Belmont Executive Center
Building A
Ashburn, VA

Technical Report #1
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Executive Summary

The purpose of Technical Report 1 is to analyze and describe the existing structural system of The Belmont Executive Center; Building A. Descriptions of the foundation system, floor system, column layout, roof system, and lateral system are provided. A list of materials and design codes are also provided. Wind loads are calculated using the analytical method 2, and seismic loads were calculated in accordance to seismic design category A. The wind loads I calculated were reasonable for a 5 story office building. They were within 10% of the wind loads noted on the structural drawings. Seismic calculations resulted in very low base shear, and wind is the controlling load for the lateral system. Beam and column spot checks were also completed and it was concluded that the structural members provide adequate support.

Introduction

The Belmont Executive Center; Building A is located in the Belmont Executive Center, which will include office, retail, restaurant, daycare, and hotel spaces. Residents of the Dulles North area will be offered daily shopping, specialty shopping, and dining choices.

Building A is a 125,000 SF, 5-story office building designed to accommodate multiple tenants. The façade of the building is constructed primarily of brick on light gage metal studs. Vertical brick columns are spaced around the perimeter façade, some of which enclose structural columns, others which do not support any load. A large curtain wall system distinguishes the entrance of the building, and the corners of the building also have a curtain wall system. The structural system of the building is constructed of steel framing with light weight concrete on composite deck as the floor system. Lateral bracing is provided by four concentrically braced frames.

Structural System

Foundation System

The foundation system is made up of spread footings located at the base of the steel columns, and range from 19'-6" square to 10'-6" square, depending on the location. Larger footings are located in the center right part of the building, to support a mechanical room and the restrooms. Smaller foundations are located at the exterior columns. All larger foundations are shown in yellow in the Figure 1 below. The perimeter footings are connected by grade beams that support the masonry facade. A stepped grade beam is located just to the left of the entrance to allow a connection to the sanitary line. There is also a stepped grade beam on the right side of the building for the domestic water line and fire service line connection. The ground floor is a 5" thick concrete slab on grade reinforced with #3 rebar @ 15"o.c. running both directions. A 6" slab on grade is located to the right side of the building to support a 30 yard trash compactor, and is highlighted in purple in Figure 1. It is reinforced with #3 rebar @ 12"o.c. each way. The slabs are supported by 4" granular material, on top of compacted soil.

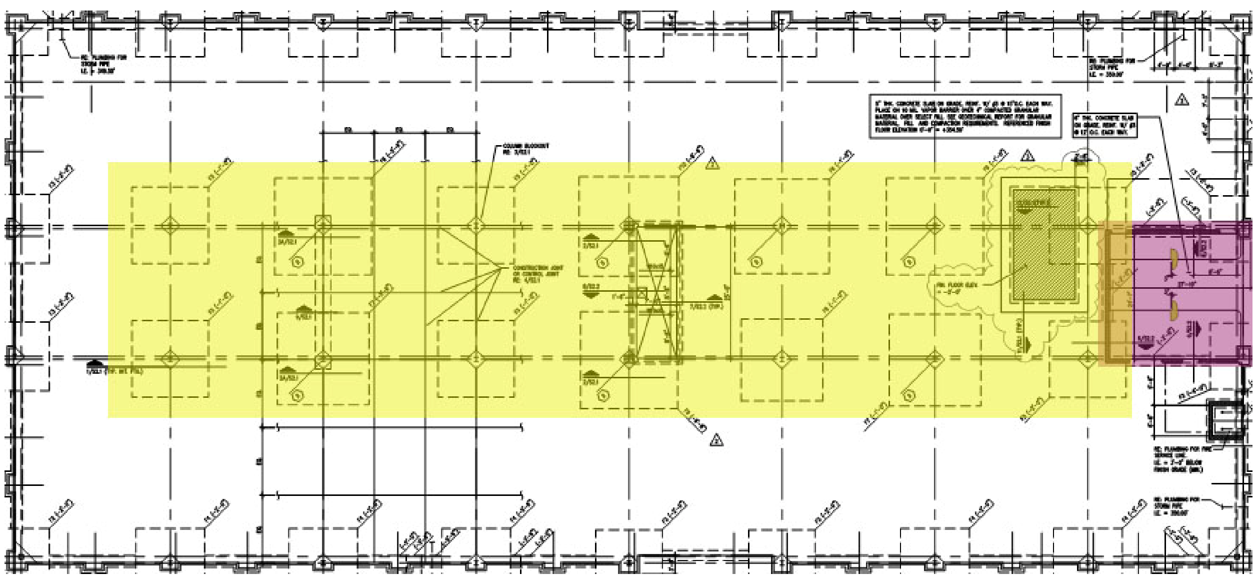


Figure 1 showing the foundation layout

Column System

The floor and roof system are supported by three column lines in the north-south direction and nine rows of columns in the east-west direction. Because the exterior column spacing is dictated by the architecture of the building, the columns on the left and right side of the building are offset from those in the interior. At the corners of the building they are offset by 1'-3" and the interior columns are offset by 7 ¼". This offset creates a slight skew in the beams spanning from the exterior to the interior. Figure 3 shows the column offset. Most of the columns are W shape steel beams, and a few are HSS columns. Hollow structural steel columns are located at the front left and right corners of the building. They are also used in the left rear and right rear corners, on floors three to five, and to provide intermediate bracing below the exterior terrace on the fifth floor. The typical bay sizes for each floor is 38'x 30' and 26'x30'. Figure 3 shows the typical column layout.

