Voorhees Replacement Hospital

Voorhees, New Jersey

Technical Report #1

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Structural Option

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Executive Summary

In the first technical report of the Voorhees Replacement Hospital the building is introduced through a brief explanation of function and a detailed description of the structural system including foundations, floor system, columns, lateral system, and the roof system. A list of the materials and governing building codes are also provided. Gravity loads are calculated using ASCE 7-05 and spot checks in typical bays are preformed. The member sizes found in these calculations are similar to those specified by the designer. Wind and Seismic loads are also calculated using ASCE 7-05 and are compared to the structural engineer's lateral loads. It was determined that the seismic loads were similar in Building A and found to control over wind loads. It was also determined that the seismic loads in Building B were off by a considerable amount, but still controlled over the wind loads.

Introduction

The Virtua Voorhees Replacement hospital is located in Voorhees, New Jersey (Latitude: 39.84° Longitude: -74.93°), immediately off Rt. 73. It will be replacing the old Voorhees hospital because of its inability to be renovated. The new hospital will have 9 floors, starting with a Garden Level continuing up through Floor 8. The building is broken up into two main areas, the main bed tower (referred to as Building A, or Northern Building in this report), and a services building (referred to as Building B, or Southern Building in this report). The building is also broken up into 7 smaller zones, for ease of reference in the drawings. Figure 1 shows how the building is split up.

The main bed tower is 8 levels and holds zones 1 through 3, holding 350 beds, each in individual rooms. It is a curved building with a curtain wall facing the majority of the site. This curtain wall allows residents to get an excellent view of the site and the wetlands on it that were protected during construction. The majority of the 8 floors in the main tower have the same floor plan with minor differences.

The services building, which holds zones 4 through 7, is attached to the main bed tower via a thin corridor. The services building houses most of the labs, offices, and surgical spaces needed in the hospital. The services are located on the ground floor through the 5th floor. Above the 5th floor, the building narrows, to match the width of the corridor connecting the bed tower and the services building. Mechanical spaces start on the 6th floor and continue up until the 9th floor. The services building also allows for future growth, adding more space on top zone 6.

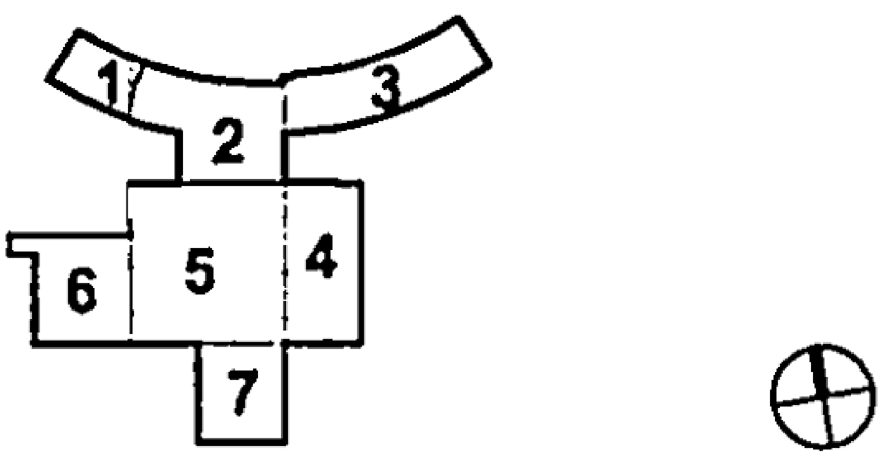


Figure 1: Key plan

Structural System Overview

Foundations

The soil for the Voorhees Replacement Hospital is mainly a sandy soil. To prevent these loose soils from liquefaction, stone piers, or geopiers were required to be put in to densify the soil. These geopiers were required to increase the bearing pressure of the soil to 6,000 psi for the soil below all the building's foundations, canopy foundations, and utility structures. For any soil below the site's retaining walls, the bearing pressure was required to be increased to 3,000 psi. The minimum required equivalent coefficient of friction equals 0.36 for sliding resistance across the entire footing bottom area for the retaining walls, and brace frame foundations.

The foundation system is a series of concrete footings either resting on concrete piers, or resting on grade. The exterior columns are concrete footings with sizes ranging from 4' x 4' x 1' – 6" to 13' x 13' x 3' – 4" with rebar sizes ranging from #6 - #10 both ways. The columns that rest on concrete piers range in size from 2' – 4" x 2' – 4" to 3' x 4' – 6" with rebar sizes ranging from #9 - #11 for the vertical reinforcement, and #4's or #5's for the ties. See Figure 2 and Figure 3 below for typical footing and pier details.

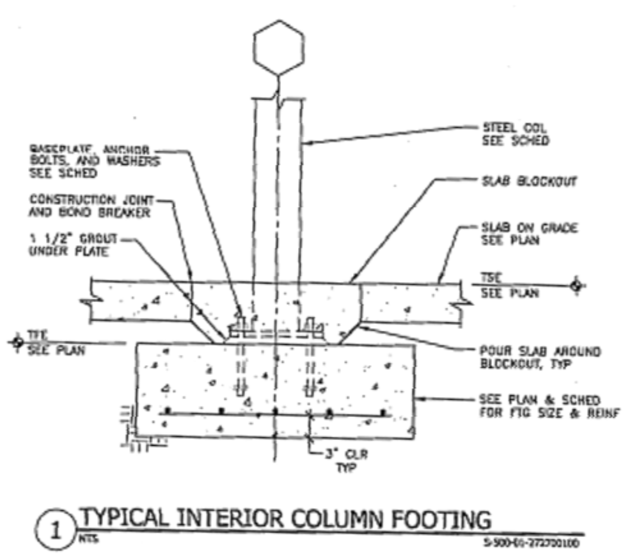


Figure 2: Typical Concrete Foundation Footing

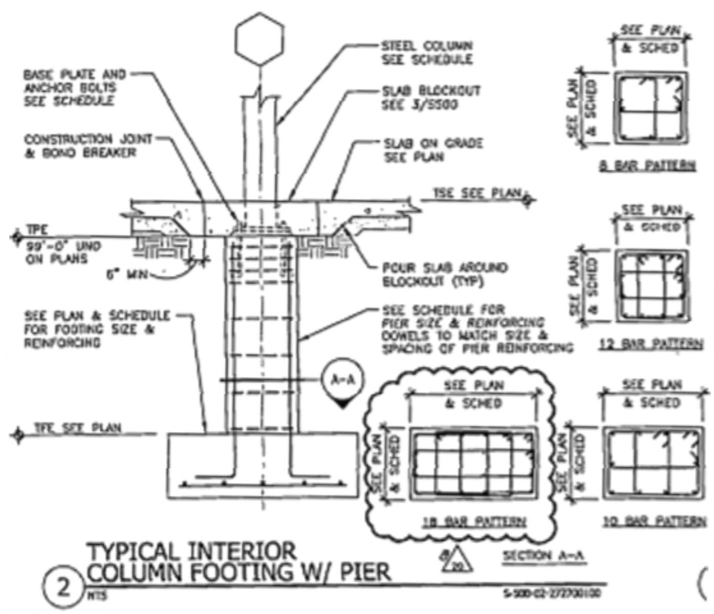


Figure 3: Typical Concrete Foundation Footing With a Concrete Pier

The garden level floor system is constructed, in most places, using a 5” concrete slab on grade, with 6 x 6 – W2.9xW2.9 WWF. In specified spots the size of the concrete slab is increased for specialized equipment, such as refrigerator equipment required for the kitchen and dietary section of the hospital. In zones 4 and 5, a grade beam travels along the perimeter. The grade beam is shown in Figure 4 below.

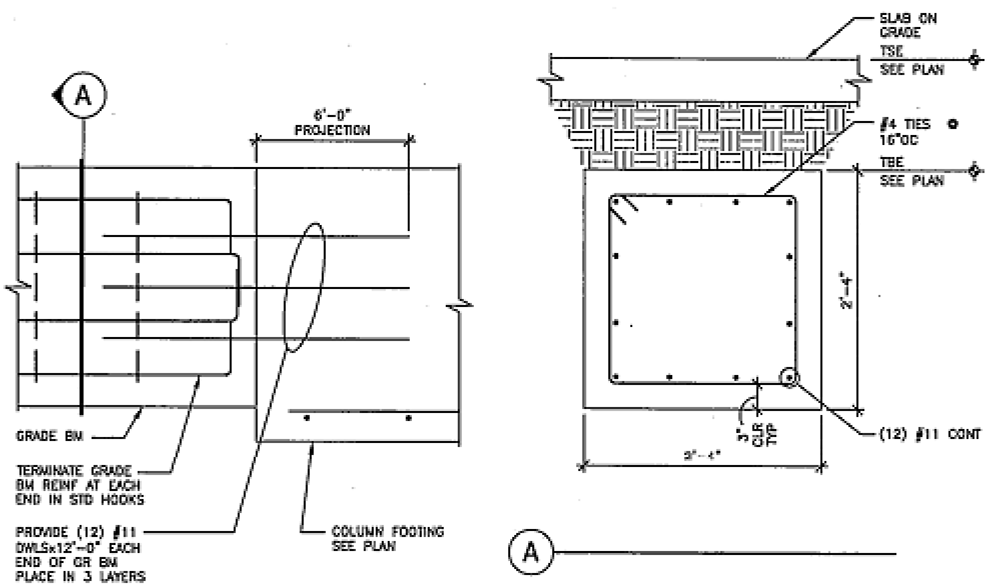


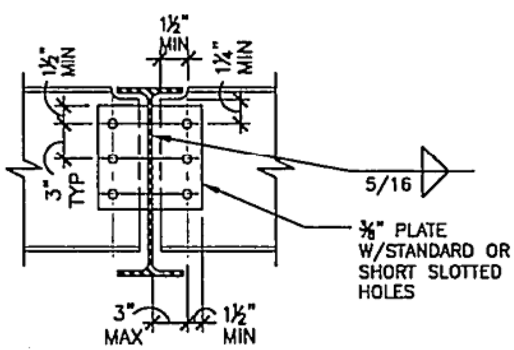
Figure 4: Typical Concrete Grade Beam Detail

Superstructure

Floor System

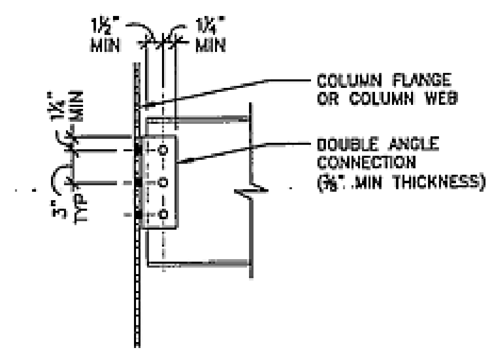
The floor system of the Voorhees hospital is a composite steel/concrete system. In Building A the typical bay sizes are around 30' x 30' or 30' x 10', depending on what area of the building they are located in. In Building B the bay sizes are typically 31' – 4" x 31' – 4" or 31' – 4" x 29' – 4". 3 – ½" light weight concrete sits on top of 3" x 18 Gage composite steel deck. The total thickness of the concrete is 6 – ½" with 6x6-W2.1xW2.1 WWF.

The steel deck is connected to the W-shape beams by ¾" diameter x 5" long shear studs allowing the two systems to work together in composite action. The beams then frame into larger W-shape girders via a single angle connection or a single plate connection. The beams are coped allowing them to connect to the girder's web so that the composite deck can sit on both the beams and the girders. A typical beam to girder connection is pictured below in Figure 5.



CONNECTION SCHEDULE	
BEAM SIZE	NUMBER OF ¾"ø A-325N BOLTS
W12, W14, C12	3
W16	4
W18	5
W21	6
W24, W27	7
W30 & UP	8

Figure 5: Typical Single Plate Beam to Beam Connection Detail



CONNECTION SCHEDULE	
BEAM SIZE	NUMBER OF ¾"ø A-325N BOLTS
W8	2
W12, W14, W16	3
W18, W21x44, 50, 57	4
W21x62 & UP W24x55, 62, 68	5
W24x76 & UP	6
W30, W33, W36	7

Figure 6: Typical Double Angle Beam to Column Connection Detail

The W-shape girders frame into W-shape columns by either double angle connections, or by moment connections. The double angle connection is shown above in Figure 6.

Columns

Typical columns for the Voorhees Replacement Hospital were W14's. The gravity columns were much lighter than the lateral columns. This is due to the added lateral force that the lateral columns must take. The columns are spliced every two floors, 4'-0" above the floor with either a bolted column splice or a welded splice. The columns located in zone 6 were designed for future expansion to be built above.

Lateral System

The Voorhees Replacement Hospital uses a combination of braced framing and moment connections for its lateral system. Though in both buildings the composite floor system and the roof deck acts as a diaphragm to transfer loads to either the braced frames, or the moment connections. In building A the braced frame supports the N-S lateral forces while the moment connections brace the E-W lateral system. The braced system consists of diagonal, square, HSS connected to W shapes. The braced frames are of two different styles, the bracing either frames from corner to corner, or from lower corner to the midpoint of the top beam. Typical frames can be seen in the Figures 8 and 9 on the next page. The moment frames in the Northern Building support the E-W lateral forces. The moment connections are located at the columns at the perimeter of the building, see Appendix B for a typical floor plan. A typical moment connection can be seen below in Figure 7.

