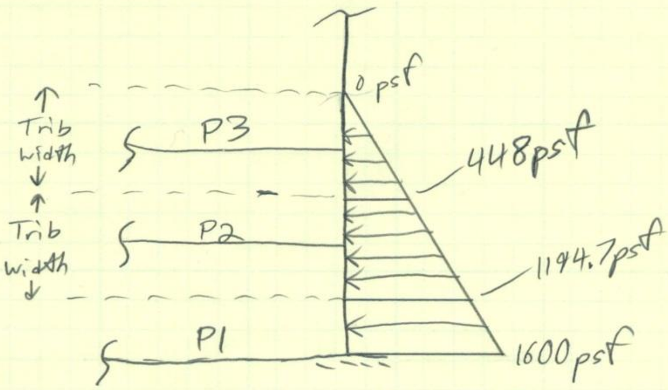
APPENDIX: A Soil Load Calculations

Soil Loading   Tech 3

Per Geotech Report

Soil load = 64 psf/ft depth

Average Grade is about 25 ft above the P1 Parking Level



Force resisted by each floor level = Area · width of building

$R_{P3} = \frac{1}{2}(7ft \cdot 448psf) \cdot \underline{200ft}$ ↳ building width in this direction

$R_{P3} = 313.6$ Kips

≈ 40k @ each integer column line

$R_{P2} = \frac{1}{2} 11.6667(448 + 1194.7) \cdot 200ft$

$R_{P2} = 1916.5$ kips

≈ 320k to each shear wall

Using ASCE 7-10 2.3.2

- Apply H with load factor of 1.6 when H adds to the primary load effect
- Apply H with load factor of 0.9 when H resists the primary load effects

APPENDIX: B Wind Load Calculations

James Chavanic Tech III Wind Loads

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

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

↳ 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

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

$$p = qGC_p - q_i(GC_{pi})$$

James Chavanic | Tech III | Wind Loads Cont.