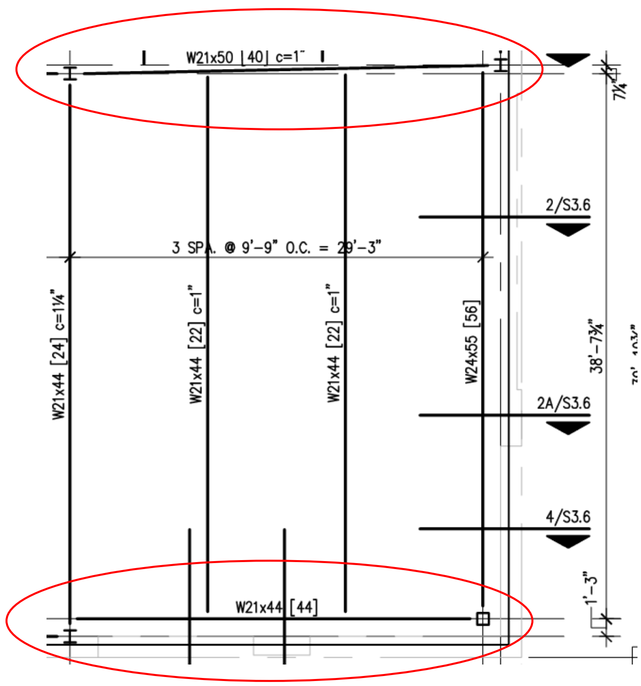


Figure 2: Column Offset

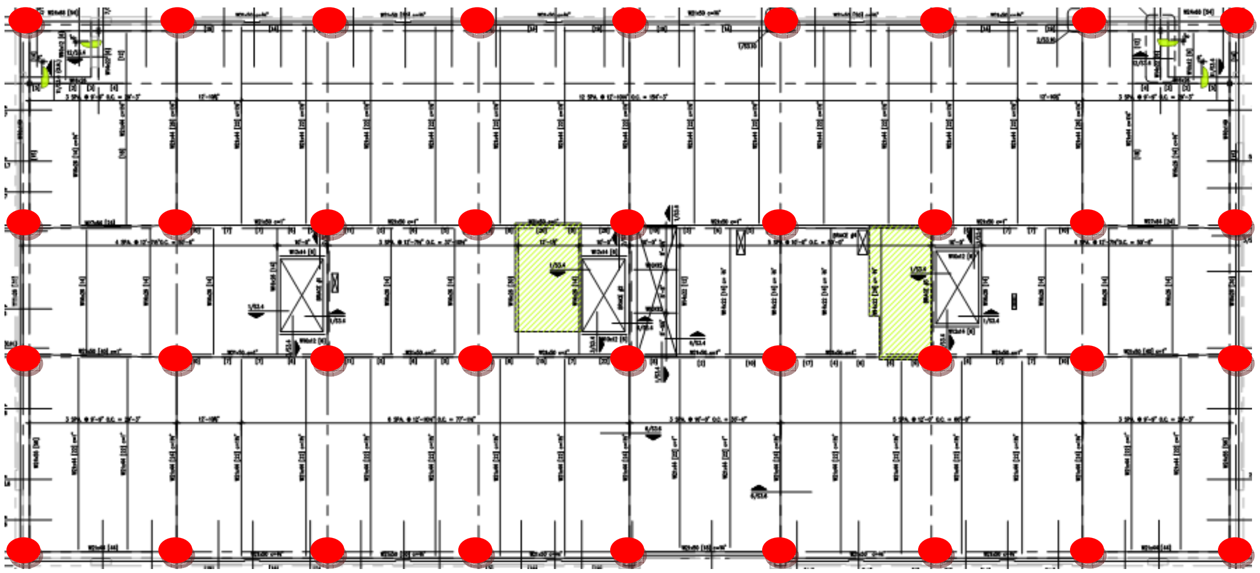


Figure 3: Column Layout

Floor System

Floors 2-4 are constructed of 3-1/4” light weight concrete, on 3” composite metal deck. The deck is reinforced by 6 x6 - W1.4 x W1.4 welded wire fabric, and supported by W-shape steel beams. There are three bays in the north-south direction, and ten in the east-west direction of the building. For reference, the outer lying bays are highlighted in red, and the middle bay is highlighted in green, see Figure 4. Typically, there are W21x44 beams spaced 12’-10 ¼” to 9’-9”, on floors 2 through 5, in the two outside bays. In the middle bay the beams are typically W16x26. Between the elevators and stairwell three, the steel beams are W14x22. Composite action is provided shear studs, and most beams also have upward camber ranging from ¾” to 1” to compensate for service and live load deflections. W 21x50 girders support the load reactions from the beams. On the second floor there is no framing at the main entrance, because this area is open to the ground floor. Floors 3-5 are framed similarly. On the fifth floor the exterior terrace floor is supported by W10x12 steel beams.

The mechanical equipment in the penthouse is supported by a typical concrete floor, constructed of lightweight concrete on composite metal deck. This is the only concrete slab on the roof level. W16x26 beams span across the bay to support the floor.

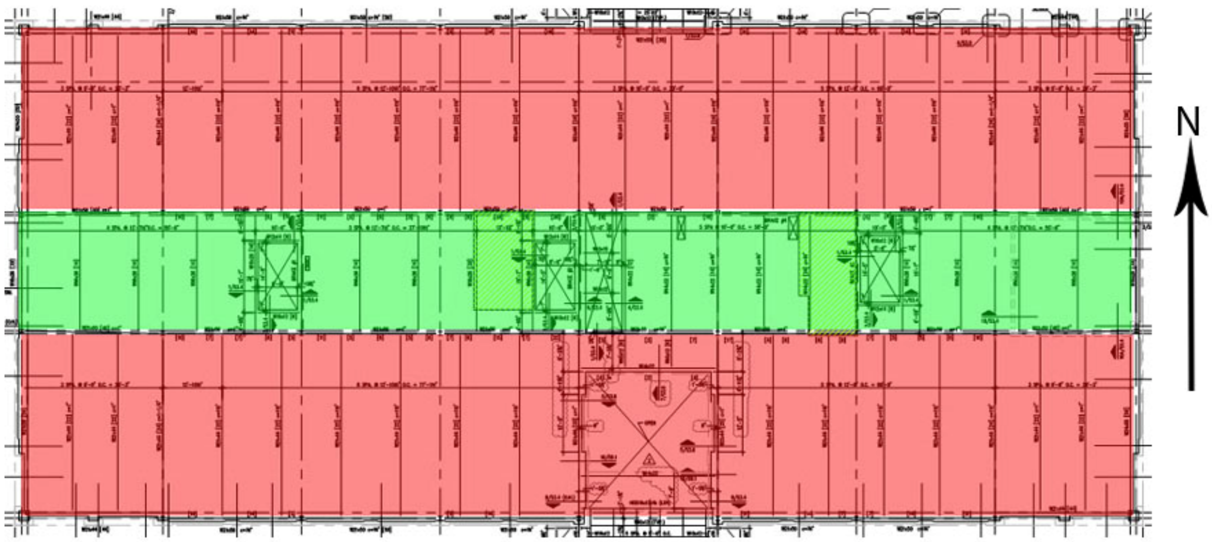


Figure 4 typical beam size and spacing

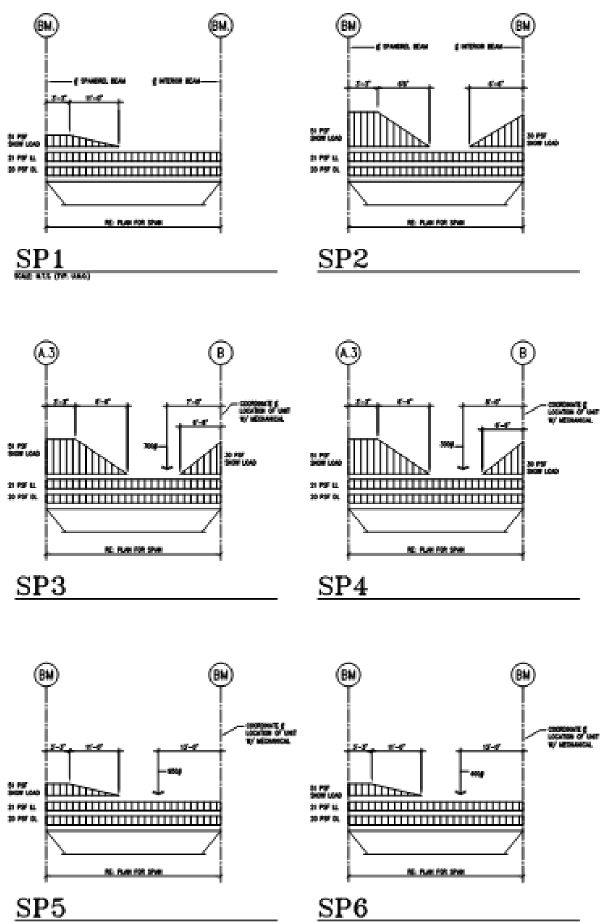


Figure 5: Special Loading Conditions

Roof System

The roof system is supported by K-series joist, spanning across the three bays in the north-south direction. All the joists in the outside two bays are spaced at 6'-0" on center. Joists in the front and rear bays were designed for specifically by the joist manufacturer for snow drifting, because this can be a critical load failure for open web joists. All joists that were specially designed are denoted by SP, and there are 6 different loading conditions. Each loading condition is shown in Figure 5. Three rows of bracing are provided in the rear bay, to prevent lateral torsional buckling. Regular K series joists ranging from 22K5 to 18K3 support the roof in the middle bay. The penthouse roof is supported by 20K3 spaced at 6'-0", with 3 rows of bridging.