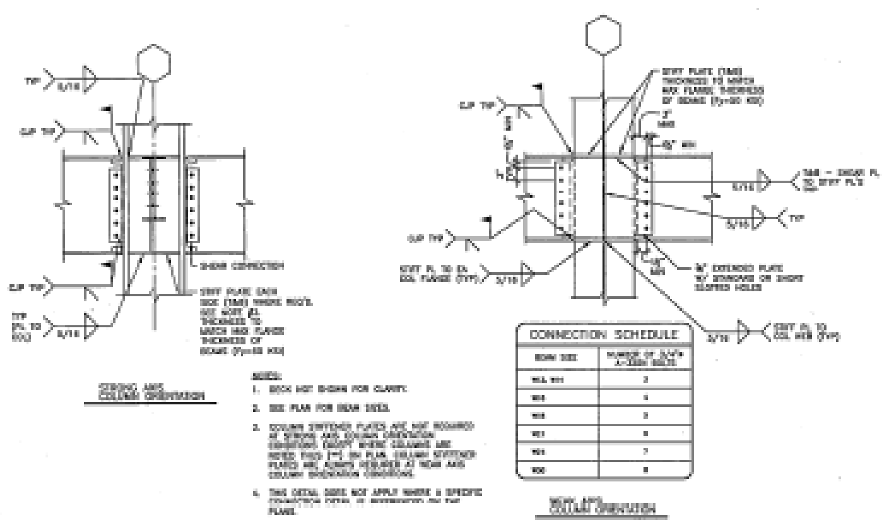


Figure 7: Typical Moment Connection Detail

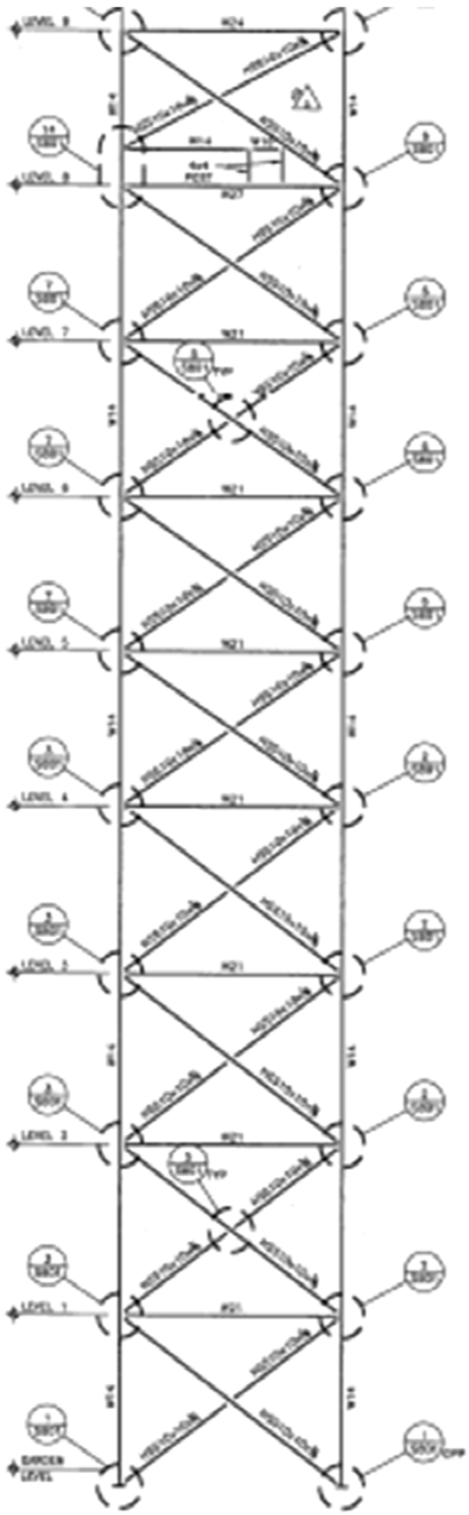


Figure 8: Typical Braced Frame

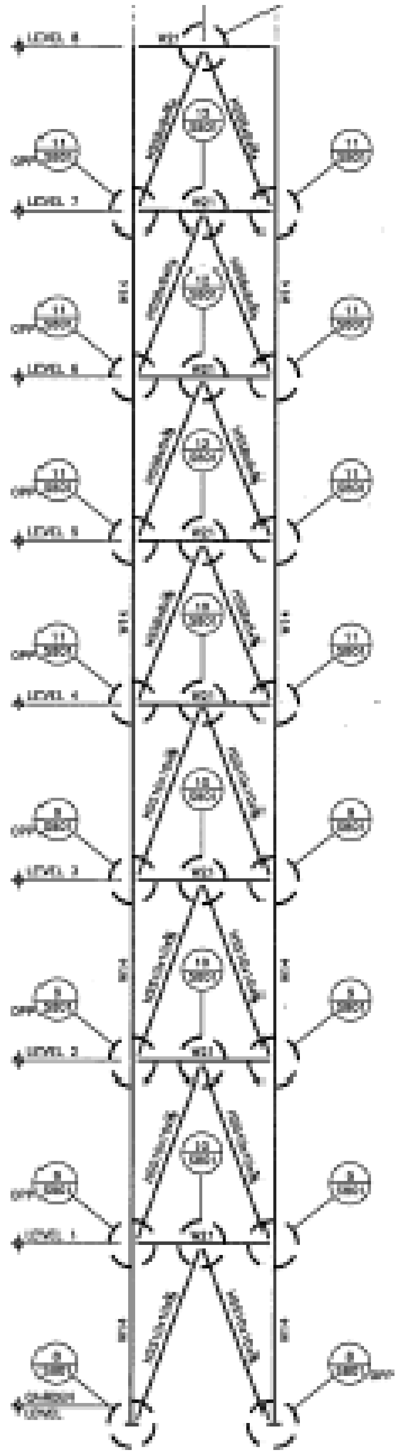


Figure 9: Typical Braced Frame

In Building B a combination of systems is used. In the N-S direction braced frames are used to resist the lateral forces. In the E-W direction, both braced frames, and moment connections resist the loads. The moment connections, again, are typically exterior columns running along the perimeter of the building. The diagonal braces are typically, like in Building A, diagonal HSS's connected to W shapes.

Roof System

The roof system is composed of 3" x 20 Gage steel roof deck topped with a concrete slab, vapor retarder, and insulation system. In certain areas the roof deck must support the green roof. To support the extra 100 psf of added weight from the green roofs, W shapes were added with a short beam to beam span. A section with added beams can be seen in the Figure 10 below.

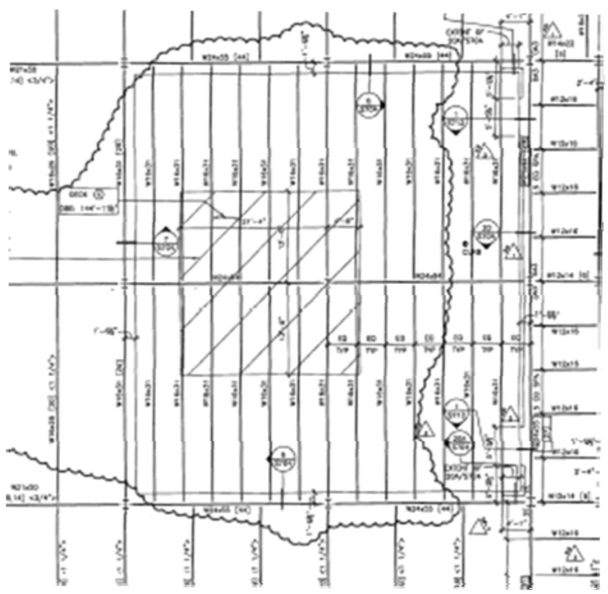


Figure 10: Typical Framing Plan for Garden Roofs

Codes & Design Standards

Applied to Original Design:

International Building Code (IBC) 2000, New Jersey Edition

American Society of Civil Engineers (ASCE 7-98), Minimum Design Loads for Buildings and Other Structures, 1998

American Concrete Institute (ACI 318), Building Code Requirements for Structural Concrete

American Institute of Steel Construction (AISC), Steel Construction Manual

Substituted for Thesis Analysis:

International Building Code (IBC) 2006

American Society of Civil Engineers (ASCE 7-05), Minimum Design Loads for Buildings and Other Structures, 2005

Material Strength Requirement Summary:

Cast-in-place Concrete:

f'_c = 3,500 psi @ 28 days for all lightweight concrete on metal decking

f'_c = 4,000 psi @ 28 days for all other concrete types

Concrete Masonry:

Concrete Masonry Units: ASTM C90 Type “N-1”

Masonry Grout: f'_c = 3,000 psi @ 28 days

Masonry Mortar: ASTM C270 (Type S uno)

Steel Reinforcing:

Reinforcing Bars: ASTM A615 (Grade 60)

Welded Bars & Anchors: ASTM A706 (Grade 60)

Deformed Bar Anchors: ASTM A496

Epoxy-Coated Reinforcing Bars: ASTM A775 or ASTM A934

Welded Wire Fabric: ASTM A185

Structural Steel:

W & WT Shapes: ASTM A992, $F_y = 50$ ksi

Plates & Shapes Other Than W: ASTM A36, $F_y = 36$ ksi

Rectangular HSS: ASTM A500, Grade B, $F_y = 46$ ksi

Round HSS: ASTM A500, Grade B, $F_y = 42$ ksi

Pipes: ASTM A53, Type E or S, Grade B, $F_y = 35$ ksi

Bolts: ASTM F1554, $F_y = 36$ ksi

Expansion Bolts: Hilti, Rawl, Thunderstuds, or National Fasteners

Adhesive Anchors/Grout: Sika, Hilti, Epcon

Headed Studs/Shear Connectors: ASTM A108

Welds:

All Types: E70XX

Building Load Summary

Building live loads were determined by referencing ASCE 7-05 and comparing that to the loads specified by the designer. Table 1 below outlines the findings.

Live Loads					
Load Description	**Design Load (psf)**	**ASCE 07-05 Load (psf)**	**Reduced?**	**Reducible by Code?**	**Assumed Partition Load (psf)**
Labs	60	60	Yes	Yes	20
Operating Rooms	60	60	Yes	Yes	20
Private Rooms/Wards	40	40	Yes	Yes	20
Offices	50	50	Yes	Yes	20
Corridors above the 1st floor	80	80	Yes	Yes	N/A
Lobbies/1st floor corridors	100	100	Yes	No	N/A
Stairs and Exits	100	100	No	No	N/A
Storage	125	125	No	No	N/A
Mechanical Room	125	125	No	No	N/A
Roof Garden	N/A	100	N/A	No	N/A
Roof	N/A	20	N/A	Yes	N/A

Table 1: Live Loads

The designer's codes and ASCE 7-05's codes were, for the most part, the same. The only difference comes with the reduction of the lobbies and 1st floor corridor. It has been assumed that, according to ASCE 7-05 4.8.4, the 1st floor corridor and lobbies are a public assembly area, and therefore cannot be reduced. Also, the designer did not provide a specified roof load; therefore this report shall use the load specified in ASCE 7-05.

Building snow loads were determined by referencing ASCE 7-05 and comparing that to the loads specified by the designer. The load values were found to be the same. Table 2 below outlines the findings.

Snow Loads		
	Found	**Designer**
Ground Snow Load, P_g	25 psf	25 psf
Flat Roof Snow Load, P_f	24 psf	24 psf
Snow Importance Factor	1.2	1.2
Snow Exposure Factor, C_e	1.0	1.0
Thermal Factor, C_t	1.0	1.0

Table 2: Snow Loads

Design Analysis

Wind Load Summary

For the wind load calculations, ASCE-07 2005 was used. It was assumed that the two buildings acted differently when subjected to wind. The assumption was justified because of the different shapes of the buildings. The northern building has its longest wall facing the N-S direction, while the E-W building has its longest wall facing the E-W direction. It was also justified, because of the two different lateral systems acting in the building. The values for the separate buildings are listed in Tables 3 through 4 and Figures 11 through 18 below. The calculations can be found in Appendix A.

Level	Height Above Ground (ft)	Wind Forces					
		Load (kip)		Shear (kip)		Moment (ft-kip)	
		N-S	E-W	N-S	E-W	N-S	E-W
9	139.33	103.1	31.7	0	0	14364.9	4416.8
8	117.33	166.8	51.5	103.1	31.7	19570.6	6042.5
7	103.33	126.3	39.2	269.9	83.2	13050.6	4050.5
6	89.33	123.6	38.5	396.2	122.4	11041.2	2545.9
5	75.33	120.6	37.7	519.8	150.9	9084.8	2839.9
4	61.33	122.7	38.7	640.4	188.6	7525.2	2373.5
3	46.00	123.8	39.3	763.1	227.3	5694.8	1807.8
2	30.66	118.0	37.9	886.9	266.6	3817.9	1162.0
1	15.33	109.9	35.9	1004.9	304.5	1684.8	550.3
Total		**1114.8**	**350.4**	**1114.8**	**350.4**	**85834.8**	**25789.2**

Table 3: The Northern Building, Wind Loads, Shears, and Moments by Level

Level	Height Above Ground (ft)	Wind Forces					
		Load (kip)		Shear (kip)		Moment (ft-kip)	
		N-S	E-W	N-S	E-W	N-S	E-W
9	139.33	122.9	124.6	0	0	17123.7	17360.5
8	117.33	199.3	202.1	122.9	124.6	23383.9	23712.4
7	103.33	151.7	153.9	322.2	326.7	15902.5	15902.5
6	89.33	149.2	151.4	473.9	480.6	13328.0	13524.6
5	75.33	146.4	148.4	623.1	632.0	11028.3	11179.0
4	61.33	149.9	151.9	769.5	780.4	9193.4	9316.0
3	46.00	152.5	154.6	919.4	932.3	7015.0	7111.6
2	30.66	146.9	148.9	1071.9	1086.9	4504.0	4565.3
1	15.33	139.1	141.0	1218.8	1235.8	2132.4	2161.5
Total		**1357.9**	**1376.8**	**1357.9**	**1376.8**	**103611.2**	**104833.4**

Table 4: The Southern Building, Wind Loads, Shears, and Moments by Level

It was found that for the northern building, the N-S direction controlled. This is expected due to the differing lengths for the building. The wall facing the N-S direction is much longer as compared to the E-W wall. For the southern building, the E-W direction controls. Again this is expected due to the larger wall length along that direction when compared to the N-S direction. The values are very similar because when the wind loads were calculated, it was assumed that the lengths of the building did not change as the heights increased. This is true for the E-W walls, but not true for the N-S walls. It is believed that this assumption was valid because the E-W direction controlled, not the N-S.