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

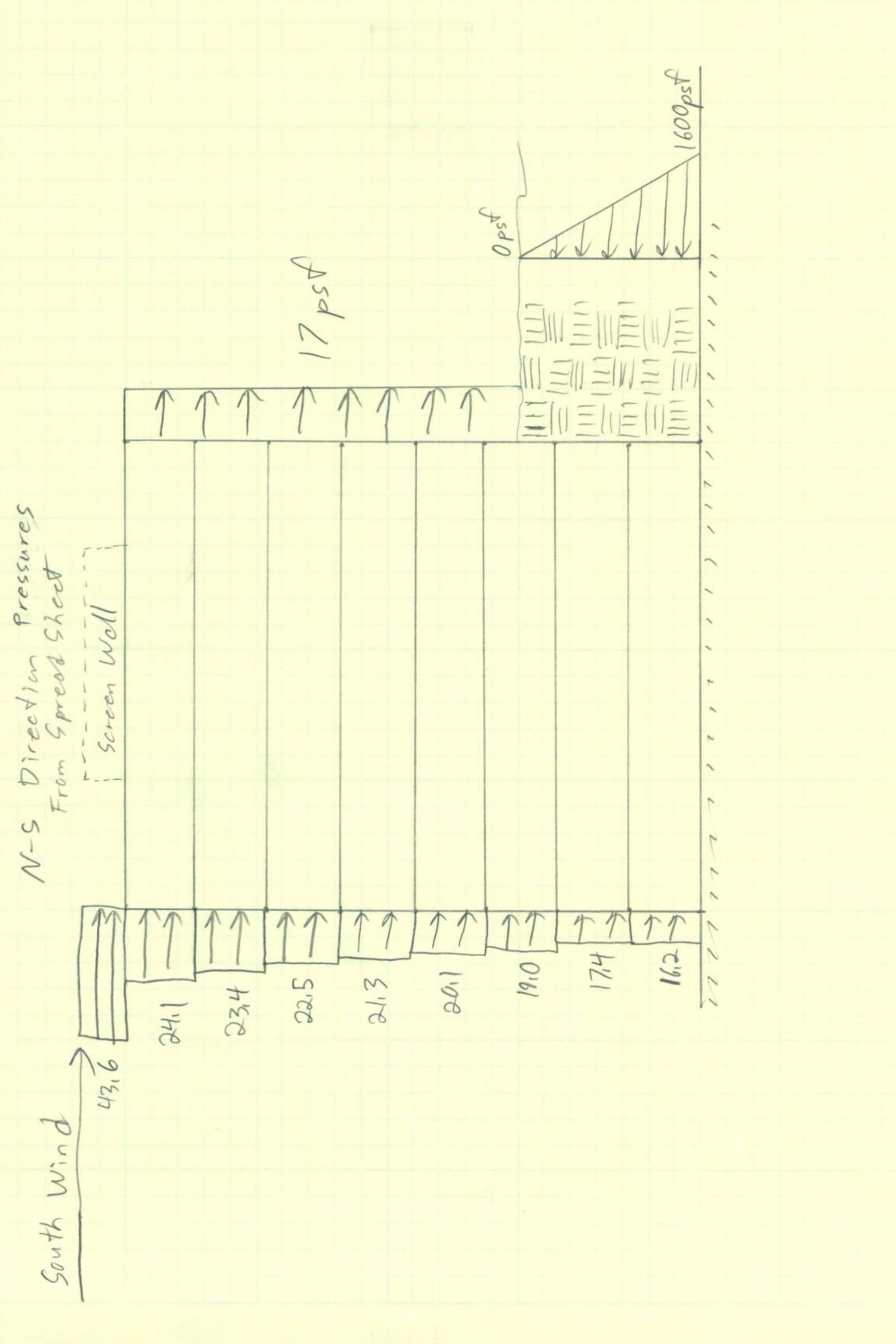
$L/B = \frac{202}{127} = 1.59$

WW wall $C_p = 0.8$ use w/ q_z

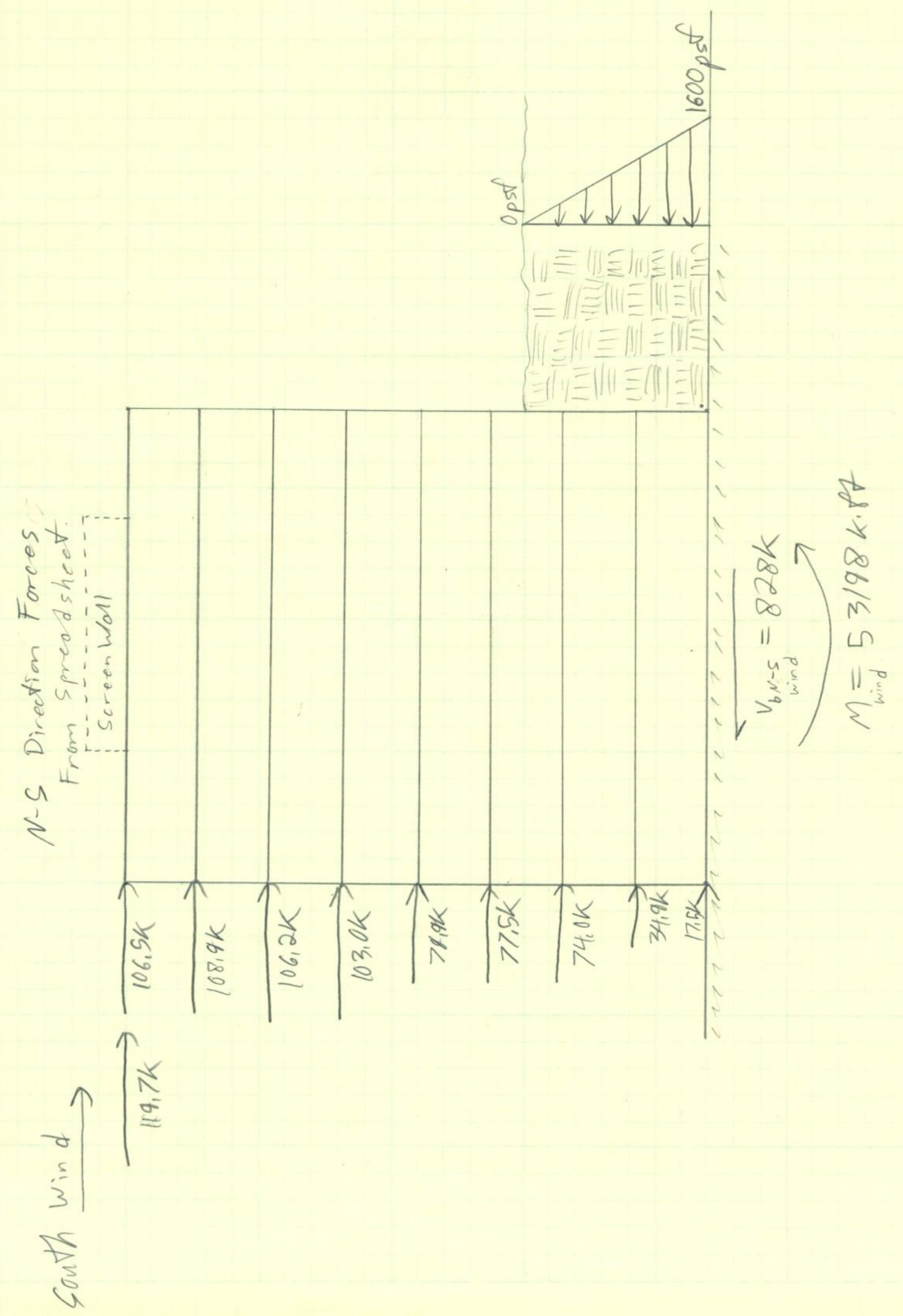
LW wall $C_p = -0.4$ use w/ q_h

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

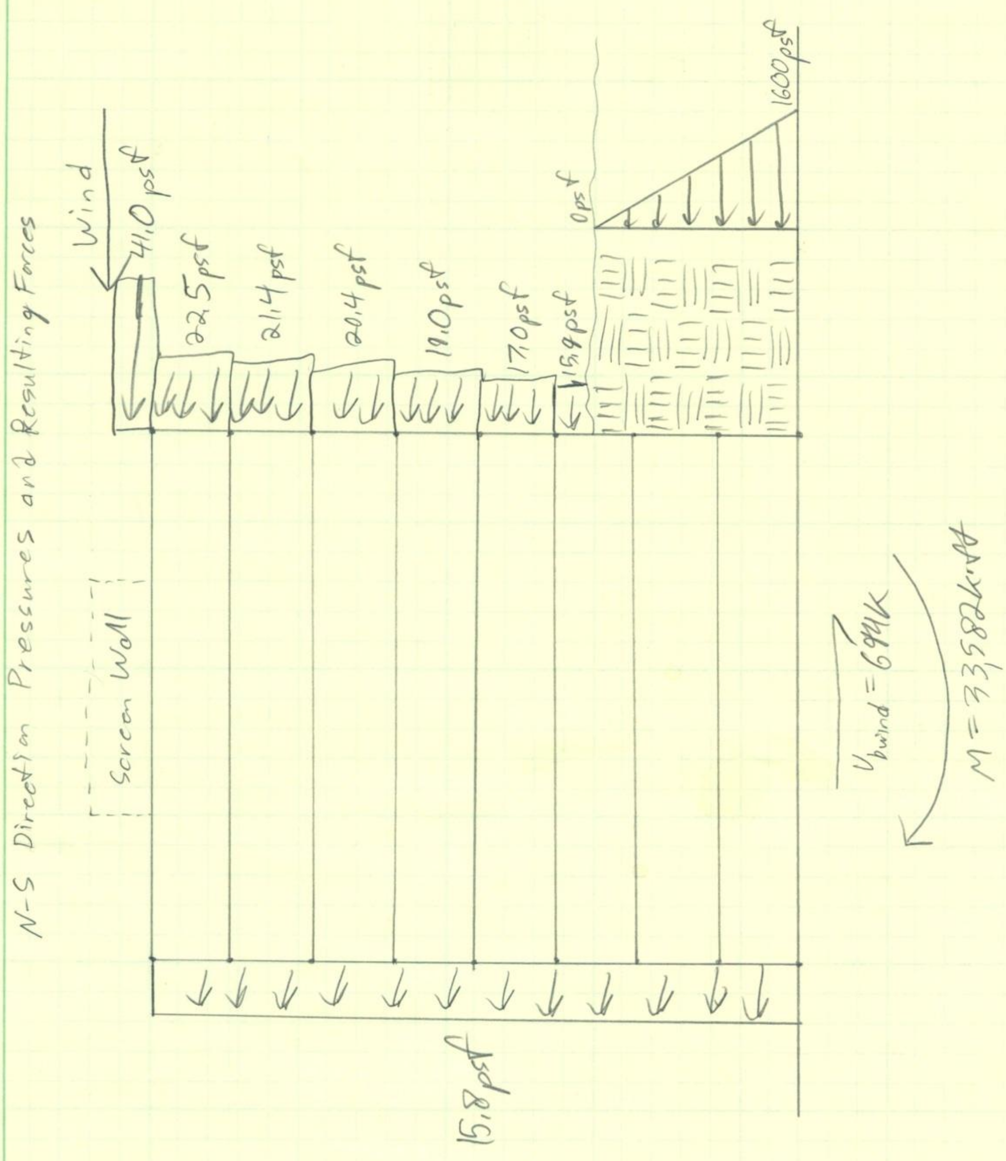
James Chavanic | Tech III | Wind loads Cont