The standing seam metal roof screen that shields the penthouse from view is supported by a combination of K Series joists and W shape beams. At roughly every 30’ W shaped steel beams are angled at 45 degrees, and are supported by steel posts. Between the beams, four K series joists run parallel to the building perimeter. L 2 x 2 x1/8” angle provides bracing at 6’, between the joists. Figure 6 shows the angled beams, highlighted in yellow, and the joists can be seen spanning between them. Figure 7 shows a typical cross section of the roof screen.

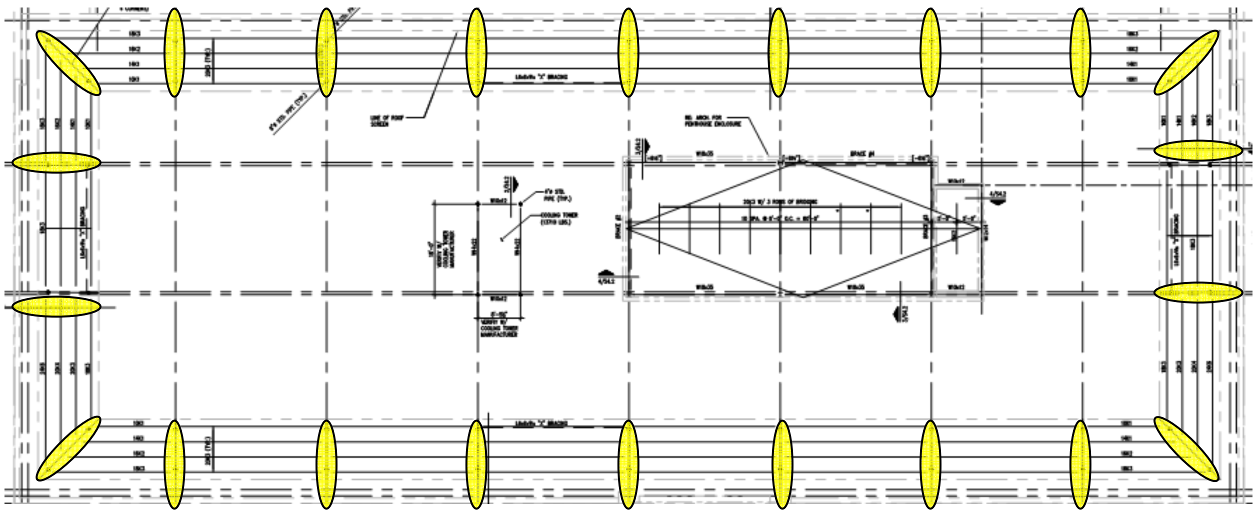


Figure 6: Angled W Shape Beams

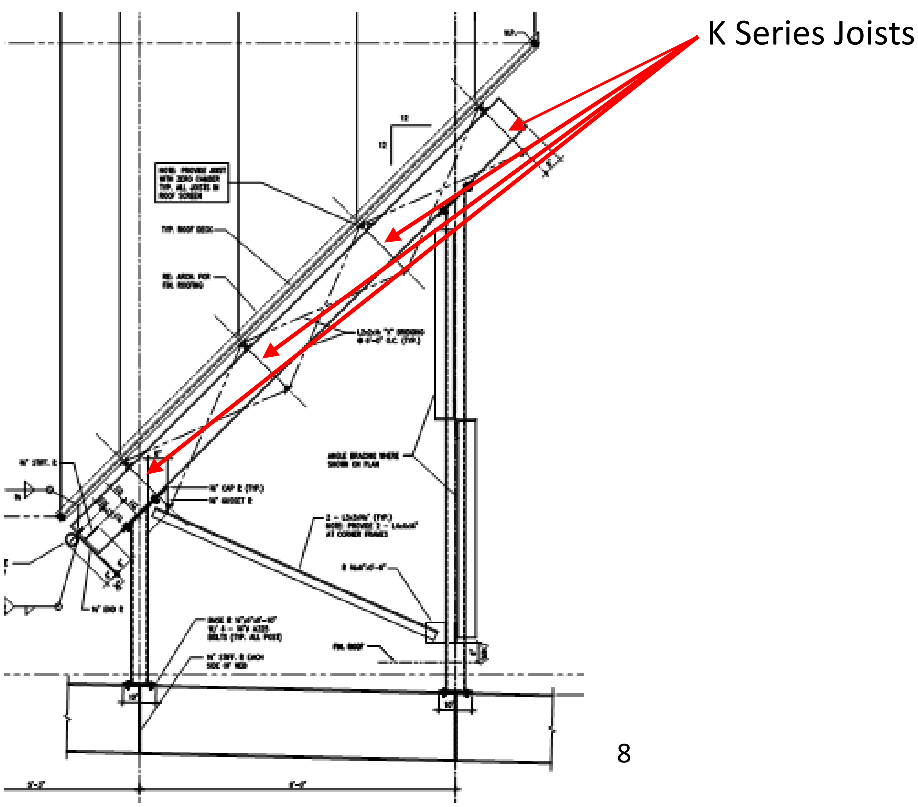


Figure 7: Roof Screen Support

Lateral System

Lateral loads on the building are supported by four concentrically braced frames. Three of the frames are located in the north-south direction to support higher wind loads from the broad side of the building, and one frame is located in the east-west direction. The three frames in the north-south direction are located on the column lines, adjacent to stairwell one and two. The other is located to the left of stairwell three. In the east-west direction the frame is located between columns B6 and B7. All frames are braced with hollow structural steel ranging in size 8 x 8 x ¼ at the first floor to 4 x 4 x ¼ on the fifth floor. Figure 8 shows the elevations of each braced frame, and Figure 9 shows the location of each frame.

