HAMAKER COURT OFFICE BUILDING

DEVELOPMENT PHASE A
3023 HAMAKER COURT

Technical Report One

Rendering courtesy of PSA Dewberry INC.

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Executive Summary

The purpose of this report is to analyze and assess the existing structural support system in the new construction at 3023 Hamaker Court in Fairfax, VA. Building construction methods, as well as the architecture and design, are also gathered to illustrate the complete building method. The structure of 3023 Hamaker Court is a 6-story, braced-frame steel construction, utilizing composite deck for the building's floors and spread footings for the foundation. The roof of the building is steel roof deck supported by K-series steel joists and houses a mechanical penthouse.

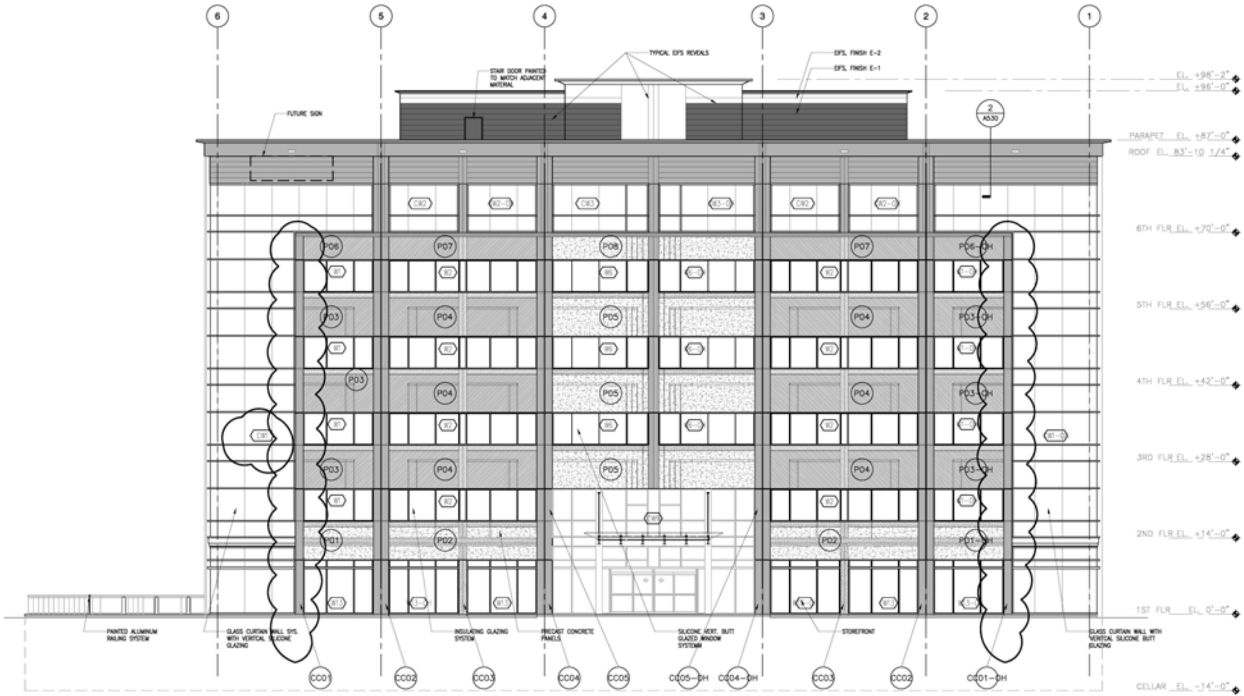
The building project is construction of a 130,000 square foot office space in conjunction with a 170,000 square foot parking structure to accommodate 860 vehicles within a complex already partially developed by the building owner. The ground floor of the building is home to rental space that will be available to stores and other shops, while the stories above ground will be exclusive to the medical office space of Hamaker MOB, LLC. The building offers views of the business district of Fairfax, VA through a precast and curtain wall glazing system suspended from the structure of each floor. The building floor plan outside of its core is open with long spans and exterior columns to offer the most freedom within the space. The building offers accentuated corner glazing with emphasis on open space and feel, and a classic tie-in to the 'corner-office' appeal from the inside.

Analysis of the existing system, including gravity loading, lateral systems, and construction were reviewed and studied in conjunction with material properties and strengths of materials to create an understanding of the building system. The resulting building analysis is provided as reviewed using the provided list of building codes. Wind and seismic investigation were completed using ASCE 7-05 as opposed to 7-10 due to the most current International Building Code has not yet adopted the standard. The MWFRS is a centralized braced frame steel building core and not specifically designed for seismic loading. The analysis of the building within category III, however found that the seismic design parameters control with a base shear of nearly 345 kips.

Spot checks were completed on a typical composite beam supporting a floor deck and an interior column that is not part of the braced frame to verify member sizes, orientation, and design by the structural engineer. The spanning members were checked on the third (a typical) floor and the column was completed just below this floor due to splice location being just above.

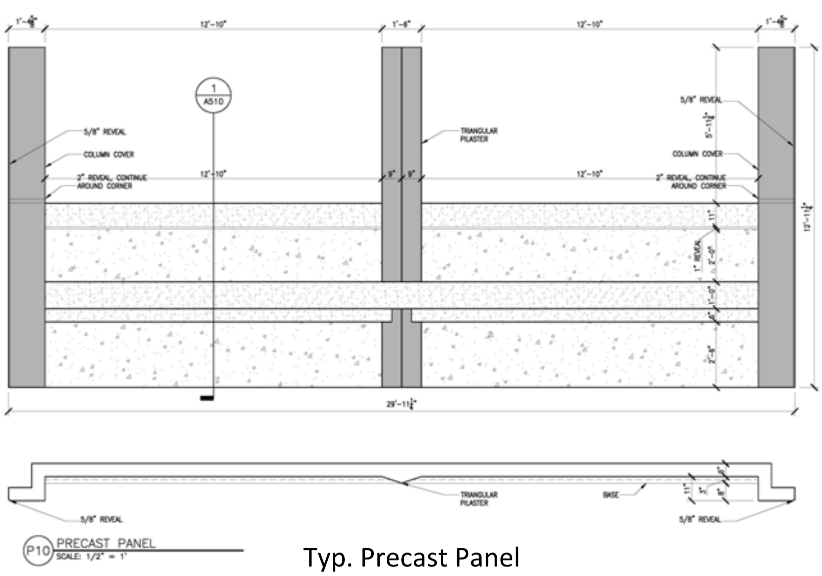
Appendices attached to the report detail load case generations, preparation and determination of snow, seismic, and wind loads, and the spot checks for reference. Framing details are located throughout the report to illustrate the building design and structural system.