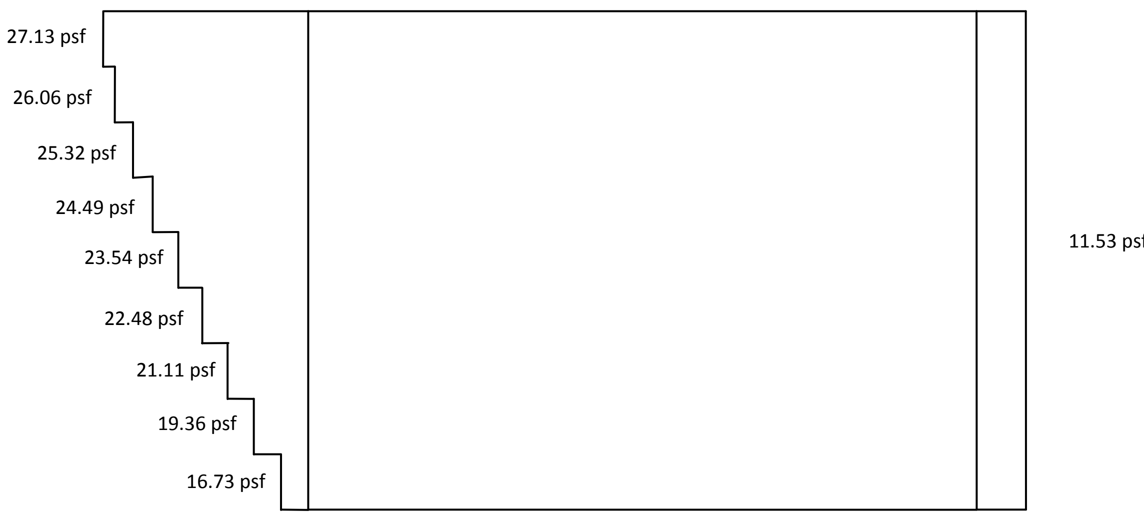


Figure 11: The Northern Building N-S Wind Loads

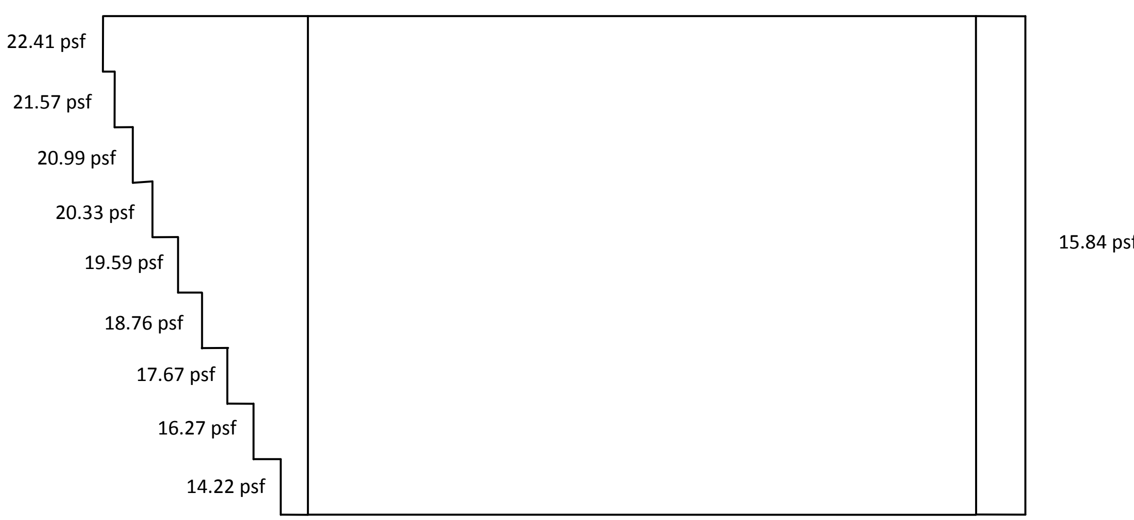


Figure 12: The Southern Building N-S Wind Loads

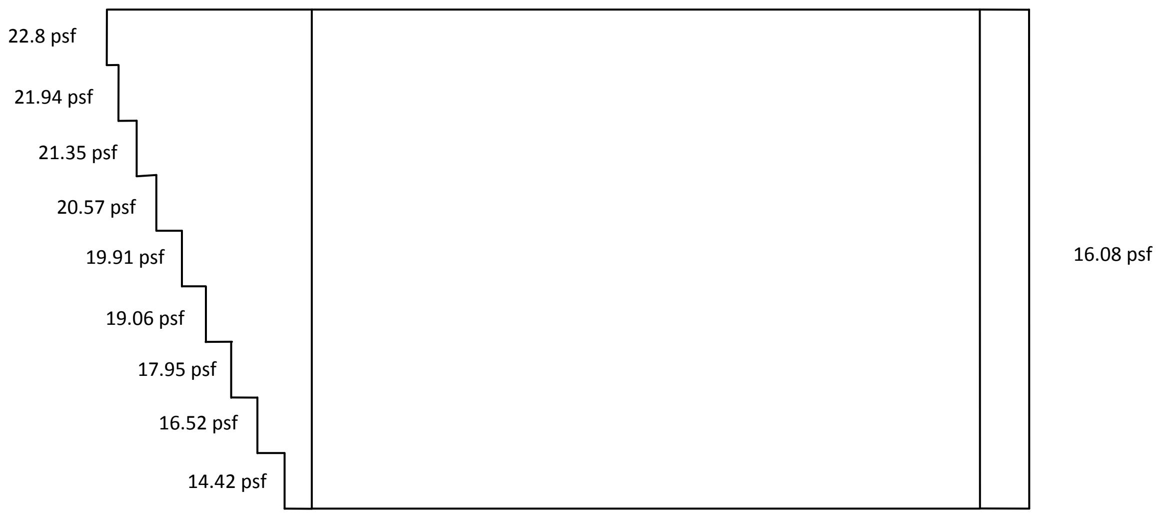


Figure 13: The Northern Building E-W Wind Loads

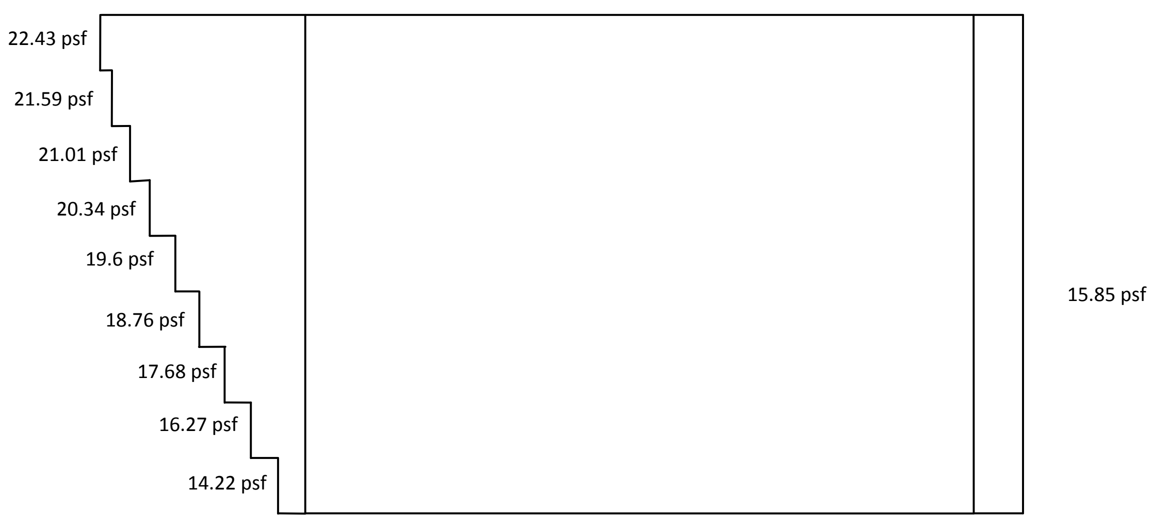


Figure 14: The Southern Building E-W Wind Loads

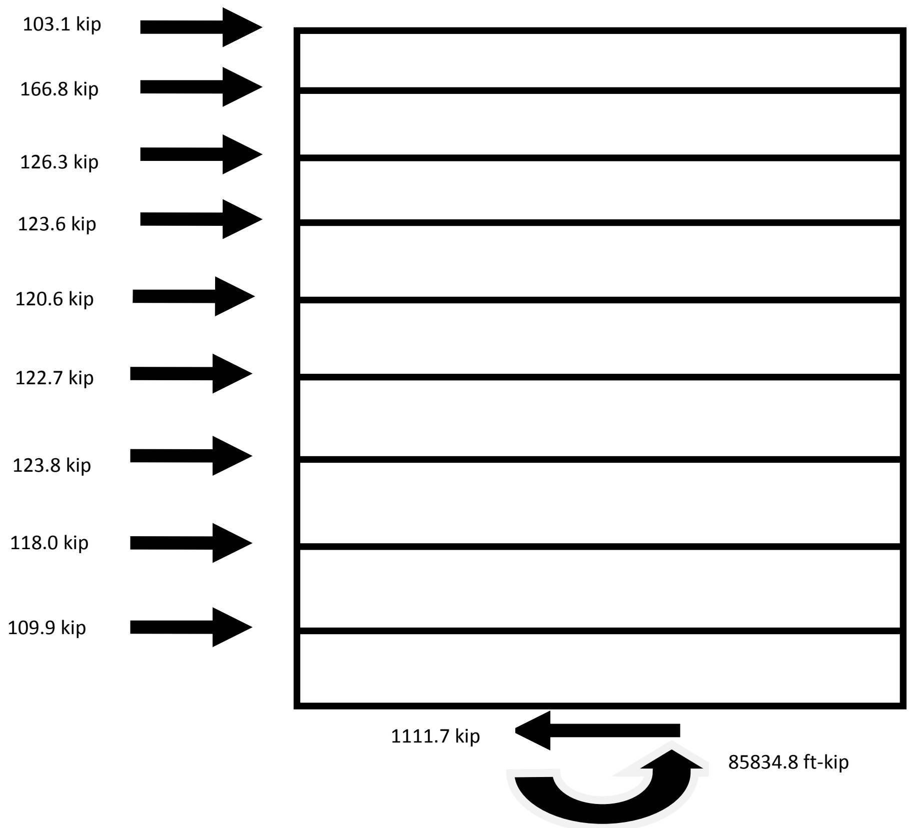


Figure 15: The Northern Building N-S Wind Loads

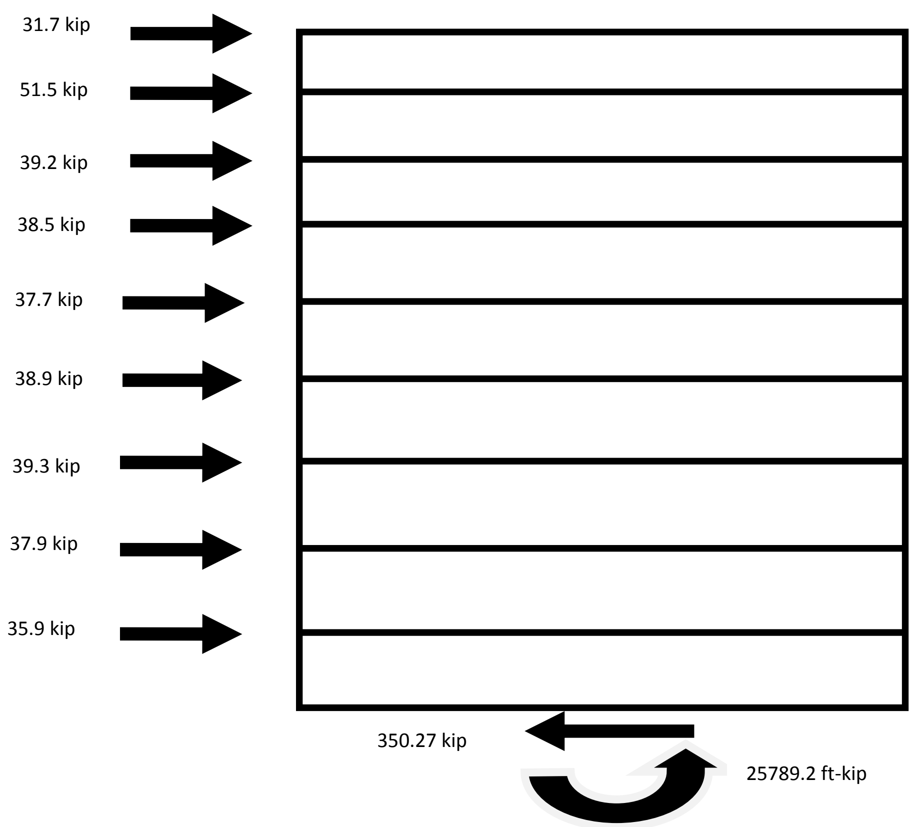


Figure 16: The Northern Building E-W Wind Loads

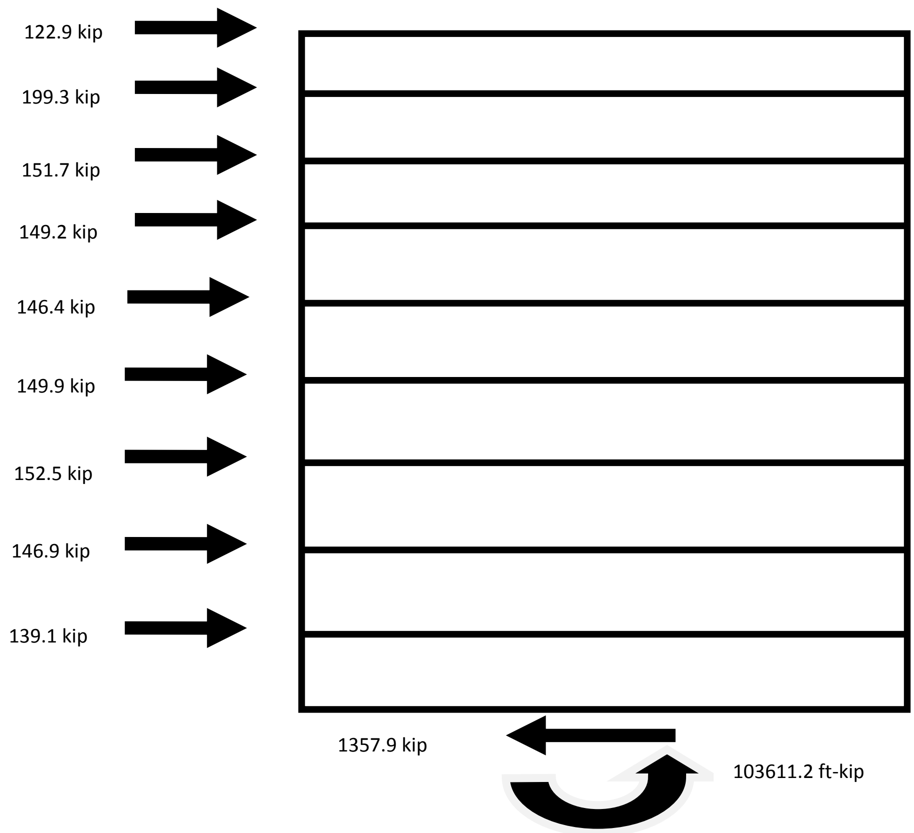


Figure 17: The Northern Building E-W Wind Loads

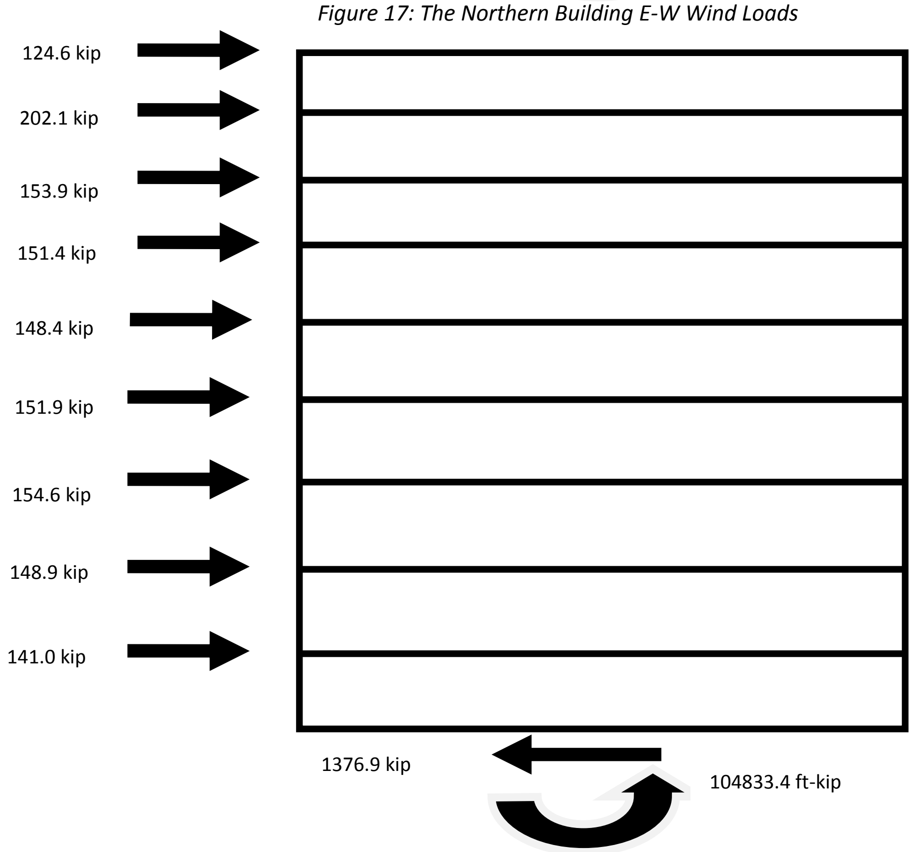


Figure 18: The Northern Building E-W Wind Loads

Seismic Load Summary

The seismic loads for this report were calculated using ASCE 7-05. Again it was assumed that the two buildings acted differently when subjected to a seismic load. The main justification for this assumption was that the two areas have different lateral systems. It was also assumed that due to the added geopiers used to densify the soil, that the site class was now D. The found seismic loads are listed in the Tables 5 through 8 below. The calculations can be found in Appendix A.

Building A N-S Seismic Loads							
Level	**Story Weight (kip)**	**Height (ft.)**	**$w_x h_x^k$**	**C_{vx}**	**Lateral Force (kip)**	**Story Shear (kip)**	**Moment (ft-kip)**
R	469.9	139.33	140726.9	0.037	55.2	55.2	7685.1
8	3497.1	117.33	858767.4	0.223	336.6	391.7	39492.1
7	3497.1	103.33	741548.5	0.192	290.6	682.4	30032.5
6	3497.1	89.33	626772.6	0.163	245.7	928.1	21944.9
5	3497.1	75.33	514761.5	0.134	201.8	1129.8	15198.5
4	3497.1	61.33	405947.9	0.105	159.1	1288.9	9758.2
3	3497.1	46	291201.3	0.0756	114.1	1403.1	5250.2
2	3497.1	30.66	182263.2	0.0473	71.4	1474.5	2190.3
1	3866.5	15.33	90493.9	0.0234	35.5	1510.0	543.7
Sum	**28816.1**		**3852483.1**	**1**	**1510.0**		**132095.4**

Table 5: The Northern Building N-S Seismic Loads

Building A E-W Seismic Loads							
Level	**Story Weight (kip)**	**Height (ft.)**	**$w_x h_x^k$**	**C_{vx}**	**Lateral Force (kips)**	**Story Shear (kip)**	**Moment (ft-kip)**
R	469.9	139.33	683079.6	0.0431	33.8	33.8	4703.6
8	3497.1	117.33	3945358.1	0.249	195.0	228.7	22877.8
7	3497.1	103.33	3271086.5	0.206	161.7	390.4	16704.6
6	3497.1	89.33	2638937.6	0.166	130.4	520.8	11650.5
5	3497.1	75.33	2052276.2	0.129	101.4	622.3	7640.5
4	3497.1	61.33	1515394.4	0.0956	74.9	697.1	4593.2
3	3497.1	46	991461.5	0.0625	49.0	746.1	2254.0
2	3497.1	30.66	545006.9	0.0344	26.9	773.1	825.8
1	3866.5	15.33	216766.8	0.0147	10.7	783.8	164.2
Sum	**28816.1**		**15859367.5**	**1**	**783.8**		**71414.3**

Table 6: The Northern Building E-W Seismic Loads

Building B N-S Seismic Loads							
Level	**Story Weight (kip)**	**Height (ft.)**	**$w_xh_x^k$**	**C_{vx}**	**Lateral Force (kip)**	**Story Shear (kip)**	**Moment (ft-kip)**
R	1556.8	139.33	466234.47	0.164	252.125	252.125	35128.580
8	788.8	117.33	193702.12	0.068	104.748	356.873	12290.089
7	1294.5	103.33	274494.46	0.096	148.438	505.311	15338.101
6	515.75	89.33	92436.0	0.032	49.987	555.298	4465.294
5	3350.9	75.33	493241.3	0.173	266.729	822.027	20092.733
4	4709.4	61.33	546673.3	0.192	295.624	1117.651	18130.609
3	5212.5	46	434041.6	0.152	234.716	1352.367	10796.942
2	4133.2	30.66	215415.7	0.076	116.490	1468.857	3571.585
1	5592.2	15.33	130883.2	0.046	70.778	1539.635	1085.020
Sum	**27154.05**		**2847122.1**	**1.000**	**1539.635**		**120898.952**

Table 7: The Southern Building N-S Seismic Loads

Building B E-W Seismic Loads							
Level	**Story Weight (kip)**	**Height (ft.)**	**$w_xh_x^k$**	**C_{vx}**	**Lateral Force (kip)**	**Story Shear (kip**	**Moment (ft-kip)**