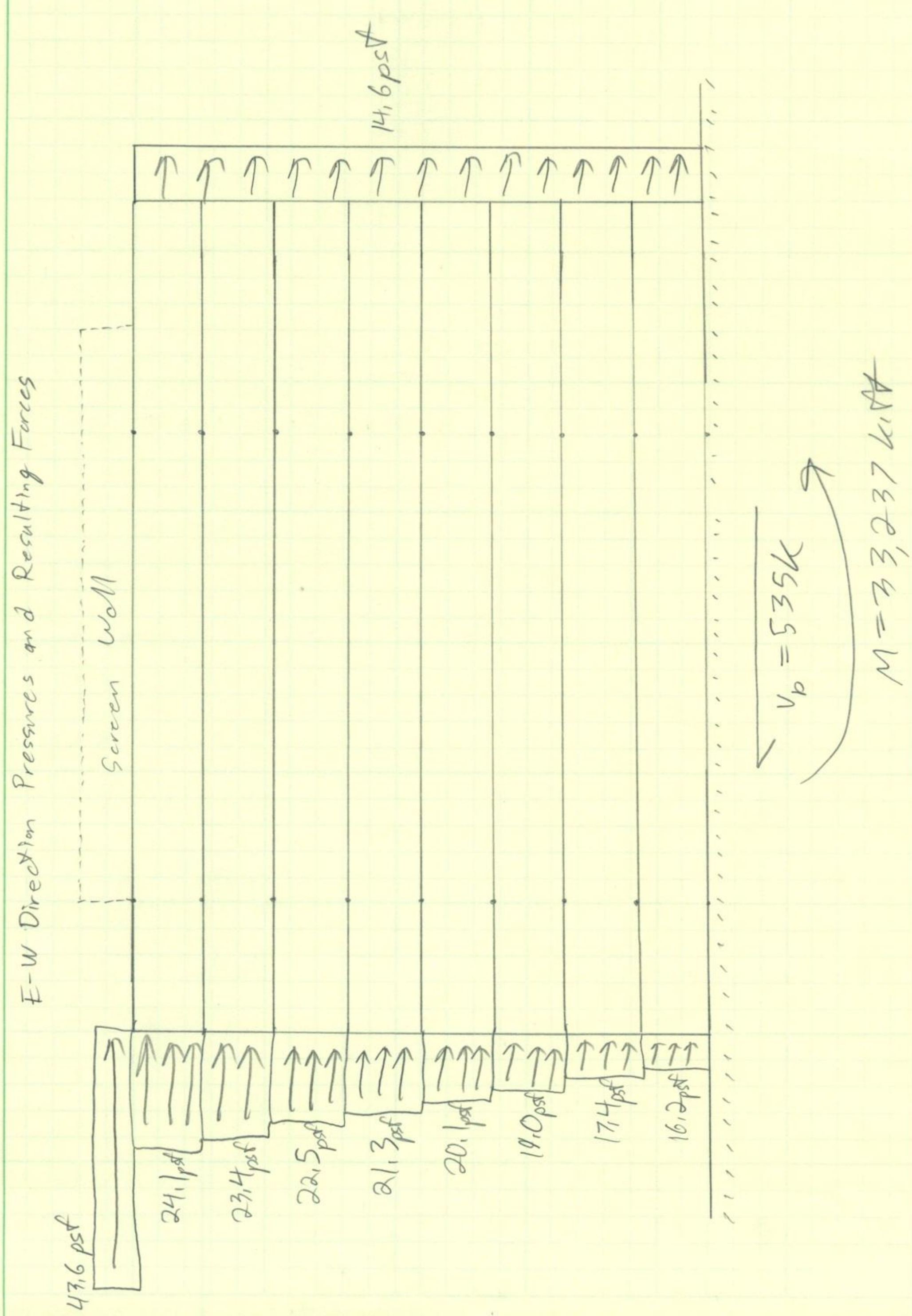
James Chavanic Tech III Wind Loads Cont.



James Chavanic | Tech III | Wind Loads Cont.



James Chavanic | Tech III | Wind Loads Cont.



APPENDIX: C Seismic Load Calculations

James Chavanic | Tech III | Seismic Loads

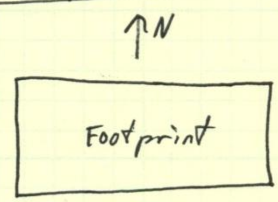
Site Class D (Geotech) Location: Kingstowne, VA

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 III | Seismic Loads Cont.

$V = C_s \cdot W$

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

$C_t = 0.02$
$x = 0.75$ > (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)

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

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

$$\therefore C_{s,min} = \begin{cases} \dfrac{S_{DS}}{\frac{R}{I}} = \dfrac{0.1387}{3/1} = 0.0462 \\[2ex] \dfrac{S_{D1}}{T\left(\frac{R}{I}\right)} = \dfrac{0.0832}{\cancel{0.611} \; 1.039 \cdot \frac{3}{1}} = 0.0267 \; \cancel{0.0454} \end{cases}$$

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

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

$F_x = C_{vx} \cdot V$

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

w_x = weight of each story

h_x = height from base to story of concern

k = exponent related to structure period

– see following spreadsheet –

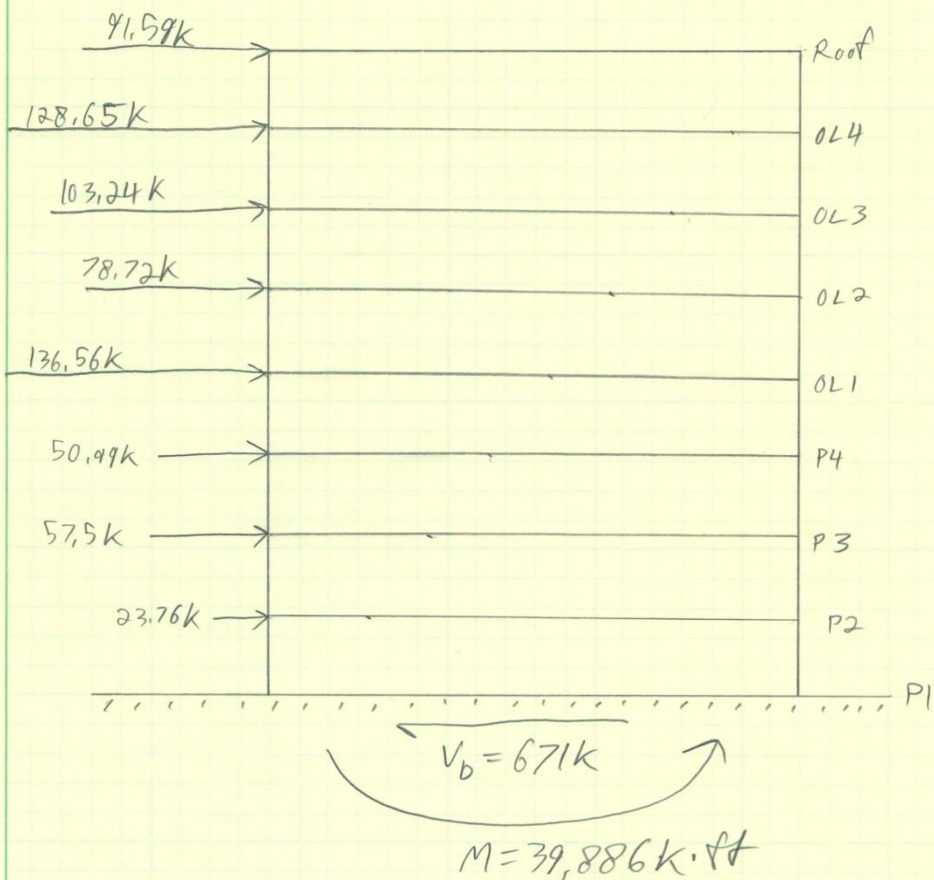
Floor Self Weight Calcs

	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 III | Seismic Loads cont.