3023 Hamaker Court:



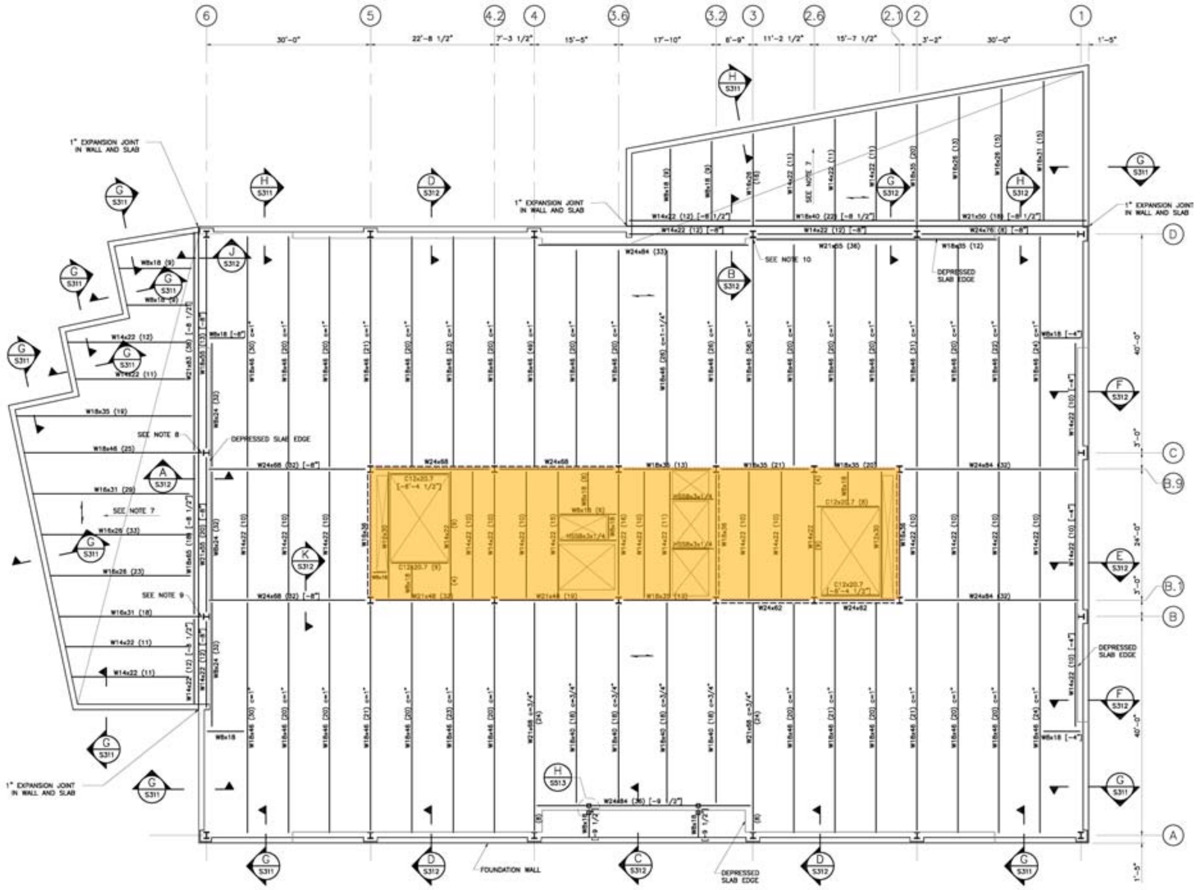
South Elevation

The building elevation shown above is of the building's long face showcasing the glazing storefront and architectural precast panels. Details for one typical piece of the many panel types are shown here.



Typ. Precast Panel

These precast panels, in conjunction with glazing that wraps the corners and in between the pieces, constitute the exterior of the building and allow the construction process to be accelerated due the ease of construction associated with 'hanging' the exterior of the building from the structure. The panels vary in size, shape, depth, and color (between a beige and deep clay color), but are all constructed of precast concrete.



First Floor Framing

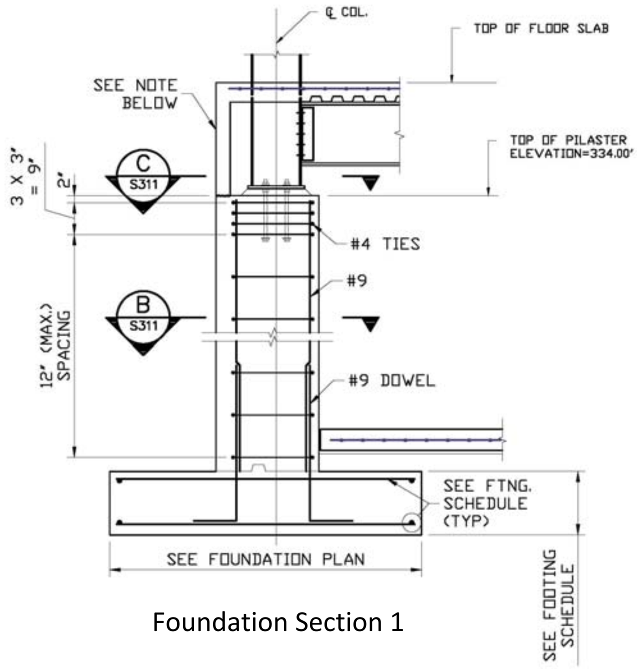
The floor structure shown here is the footprint of the building for the basement and foundation walls and illustrates framing members on the first floor. The building has a core construction that houses the chases for mechanical equipment, the corridors for egress, elevators, stairways, and the building's main lateral resisting members. The building core is highlighted above in orange.

Structural System

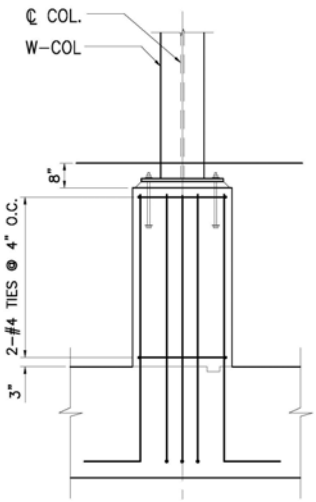
Foundation

The foundation of the building consists of a slab on grade basement level built atop spread footings ranging in footprint from 8’-0” – 13’-0” square for individual piers, to long dimensions over 36’-0” for select combined footings for the braced bays at column lines 2.1, 3.2, and 5.

The column schedule for the basement level varies due the difference in loading and the construction of the exterior walls. The basement walls are typically built integral to the building’s support structure and feature the pilasters for the building framing extending up to the first floor with haunches for the first floor framing above. This detail is pictured right and a very general typical interior spread footing detail is shown below for reference.



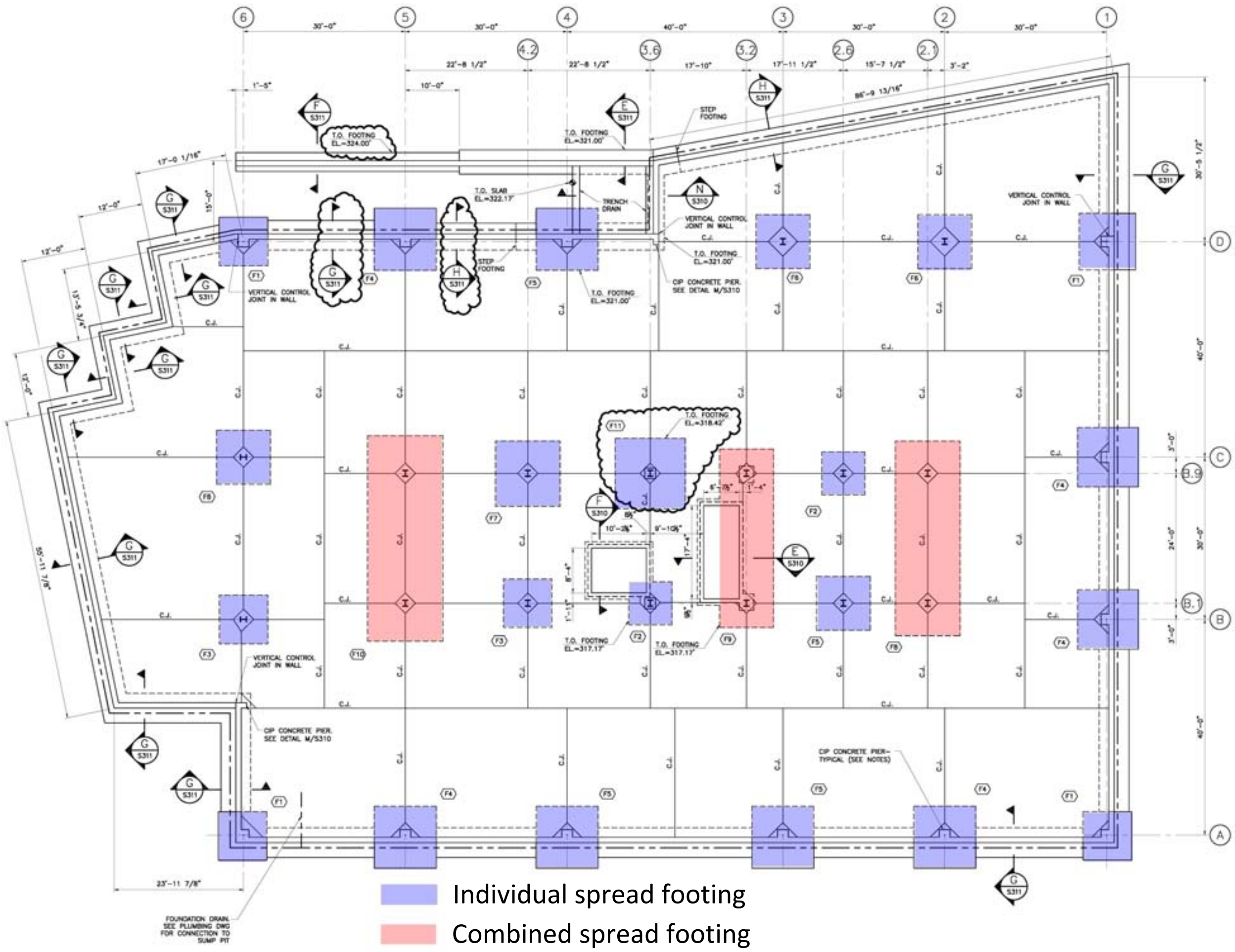
Foundation Section 1



Foundation Section 2

The basement footprint is larger and extends outward slightly beyond those of the upper levels for receiving and infrastructure purposes. The slab is a 6” slab on grade continuous across the whole building with the exception of sumps located in the center of the building for water retention and access to the elevator cores. The soil base below these footings and slab is generally graded at site class C with bearing capacities of 10,000 psf for footings on weathering rock, and 3,000 psf for footings on structural fill or natural soils.