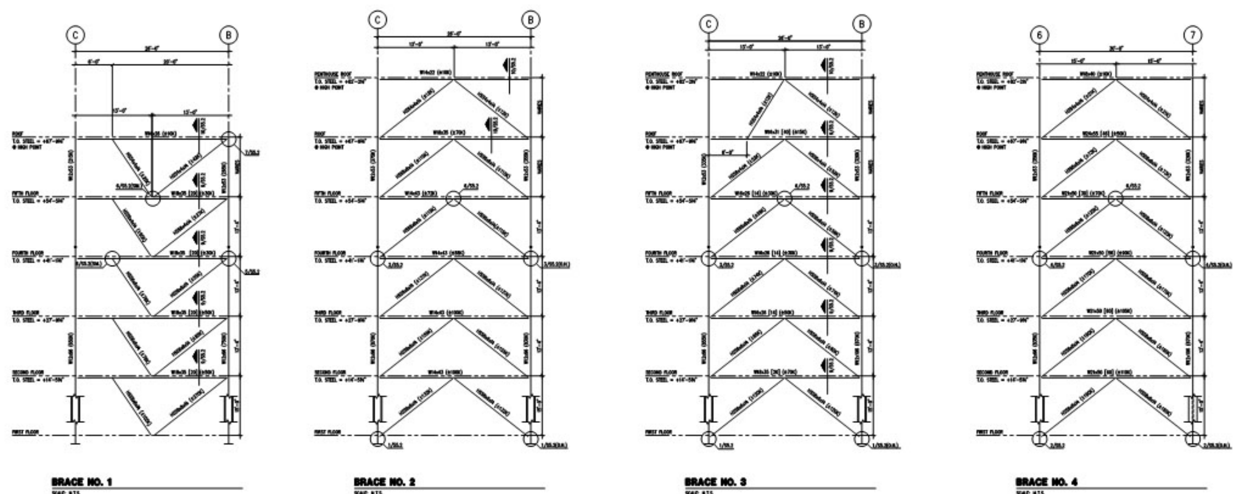


Figure 8: Braced Frame Elevations

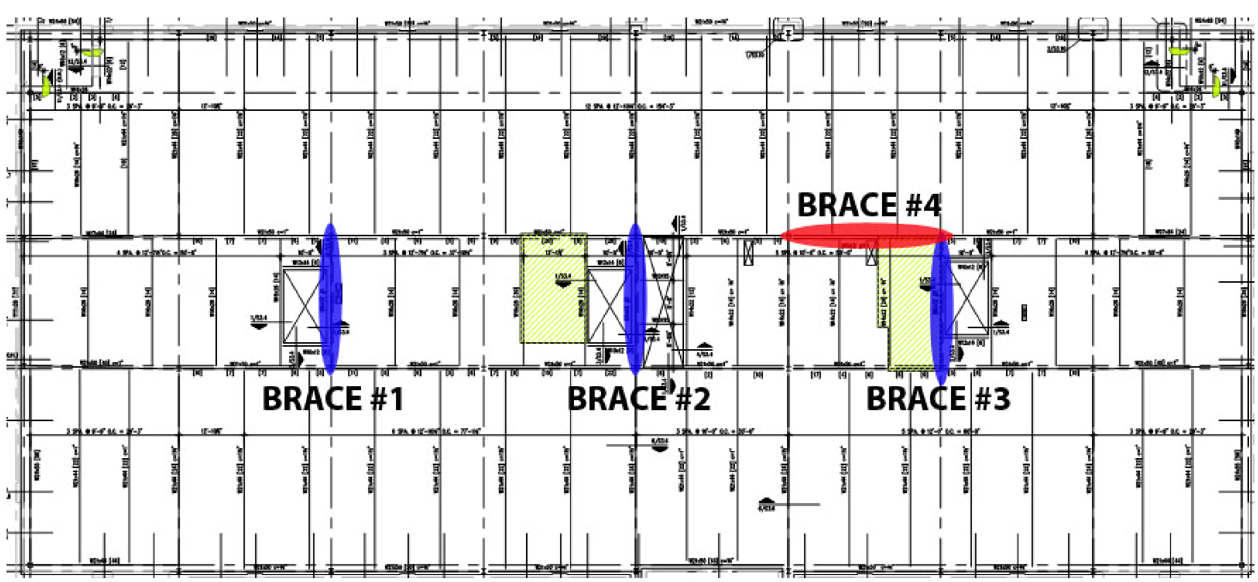


Figure 9: Location of Braced Frames

Materials

Concrete – All concrete shall have natural sand fine aggregate, and Type I Portland Cement conforming to ASTM C150. Concrete in the footings, pilasters, and slabs on grade shall be prepared with normal weight course aggregates conforming to ASTM C33. The concrete in the composite slabs shall have lightweight course aggregates conforming to ASTM C330, and a maximum unit weight of 115 pounds per cubic foot.

Compressive Strength

Footings	3000 psi
Pilasters	3000 psi
Slabs on Grade	4000 psi
Composite Slabs	3500 psi

Reinforcing Bars – Must conform to ASTM A615, grade 60.

Welded Wire Fabric – Must conform to ASTM A185.

Roof Deck – All Type B deck shall be 22 gage cold formed steel conforming to ASTM A653 SQ Grade 33. The deck shall be 1 – ½ inches deep and have a minimum section modulus of 0.186 inches cubed per foot of width.

Composite Steel Deck - Composite steel deck shall be 18 gage minimum cold-formed steel conforming to ASTM A611, Grade D and shall have a phosphatized and painted lower surface and a phosphatized only top surface. The deck shall be 3 inches deep and shall have a minimum section modulus of 0.803 inches cubed per foot of width.

Structural Steel

W Shapes – Shall conform to ASTM A992
Other Steel Shapes, Plates, Angles and Channels – Shall conform to ASTM A36
Steel Pipe – Shall conform to ASTM A53, Grade B
Steel Tubing – Shall conform to ASTM A500, Grade B
Anchor Bolts – Shall conform to ASTM F1554, Grade 36
Bolts – Shall meet or exceed the requirements of ASTM A325, Type N, X, or F

Concrete Masonry

Concrete masonry shall have a minimum compressive strength of 1500 PSI on the net cross sectional area at 28 days

Masonry Units – Shall be grade N, Type I light weight or medium weight hollow concrete units meeting fire rating requirements and conforming to the requirements of ASTM C90

Mortar – shall conform to the requirements of ASTM C270, type M or S

Grout – shall conform to ASTM C476

Codes

Building Code

Virginia USBC (IBC 2000)

Structural Steel

AISC Specification for Structural Steel Buildings

AISC Code of Standard Practice for Steel Buildings and Bridges

*Exception of paragraph 4.2.1 – Deletion of the following sentence: "This approval constitutes the owner's acceptance of all responsibility for the design adequacy of any connections designed by the fabricator as part of his preparation of these shop drawings."

AISC Manual of Steel Construction – Allowable Stress Design, 9th Addition

Steel Joist Institute Standard Specifications for Open Web Steel Joists

AISI Specification for the Design of Cold-Formed Steel Structural Members

Concrete

ACI Details and Detailing of Concrete Reinforcement, ACI 315

ACI Detailing Manual, ACI SP-66

ACI Manual of Engineering and Placing Drawings for Reinforced Concrete Structures, ACI 315R

CRSI Manual of Standard Practice

Concrete Masonry

ACI Building Code Requirements for Concrete Masonry Construction, ACI 530

ACI Specifications for Masonry Structures, ACI 530.1

Design Loads

International Building Code 2000

American Society of Civil Engineers (ASCE), ASC- 7

Gravity Loads

Snow Load

Snow loads were calculated in accordance with ASCE 7-05 Chapter 7. As mentioned earlier, special snow drift conditions were considered for the K series joists supporting the roof. Snow drifting was considered against the parapet, and the penthouse. One calculation determined that a load of 49 – 50 pounds per square foot should be applied where drifting occurs. This matches the loading of the structural engineers, who calculated a load of 51 psf. The calculated ground snow load 21 PSF also matched the load listed in the structural notes. See Appendix A for calculations.

Dead/Live Loads

Live Loads	
Area	**Design Load**
Office Space	100
Permanent Corridors	100
Lobbies, Stairs, and Assembly Areas	100
Mechanical Space	125
Light Storage (Mechanical Rooms)	125
Roof	30
Dead Loads	
MEP	5
Ballasted Single Ply Roof	11
Finishes/Partitions	20
3 1/4" Lightweight Concrete on 3" Metal Deck	60

Table 1: Design Gravity Loads

Lateral Loads

Wind Loads

Wind loads on the building were calculated in accordance to ASCE 7-05, Chapter 6. Analytical method number two was used to determine wind loads in both the north-south direction and in the east-west direction. For the purpose of this report, I made a few assumptions to simplify the calculations. The assumptions made were to include the roof screen height into the total building height, and to assume the building horizontal projections to be rectangular. Wind effects on the building parapets and roof screen were not taken into consideration. A more detailed and accurate analysis will be performed in future technical reports.