R	1556.8	139.33	466234.470	0.164	216.107	216.107	30110.211
8	788.8	117.33	193702.122	0.068	89.784	305.891	10534.362
7	1294.5	103.33	274494.458	0.096	127.233	433.124	13146.943
6	515.75	89.33	92436.003	0.032	42.846	475.969	3827.395
5	3350.9	75.33	493241.341	0.173	228.625	704.595	17222.343
4	4709.4	61.33	546673.288	0.192	253.392	957.986	15540.522
3	5212.5	46	434041.608	0.152	201.185	1159.172	9254.522
2	4133.2	30.66	215415.651	0.076	99.849	1259.020	3061.358
1	5592.2	15.33	130883.194	0.046	60.666	1319.687	930.017
Sum	**27154.05**		**2847122.135**	**1.000**	**1319.687**		**103627.673**

Table 8: The Southern Building E-W Seismic Loads

In the northern building, the base shear in the N-S direction was found to control. The shear was approximately twice as much as in the E-W direction. These values are similar to the values that the designer specified. The designer specified a load of 1528 kips in the N-S direction and 873 kip. These values are expected to be higher than the calculated due to the lack of mechanical equipment weights in the calculated weights.

In the southern building, the base shear in the N-S direction was also found to control. These calculated values were found to differ from the designed loads. The designer specified values of 1710 kip in the N-S direction and 1235 kip in the E-W direction. The calculated value in the N-S direction was found to be lower than the specified value, while the E-W calculated value was found to be higher than the specified value. A possible reason for N-S values not corresponding is the lack of mechanical equipment weights in the calculated values.

In both buildings, the seismic values control over the wind values. This is to be expected because of the hospital having such a high importance factor, and the site having poor soil quality.

Spot Check of Typical Gravity Load Area

Spot checks were performed at random areas throughout the Voorhees Hospital. For exact calculations please refer to Appendix A.

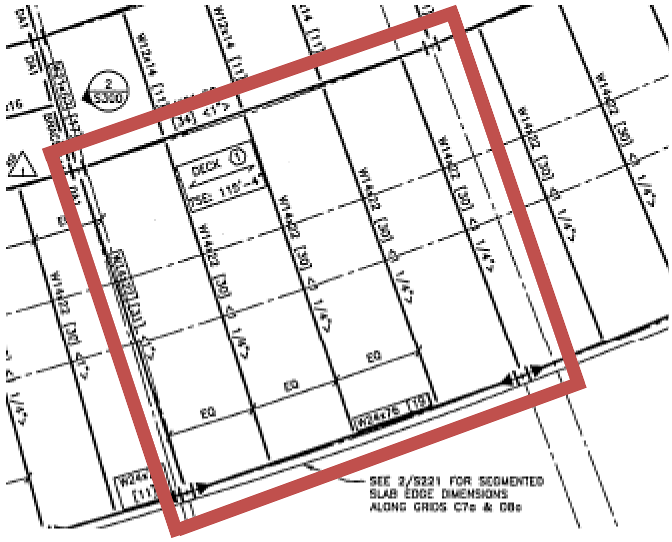


Figure 19: Typical Bay Selected for Metal Decking Spot Check

Metal Decking:

In Figure 19 above is the bay that was analyzed for the lightweight concrete slab on metal deck. This bay is located on level 1 of zone 3. It was determined from the structural design criteria that the designer used 3" deep metal deck with a 40 ksi minimum yield strength, and a minimum thickness of 18 gage. It was determined from the CMC Joist & Deck, 2008 Design Manual and Catalog of Steel Deck Products, that the specified steel deck could support 400 psf. The total load required for the deck to support is 238.4 psf. Therefore the specified metal deck is adequate.

Figure 20: Typical Beam Selected for Composite Beam Spot Check

Typical Composite Beam:

In Figure 20 above is a typical composite beam that was analyzed. This particular beam came from level 2 in zone 5. The beam is specified as a W16x26 with 16, ¾"diameter x 5" long, shear studs, and a 1 ¼" camber. This beam supports restroom and locker rooms, therefore a live load of 60 psf will be assumed. The beam was checked for bending, construction loads, and deflection. The beam was not checked for shear; it was assumed that shear will not control this average length beam.

After designing a typical composite beam, it was verified that a W16x26 without a camber meets the requirements for strength, and serviceability. The maximum deflection was 0.845" which is smaller when compared to the $^{L}/_{360}$ value of 1.04". With a camber of 1 ¼" the deflection was also found to be acceptable with a deflection of -0.4", or 0.4" in the upward direction. The minimum amount of shear studs was found to be 10, which is acceptable when compared to the specified 16.