From spreadsheet

E-W or N-S



James Chavanic | Seismic Considerations | Tech III

✓ SDC = B

✓ ASCE 7-10 12.5.2

Ortho combo of seismic loads not required for SDC = B

✓ Eccentricity of building used to dermine critical direction to apply accidental torsion

✓ ASCE 7-10 12.7.3

✓ a) consider effects of cracked concrete

cols $= 0.7 I_g$
shear walls $= 0.7 I_g$ > from ACI 318-11 8.8.2

✓ b) Steel Moment frames should account for panel zones

✓ ASCE 7-10 12.8.4.2

Consider Accidental Torsion

✓ ASCE 7-10 12.8.4.3

Accidental torsional Moment amplification not required for SDC = B

ASCE 7-10 12.12.1

Table 12.12-1
Allowable story Drift $\Delta_A = 0.020\, h_{sx}$ for risk cat = II

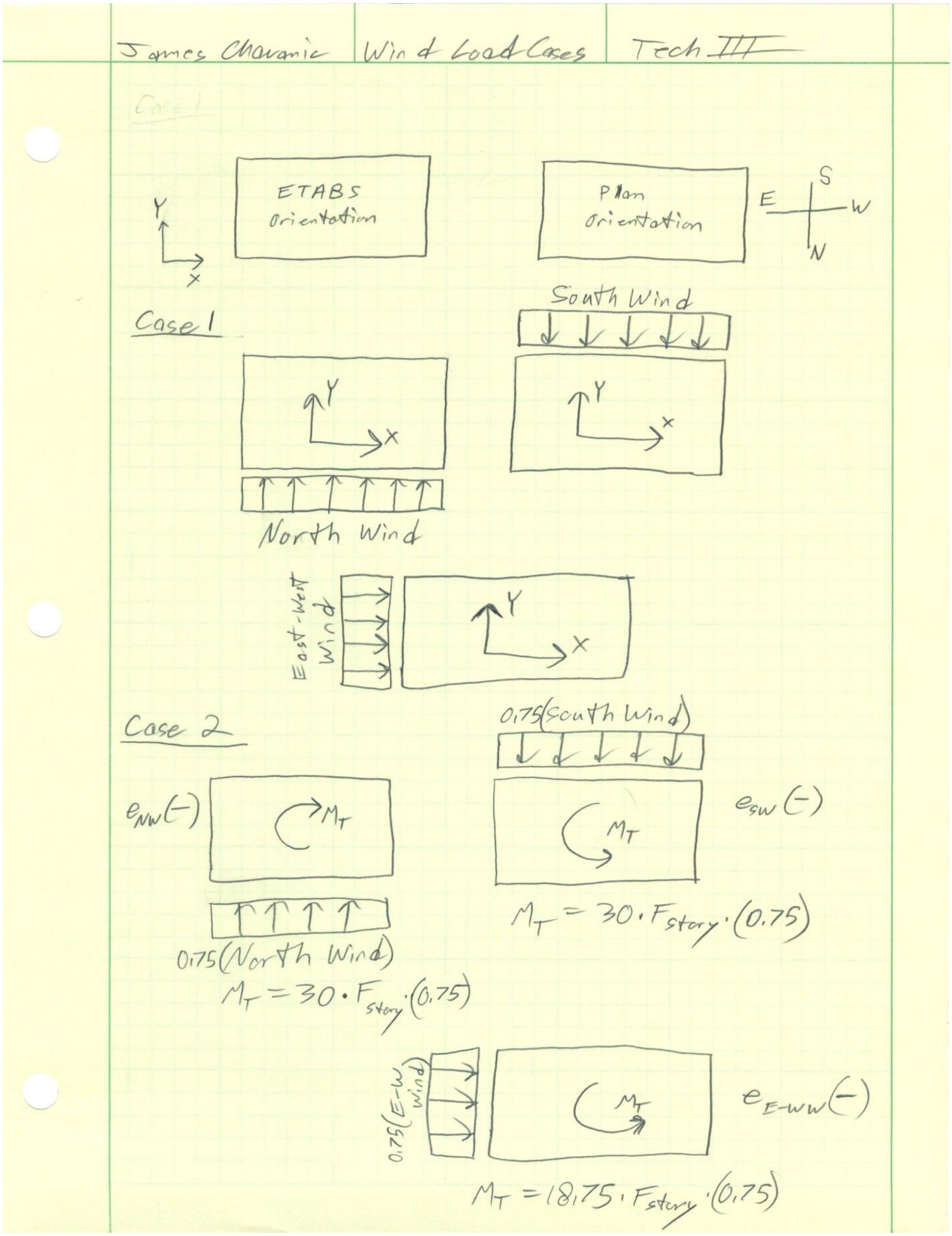
ASCE 7-10 12.8.6

Deflection Amplification

Equation 12.8-15 $\delta_x = \frac{C_d}{I} \delta_{xe}$

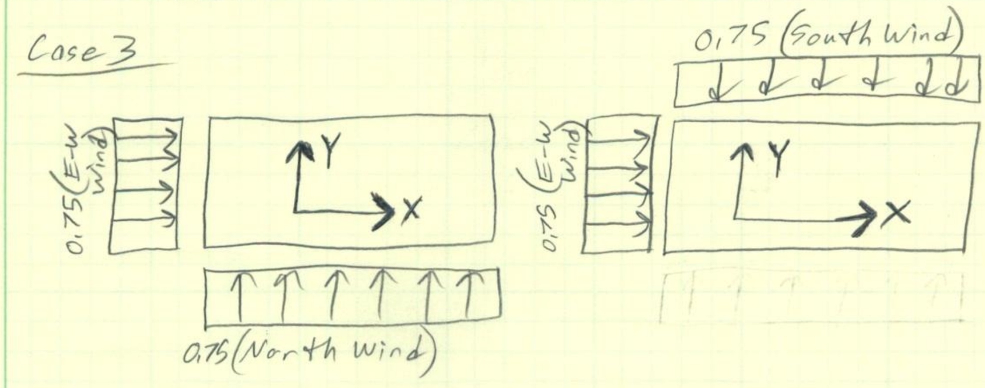
$I = 1.0$ for risk category II
$C_d = 3$ Table 12.2-1
Steel Not Detailed for Seismic