All the variables and coefficients used in the calculations are located in the Appendix B. Table 2 summarizes the wind pressures in the North-South direction. As mentioned earlier the wind loads in this direction are higher than those in East-West direction because this is the broader side of the building. Table 3 summarizes the wind pressures in the East-West direction. It should be noted that internal pressures caused by openings in the building façade were not incorporated in the total wind pressure. An internal pressure of ±3PSF will be added when determining the worst possible loading for the design of the exterior walls.

North-South Direction							
Story	Height (ft)	Wind Pressures: Windward Walls	Wind Pressures: Leeward Walls	Total (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (ft-kips)
1	15	6.89	-6.64	13.52	24.7	357.1	371
2	28.33	8.26	-6.64	14.90	49.0	332.3	1387
3	41.67	9.22	-6.64	15.86	50.0	283.3	2084
4	55	9.98	-6.64	16.62	52.8	233.3	2905
5	68.21	10.62	-6.64	17.25	54.8	180.5	3740
Roof	84.5	11.29	-6.64	17.92	125.7	125.7	10619
				Total	357.1	1512	21107
				Internal Pressure =		±3PSF	

Table 2 wind pressures (North-South Direction)

East-West Direction							
Story	Height (ft)	Wind Pressures: Windward Walls	Wind Pressures: Leeward Walls	Total (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (ft-kips)
1	15	6.89	-3.72	10.60	19.4	292.5	291
2	28.33	8.26	-3.72	11.98	38.9	273.1	1102
3	41.67	9.22	-3.72	12.94	40.5	234.3	1688
4	55	9.98	-3.72	13.70	43.3	193.7	2383
5	68.21	10.62	-3.72	14.33	45.4	150.4	3096
Roof	84.5	11.29	-3.72	15.00	105.0	105.0	8875
				Total	292.5	1249	17435
				Internal Pressure =		± 3PSF	

Table 3 wind pressures (East-West Direction)