The exterior walls are also poured on continuous spread footings and the schematic for the groundwork is shown in the figure on the following page.



Building Columns

Hamaker Court is a steel framed building with W12 and W14 shapes composing the vertical support members. The steel sections begin at the basement level with the exception of the building's southern face integral to the building foundation in which the columns begin at the first floor level. The columns are continuous throughout the building's six floors as wide flange columns and frame into hollow structural sections for the penthouse at the building's core.

The building core consists of moment frames and the base plates connecting them to the foundations are in excess of 2" thick and two feet (2') square; thus denoting moment connections. These base plates exist on the combined footings discussed above and distribute the moment from the frames into the foundation. A column schedule for the building can be found on the next page without the base plate schedule for clarity.

COLUMN LINE	6-A 6-D 1-A 1-D	6-B 2-D	6-C	5-A 5-D 2-A	4-A 4-D 3-A	3-D	1-B 1-C	2.6-B.9	3.6-B.1	4.2-B.1
COLUMN CAP PLATE T										
PENTHOUSE ROOF EL. VARIES										
ROOF EL. VARIES								HSS12x8x1/4	HSS12x8x1/4	
6TH. FL. EL. 84'-0"	W12X50	W14X61	W12X53	W12X53	W14X61	W14X61	W12X53	W12X40	W12X53	W14X61
5TH. FL. EL. 70'-0"										
4TH. FL. EL. 56'-0"	W12X65	W14X82	W12X72	W12X72	W14X90	W14x90	W12X79	W12X58	W12X65	W14X82
3RD FL. EL. 42'-0"										
2ND FL. EL. 28'-0"	W12X87			W12X106	W14X120		W12X120			
1ST FL. EL. 14'-0"		W14X132	W12X152			W14X145		W12X87	W12X120	W14X120
F. FL. EL. 0'-0"	CONCRETE			CONCRETE	CONCRETE		CONCRETE			
B/COL. SEE FTG. PLAN										
BASE PL. TYPE	A	A	A	A	A	A	A	A	A	A
BASE PL. NxBxT	15x15x1⅝"	20x18x2"	20x18x2"	20x20x2¾"	22x22x2¾"	20x20x2¾"	20x20x2¾"	16x15x2"	16x15x1⅝"	19x17x2"
ANCHOR BOLT DIA.	3/4"	3/4"	3/4"	3/4"	3/4"	3/4"	3/4"	3/4"	3/4"	3/4"
ANCHOR BOLT LENGTH L	1'-4"	1'-4"	1'-4"	1'-4"	1'-4"	1'-4"	1'-4"	1'-4"	1'-4"	1'-4"

COLUMN LINE	5-B.1	5-B.9	4.2-B.9	3.6-B.9	3.2-B.1	3.2-B.9	2.6-B.1	2.1-B.1	2.1-B.9
COLUMN CAP PLATE T									
PENTHOUSE ROOF EL. VARIES									
ROOF EL. VARIES				HSS12x8x1/4	HSS12x8x1/4	HSS12x8x1/4	HSS12x8x1/4	HSS12x8x1/4	HSS12x8x1/4
6TH. FL. EL. 84'-0"	W14X61	W14X53	W14X82	W14X43	W14X43	W14X43	W14X43	W14X43	W14X43
5TH. FL. EL. 70'-0"	W14X82	W14X82	W14X82	W14X68	W14X61	W14X61	W14X53	W14X61	W14X61
4TH. FL. EL. 56'-0"									
3RD FL. EL. 42'-0"	W14X176	W14X159	W14X109	W14X109	W14X90	W14X99	W14X82	W14X109	W14X120
2ND FL. EL. 28'-0"									
1ST FL. EL. 14'-0"	W14X233	W14X193	W14X176	W14X159	W14X211	W14X233	W14X132	W14X176	W14X193
F. FL. EL. 0'-0"									
B/COL. SEE FTG. PLAN									
BASE PL. TYPE	C	C	B	B	C	C	B	C	C
BASE PL. NxBxT	32x27x3¾"	32x27x3½"	27x27x2¾"	27x27x3½"	32x27x2¾"	32x27x3"	27x27x2¾"	32x27x3"	32x27x3½"
ANCHOR BOLT DIA.	2"	2"	2"	2"	2"	2"	2"	2"	2"
ANCHOR BOLT LENGTH L	2'-8"	2'-8"	1'-4"	3'-0"	3'-0"	3'-0"	1'-4"	1'-10"	1'-10"

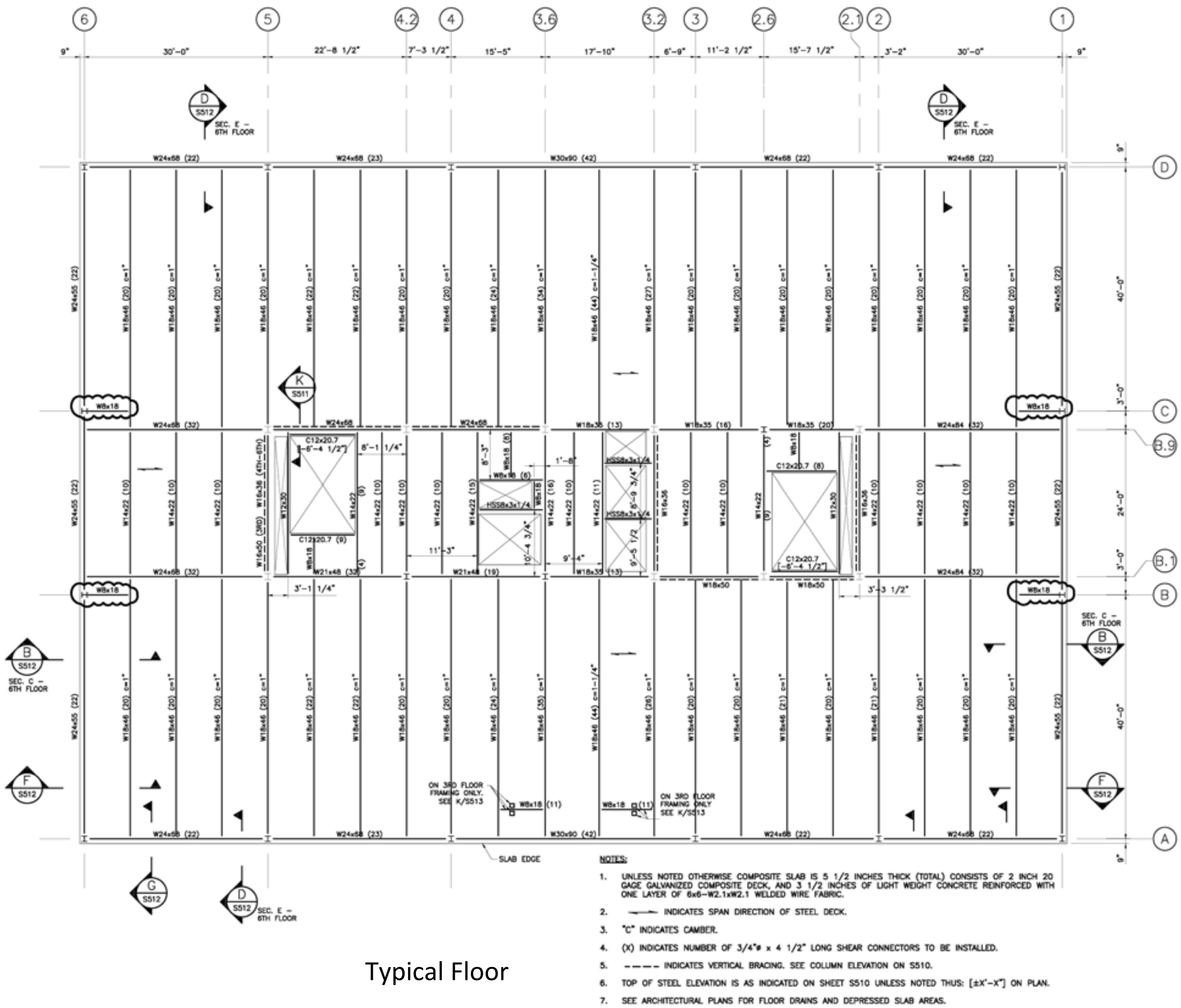
Column Schedule

Floor System

The floors of the Hamaker Court Building are rectangular with the underlying exception of the first floor just above the building's basement. The floor plan consists of a regular stacked construction with building critical elements delegated to the core and center of the each floor and the flexible, open office space in a rectangular shape around it. The building core is offset to a slightly different grid than the exterior walls and causes a typical bay to be enclosed by three (3) columns and the extension of a floor girder in any given bay. The second floor of the building allows for two (2) story entry atriums within the building north and south with rounded balconies. From the third story up, the framing becomes regular and the floors contain no openings besides the stairs and elevators in the building core.