APPENDIX: D Wind Load Cases

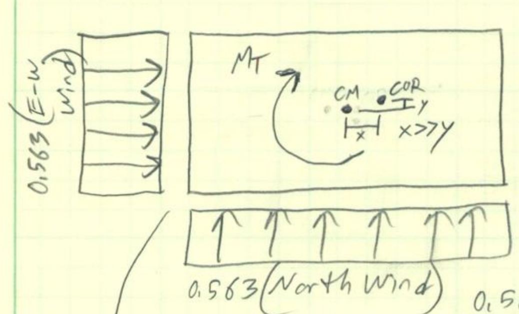


James Chavanic | Wind Load Cases | Tech III

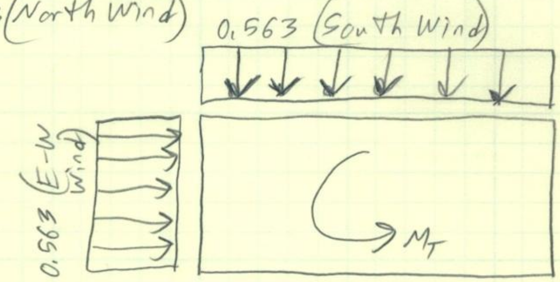
Case 3



Case 4



By inspection
$e_{NW}(-)$
$e_{E\text{-}WW}(+)$
∴ Apply moment in (−) direction



By Inspection
$e_{SW}(-)$
$e_{(E\text{-}WW)}(-)$
∴ apply Moment (+)

$$M_T = 0.563\left[F_{story\,SW}\cdot 30 + F_{story_{E\text{-}W}}\cdot 18.75\right]$$

$$M_T = 0.563\left[F_{story\,NW}\cdot 30 + F_{story_{EW}}\cdot 18.75\right]$$

All Wind Load Cases also consider the effects of the lateral soil load when Modeled.

Story	Load	UX	UY	|UX|	|UY|
ROOF	CASE1NW	-0.0036	0.9603	0.0036	0.9603
ROOF	CASE1SW	0.0032	-0.9759	0.0032	0.9759
ROOF	CASE1EWW	3.3613	0.0248	3.3613	0.0248
ROOF	CASE2NW	-0.0029	0.7289	0.0029	0.7289
ROOF	CASE2SW	0.0023	-0.7269	0.0023	0.7269
ROOF	CASE2EWW	2.5209	0.0248	2.5209	0.0248
ROOF	CASE3NW	2.5184	0.7249	2.5184	0.7249
ROOF	CASE3SW	2.5235	-0.7272	2.5235	0.7272
ROOF	CASE4NW	1.8903	0.553	1.8903	0.553
ROOF	CASE4SW	1.8943	-0.541	1.8943	0.541

ROOF		Controlling Case
Max X (in)	3.3613	CASE1EWW
Max Y (in)	0.9759	CASE1SW

OL4	CASE1NW	-0.0037	0.7694	0.0037	0.7694
OL4	CASE1SW	0.0031	-0.7757	0.0031	0.7757
OL4	CASE1EWW	2.777	0.0234	2.777	0.0234
OL4	CASE2NW	-0.003	0.5843	0.003	0.5843
OL4	CASE2SW	0.0023	-0.5762	0.0023	0.5762
OL4	CASE2EWW	2.0827	0.0236	2.0827	0.0236
OL4	CASE3NW	2.0802	0.5817	2.0802	0.5817
OL4	CASE3SW	2.0853	-0.5772	2.0853	0.5772
OL4	CASE4NW	1.5614	0.4439	1.5614	0.4439
OL4	CASE4SW	1.5653	-0.4277	1.5653	0.4277

OL4		Controlling Case
Max X (in)	2.777	CASE1EWW
Max Y (in)	0.7757	CASE1SW

OL3	CASE1NW	-0.0037	0.541	0.0037	0.541
OL3	CASE1SW	0.003	-0.5349	0.003	0.5349
OL3	CASE1EWW	1.9164	0.0219	1.9164	0.0219
OL3	CASE2NW	-0.003	0.4113	0.003	0.4113
OL3	CASE2SW	0.0022	-0.3948	0.0022	0.3948
OL3	CASE2EWW	1.4372	0.0225	1.4372	0.0225
OL3	CASE3NW	1.4347	0.4103	1.4347	0.4103
OL3	CASE3SW	1.4398	-0.3966	1.4398	0.3966
OL3	CASE4NW	1.0768	0.3135	1.0768	0.3135
OL3	CASE4SW	1.0808	-0.2914	1.0808	0.2914

OL3		Controlling Case
Max X (in)	1.9164	CASE1EWW
Max Y (in)	0.541	CASE1NW

OL2	CASE1NW	-0.0036	0.2988	0.0036	0.2988
OL2	CASE1SW	0.0028	-0.2784	0.0028	0.2784
OL2	CASE1EWW	0.8865	0.0204	0.8865	0.0204
OL2	CASE2NW	-0.0029	0.2293	0.0029	0.2293
OL2	CASE2SW	0.0021	-0.2031	0.0021	0.2031
OL2	CASE2EWW	0.6648	0.0208	0.6648	0.0208
OL2	CASE3NW	0.6624	0.2286	0.6624	0.2286
OL2	CASE3SW	0.6673	-0.2043	0.6673	0.2043
OL2	CASE4NW	0.497	0.1767	0.497	0.1767
OL2	CASE4SW	0.5008	-0.1477	0.5008	0.1477

OL2		Controlling Case
Max X (in)	0.8865	CASE1EWW
Max Y (in)	0.2988	CASE1NW

OL1	CASE1NW	-0.0027	0.0745	0.0027	0.0745
OL1	CASE1SW	0.0019	-0.0406	0.0019	0.0406
OL1	CASE1EWW	0.0854	0.0187	0.0854	0.0187
OL1	CASE2NW	-0.0022	0.0612	0.0022	0.0612
OL1	CASE2SW	0.0013	-0.0261	0.0013	0.0261
OL1	CASE2EWW	0.064	0.0187	0.064	0.0187
OL1	CASE3NW	0.0623	0.0601	0.0623	0.0601
OL1	CASE3SW	0.0657	-0.0262	0.0657	0.0262
OL1	CASE4NW	0.0466	0.0505	0.0466	0.0505
OL1	CASE4SW	0.0493	-0.0153	0.0493	0.0153

OL1		Controlling Case
Max X (in)	0.0854	CASE1EWW
Max Y (in)	0.0745	CASE1NW

P4	CASE1NW	-0.0015	0.0492	0.0015	0.0492
P4	CASE1SW	0.0007	-0.0182	0.0007	0.0182
P4	CASE1EWW	0.0413	0.0165	0.0413	0.0165
P4	CASE2NW	-0.0013	0.0413	0.0013	0.0413
P4	CASE2SW	0.0005	-0.0096	0.0005	0.0096
P4	CASE2EWW	0.0309	0.0166	0.0309	0.0166
P4	CASE3NW	0.0301	0.0408	0.0301	0.0408
P4	CASE3SW	0.0317	-0.0098	0.0317	0.0098
P4	CASE4NW	0.0224	0.035	0.0224	0.035
P4	CASE4SW	0.0238	-0.0033	0.0238	0.0033