Wind Pressure Diagrams

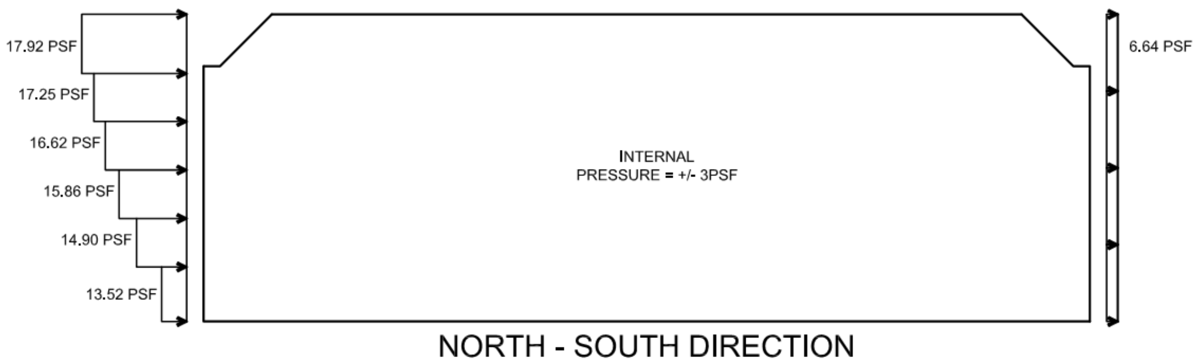


Figure 10 Pressure Distribution along E-W Face

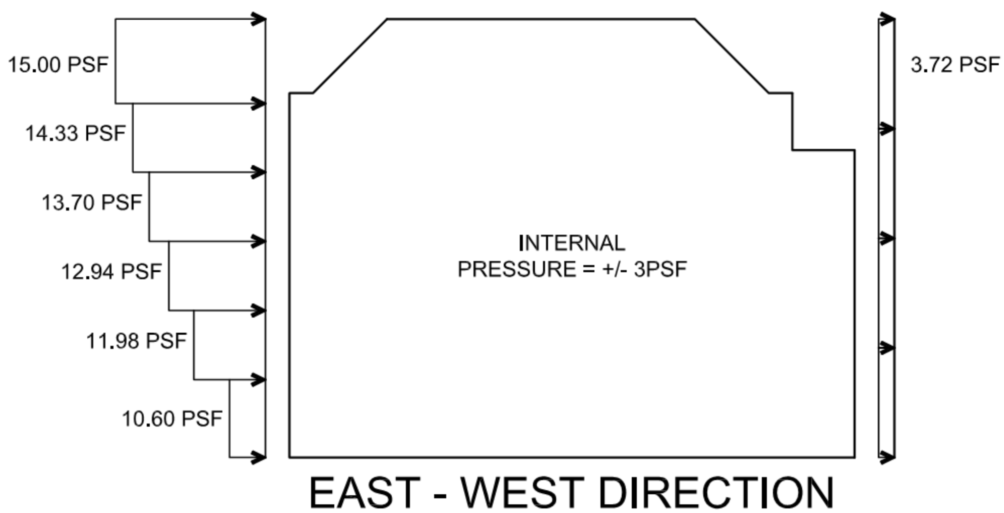


Figure 11 Pressure Distribution along N-S Face

Wind Story Load Diagrams

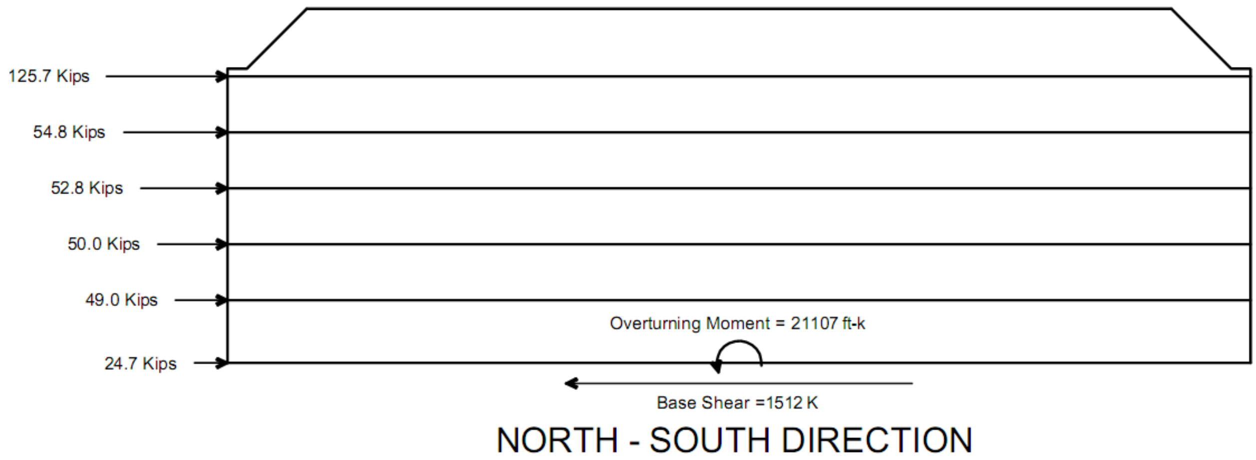


Figure 12: Load Distrubution along E-W Face

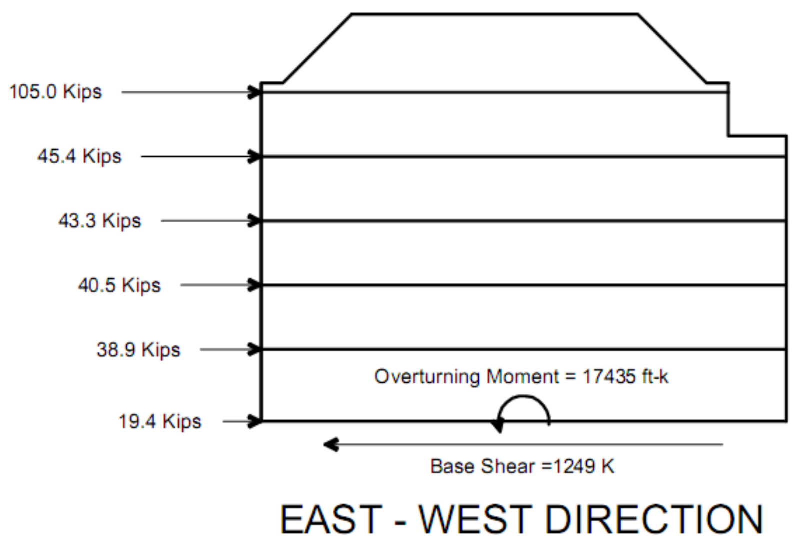


Figure 13: Load Distrubtion along N-S Face

The highest wind pressure calculated is 17.92 PSF. Because the roof screen was assumed to be another story, this value will be conservative. Overall the values obtained were reasonable for a five story office building. In the structural notes, the structural engineers listed a design value of 18.3 PSF. If the engineers used the highest value as a blanket load on the entire façade, it is within 10% of my calculated value.

Seismic Loads

Seismic loads were analyzed using chapters 11 and 12 of ASCE 7-05. Values for the short period response accelerations and the one second period response accelerations differed from those of the structural engineers. Both values I found were from the USGS computer program and the USGS seismic maps. A reason for the variance in the numbers could be caused by the fact that the structural engineers followed IBC 2000 when designing the building. Also, there could be a local provision in the Virginia area, which requires the response accelerations to be a certain value. Although research was done to find such a provision, none was discovered.

Because of the building height, soil class, and response accelerations the building fell into seismic design category A. Buildings in this category need only to be designed in accordance to section 11.7 of ASCE 7-05. Table shows the equivalent force on each floor. The base shear created by the seismic load is much less than the base shear created by the wind loads. Therefore wind will be the controlling lateral load.

Seismic Loads				
Floor	**h_x (ft)**	**Weight (kips)**	**$F_x = 0.01\ w_x$ (kips)**	**M_x (ft-kips)**
2	15	2279	23	342
3	28.33	2256	23	639
4	41.67	2256	23	940
5	55	2042	20	1123
Roof	68.21	682	7	465
		Total	95	3509

Table 4: Equivalent Seismic Load

Appendix A

Snow Load Calculations: Flat Roof Snow Load, Snow Drifting

NICK ZIEGLER | SNOW LOAD CALCULATIONS | (1)

FLAT ROOF SNOW LOAD

ROOF SLOPE = 1/4 in PER FOOT

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

THERMAL FACTOR: $C_t = 1.0$ (TABLE 7-3)

IMPORTANCE FACTOR: I

BUILDING CATEGORY = II → $I = 1.0$ (TABLE 7-4)

EXPOSURE FACTOR: C_e

TERRAIN CATEGORY = B
PARTIALLY EXPOSED

$C_e = 1.0$ (TABLE 7-2)

GROUND SNOW LOAD: p_g

$p_g = 30$ psf (FIGURE 7-1)

$p_f = 0.7(1.0)(1.0)(30)(1.0) = 21$ psf

DRIFT SNOW LOADS FOR CERTAIN JOISTS

2LKSP1 – DRIFT ACCUMULATES AGAINST PARAPET

$\gamma = 0.13(30) + 14 = 17.9$ lb/ft³

WINDWARD DRIFT

h_c = HEIGHT OF PARAPET
= 4'-4 3/8"

$h_d = 0.75\left[0.43(l_u)^{1/3}(p_g+10)^{1/4} - 1.5\right]$

$= 0.75\left[0.43(107)^{1/3}(30+10)^{1/4} - 1.5\right]$

$= 2.73$ ft $< h_c \rightarrow w = 4(2.73) = 10.92$ ft

$p_d = 17.9(2.73) = 48.867 \approx 49$ lb/ft

Appendix B

Wind Load Variables, Coefficients, Calculations

Variables	
Basic Wind Speed	V = 90MPH
Wind Directionality Factor	K_d = 0.85
Importance Factor	I = 1.0
Exposure Category	B
Guss Factor	G=0.85
External Pressure Coefficient Windward Wall	C_p=0.8
External Pressure Coefficient (N-S Leeward Wall)	C_p= -0.5
External Pressure Coefficient (E-W Leeward Wall)	C_p= -0.28
Internal Pressure Coefficient	GC_p= ±0.18

Table 5: Wind Variables

Velocity Pressure Coefficients: $K_{z,}$ K_h			
Story	**Height**	**K_z**	**q_z**
1	15	0.57	10.13
2	28.33	0.69	12.15
3	41.67	0.77	13.56
4	55	0.83	14.68
5	68.21	0.89	15.61
Roof Screen	84.5	0.94	16.60

Table 6: Pressure Coefficients

NICK ZIEGLER | WIND LOAD CALCULATIONS | (1)

METHOD 2

BASIC WIND SPEED: V = 90 MPH
WIND DIRECTIONALITY FACTOR: $K_d = 0.95$
IMPORTANCE FACTOR: I = 1.0
EXPOSURE CATEGORY: B
TOPOGRAPHIC FACTOR: $K_{zt} = 1.0$
VELOCITY PRESSURE COEFFICIENTS: K_h

- CASE 2
- EXPOSURE B
- h = 84'-6"

- INTERPOLATE VALUE FROM TABLE 6-3

$$K_h = \frac{(84.5-80)(0.96-0.93)}{(90-80)} + 0.93$$
$$= 0.9435$$

VELOCITY PRESSURE COEFFICIENTS: K_z

$K_z = 0.59 \quad z < 15 ft$
$K_z = 2.01\ (z/z_g)^{2/\alpha}$
$z_g = 1200$
$\alpha = 7.0$

- CASE 2
- EXPOSURE B

HEIGHT	K_z	q_z
0'-15'	0.59	10.05
20'	0.62	10.93
25'	0.66	11.63
30'	0.70	12.34
40'	0.76	13.40
50'	0.81	14.28
60'	0.85	14.98
70'	0.89	15.69
80'	0.93	16.39
84.5'	0.9435	16.63

* DIFFERENT HEIGHTS IN EXCEL FILE/TABLE CALCULATED BY THE SAME METHOD

VELOCITY PRESSURES @ h & z

$$q_h = 0.00256 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 \cdot I$$
$$= 0.00256(0.9435)(1.0)(0.85)(90)^2(1.0) = 16.63$$

- SAME FORMULA USED FOR q_z: SEE TABLE ABOVE FOR VALUES

GUST FACTOR:

$$n_1 = \frac{100}{H} = \frac{100}{84.5} = 1.18 > 1.0 \quad \therefore G = 0.85$$

NICK ZIEGLER WIND LOAD CALCULATIONS (2)