The floor system consists of 3 ½" of lightweight concrete on a 2" 20 gauge composite deck and a single layer of 6x6-W2.1xW2.1 welded wire fabric. The deck spans a maximum distance of 9'-0" across W18x46 wide flange beams. The composite system is engaged to the floor framing by ¾" diameter, 4 ½" long shear studs specified independently for each member. The beams frame into the interior and exterior of the building by means of girders spanning from W18x35 to W30x90 and 15'-6" to 40'-0". There are 12 atypical bays spanning either 24'-0" or 40'-0" that surround the building core which is 100'-0" by 24'-0" with the same deck on varying members.

The typical floor framing is shown on the following page.

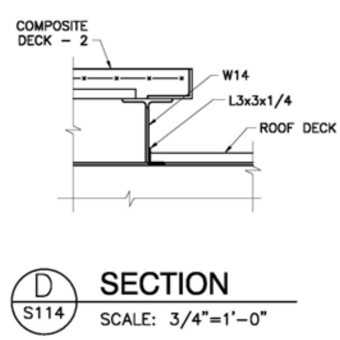


Typical Floor

Roof System

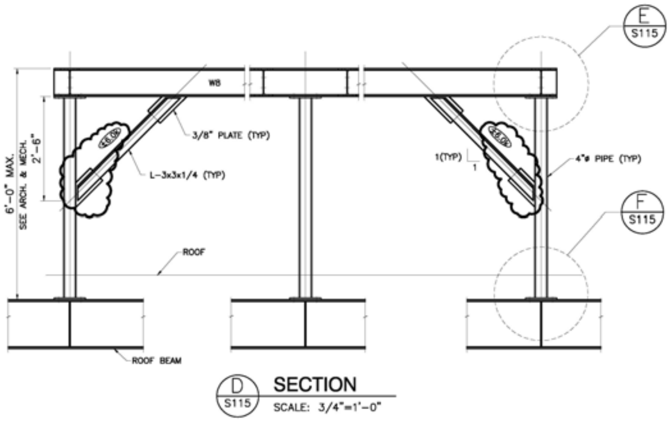
Typical to most office spaces and buildings of the same type built in the northeast area of the US, the building has a lighter construction roof to support only roof loads due to how rarely it will be occupied. The roof is constructed primarily of KCS series joists and wide flange members spanning in the same direction as the support members of the lower floors on both sides of the 160’-0” long dimension of the building.

The framing outside the building core supports the same 5 ½” 20 gauge steel composite deck with LW concrete as well as a heavier 18 gauge deck and 1 ½” deep roof deck in different areas. The deck is not, however, engaged to the structure as on lower floors and is simply composite itself, acting with the concrete placed.



Roof deck Section

The core of the building (30’-0” wide by 100’-0” long) is extended another level to enclose mechanical equipment. The framing for this level varies from all others due to having variations in finishes, elevations, and the addition of HSS shapes in the framing. The braced frames also stop at the roof level and lateral bracing is achieved by additional angles in the corners of the frames to resist lateral loads. A section from the building is shown here.

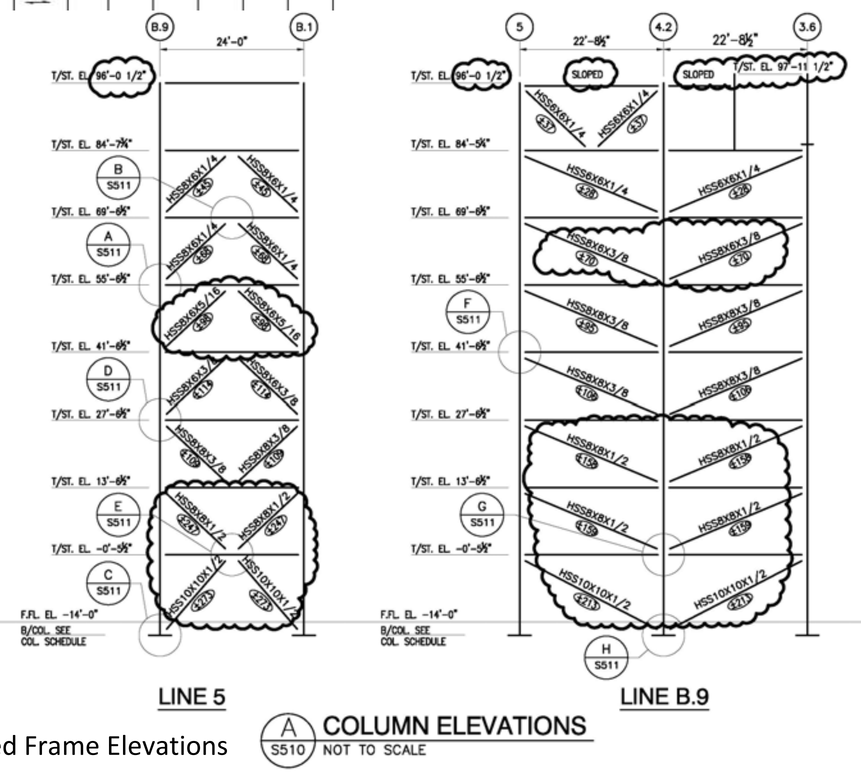


Penthouse Bay Framing

Lateral System

The primary lateral load resisting system for the building is HSS shape braced frames within selected bays of the building. The primary frames are all wide shapes for the building columns and beams but are reinforced with HSS 6x6 and 8x8 shapes in five bays denoted on the plan below. The braced frames shown are of two designs incorporating either a single or two adjacent bays. The bays resist the lateral loads in both the north-south and east-west directions based on their orientation shown on the plan.

Braced Bay Configuration



1 and 2 bay Braced Frame Elevations

Design Codes and References

By Original Building Designers

- 2003 International Building Code – International Code Council
- 2003 Virginia Uniform Statewide Building Code
- AISC Minimum Design Loads for Buildings and Other Structures ASCE 7-02
- American Institute of Steel Construction 9th Edition
- Building Code Requirements for Structural Concrete ACI 318-02

Thesis Codes Referenced

- 2009 International Building Code – International Code Council
- ASCE 7-05 – American Society of Civil Engineers
- ACI 318-08 – American Concrete Institute

Gravity Loads

The following charts are a snapshot representation of the loading diagrams and code referenced values used for analysis of the structure. See appendices for further detail of loadings

Superimposed Dead Loads	Designer	Analysis
Floor Areas		10
Mechanical Equipment		15
Precast Walls		10
Lightweight Concrete	110	110

Live Loads	ASCE 7-02	ASCE 7-05
Lobbies and 1st Floor Corridors	100	100
Office Space	50	50
Corridors above 1st floor	80	80
Storage, Mechanical	150	125
Flat Roof	30	20

Snow Loads	ASCE 7-02	ASCE 7-05
Ground Snow Load Pg	30	30
Exposure Factor	1	1
Importance Factor	1	1
Thermal Factor	1	1
Flat Roof Snow Load p_f	21.0	21.0
Drift Surcharge p_d		86.1

Lateral Loads

Wind

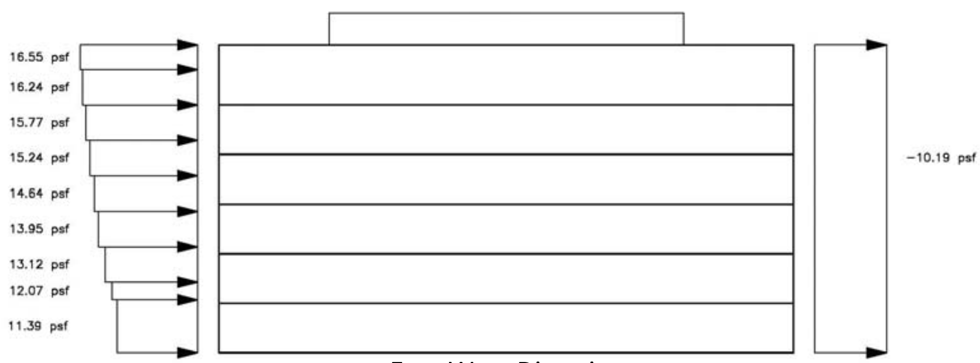
Following IBC 2009, the method for wind analysis is described in ASCE 7-05 chapter 6. The lateral wind loads were analyzed in the north-south and east-west direction by means of the MWFRS Directional Procedure. Using a design wind speed of 90 MPH for the area of Fairfax, VA, and assuming the building exhibits a regular rectangular shape, the building was found to have the following values. The details of this part of the building analysis can be found in the appendices for further information regarding outcome from the calculations.