P4		Controlling Case
Max X (in)	0.0413	CASE1EWW
Max Y (in)	0.0492	CASE1NW

P3	CASE1NW	-0.001	0.0331	0.001	0.0331
P3	CASE1SW	0.0002	-0.0039	0.0002	0.0039
P3	CASE1EWW	0.0256	0.0153	0.0256	0.0153
P3	CASE2NW	-0.0008	0.0288	0.0008	0.0288
P3	CASE2SW	0.0001	0.0008	0.0001	0.0008
P3	CASE2EWW	0.0191	0.0153	0.0191	0.0153
P3	CASE3NW	0.0187	0.0285	0.0187	0.0285
P3	CASE3SW	0.0196	0.0007	0.0196	0.0007
P3	CASE4NW	0.0139	0.0254	0.0139	0.0254
P3	CASE4SW	0.0146	0.0043	0.0146	0.0043

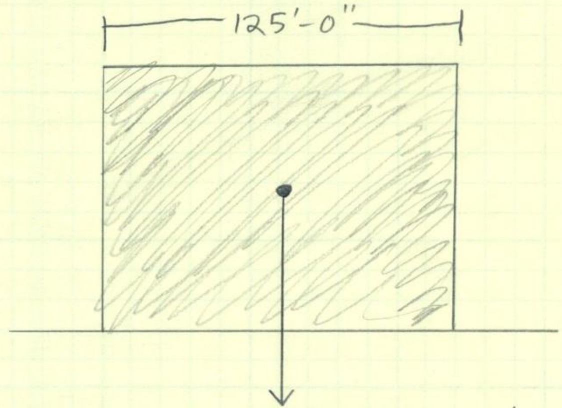
P3		Controlling Case
Max X (in)	0.0256	CASE1EWW
Max Y (in)	0.0331	CASE1NW

P2	CASE1NW	-0.0004	0.0187	0.0004	0.0187
P2	CASE1SW	0	0.0067	0	0.0067
P2	CASE1EWW	0.0114	0.013	0.0114	0.013
P2	CASE2NW	-0.0004	0.0173	0.0004	0.0173
P2	CASE2SW	0	0.0083	0	0.0083
P2	CASE2EWW	0.0085	0.013	0.0085	0.013
P2	CASE3NW	0.0083	0.0173	0.0083	0.0173
P2	CASE3SW	0.0087	0.0083	0.0087	0.0083
P2	CASE4NW	0.0062	0.0162	0.0062	0.0162
P2	CASE4SW	0.0065	0.0094	0.0065	0.0094

P2		Controlling Case
Max X (in)	0.0114	CASE1EWW
Max Y (in)	0.0187	CASE1NW

APPENDIX: E Overturning Moment Check

James Chavanic | Overturning Moment | Tech III



Bldg wt = 25,132k

Worst Case Overturning Moment

Soil + Earthquake

$$M = 31{,}593\ \text{k·ft} \quad + \quad M = 39{,}886\ \text{k·ft}$$

$$= \quad M_{comb} = 71{,}479\ \text{k·ft}$$

$$M_{resisting} = \frac{2}{3} \cdot \text{Bldg. wt} \cdot \frac{\text{Least Dim}}{2}$$

$$= \frac{2}{3} \cdot 25{,}132\text{K} \cdot \frac{125\ \text{ft}}{2}$$

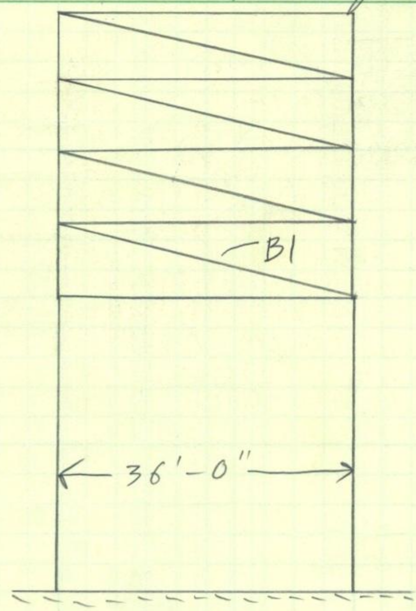
$$= 1{,}047{,}167\ \text{k·ft}$$

$$F.S. = \frac{M_{resisting}}{M_{overturning}} = \frac{1{,}047{,}167}{71{,}479}$$

$$\boxed{F.S = 14.65}$$ ∴ Building will resist overturning Moment

APPENDIX: F Strength Spot Checks

James Chavanic | Strength Checks | Tech III



SW1 / BF1

From ETABS

Max Shear in SW1

$V_u = 665k$ (EQYTSOIL)

$f'_c = 5000psi$

Brace B1 HSS 10x10x5/8

Max forces from ETABS

Tension 247k (Case 1 SW)

Compression 228.1k (Case 1 NW)

Unbraced Length = 38.4 ft

$\phi P_n = 328k > 228.1k$ ✓

$\phi P_{n\,yielding} = 869k > 247k$ ✓

$\phi P_{n\,rupture} = 687k > 247k$ ✓

∴ Brace is adequate

Shear Wall check

Max Shear Strength

$V_u < \phi V_n = \phi \cdot 10\sqrt{f'_c} \cdot hd$

$d = 0.8 \cdot \ell_w = 0.8 \cdot 36' \cdot 12'' = 345.6''$

$\phi V_n = 0.75 \cdot 10 \cdot \sqrt{5000} \cdot 12'' \cdot 345.6''$

$\phi V_n = 2{,}200k > 635k$ ✓

Shear strength Provided by V_c

$V_c = 2\sqrt{f'_c} \cdot hd$

$= 2 \cdot \sqrt{5000} \cdot 12'' \cdot 345.6''$

$= 586k > 635k$ ✗

However, argue that wall will see some amount of axial load

$V_c = 3.3\sqrt{f'_c} \cdot hd + \frac{N_u d}{4\ell_w}$ conservatively say 0

$V_c = 3.3\sqrt{5000} \cdot 12'' \cdot 345.6''$

$V_c = 968k$ $\quad \phi V_c = 725k > 635k$ ✓ ∴ Shear wall is sufficient

APPENDIX: G Center of Rigidity Calculation

X COR Story P1		
Element	Stiffness (% K_{story})	Dist. From Origin (ft)
SW1	14.32	60.75
SW2	18.42	114.5
SW3	17.53	143
SW7	6.01	28.5
SW8	8.68	57
SW9	8.62	85.6
SW10	8.61	114.5
SW11	8.79	143
SW12	8.7	171.5
	X COR =	106.59

Y COR Story P1		
Element	Stiffness (% K_{story})	Dist. From Origin (ft)
MF1	1.71	125
MF2	1.48	0
SW4	84.14	62.75
SW5	5.48	74
SW6	5.61	45
	Y COR =	62.50

X COR Story P2		
Element	Stiffness (% K_{story})	Dist. From Origin (ft)
SW1	31.49	60.75
SW2	37.44	114.5
SW3	30.67	143
	X COR =	106.28

Y COR Story P2		
Element	Stiffness (% K_{story})	Dist. From Origin (ft)
MF1	1.38	125
MF2	1.22	0
SW4	95.28	62.75
SW5	0.58	74
SW6	1.02	45
	Y COR =	62.73