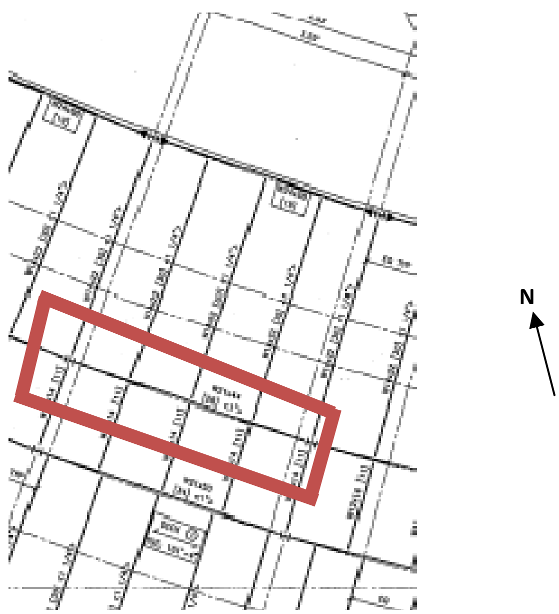


Figure 21: Typical Composite Girder Selected for Spot Check

Typical Composite Girder:

In Figure 21 above is a typical composite girder that was analyzed. This particular girder came from level 4 in zone 1. The beam is specified as a W21x44 with 28, ¾" diameter x 5" long, shear studs, and a 1" camber. This girder supports the weight of the beams that are supporting a corridor above the first floor, a patient's room, and a medicine room. Therefore a live load of 80 psf on the south side of the beam, and 40 psf on the north side of the beam, were assumed in this calculation. Live load reduction was preformed for these live loads. The girder was checked for bending, and deflection. It was not checked for shear; it was assumed that the shear strength would not control due to the average length of this girder.

After designing a typical composite girder, it was verified that a W21x44 would be sufficient without the camber. The maximum deflection found in the girder was 0.22", while the $^{L}/_{360}$ value was found to be 1.08". With a camber of 1" on the girder the maximum deflection was found to be -0.78", or 0.78" in the upward direction, which is also below the $^{L}/_{360}$ value of 1.08". The minimum amount of shear studs needed for the girder was 16, which is less than the 28 specified, therefore it is acceptable.

Figure 22: Typical Column Selected for Spot Check

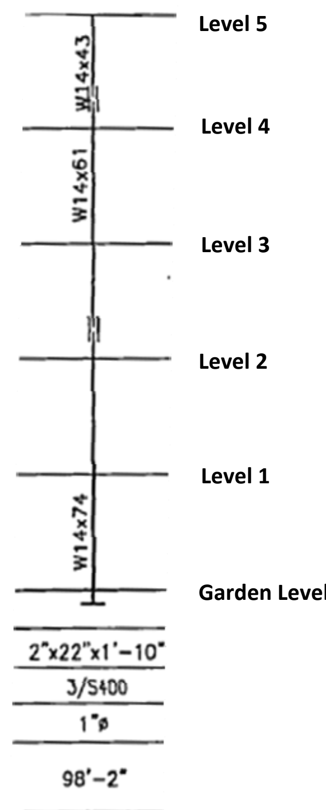


Figure 23: Typical Column Elevation Selected for Sport Check

Typical Column:

In Figures 22 and 23 above is a typical column that was analyzed. This particular column, Qa-7a, is located on level 4 in zone 4. This column is located in an area primarily surrounded by offices and labs, therefore a live load of 60 psf was assumed. The columns were spliced 4' above level 2 and 4. The column continues to the bottom of level 5. The specified columns are a W14x43at the top, a W14x61 in the middle, and a W14x74 at the bottom. The unbraced lengths were taken to be the floor to floor height, or typically 15'-4".

Using table 4-1 in AISC the beam sizes were found to be W14x43 at the top, W14x61 in the middle, and W14x82 at the bottom. The top column, W14x43, was found to be to carry a much larger load, 397 kip, compared to the calculated load of 100.9 kip. This is most likely because the designer wanted to keep a constant beam size of W14 the entire way up to avoid expensive splice connections. The middle column, W14x61, was calculated to be 100 kip over the capacity because the next smallest size would not be able to take the full amount of load. The bottom column was calculated to be a W14x82, which is larger than the specified column. This probably occurs since the calculated value did not take into account live load reductions.

Conclusion

In the first technical report of the Voorhees Replacement Hospital, the existing building conditions are investigated. A detailed description of the building's foundations, floor system, columns, lateral system, and roof system, as well as typical floor framing plans and other images were provided to introduce the building and structure. Gravity loads were calculated from ASCE 7-05 and were used to verify typical bay member sizes. Lateral loads were also examined using ASCE 7-05 and were compared to the forces used by the designers.

Spot checks for random typical bays verified the structural engineer's results shown in the structural drawings. It was determined that the checked member sizes were similar to those specified by the designer. Seismic loads in Building A calculated in this report were similar to those calculated by the structural engineer, though the loads in Building B were off by a considerable amount. It was found that in both cases the Seismic loads controlled over the Wind loads.

Appendix A

Paul Stewart | Virtua Health Hospital | Tech #1 | Seismic | 1

Design Data:

Location Voorhees, New Jersey (Latitude: 39.84°, Longitude -74.93°)

Soil Classification: D

Occupancy: Medical

Material: Structural Steel

Structural System: Moment-resisting frames & Braced Frames

1)
a) $S_s = 0.249$
$S_1 = 0.057$ } According to the USGS Ground Motion Parameter Calculator

b) Site Class D

c) $S_{MS} = 0.398$

$S_{M1} = 0.137$

d) $S_{DS} = 0.265$

$S_{D1} = 0.092$

2) a) $S_1 = 0.057 \geq 0.04$ & $S_s = 0.249 > 0.15$ $\therefore$ can not be assigned to SDC A

b) For a health care facility or hospital having surgery or emergency treatment facilities, occupancy category is $\underline{IV}$

c) $S_1 < .75$ $\therefore$ not assigned to SDC E or SDC F

d) $\therefore$ determin SDC from 11.6-1 & 11.6-2

$.167 \leq S_{DS} \leq .33$ $\therefore$ use SDC = C

e) In N-S of Building A, uses concentrically braced frames

$T_a = C_t h_n^x = .02(139.33)^{.75} = 0.811$

In N-S of Building B, uses concentrically braced frames & moment frames:

$T_a = .02(139.33)^{.75} = .811$

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In E-W of Building A, Moment frames are used

$T_a = .028(139.33)^{.8} = 1.45$

In E-W of bldg B, both systems are used

$T_a = .811$

$T_S = S_{D1}/S_{DS} = 2.88\text{ sec}$

3) check if $T < 3.5T_s$

T for B. A. N-S $= 0.811 > 1.008$
T for B. B N-S $= 0.811 > 1.008$ $\quad .35T_s = .35(2.88) = 1.008$
T for B A E-W $= 1.45 < 1.008$
T for B B E-W $= 0.811 > 1.008$

a) determine R

$R_{BANS} = 3.25$ ← ordinary steel concentrically braced frames

$R_{BBNS} = 3$ ← not described

$R_{BAEW} = 3.5$ ← ordinary steel moment frames

$R_{BBEW} = 3.5$ ← ordinary moment frames

b) $I = 1.5$

c) $T_L = 6\text{ sec}$ from Fig 22-15

d) $C_{SANS} = \dfrac{S_{D1}}{T(R/I)} = \dfrac{0.092}{.811(3.25/1.5)} = 0.0524$

$C_{SBNS} = \dfrac{0.092}{.811(3/1.5)} = 0.0567$

$C_{SAEW} = \dfrac{.092}{1.45(3.5/1.5)} = 0.0272$

$C_{SBEW} = \dfrac{.092}{.811(3.5/1.5)} = 0.0486$

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e) Dead Load for Building A:

Area #1: Area of 1 = 30.083'

4.03°
$R_1 = 429.25'$
$R_2 = 459.33'$

$C = \frac{2\pi r}{360} = 7.492\ ft/°$

$C = \frac{2\pi r}{360} = 8.0168\ ft/°$

$d = \frac{8.0168 + 7.492}{2} = 7.754\ ft/° \times 4.03° = 31.25\ ft$

Area of #1 = $31.25 \times 30.083 = 940.1\ ft^2$

concrete = 115 pcf = $115(6.5/12) = 62.3\ psf$
metal deck = 12 pcf = $12(3/12) = 3\ psf$

65.3 psf

beam weight = $\frac{31}{2} + 22 + 22 + 22 + \frac{22}{2} = \frac{92.5\ plf}{31.25\ ft} = 2.96\ psf$

girder weight = $76 + \frac{44}{2} = \frac{98}{30.083} = 3.26\ psf$

Total = $65.3 + 2.96 + 3.26 = 71.5\ psf$

column weights

Assume 7 psf

Total = $71.5\ psf + 7\ psf = 78.5\ psf$

Area of building A = $38547\ ft^2/floor$

Total weight of Building A:

$78.5\ psf(38547\ ft^2)(9) = 27233.46^k$

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Building B

Deck weight = 65.3psf

beam weight = $26+26+26\text{plf} = \frac{78\text{plf}}{29.33\text{ft}} = 2.66\text{psf}$

girder weight = $35+\frac{50}{2} = \frac{60}{31.33} = 1.92\text{psf}$

assume 7psf column weight

Total weights = $65.3+2.66+1.92+7 = 76.9\text{psf}$

Area of Building B = $63634\text{ft}^2 + 85293 + 68547 + 50972 + 47287 + 14625 + 14625 + 9893 + 2622 + 2622$

$= 360120\text{ft}^2$

Total weight of Building B:

$360120(76.9) = 27693.2^{k}$

$V_{ANS} = .0524(27233.5) = 1427.03^{k}$

$V_{BNS} = .0567(27693.2) = 1570.2^{k}$

$V_{AEW} = .0272(27233.5) = 740.75^{k}$

$V_{BEW} = .0567(27693.2) = 1570.2^{k}$

f) $.5 < T < 2.5\text{sec}$

$k_{ANS} = .75+.5T = 1.155$

$k_{BNS} = 1.155$

$k_{AEW} = 1.475$

$k_{BEW} = 1.155$

g) See excel sheet for Lateral Forces & Story shear

$k_{ANS} = 1.155$ — @ .5 = 1, @ 2.5 = 2

$K_{BNS} = 1.155$

$K_{AEW} = 1.475$

$K_{BEW} = 1.155$

$p_g = 25\,psf$ From ASCE7-05 Figure 7-1

Importance Factor = 1.2

Thermal Factor, $C_t = 1.0$

Exposure Factor, $C_e = 1.0$

$p_f = .7(1)(1)(1.2)(25) = 21\,psf$

$p_f \geq 20I = 24\,psf > 21\,psf$

$\therefore p_f = 24\,psf$

Paul Stewart	Virtua Health Hospital	Tech #1 Spot checks (1)

check Metal deck

3½" lightweight concrete slab on 3"x18 Ga composite steel deck

w/ 6x6-W2.1xW2.1 WWF

$f'_c = 3.5$ ksi $\quad F_y = 40$ ksi

115 pcf concrete

$$\text{Span of Deck} = \left[\left(\frac{2\pi(471.33)}{360}\right)3.59 + \left(\frac{2\pi(501.42)}{360}\right)3.59\right] / 2 = 30.48\ \text{ft/bay}$$

$$\text{Span of Deck} = 30.48\ \text{ft/bay} / 4\ \text{beam spans} = 7.62\ \text{ft}$$

Span of Deck = 7.62 ft

Per chart @ 9' span, w = 400 psf

Actual Floor Loading: LL 100 psf

$DL = W_{deck} + W_{concrete} = 3\text{psf} + 115(6.5/12)$

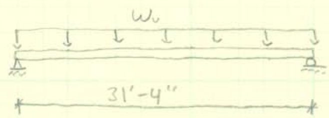
$= 65.3$ psf

$w_u = 1.2(65.3) + 1.6(100) = 238.35$ psf

$w = 400\text{psf} > w_u = 238.35$ psf $\therefore$ ok ✓

Paul Stewart | Virtua Health Hospital | Tech #1 Spot checks (2)

Beam Check:



Lab space

$w_u = 1.2D + 1.6L$

$L = 60\,psf\,(29.33ft/3) = .586\,klf$

$D = 3 + 115(6.5/12) + 5 = 70.3\,psf\,(9.78ft)$

deck concrete fireproofing

$D = .6872\,klf$

$w_u = 1.2(.687) + 1.6(.586) = 1.76\,klf$

$M_n = \frac{1.76(31.33)^2}{8} = 215.95\,ft\text{-}k$ $\qquad V_u = \frac{1.76(31.33)}{2} = 27.6^k$

Assume $a = 1"$

$Y_2 = 6.5 - 1/2 = 6.0"$

Try a W16x26:

$M_u = 252\,ft\text{-}k$ @ PNA 7

$b_{eff} = 9.78' = 117.4"$
$31.33/4 = 7.8325' = 94"$ ← controls

$a = \frac{96}{.85(3.5)(94)} = .34" \leq 1$ ∴ good assumption

$Y_2 = 6.5 - \frac{.34}{2} = 6.33" > 6.0"$ ∴ ok

$\#\,studs = \frac{\Sigma Q_n}{Q_n} \times 2 = \frac{2(96)}{21} = 9.14 = 10$ studs

check self-weight: $1.76 + 1.2(.026) = 1.79$

$M_n = \frac{1.79(31.33)^2}{8} = 219.78\,ft^k < M_u$ ∴ ok

Paul Stewart | Virtua Health Hospital | Tech #1 | Spot checks (3)

Construction loads:

concrete = 51psf

deck = 3psf

total = (54 × 9.78) + 26 = .554 klf

$LL_{construction}$ = 22psf

$w_u = 1.2(.554) + 1.6(.022 \times 9.78) = .99\text{klf}$

$M_u = \frac{.99(31.33)^2}{8} = 121.9\text{ft-k} < \phi M_p = 166\text{ft-k}$

check deflection:

$\Delta_{const} = \frac{5(.554)(31.33)^2(1728)}{29000(301)(384)} = 1.376'' \geq 1''$ ∴ no good

use 1-1/4" chamber $\quad \Delta_{const} = 1.376 - 1.25 = .126''$ ∴ ok

$E_s = 29000\text{ksi} \quad E_c = (115)^{1.5}\sqrt{3.5} = 2307.2\text{ksi} \quad a = .34$ -

$n = \frac{29000}{2307.2} = 12.57 \quad b_{eff} = 94'' \quad w_{LL} = 60(31.33) = 1.88\text{klf}$

$\frac{b_{eff}}{n} = 7.48\text{in} \quad I_s = 301\text{in}^4 \quad I_t = \frac{7.48(.34)^3}{12} = .0245\text{in}^4$

$A_t = 7.48(.34) = 2.54\text{in}^2 \quad A_s = 7.68\text{in}^2$

$\bar{y} = 7.68(15.7/2) + 2.54(15.7 + 6.5 - .34/2) / (7.68 + 2.54) = 11.37\text{in}$

$I_{tr} = 301 + 7.68(11.37 - 15.7/2)^2 + .0245 + 2.54(15.7 + 6.5 - .34/2)^2$

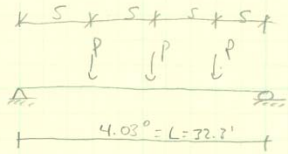
$= 1662.7\text{in}^4$

$\Delta_{LL} = \frac{5(1.88)(31.33)^4(1728)}{384(29000)(1662.7)} = .845\text{in} - 1.25 = -.41'' < +/- L/360 = +/- 1.04''$

W16×26 [10] <1-1/4"> good ✓

Paul Stewart | Virtua · Health Hospital | Tech #1 | Spot Checks (4)

Girder Check:



$L = \dfrac{2\pi(459.33)}{360}(4.03) = 32.3'$ $\qquad S = \dfrac{L}{4} = 8'$

$P = DL_N + DL_S + LL_N + LL_S$ $\qquad DL_N = (115(6.5/12) + 3 + 5)\,8ft + 22 = 584.33\,plf$

$LL_{red} = .25 + \dfrac{15}{\sqrt{2(12)(8)}}(80) = 106 > 80 \times$ $= 584.33(30.083) = \dfrac{17.58}{2} = 8.79^K$

$= 80(12)(8) = 7.68/2 = 3.84$ $\qquad DL_S = (115(6.5/12) + 3 + 5)\,8ft + 14 = 576.33\,plf$

$LL_{red} = .25 + \dfrac{15}{\sqrt{2(8)(30.08)}}(40) = 37.34$ $\qquad = 576.33(12ft) = \dfrac{6.92^K}{2} = 3.46^K$

$= 37.34(30.08)(8)(1/2) = 4.49$

$P = 1.2(8.79 + 3.46) + 1.6(4.49 + 3.84) = 28.028$

$M = \dfrac{PL}{4} + Pa = \dfrac{28.03(32.3)}{4} + 28.03(8) = 450.6\ ft\text{-}K$

$V = P + \dfrac{P}{2} = 42.05^K$

Assume $a = 1''$ $\qquad Y_2 = 6.5 - 1/2 = 6''$

$b_{eff} = S = 8(12) = 96''$
$= L/4 = \dfrac{32(12)}{4} = 96''$

Try W21x44 @ PNA = 7 $\quad \phi M_n = 528\ ftK$
must use W21 b/c W14 frames into it.

$a = \dfrac{162}{.85(3.5)(96)} = .57'' < 1''$ $\therefore$ ok to assume $a = 1''$

$Y_2 = 6.5 - \dfrac{.57}{2} = 6.22''$ $\qquad \#studs = \dfrac{2\Sigma Q_n}{Q_n} = \dfrac{2(162)}{21} = 15.4 = 16$

check self-weight $\dfrac{(1.2)44(32)^2}{8} = 6.76\ ft\text{-}K$ $+ M_u = 6.76 + 450.6 = 457.36\ ftK$
$< 528 \therefore$ ok

Paul Stewart | Virtua Health Hospital | Tech #1 Spot Checks (5)

check Δ_{LL}:

$E_s = 29000$ ksi $\quad E_c = 2307.2$ ksi $\quad a = .57"$ $\quad n = 12.57$ $\quad b_{eff} = 96"$

$w_{LL} = \frac{80(12) + 40(30.08)}{2} = 1.08$ klf

$\frac{b_{eff}}{n} = 7.64"$ $\quad I_s = 843 \text{ in}^4$ $\quad I_t = \frac{7.64(.57)^3}{12} = .118 \text{ in}^4$

$A_t = 7.64(.57) = 4.35$ $\quad A_s = 13 \text{ in}^2$

$\bar{y} = 7.64(20.7/2) + 4.35(20.7 + 6.5 - .57/2) / (4.35 + 13) = 11.31"$

$I_{tr} = 843 + 13(11.31 - 20.7/2)^2 + .118 + 4.35(20.7 + 6.5 - .57/2)^2 = 4006.3 \text{ in}^4$

$\Delta_{LL} = \frac{5(1.08)(32)^4(1728)}{384(29000)(4006.3)} = .22 \text{ in} < L/360 = 1.08"$

use W21x44 [16] <0>

Column check:

W14x43 from 4' above level 4

W14x61 from 4' above level 2

W14x74 from foundation up

Calculated below

Level	Total floor load (kip)	Takedown (Kip)
5	100.9^K	100.9^K
4	146.88^K	247.78^K
3	146.88^K	394.66^K
2	146.88^K	541.54^K
1	146.88^K	688.42^K

Level 5:

$RLL = 20\ psf$

$SL = 25\ psf$

$DL_{on\ bm} = [7.33ft(31.33ft)((115 \times \frac{6.5}{12}) + 5 + 3)] + 31.33(26) = 16.96^K$

$R_{Beam} = \frac{16.96}{2} = 8.48^K$

$DL_{on\ g} = 8.48(3) = 25.44^K$ $\quad R_g = \frac{25.44}{2} = 12.72^K$

$DL_{on\ column} = 8.48(2) + 12.72(2) = 42.4^K$

$TL = 1.2(42.4) + 1.6(20) + .5(25) = 95.4^K$
or $1.2(42.4) + 1.6(25) + .5(20) = 100.9^K \leftarrow$

Level 4:

$TL_4 = 1.2(42.4) + 1.6(60) = 146.88^K$

Level 3:

$TL_3 = TL_4 = 146.88^K$

Level 2:

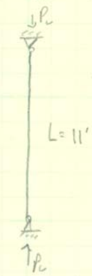
$TL_2 = TL_3 = TL_4 = 146.88^K$

Paul Stewart | Virtua Health Hospital | Tech #1 Spot Checks (7)

Level 1:

$TL_1 = TL_2 = 146.88^k$

Design of column Top splice:



$P_u = 100.9^k$

$K = 1.0$

$L = 11'$

From Table 4-1 in AISC use W14x43 $\phi P_n = 397^k$

Design of column middle splice:

$P_u = 394.66^k$

$K = 1.0$

$L = 15'\text{-}4''$ ← unbraced height is floor to floor height

use W14x61 $\phi P_n = 515^k$

Design of column lower splice:

$P_u = 688.42^k$

$K = 1.0$

$L = 15'\text{-}4''$ ← unbraced height is floor to floor height

use W14x82 $\phi P_n = 698^k$

Paul Stewart | Virtua Health Hospital | Tech #1 Wind Loads

According to ASCE 07-05:

$V = 100$ mph per Fig 6-1

$K_d = .85$ per Table 6-4

$I = 1.15$ per Table 6-1

Exposure Category = B per 6.5.6

6.5.7.1: all points not met

→ $K_{zt} = 1.0$

$\beta = .01$

N-S:
Area A = Areas 1-3: $B = 74.166'$, $h = 139'-4''$
Area B = Areas 4-7: $B = 295.875'$, $h = 139'-4''$

E-W:
Areas A: $B = 242.4'$, $h = 139'-4''$
Areas B: $B = 292'$, $h = 139'-4''$

Story heights = 15'-4"

roof mean height = 139.33 ft

For exposure type B per Table 6-2:

$\alpha = 7.0$
$z_g = 1200'$

$\hat{a} = 1/7$
$\hat{b} = 0.84$
$\bar{\alpha} = 1/4.0$
$\bar{b} = 0.45$
$c = 0.30$
$\ell = 320'$
$\bar{\epsilon} = 1/3.0$

· Find K_z per Table 6-3:

$$K_z = 2.01(z/z_g)^{2/\alpha}$$

Level	Height (ft)	K_z
0	0	0
1	15.33	0.578
2	30.66	0.705
3	46	0.792
4	61.33	0.859
5	75.33	0.911
6	89.33	0.957
7	103.33	0.998
8	117.33	1.034
9	139.33	1.086

$$K_h = 2.01(h/z_g)^{2/\alpha} = 1.086$$

Paul Stewart | Virtua · Health Hospital | Tech #1 Wind Loads (2)

· Find velocity pressures:

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

Height	q_z
0	0
15.33	14.463
30.66	17.642
46	19.819
61.33	21.496
75.33	22.797
89.33	23.948
103.33	24.974
117.33	25.875
139.33	27.176

$$q_h = .00256 K_h K_{zt} K_d V^2 I = .00256(1.086)(1)(.85)(100)^2(1.15) = 27.176$$

Gust Effects Factors:

$$n_1 = 22.2/H^{0.8} = 22.2/(139.33)^{.8} = .4276 \quad \text{per C6.5.8}$$

$n_1 < 1$ ∴ building is flexible per 6.2

$$g_Q = g_v = 3.4$$

$$g_r = \sqrt{2\ln(3600 n_1)} + \frac{.577}{\sqrt{2\ln(3600 n_1)}} = 3.982 \qquad \text{Eq 6-9}$$

$$\bar{z} = .6h = .6(139.33) = 83.598$$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = .30\left(\frac{33}{83.6}\right)^{1/6} = .2569 \qquad \text{Eq 6-5}$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{83.6}{33}\right)^{1/3} = 436.23 \qquad \text{Eq 6-7}$$

$$Q = \sqrt{\frac{1}{1 + .63\left(\frac{B+h}{L_{\bar{z}}}\right)^{.63}}} \qquad \text{Eq 6-6}$$

$Q_{A\,N\text{-}S} = 0.845$

$Q_{B\,N\text{-}S} = 0.783$

$Q_{A\,E\text{-}W} = 0.796$

$Q_{B\,E\text{-}W} = 0.784$

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$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = 83.265$$ Eq 6-14

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = 2.24$$ Eq 6-12

$$R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}} = 0.083$$ Eq 6-11

$$\eta_1 = \frac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = 3.29$$

$$R_h = \frac{1}{\eta_1} - \frac{1}{2\eta_1^2}\left(1-e^{-2\eta_1}\right) = .2578$$

$$\eta_2 = \frac{4.6 n_1 B}{\bar{V}_{\bar{z}}} =$$ $\eta_{2A\,N\text{-}S} = 1.752$ $\eta_{2A\,E\text{-}W} = 5.724$

$\eta_{2B\,N\text{-}S} = 6.989$ $\eta_{2B\,E\text{-}W} = 6.897$

$$R_B = \frac{1}{\eta_2} - \frac{1}{2\eta_2^2}\left(1-e^{-2\eta_2}\right) =$$ $R_{BA\,N\text{-}S} = .4127$ $R_{BA\,E\text{-}W} = .1594$

$R_{BB\,N\text{-}S} = .1328$ $R_{BB\,E\text{-}W} = .1345$

$$\eta_3 = \frac{15.4 n_1 L}{\bar{V}_{\bar{z}}} =$$ $\eta_{3A\,NS} = 19.165$ $\eta_{3A\,EW} = 5.865$

$\eta_{3B\,NS} = 23.093$ $\eta_{3B\,EW} = 23.399$

$$R_L = \frac{1}{\eta_3} - \frac{1}{2\eta_3^2}\left(1-e^{-2\eta_3}\right) =$$ $R_{LA\,NS} = .0508$ $R_{LA\,EW} = .1559$

$R_{LB\,NS} = .0423$ $R_{LB\,EW} = .0418$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (.53 + .47 R_L)}$$ Eq 6-10

$R_{A\,NS} = .6994$ $R_{A\,EW} = .4536$

$R_{B\,NS} = .3953$ $R_{B\,EW} = .3977$

$$G_f = .925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$$ $G_{FA\,NS} = 1.0227$ $G_{FA\,EW} = .8237$ Eq 6-8

$G_{FB\,NS} = .8059$ $G_{FB\,EW} = .8065$

The building is enclosed with no parapet, & is not a low-rise building.