Building N-S

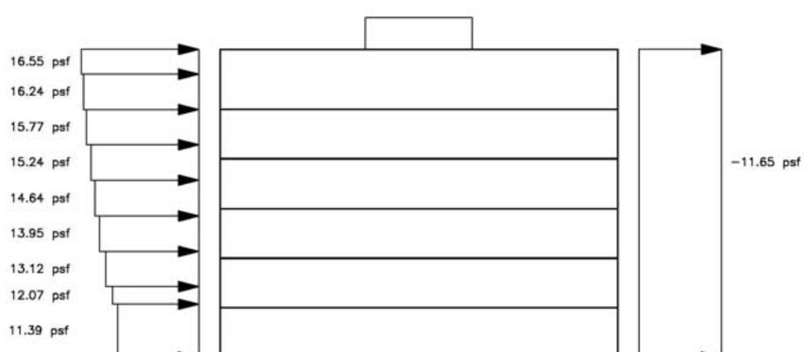
Bldg. Story*	Height (z) (ft)	Hght. Fact. (kz)	Vel. Press. (qz) (psf)	Des. Press. (p) Windward (psf)	Leeward (psf)	Per ft eq. Windward (plf)	Leeward (plf)	Eq. Shear Windward (kip)	Leeward (kip)	Story Shear (kip)	Eq. Moment (kft)
6	87	0.95	19.25	23.68	-18.77	3835.66	-3040.46	32.60	-25.84	58.45	5084.89
5	70	0.89	18.09	22.89	-18.77	3707.95	-3040.46	25.96	-21.28	47.24	3306.72
4	56	0.84	16.97	22.13	-18.77	3584.86	-3040.46	25.09	-21.28	46.38	2597.13
3	42	0.77	15.63	21.22	-18.77	3437.32	-3040.46	24.06	-21.28	45.34	1904.47
2	28	0.69	13.92	20.06	-18.77	3248.94	-3040.46	22.74	-21.28	44.03	1232.72
1	14	0.56	11.42	18.35	-18.77	2973.37	-3040.46	20.81	-21.28	42.10	589.36
* Penthouse height =96 ft. Values not shown for clarity										283.53	14715.29

Building E-W

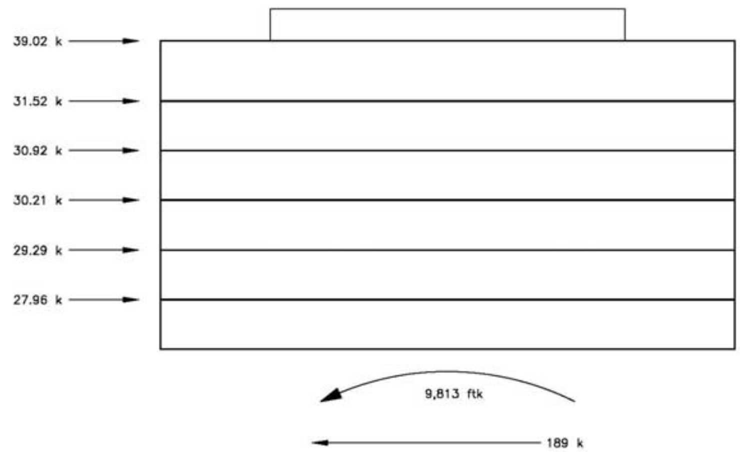
Bldg. Story*	Height (z) (ft)	Hght. Fact. (kz)	Vel. Press. (qz) (psf)	Des. Press. (p) Windward (psf)	Leeward (psf)	Per ft eq. Windward (plf)	Leeward (plf)	Eq. Shear Windward (kip)	Leeward (kip)	Story Shear (kip)	Eq. Moment (kft)
6	87	0.95	19.25	23.68	-17.31	2651.82	-1938.72	22.54	-16.48	39.02	3394.70
5	70	0.89	18.09	22.89	-17.31	2563.52	-1938.72	17.94	-13.57	31.52	2206.10
4	56	0.84	16.97	22.13	-17.31	2478.42	-1938.72	17.35	-13.57	30.92	1731.52
3	42	0.77	15.63	21.22	-17.31	2376.42	-1938.72	16.63	-13.57	30.21	1268.65
2	28	0.69	13.92	20.06	-17.31	2246.18	-1938.72	15.72	-13.57	29.29	820.24
1	14	0.56	11.42	18.35	-17.31	2055.66	-1938.72	14.39	-13.57	27.96	391.45
* Penthouse height =96 ft. Values not shown for clarity										188.92	9812.66



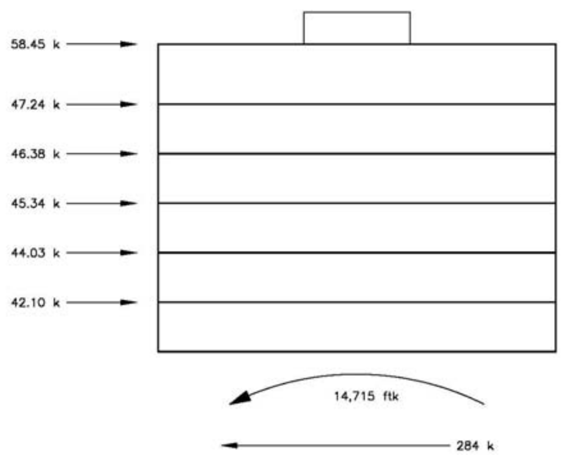
East-West Direction



North-South Direction



East-West Direction



North-South Direction

The structure was found to have a controlling base shear of 283 k in the building's east-west direction. The designers did not specifically design the building for seismic so this was the value determined to control for design. Further information regarding wind design is provided in the seismic section.

Seismic

Seismic analysis was completed using the relevant sections of chapter 11 and 12 of ASCE 7-05. The building was found to have a controlling base shear of 344 k and this varies from the designed value from the drawings of 308 k for a few reasons. First, the weight of the building was over-estimated to remain conservative, and the building was deemed a class III importance factor which raised all the design values due to the occupant load. The original design was for an importance factor of 1.00 and this was completed for 1.25. Detailed calculations showing more information relating to seismic design can be found in the appendices.

Deck and Superimposed Dead Load					Curtain Wall*			
Floor	Area (ft^2)	Deck Weight (psf)	Mech, finishes, etc. (psf)	Area Loads (lbs)	Length (ft)	Const. Weight (psf)	Lin. Loads (plf)	Per Floor (k)
Roof	18144	10.00	30	730000	548	56	31000	761
6th	18144	50.42	30	1460000	548	56	31000	1491
5th	18144	50.42	30	1460000	548	56	31000	1491
4th	18144	50.42	30	1460000	548	56	31000	1491
3rd	18144	50.42	30	1460000	548	56	31000	1491
2nd	14704	50.42	30	1180000	548	56	31000	1211
1st	16524	50.42	30	1330000	548	56	31000	1361
			Totals	9080000			217000	9297

*Curtain wall details taken as 5-10 lbs/ft2 for 14' building height or <5 lbs/ft2 for glazing and storefront. 4lbs/ft2 avg used.