PRESSURE COEFFICIENTS: C_p

- NORTH - SOUTH DIRECTION

$B = 244'$ $\quad L/B = \frac{109}{244} = 0.447$

$L = 109'$

WINDWARD WALL: $C_p = 0.8$

LEEWARD WALL: $C_p = -0.5$

- EAST - WEST DIRECTION

$B = 109'$ $\quad L/B = \frac{244}{109} = 2.24$

$L = 244$

WINDWARD WALL: $C_p = 0.8$

LEEWARD WALL: $C_p = -0.29$ - INTERPOLATED

INTERNAL PRESSURE COEFFICIENTS: GC_{pi}

$GC_{pi} = \pm 0.18$

DETERMINE DESIGN WIND PRESSURES

- WINDWARD WALLS: $p_z = q_z G C_p - [q_h (GC_{pi})]$ — NEGLECT FOR WINDWARD & LEEWARD PRESSURE
- LEEWARD WALLS: $p_h = q_h G C_p - [q_h (GC_{pi})]$

* SEE TABLE FOR VALUES

Appendix C

Seismic Load Calculations

SEISMIC CALCULATIONS

SPECTRAL RESPONSE ACCELERATIONS:

$S_s = 0.158$ (USGS MAPS)
$S_1 = 0.051$

SITE CLASS OF SOIL: C

$S_{MS} = F_a \cdot S_s$ (EQ. 11.4-1)
$S_{M1} = F_v \cdot S_1$ (EQ. 11.4-2)

$F_a = 1.2$ (TABLE 11.4.1)
$F_v = 1.7$ (TABLE 11.4.2)

$S_{MS} = 1.2(0.158) = 0.1896$
$S_{M1} = 1.7(0.051) = 0.0867$

$S_{DS} = 2 \frac{S_{MS}}{3}$ (EQ. 11.4-3)

$S_{D1} = 2 \frac{S_{M1}}{3}$ (EQ. 11.4-4)

$S_{DS} = 2 \frac{0.1996}{3} = 0.1331$

$S_{D1} = 2 \frac{0.0867}{3} = 0.0578$

OCCUPANCY CATEGORY: II

$T_a = C_t \cdot h_n^x$ Eq. 12.8-7

$= (0.02)(82.6)^{0.75}$ * CONCENTRICALLY BRACED FRAME
$= 0.548$ $h_n = 82.6'$

$T_s = \frac{S_{D1}}{S_{DS}} = \frac{0.0578}{0.1331} = 0.434$

$0.8T_s = 0.8(0.434) = 0.3472 < T_a$

$T_a > 0.8 T_s$

$S_s \leq 0.15$ & $S_1 \leq 0.04$

SITE DESIGN CATEGORY A

Building Weight

To find the weight of the steel structure a typical bay was analyzed and the average weight per square foot was determined. A load of 15 PSF was assumed for the exterior façade, large MEP equipment was added in, and the weight of the roof screen was included. All applicable dead loads were also applied.

Average Weight Per Floor	
Material	**PSF**
Steel Framing	8
3 1/4" LWC on 3" comp. metal deck	60
MEP	5
Finishes	5
Partitions	15
Total	93

Table 7: Average Weight Per Floor

Building Weight Per Floor			
Load (PSF)	**Floor**	**Floor Area (SF)**	**Weight (kips)**
93	2	25707	2391
93	3	25435	2365
93	4	25435	2365
93	5	22850	2125
22	Roof	22850	503
		Total	9749

Table 8: Total Weight Per Floor

Weight of Mechanical Equipment	
Equipment Name	**Equipment Weight (lbs)**
Cooling Tower	13710.0
EF-1	500.0
OAF-1	1300.0
EF-2	500.0
OAF-2	1300.0
EF-3	500.0
HP-1	396.0
HP-2	396.0
Total (kips)	18.6

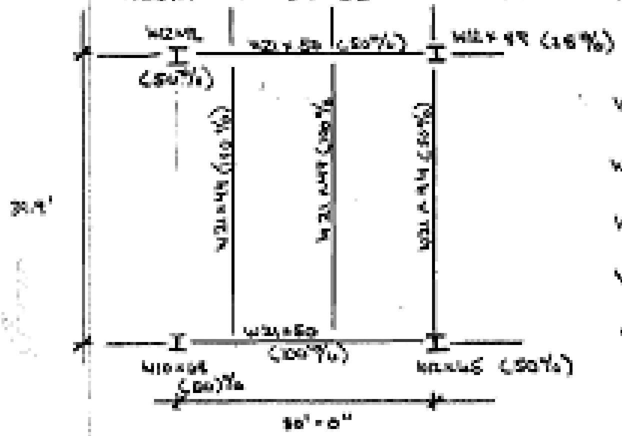
Table 9: Weight of Large Mechanical Eq.

Total Building Weight (kips)	
Total Floor Weight	9749
Exterior Façade	726
Roof Screen	15
Large MEP Eq.	18.6
Total	10509

Table 10: Total Weight

BUILDING SELF-WEIGHT 9)

WEIGHT OF STEEL IN TYPICAL BAY



TOTAL WEIGHT

W21x44: (39.9)(44)(2) + (39.9)(44)(0.50)
= 4389 lb

W21x50: (30)(50)(0.50) + (30)(50)
= 2250 lb

W10x68: (14)(68)(0.50)
= 476 lb

W12x65: (14)(65)(0.50)
= 455 lb

W12x87: (14)(87)(0.25)
= 305 lb

W12x96: (14)(96)(0.50)
= 672 lb

TOTAL WEIGHT: 8547 lb

TOTAL SF: 30(39.9) = 1197 ft²

AVERAGE lb/ft² = $\frac{8547}{1197}$ = 7.5 lb/ft²
≈ 8 lb/ft²

FLOOR AREA

2: 26,702 SF
3: 26,435 SF
4: 26,435 SF
5: 22,850 SF

DEAD LOADS

9 PSF - STEEL FRAMING
60 PSF - 3in COMP METAL DECK 3¼in LWC
5 PSF - MEP
10 PSF - PARTITIONS
83 PSF

WEIGHT PER FLOOR

2: 2134 K
3: 2111 K
4: 2111 K
5: 1897 K
8253 K

EXTERIOR FACADE

* ASSUME 15 PSF FOR AVERAGE WEIGHT

GROSS WALL AREA = 48,379 SF

TOTAL WEIGHT = $\frac{48{,}379\,(15)}{1000}$ = 726K

ROOF

* ASSUME ROOF STEEL FRAMING IS 3/4 FLOOR STEEL FRAMING

6 PSF - STEEL
5 PSF - MEP
11 PSF - ROOFING
22 PSF

WEIGHT = $\frac{22(22850)}{1000}$ = 503K

BUILDING SELF-WEIGHT (2)

ROOF SCREEN:

18 K3 - 6.6 lb/ft
16K2 - 6.3 lb/ft
14K1 - 5.2 lb/ft
10K1 - 5.0 lb/ft

W14 × 22 @ 30', 7.5' LONG

$7.5(22) = \frac{165 \text{ lb}}{30} = 5.5$ lb/ft

TOTAL - $\frac{29.6 \text{ lb/ft } (519 \text{ ft})}{1000} = 15.0^k$

LARGE MEP EQ'