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From 6-6

C_p:

GC_{pi} = +/-0.18

Windward Wall : .8

Leeward Walls: A-NS: -.2388
B-NS: -.5
A-EW: -.5
B-EW: -.5

Side walls: -.7

$P_z = q_z G_f C_p - q_h (GC_{pi})$ windward Eq 6-19

$P_h = q_h G_f C_p - q_h (GC_{pi})$ leeward Eq 6-19

Area 1,2,3 N-S

height	Floor	P_z	P_h
0	0	0	11.529
15.33	1	16.725	↓
30.66	2	19.326	
46	3	21.107	
61.33	4	22.479	
75.33	5	23.543	
89.33	6	24.485	
103.33	7	25.324	
117.33	8	26.062	
129.33	9	27.126	

Area 4,5,6,7 N-S

Floor	P_z	P_h
0	0	15.842
1	14.216	↓
2	16.266	
3	17.669	
4	18.751	
5	19.589	
6	20.331	
7	20.993	
8	21.574	
9	22.413	

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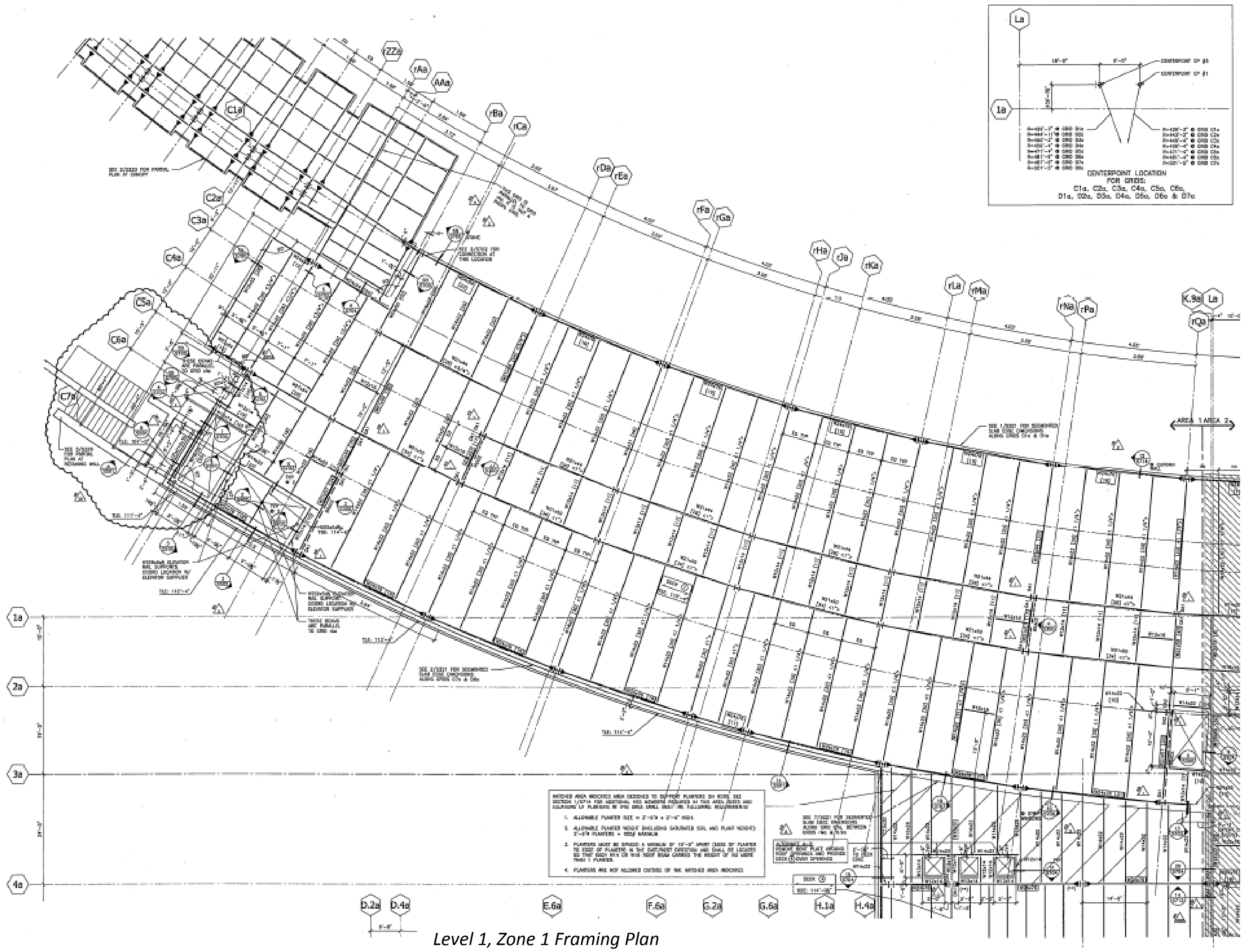
Area 1, 2, 3 E-W

Floor	Pz	Ph
0	0	16.084
1	14.422	↓
2	16.517	↓
3	17.952	↓
4	19.057	↓
5	19.914	↓
6	20.672	↓
7	21.349	↓
8	21.942	↓
9	22.800	↓

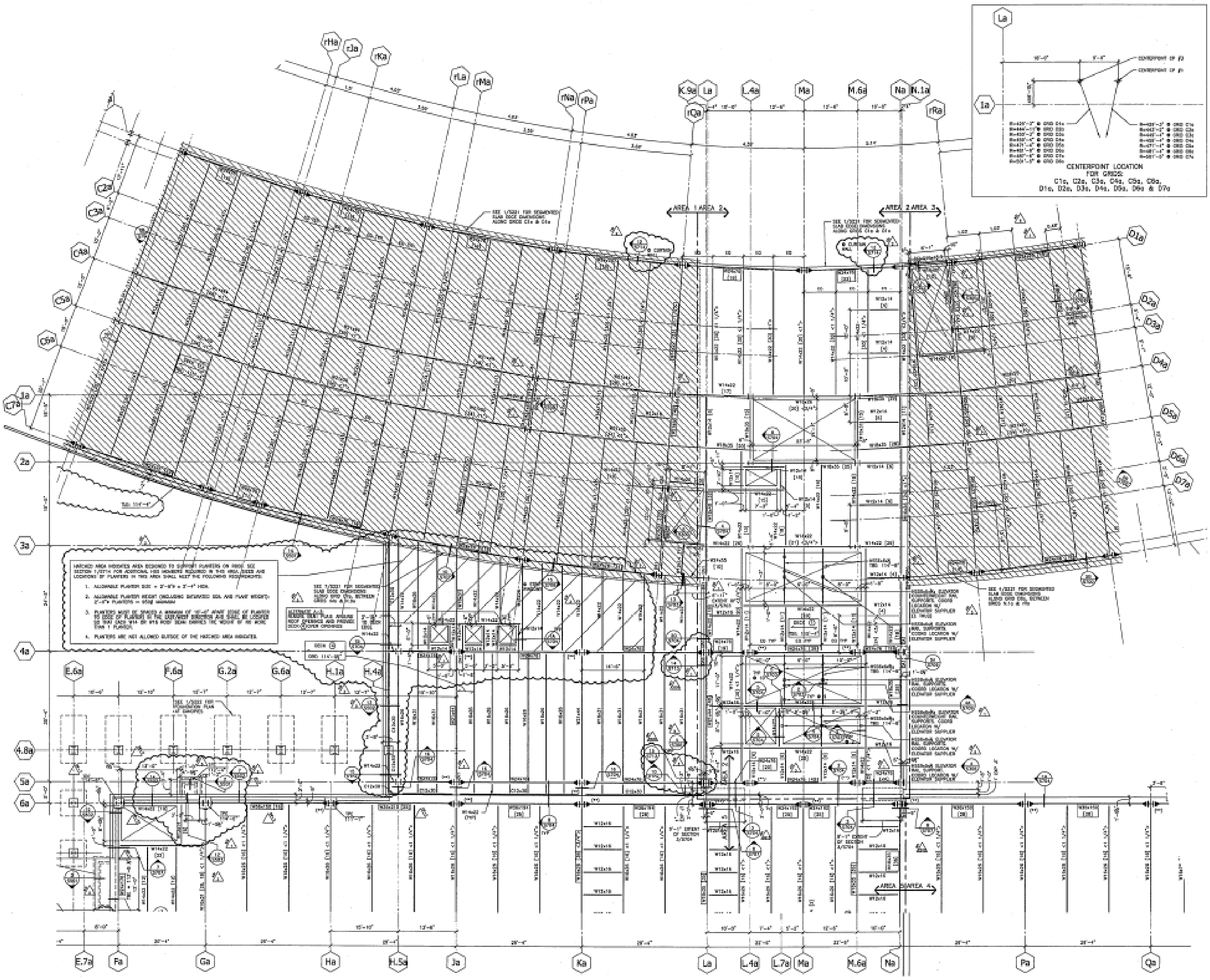
Area 4, 5, 6, 7 E-W

Floor	Pz	Ph
0	0	15.85
1	14.223	↓
2	16.274	↓
3	17.679	↓
4	18.761	↓
5	19.600	↓
6	20.343	↓
7	21.005	↓
8	21.586	↓
9	22.426	↓