Beams Typical Floor (3rd-5th Floors)						Columns Lower Levels (1st and 2nd Floors)					
Beam Type	Number #	Length (ft)	Unit Weight (plf)	Linear Weight (lbs)	Total/type (lbs)	Column Type	Number #	Story Height (ft)	Unit Weight (plf)	Linear Weight (lbs)	Total/type (lbs)
W8x18	11	8	18	144	1584	W12x87	4	14	87	1218	4872
W12x30	2	24	30	720	1440	W12x106	3	14	106	1484	4452
W14x22	18	24	22	528	9504	W12x120	3	14	120	1680	5040
W16x36	3	24	36	864	2592	W12x152	1	14	152	2128	2128
W18x35	4	18	35	630	2520	W14x120	4	14	120	1680	6720
W18x46	40	40	46	1840	73600	W14x132	3	14	132	1848	5544
W18x50	2	17	50	850	1700	W14x145	1	14	145	2030	2030
W21x48	2	21	48	1008	2016	W14x159	1	14	159	2226	2226
W24x55	4	40	55	2200	8800	W14x176	2	14	176	2464	4928
W24x56	2	30	55	1650	3300	W14x193	2	14	193	2702	5404
W24x68	10	30	68	2040	20400	W14x211	1	14	211	2954	2954
W24x69	2	23	68	1564	3128	W14x233	2	14	233	3262	6524
W24x84	2	33	84	2772	5544						
W30x90	2	40	90	3600	7200						
C12x20.7	4	7	21	145	580						
HSS8x3x1/4	3	8	25	200	600						
			Total Weight per floor (k)		144.5				Total Weight per floor (k)		52.8
			Total Estimated Building (k)		1011.6				Total Estimated Building (k)		369.8

Between the two lateral forces, wind was deemed controlling for two reasons. The student wishes to complete analysis in as similar a way as possible and the building was not specifically designed for seismic, and the seismic loads were artificially high based on post-checking values.

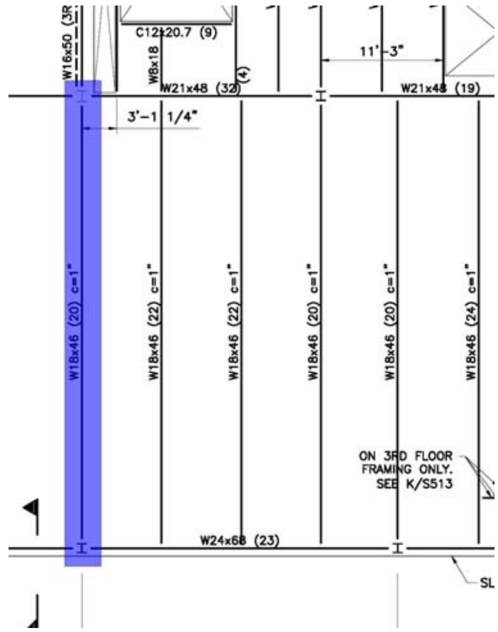
Story (level)	w_x (k)	h_x (ft)	$w_x h_x^k$* (ftk)	C_{vx}	f_i* (k)	V_i (k)	M_i (ftk)
Roof	845.21	87	85974	0.18	60.78	60.78	5288
6th	1688.33	70	137130	0.28	97.04	157.82	6793
5th	1688.33	56	108851	0.22	77.03	234.85	4314
4th	1688.33	42	80820	0.17	57.19	292.05	2402
3rd	1688.33	28	53121	0.11	37.59	329.64	1053
2nd	1408.33	14	21625	0.04	15.30	344.94	214
1st	1445.21	0	0	0.00	0.00	344.94	0
	10500	297	488000	1	345		20063

*k = 1.035 *v = 345 k

Spot Checks

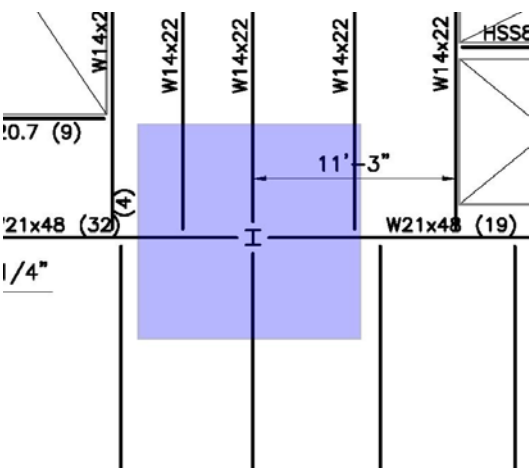
Composite Beam with Floor Slab

For the beam spot check a member was chosen from the exterior bay of the third (typical) floor and analyzed with the prescribed loading values. A W18x46's Shear and moment values were calculated and found the construction prescribed for the floor system was adequate to carry the loads. The beams were analyzed in conjunction with the slab due the composite action imposed by the shear studs on top of the supporting members. The beam was found heavier than necessary which could be the result of higher loading in another section of the building and the resulting members all being sized alike for construction simplicity. Details of the calculation can be found in the appendices.



Typical Beam

Interior Column



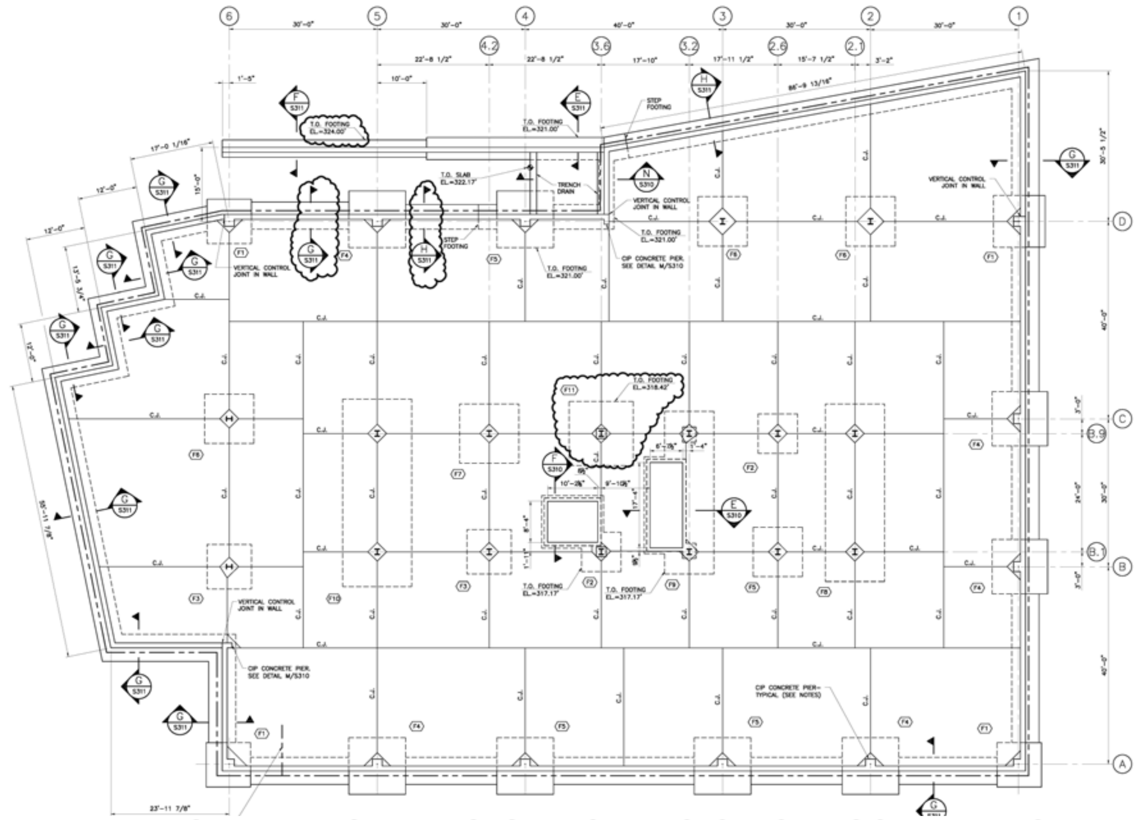
The column at grid lines 4.2-B.1 was analyzed just below the first floor to check its adequacy to carry the set loads from the above floors. The column size for this grid line at the given height is a W14x120. The building plans do not include this member in the braced frames, and the column was assumed to not be part of the lateral system, carrying only gravity loads. The member was scrutinized under pure axial, pure moment, and combined loading, and the axial case made to control due to the nature of it being a gravity column. The column loads were again lower than the designers may have seen in another similar bay or arrangement but is well within reasonable tolerances. Detailed work showed in the appendices.