18.6^k

BUILDING WT

19.6
15.0
8235
726
503

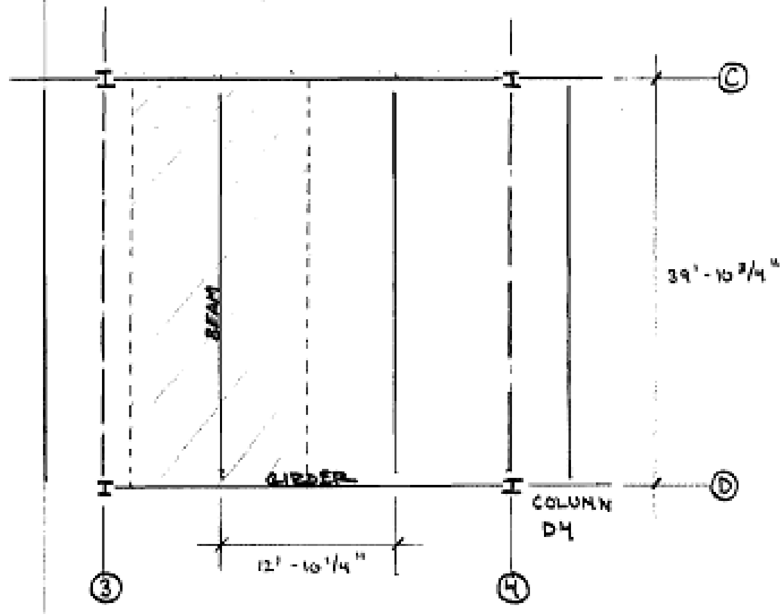
$9515^{k*} \approx 9600^k$

*I increased partitions load to 20PSF, and therefore the total building weight is listed as 1059 in Table 10.

Appendix D

Spot Checks: Beam and Column

BEAM SPOT CHECK (1)



LOADS

LIVE - 100 PSF
DEAD - 5 PSF - MEP
20 PSF - FINISHES/PARTITIONS
60 PSF - 3 1/4" LIGHTWEIGHT CONC. ON METAL DECK
85 PSF

FACTORED LOADS

$1.2D + 1.6L$
$1.2(85) + 1.6(100) = 262$ PSF

TRIB WIDTH = 12' 10 1/4"

$$w_u = \frac{262\,(12.85)}{1000} = 3.37\ ^{k}/_{ft}$$

- DECKING IS PERPENDICULAR TO BEAM

$$b_{eff} = \left| \begin{array}{l} \text{SPACING} = (12.85)(12) = 154\text{ in} \\ \text{min } \dfrac{\text{SPAN}}{4} = \dfrac{39.9(12)}{4} = 120\text{ in} \leftarrow \text{CONTROLS} \end{array} \right.$$

$$M_u = \frac{w_u \cdot l^2}{8} = \frac{3.37\,(39.9)^2}{8} = 671^{\text{'k}}$$

BEAM CHECK

ASSUME $a = 1$ in

$Y_2 = (6.25 - 1/2) = 5.75$ in $\rightarrow 5.5$ in

TRY W21 x 44

$Y_2 = 5.5$ in PNA @ TFL $\phi M_n = 771^{'k} > 671^{'k}$

CHECK ASSUMPTION $a = 1.0$

$$a = \frac{649}{0.85(3.5)(120)} = 1.91 \text{ in} \quad \therefore \text{NO GOOD}$$

PNA @ 4 => $\Sigma Q_n = 430^k$ $\phi M_n = 677^{'k} > 671^{'k}$

$$a = \frac{430}{0.85(3.5)(120)} = 1.20 \text{ in. NO GOOD}$$

DOES NOT WORK. PROBABLY BECAUSE THE STRUCTURAL ENGINEER USED LIVE LOAD REDUCTION

• TRY LIVE LOAD REDUCTION

LIVE LOAD REDUCTION

$$L = L_o\left(0.25 + \frac{15}{\sqrt{A_I}}\right)$$

$$= L_o\left(0.25 + \frac{15}{\sqrt{1025}}\right) = 0.719 \quad \text{LIMIT TO } 0.50 \cdot L_o$$

$L = 0.50(100) = 50$ psf

FACTORED LOAD

$1.2(95) + 1.6(50) = 192$ psf

$$W_u = \frac{192\,(12.85)}{1000} = 2.34\ ^k/ft$$

$$M_u = \frac{2.34\,(39.9)^2}{8} = 466^{'k}$$

BEAM CHECK

ASSUME $a = 1$ in

$Y_2 = 6.5$ in   DNA @ BFL $\phi M_n = 646^{1k} > 466^{1k}$

CHECK ASSUMPTION

$$a = \frac{357}{0.85(3.5)(120)} = 1 \text{ in}$$

IN STRUCTURAL NOTES   SHEAR STUD CAPACITY = 19.8 ^{k}/STUD

$$\# \text{ STUDS} = \frac{357}{19.8} \times 2 = 36 \text{ STUDS}$$

· CHOOSE LOWER PNA

$$7 \cdot \quad a = \frac{162}{0.85(3.5)(120)} = 0.454 \text{ in}$$

$$\phi M_n = 522^{1k} > 466^{1k}$$

$$\# \text{ STUDS} = \frac{162}{19.8} \times 2 = 16 \text{ STUDS}$$

CONCLUSION: BEAM IS ADEQUATE IF LIVE LOAD REDUCTIONS ARE CONSIDERED SIMILAR TO ENGINEERS DESIGN

Column Spot Check										
Floor	Tributary Area (ft^2)	Dead Load (psf)	Live Load (psf)	Influence Area (ft^2)	Reduction Factor	Live Load (Kips)	Dead Load (kips)	Load Combination	Load at Floor (Kips)	Accumulated Load (kips)
Roof	599	33	20	2394	0.56	6.7	19.8	1.2D + 0.5L_r	27.1	27
5	599	85	100	4788	0.47	28.0	50.9	1.2D + 1.6L	94.6	122
4	599	85	100	7182	0.43	25.6	50.9	1.2D + 1.6L	91.8	213
3	599	85	100	9576	0.40	24.2	50.9	1.2D + 1.6L	90.1	304
2	599	85	100	11970	0.40	24.0	50.9	1.2D + 1.6L	89.9	393

COLUMN SPOT CHECK

@ ROOF : $P_u = 22^k$

W10x33 $\quad h = 13'4" = 13.33'$

$A_g = 9.71 in^2$

$I_x = 171 in^4 \quad I_y = 36.6 in^4$

$r_x = 4.19 in \quad r_y = 1.94 in$

$\frac{KL}{r_x} = \frac{13.33(12)}{4.19} = 38.2 \quad \frac{KL}{r_y} = \frac{13.33(12)}{1.94} = 82.5$ ← CONTROLS

$\frac{KL}{r} \leq 4.71\sqrt{E/F_y} = 4.71\sqrt{29000/50} = 113$

82.5 < 113 INELASTIC BEHAVIOR

$F_{cr} = \left[0.658^{F_y/F_e}\right] \cdot F_y = \left[0.658^{50/41.5}\right] \cdot 50 = 30.2 \text{ ksi}$

$F_e = \frac{\pi^2 \cdot E}{(KL/r)^2} = \frac{\pi^2 (29000)}{83^2} = 41.5 \text{ ksi}$

$\phi P_n = \phi \cdot F_{cr} \cdot A_g = 0.9(30.2)(9.71) = 263^k > P_u$

CHECK W/ TABLE 4-22

$\frac{KL}{r} = 83 \quad \phi F_{cr} = 29.2$

$\phi F_{cr} = 30.2(0.9) = 27.2$ ✓

CHECK W/ TABLE 4-1

KL = 13.33' W10x33

$\phi P_n = 273^k$ OK

LARGEST LOAD = 122^k OK.

@ LEVEL 4 COLUMN CHANGES TO W10x68

$\phi P_n = 670^k > 393^k$ OK

THE REASON THE COLUMNS ARE OVERSIZED COULD BE DUE TO A MORE CRITICAL LOADING CONDITION SUCH AS WIND.