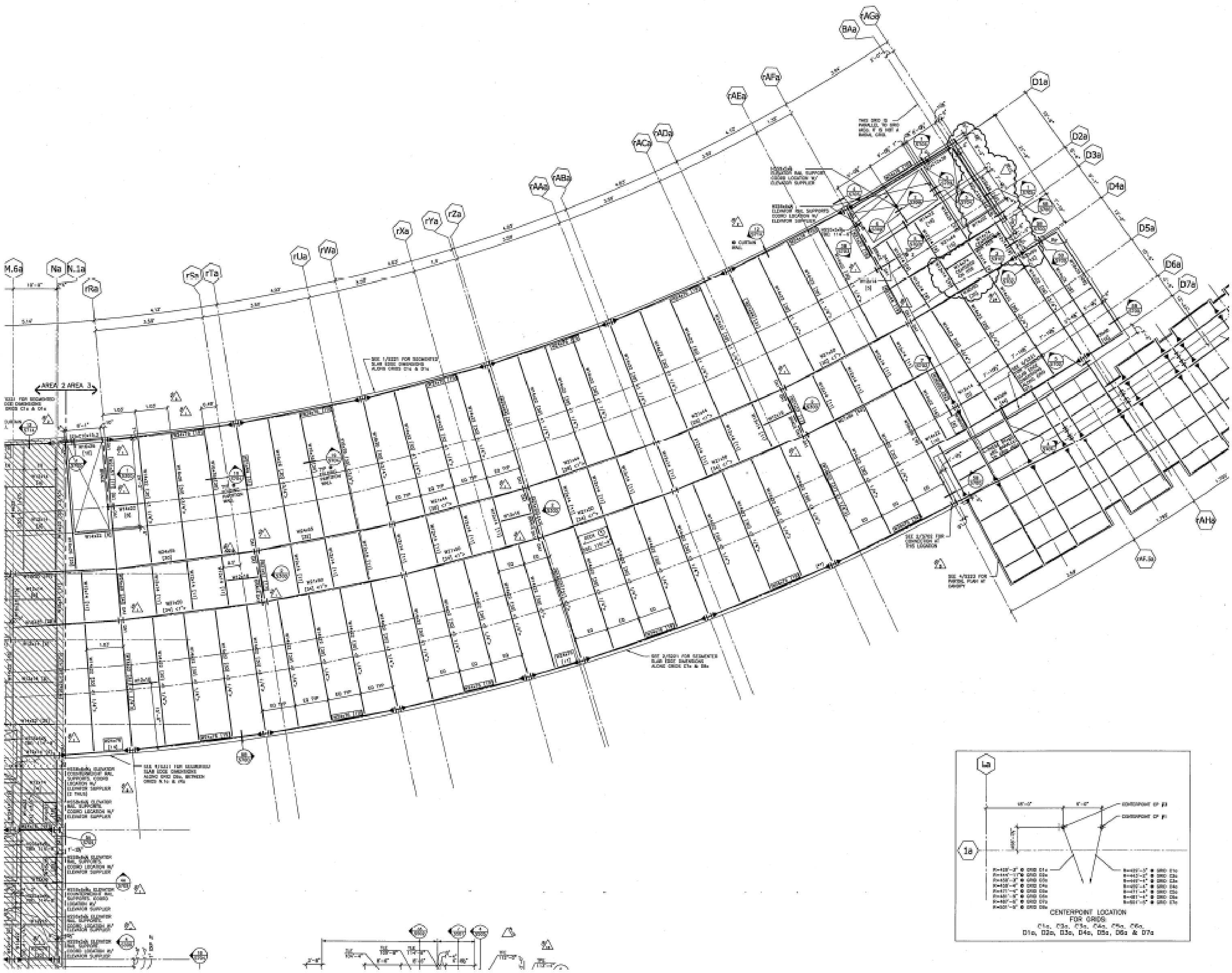
Appendix B



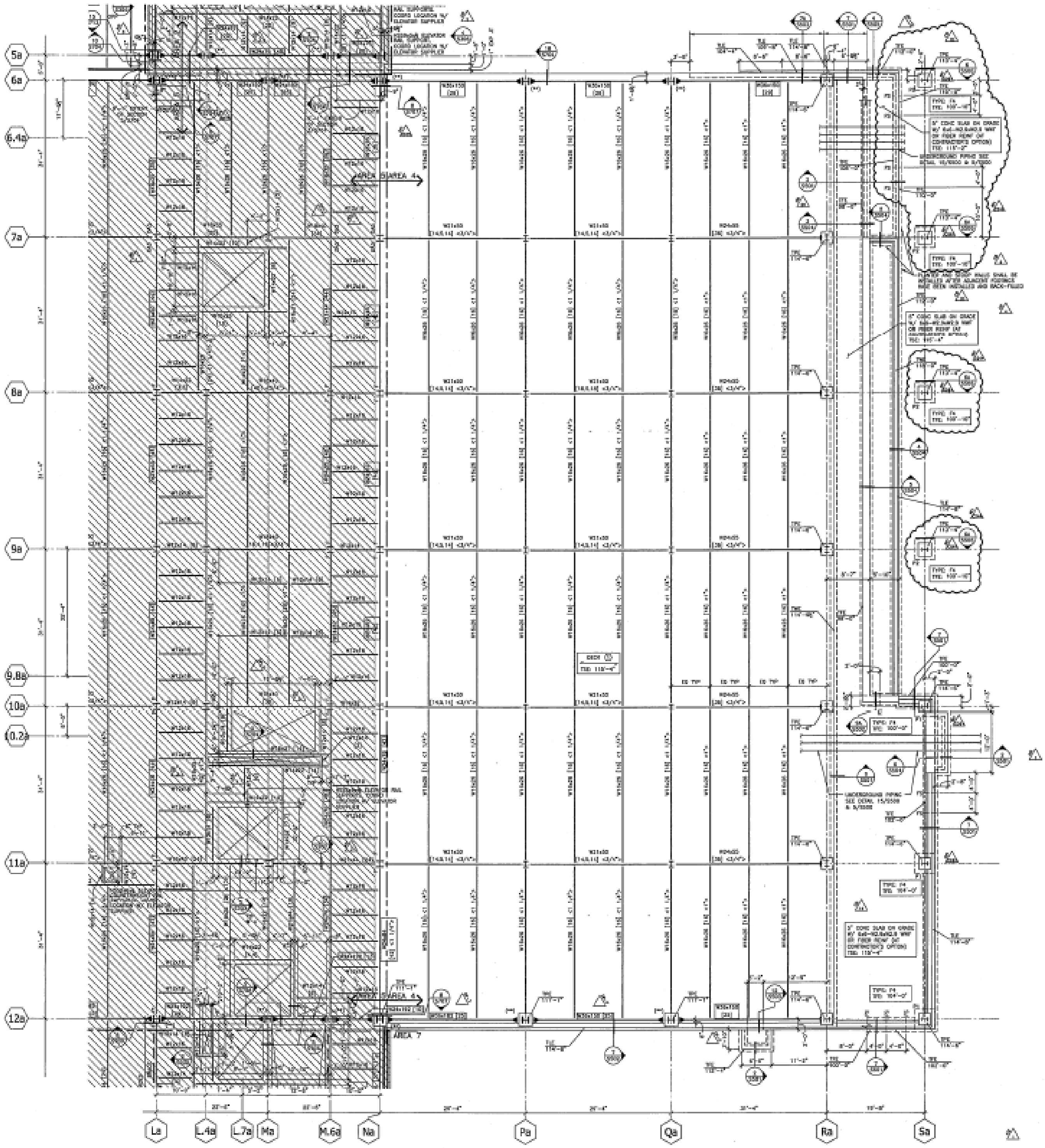
Level 1, Zone 1 Framing Plan



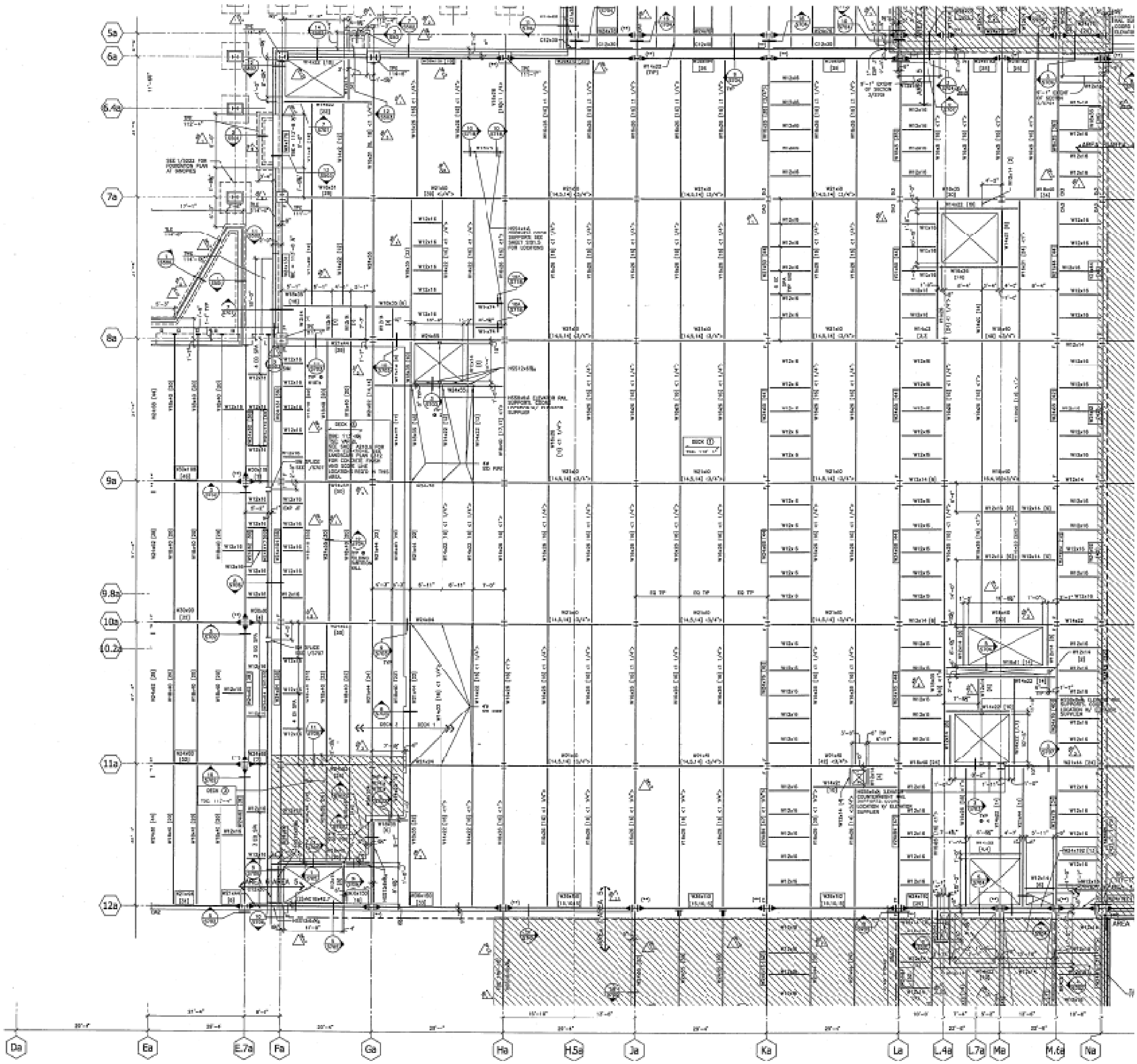
Level 1, Zone 2 Framing Plan



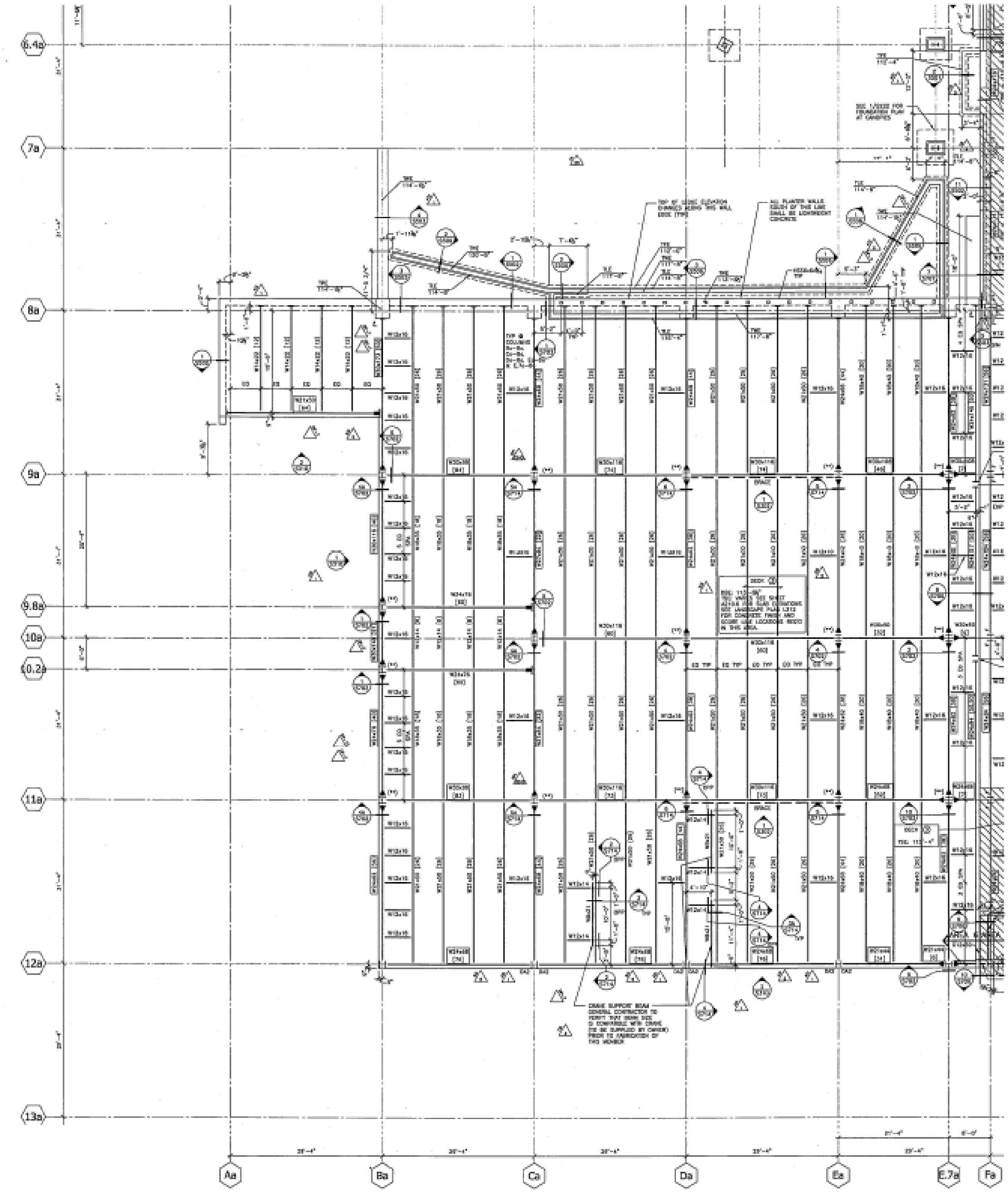
Level 1, Zone 3 Framing Plan



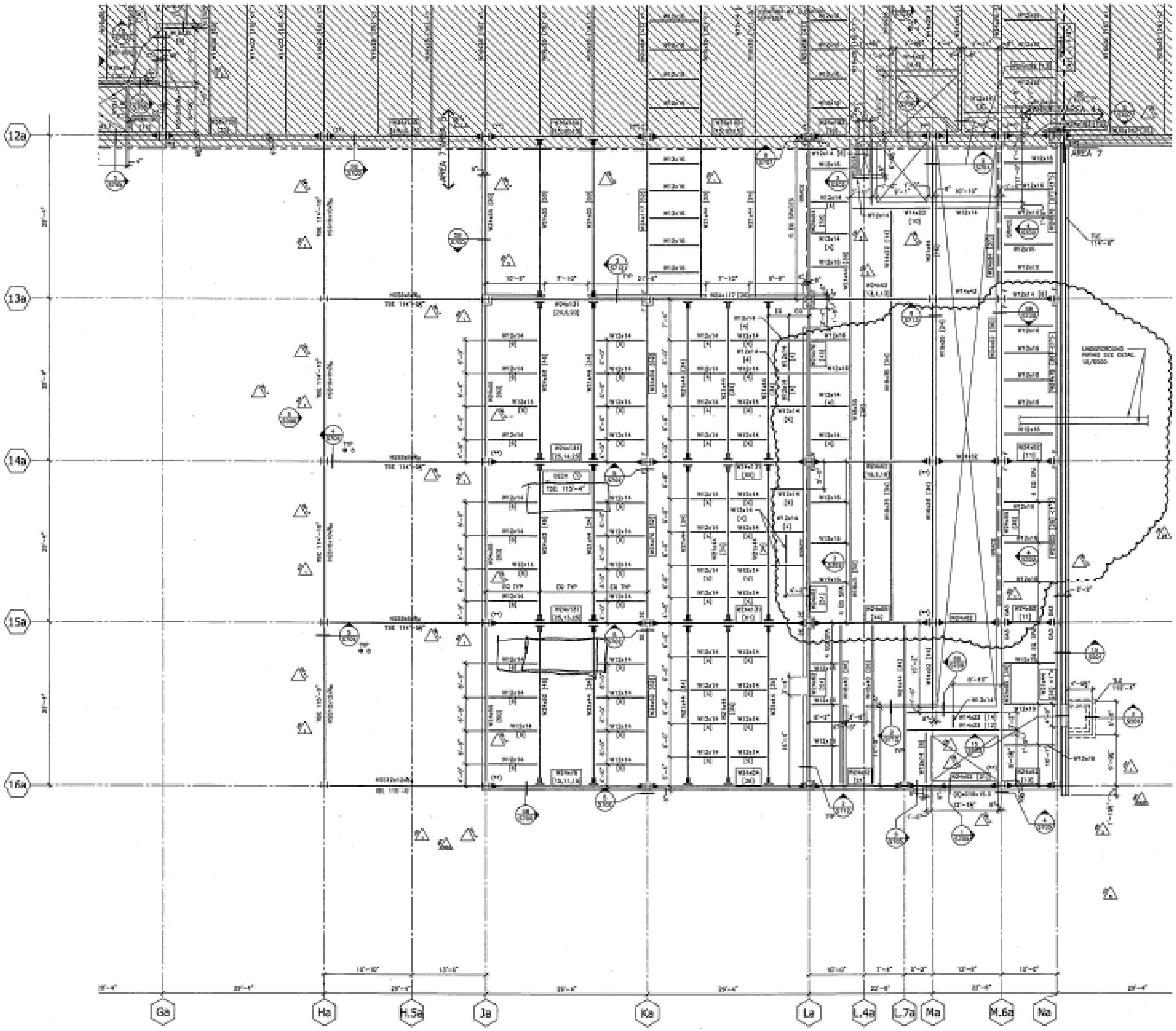
Level 1, Zone 4 Framing Plan



Level 1, Zone 5 Framing Plan



Level 1, Zone 6 Framing Plan



Level 1, Zone 7 Framing Plan

Appendix C

Aerial View from the East of the Building

Aerial View from the West of the Building

West Elevation

Overall Site Plan