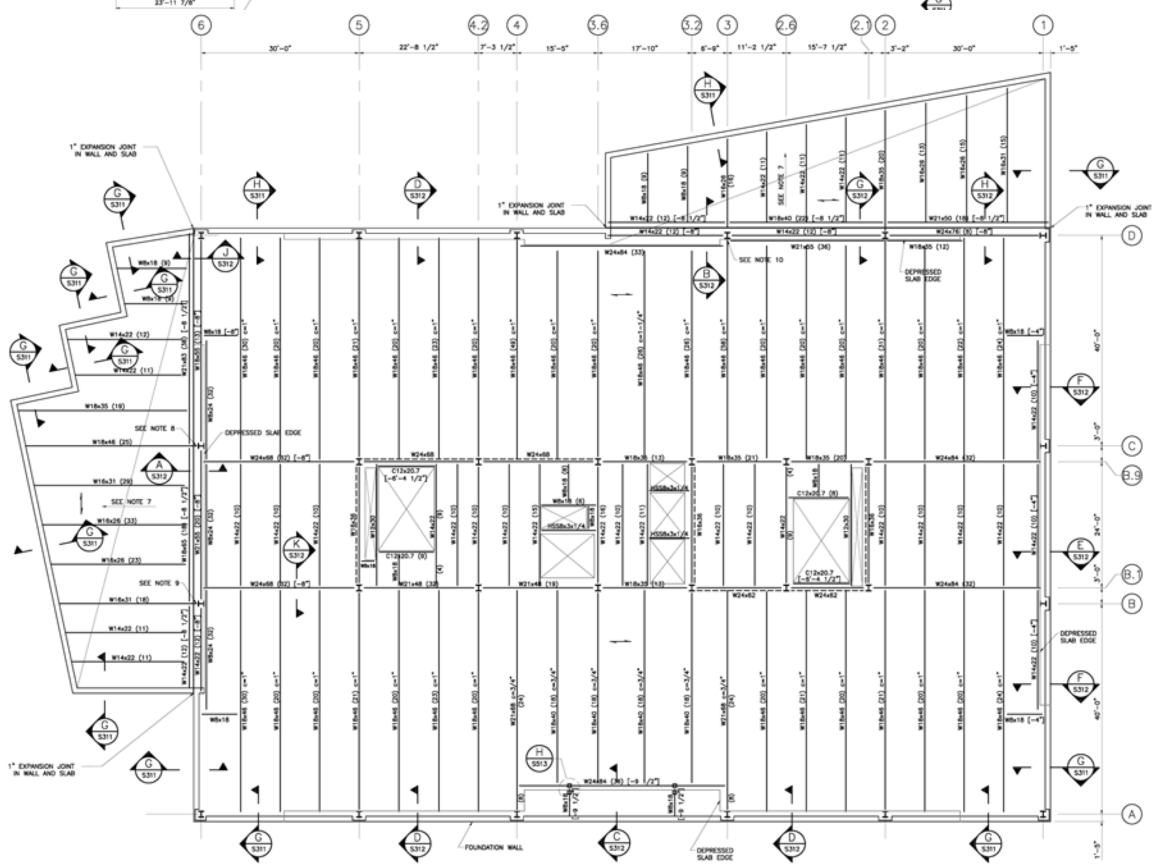
Conclusion

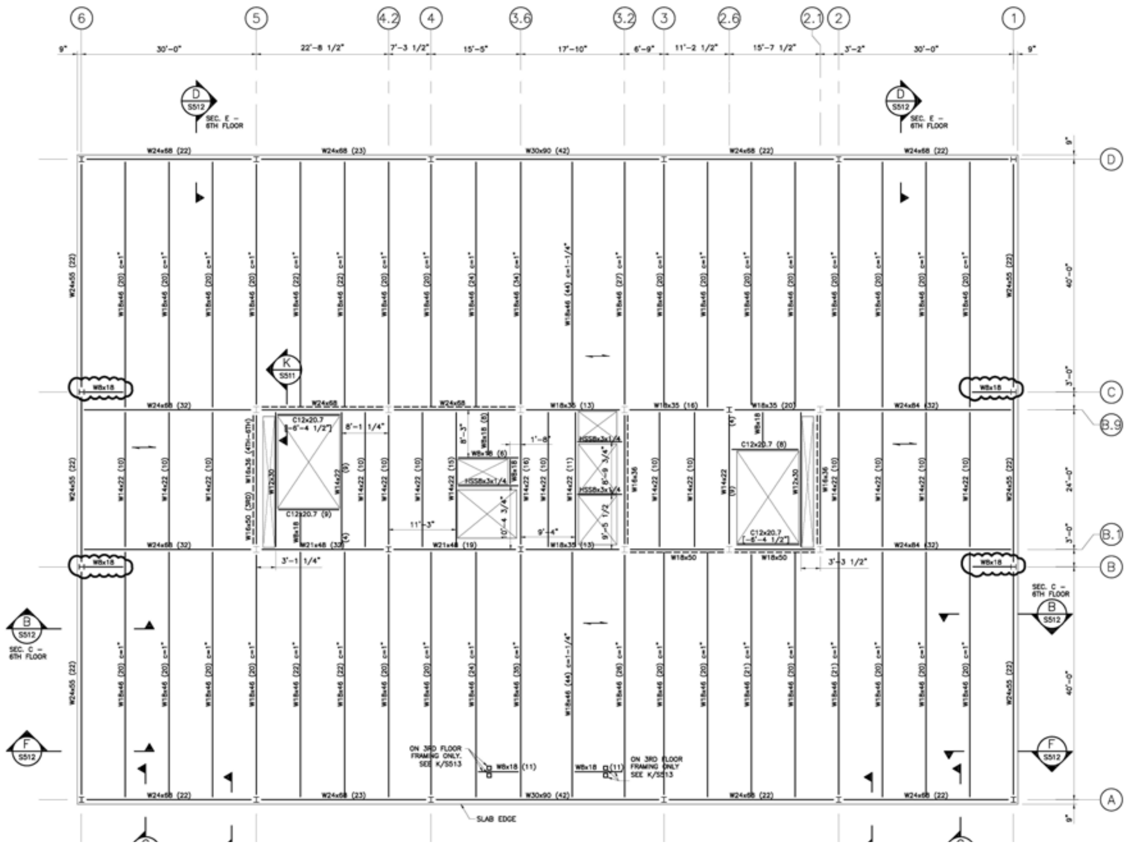
The technical report is designed to create an in depth understanding of the existing building by means of analysis and calculations. This report shows in full detail the descriptions and understanding of the building's structure, aesthetics, and load resisting members.

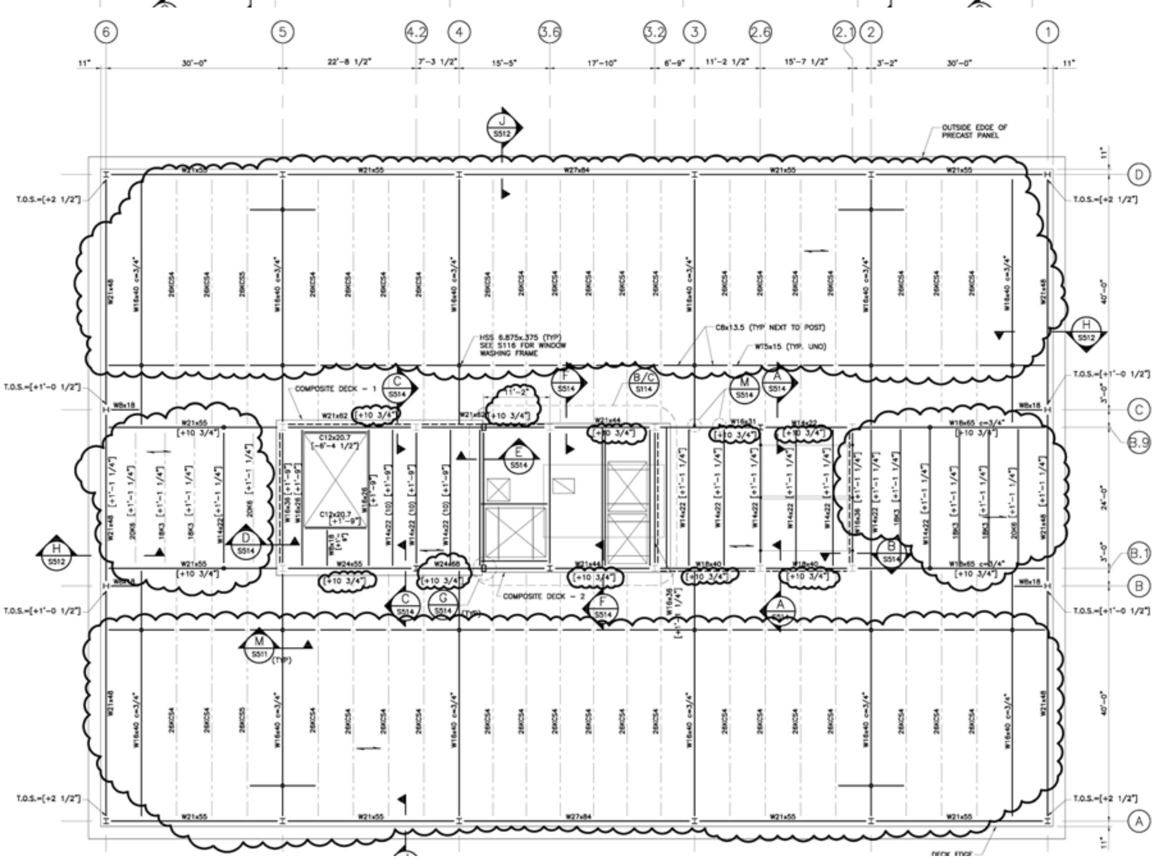
The spot checks used to verify the engineer's design were correct and validated the design parameters. The calculations explicitly show the extent of the design is sufficient in resisting both the vertical and lateral loads that may be expressed the building's frame. The floor slab of the building was not specifically checked by itself and will see further investigation in the following report to ensure proper function without issue.