X COR Story P3		
Element	Stiffness (% K_{story})	Dist. From Origin (ft)
SW1	30.27	60.75
SW2	36.68	114.5
SW3	32.41	143
	X COR =	107.42

Y COR Story P3		
Element	Stiffness (% K_{story})	Dist. From Origin (ft)
MF1	3.34	125
MF2	3.19	0
SW4	75.7	62.75
SW5	7.04	74
SW6	8.48	45
	Y COR =	62.10

X COR Story P4		
Element	Stiffness (% K_{story})	Dist. From Origin (ft)
SW1	21.81	60.75
SW2	36.28	114.5
SW3	33	143
	X COR =	111.96

Y COR Story P4		
Element	Stiffness (% K_{story})	Dist. From Origin (ft)
MF1	14.37	125
MF2	14.3	0
SW4	54.44	62.75
SW5	7.54	74
SW6	7.21	45
	Y COR =	62.28

X COR Story OL1		
Element	Stiffness (% K_{story})	Dist. From Origin (ft)
BF1	48.46	60.75
BF2	25.72	114.5
BF3	25.82	143
	X COR =	95.81

Y COR Story OL1		
Element	Stiffness (% K_{story})	Dist. From Origin (ft)
MF1	45.46	125
MF2	45.47	0
	Y COR =	62.49

X COR Story OL2		
Element	Stiffness (% K_{story})	Dist. From Origin (ft)
BF1	44.14	60.75
BF2	27.78	114.5
BF3	28.08	143
	X COR =	98.78

Y COR Story OL2		
Element	Stiffness (% K_{story})	Dist. From Origin (ft)
MF1	50	125
MF2	50	0
	Y COR =	62.50

X COR Story OL3		
Element	Stiffness (% K_{story})	Dist. From Origin (ft)
BF1	34.24	60.75
BF2	32.94	114.5
BF3	32.82	143
	X COR =	105.45

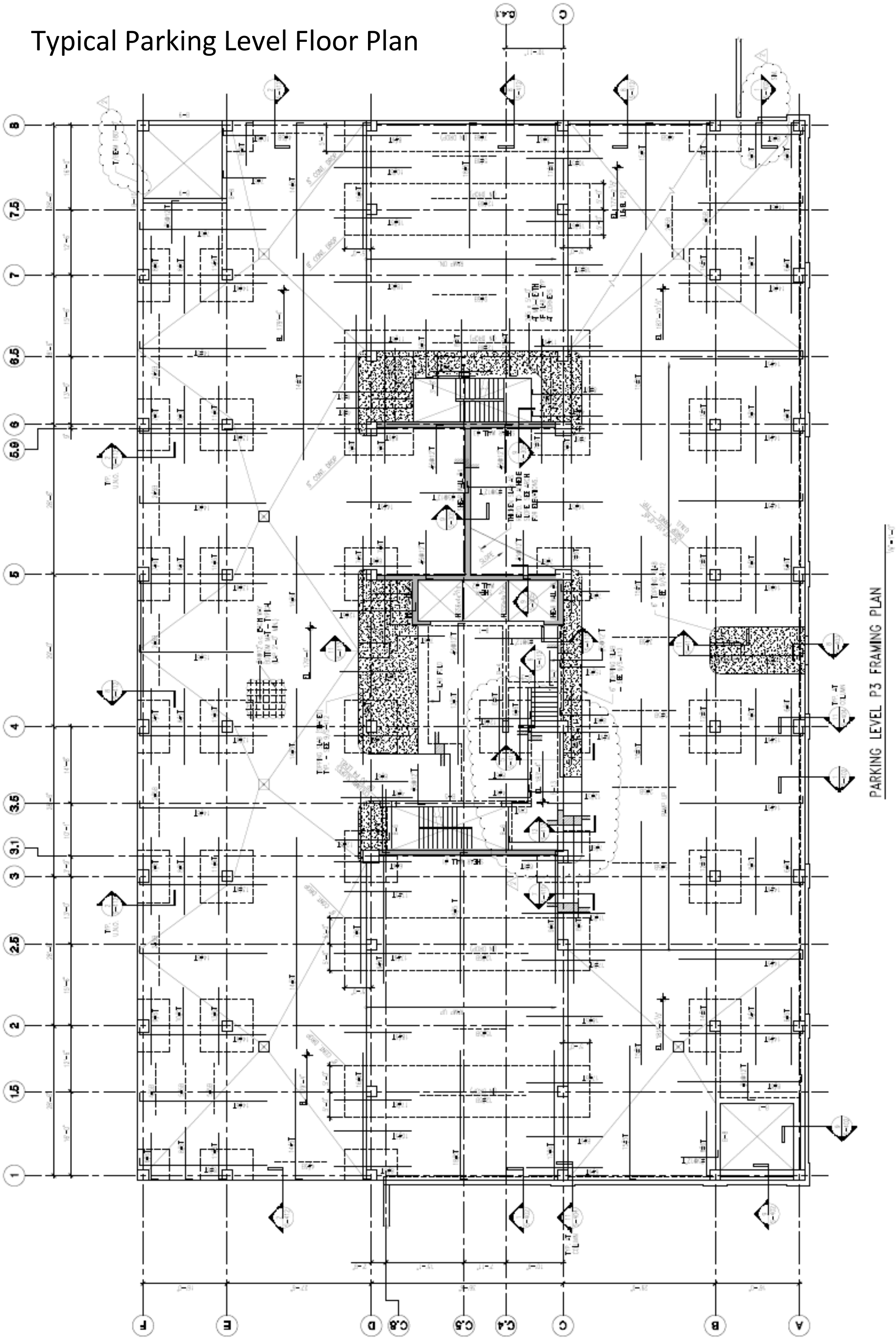
Y COR Story OL3		
Element	Stiffness (% K_{story})	Dist. From Origin (ft)
MF1	49.99	125
MF2	50.01	0
	Y COR =	62.49

X COR Story OL4		
Element	Stiffness (% K_{story})	Dist. From Origin (ft)
BF1	33.87	60.75
BF2	36.41	114.5
BF3	29.73	143
	X COR =	104.77