APPENDIX A
Floor Plans









APPENDIX B

Snow, Wind, and Seismic Calcs

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SNOW LOADS

$P_g = 30$ psf   FIG 7.1

IMPORTANCE FACTOR : III   >300 PPL

$I = 1.1$

EXPOSURE FACTOR - B PARTIALLY EXPOSED

$C_E = 1.0$ (TABLE 7-2)

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

FLAT ROOF : $p_f = .7(C_e)C_t(I)p_g$

$p_f = .7(1.0)(1.0)(1.1)30 = 23.1$

$p_f \approx 23$ psf

DRIFT

SNOW DENSITY $\gamma_s = .13(p_g) + 14 <= 30$

$\gamma_s = .13(30) + 14 = 17.9$

$\gamma_s = 18$ PCF

BALANCED HEIGHT $h_b = p_f/\gamma_s = 23/18$

$h_b = 1.28$ ft

PARAPET OBSTRUCTION $l_u = 100$ ft

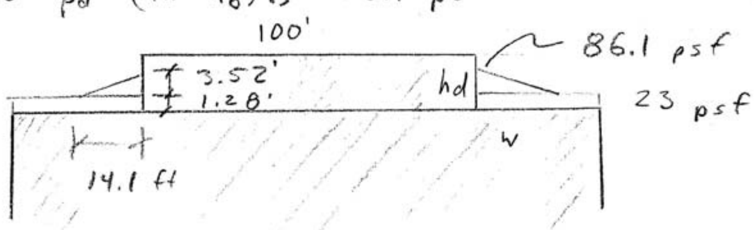
$h_o = 9$ ft

DRIFT HEIGHT $h_d = .43\sqrt[3]{l_u}\left(\sqrt[4]{p_g + 10}\right) - 1.5$

$h_d = .43\left(\sqrt[3]{100}\right)\sqrt[4]{30+10} - 1.5 = 3.52$ ft

WIDTH $w = 4(h_d) = 4(3.52) = 14.1$ ft

DRIFT LOAD $p_d = (h_d + h_b)\gamma_s = 86.1$ psf



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WIND CALCULATIONS

- ASCE 7-05

BASIC WIND SPEED $V = 90$ MPH (6.1)

EXPOSURE - GROUND ROUGHNESS - CATEGORY B (6.2)

BLDG. CATEGORY III

IMPORTANCE FACTOR $V<100 = 1.15$ (6.1)

BOUNDARY CONDITION $Z_g = 1200$ ft

GUST FACTOR $\alpha = 7$

MEAN ROOF HEIGHT $h = 87$ ft

DIRECTIONALITY FACTOR $K_d = 0.85$ (MWFRS) (6.4)

EXPOSURE COEFFICIENT $K_z = 2.01\left(\frac{z}{z_g}\right)^{2/\alpha}$ $z_{min} = 15$

$$K_z = 2.01\left(\frac{87}{1200}\right)^{2/7} = .9497$$

$K_z = .95$

TOPOGRAPHIC FACTOR $K_{zt} = 1.0$ (6.5.7)

GUST FACTOR $G = .85$ (6.5.8) (MID-RISE/ASSUMED RIGID)

$q_z = .00256(K_z K_{zt} K_d V^2 I)$ psf

$q_z = .00256(.95)1.0(.85)(90)^2(1.15) = 19.3$ psf

HEIGHT	K_z	q_z (psf)
87	.95	19.3
80	.91	18.5
70	.87	17.7
60	.83	16.9
50	.79	15.9
40	.73	14.8
30	.67	13.5
20	.58	11.7
15	.54	11.2

CONT.

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WIND (CONT.)

$n_a \geq 1\ Hz \rightarrow G = .85$ (6.5.8)

BUILDING SHORT DIM. 112 ft
LONG 162 ft

RATIO. FOR C_p $L_s/B_L = .69$ (TABLE 6.6)
$L_L/B_s = 1.45$ (TABLE 6.6)

PRESSURE COEFFICIENTS - INTERPOLATED FROM TABLE 6.6
C_p = SHORT FACE
WINDWARD = .8
LEEWARD = -.41
LONG
WINDWARD = .8
LEEWARD = -.5

INTERNAL PRESS. COE. C_{pi}

- FULLY ENCLOSED BUILDING
$GCP_i = \pm .18$ - (FIGURE 6.5)

DESIGN PRESSURE @h - WINDWARD

$$p = (q_z * G * C_p) - q_h(GC_{pi})$$

$$p = 19.25(.85).8 - 19.25(-.18)$$

$$p = 16.55\ psf$$

H(z)	LONG NORMAL WIND	LONG NORMAL LEE	SHORT NORMAL WIND	SHORT NORMAL LEE
87	16.55	-11.65	16.55	-10.19
80	16.24	↓	16.24	↓
70	15.77		15.77	
60	15.24		15.24	
50	14.64		14.64	
40	13.95		13.95	
30	13.12		13.12	
20	12.07		12.07	
15	11.39		11.39	

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SEISMIC LOADING

$S_S = 0.156$ USGS — $F_a = 1.2$ (TABLE 11.4.1)

$S_1 = 0.051$ — $F_v = 1.7$ (" 11.4.2)

SITE CLASS C (USGS)

$S_{MS} = F_a(S_S) = 1.2(.156) = .187$

$S_{M1} = 1.7(.051) = .087$

$S_{DS} = 2/3 S_{MS} = 2/3(.187) = .125$

$S_{D1} = 2/3 S_{M1} = 2/3(.087) = .058$

SEISMIC DESIGN CATEGORY

$S_{DS} < .167$ CAT III – A TABLE 11.6.1

$S_{D1} < .067$ CAT III – A 11.6.2

RESPONSE MOD. COEFFICIENT

-BRACED FRAME, CONCENTRIC $R = 3.25$ TABLE 12.2-1

SEISMIC RESPONSE COEFFICIENT

$C_S = \frac{S_{DS}}{R/I_e} = \frac{.125}{3.25/1.00}$ $I_e = 1.$ ( ) (TABLE 1.5-2)

$C_S = .038$

BASE SHEAR $V = C_S W$ $W = 10,808$ K

$V = .038(10,808\ K)$

$V = 411$ K

STORY FORCES $F_z = C_{sz} W_z$

$C_{sz} = \frac{w_z h_z^k}{\sum w_i h_i^k}$

SEISMIC (CONT'D)

FUNDAMENTAL PERIOD

- APPROX $T_a = C_t h_n^x$

$C_t = .02$ – BUILDING NOT SPECIFICALLY DESIGN-

$x = .75$ (TABLE 12.8.2) ED FOR SEISMIC.

$T_a = .02(87)^{.75}$

$T_a = .575$

$C_s = \frac{S_{D1}}{T(R/I_e)} = \frac{.058}{.57(3.25/1)} = .032 < .038$

$V = C_s W = .032(10808) = 345\ k$

$k = \frac{.57-.5}{2.5-.5}(2-1)+1 = 1.035$

$k = 1.035$

SEE SPREADSHEET

Story (level)	w_x (k)	h_x (ft)	$w_x h_x$^k* (ftk)	C_{vx}	f_i* (k)	V_i (k)	M_i (ftk)
Roof	845.21	87	85974	0.18	60.78	60.78	5288
6th	1688.33	70	137130	0.28	97.04	157.82	6793
5th	1688.33	56	108851	0.22	77.03	234.85	4314
4th	1688.33	42	80820	0.17	57.19	292.05	2402
3rd	1688.33	28	53121	0.11	37.59	329.64	1053
2nd	1408.33	14	21625	0.04	15.30	344.94	214
1st	1445.21	0	0	0.00	0.00	344.94	0
	10500	297	488000	1	345		20063

*k = 1.035 *v = 345 k

APPENDIX C
Spot Checks