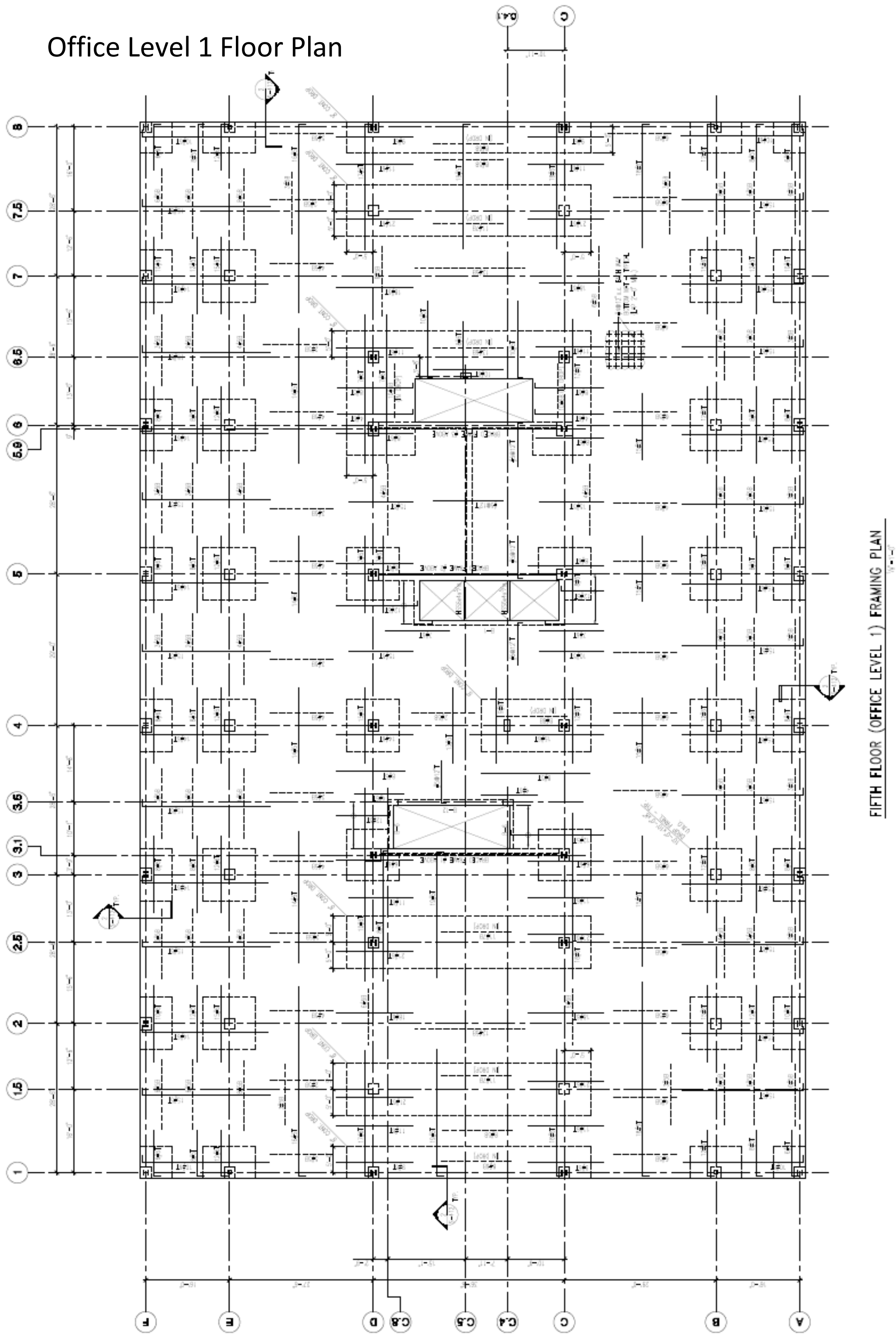
Y COR Story OL4		
Element	Stiffness (% K_{story})	Dist. From Origin (ft)
MF1	49.99	125
MF2	50.01	0
	Y COR =	62.49

APPENDIX: H Typical Building Plans

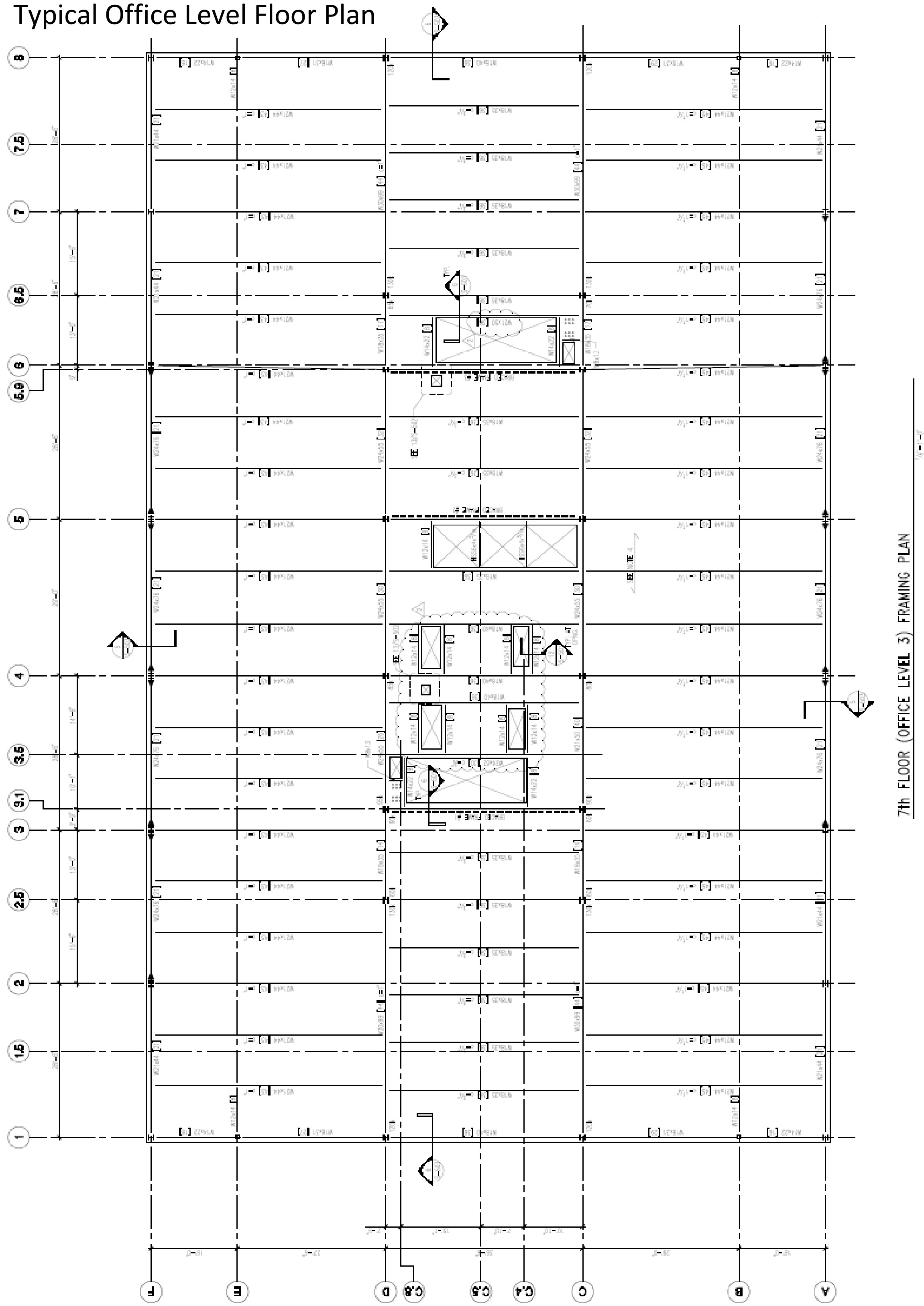
Typical Parking Level Floor Plan



Office Level 1 Floor Plan



Typical Office Level Floor Plan



APPENDIX: I Lateral Resisting Walls and Frames