St. Vincent Mercy Medical Center Heart Pavilion

Toledo, Ohio

Technical Report I

Kristen Marie Lechner | Advisor: Dr. Linda Hanagan | Structural Option | September 29, 2008

TABLE OF CONTENTS

EXECUTIVE SUMMARY

St. Vincent Mercy Medical Center Heart Pavilion is a four story hospital that provides diagnostics, surgery, and patient care. It was constructed for St. Vincent's Mercy Medical Center Campus, established in 1855, in downtown Toledo, Ohio.

The facility is approximately 144,000 square feet and reaches a height of 57'-5" above grade with a typical floor to floor height of approximately 14 feet. A typical interior bay is 30 feet by 35 feet and is comprised of composite steel with a concrete slab on deck. The lateral system utilizes steel moment frames due to limited floor space. Drilled caissons and spread footings make up the foundation system. The ground floor is a reinforced slab on grade with grade beams between caissons to transfer wall load into the foundation.

In this first technical report, the existing structural conditions of St. Vincent Mercy Medical Center Heart Pavilion are discussed through a detailed description of the foundation, floor system, columns, and lateral system. The floor framing plans and typical details are included within this report for a better understanding of how the structure works. In addition, summaries of building codes and material strengths used by the engineer of record are provided.

Spot checks of gravity loads were done within a typical bay for the composite floor, girder, and columns in an effort to check the validity of member sizes chosen. Sizes of the composite floor beams and girders were confirmed to be legitimate while column sizes seemed very large. However, since the building utilizes steel moment frames at every column, large moment must be resisted due to lateral loading. Therefore, it is seen why these column sizes were chosen by the engineer of record.

In an effort to further understand the structure, wind and seismic loads were analyzed using ASCE 7-05. The Analytical Procedure was used to determine wind loads for the structure in both directions. Wind in the North-South direction was found to control over wind in the East-West direction. This result makes sense as the building façade is longer in the North-South direction, thus required to resist greater wind pressure. Seismic loads were determined using The Equivalent Lateral Force Procedure. It was found that seismic forces control the design of this structure without considering torsion effects. The soil within the site is classified as Seismic Site Class E, "Soft Soil Profile", which means that the soil cannot take great shear force. This played a significant role in the determination of the controlling lateral force of the structure as the base shear value was considerably affected. Future technical reports will revisit this topic, taking torsion effects into account, in an effort to optimize the structural system.

INTRODUCTION: ST. VINCENT MERCY MEDICAL CENTER HEART PAVILION

St. Vincent's Heart Pavilion is one of the seven hospitals that comprise Mercy Health Partners. As Toledo's first and only facility for the treatment of vascular disease, St. Vincent's Heart Pavilion has become a staple within the community. St. Vincent's Mercy Medical Center Campus is now able to take a leadership role in providing education to its students as well as saving lives through the treatment of vascular disease.

Modernization is emphasized through the façade of St. Vincent Mercy Medical Center Heart Pavilion. As one approaches the building from the North, a beautiful curtain wall composed of curved aluminum and spandrel glass is seen, thus adding great verticality to the building. As the eye gazes along the façade, stone bands and brick veneer promote horizontal progression to an attractive vertical component of stairs wrapped in stone veneer and spandrel glass. The eye is then led to the pedestrian bridge, connecting the Heart Pavilion to a parking garage, which shows off its structure through exposed chevron bracing.

The structure of the Heart Pavilion is comprised of a composite steel floor system that utilizes steel moment frames to resist lateral forces. Drilled caissons and spread footings make up the foundation system. The ground floor is a reinforced slab on grade with grade beams between caissons to transfer wall load into the foundation.

The purpose of Technical Report I is to gain an understanding of how gravity and lateral loads are resisted by the existing structural system. Upon completion of this report, conclusions will be drawn on the validity of member sizes based on gravity loads. Future technical reports will include lateral forces with member spot checks.

STRUCTURAL DESCRIPTION

Foundations

The foundation system is made up of 80 drilled caissons and 6 spread footings that support the entrance lobby. The caisson caps are a uniform size of 4'x4'x3' thick. Between caissons are grade beams, varying in depth from 2' to 4' depending on the location, which transfer façade and wall load to the foundation system. The ground (main) floor rests on a 6" concrete slab reinforced with W/4x4-W4.0x4.0 welded wire fabric.

Floor System

St. Vincent Mercy Medical Center Heart Pavilion's typical floor system is made up of composite steel framing and normal weight concrete, creating a total floor thickness of 6½". Composite action is created by the use of 2" 20 gauge steel deck with 5½" long, ¾" diameter shear studs evenly spaced over the length of each beam. Even though a composite system is used, the girders are actually non-composite. In order to avoid coping of the infill beams, the girders are placed 2" higher than the beams on a typical floor and 1½" higher on the roof (see Figure 2 below). This system saved money and fabrication time which resulted in faster steel erection. In addition to these benefits, the deck connection to the girder automatically provides a pour-stop, making placement of the concrete easier.

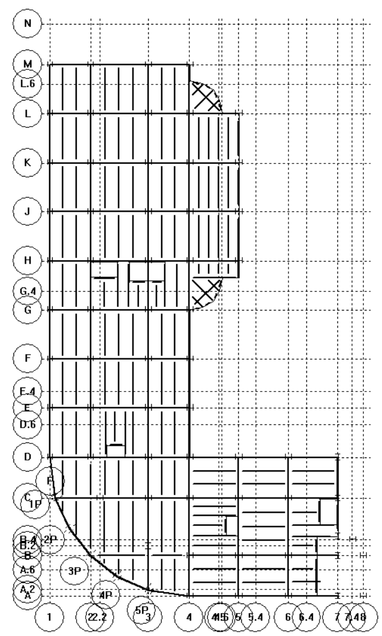


Figure 1: *Typical Floor Layout*

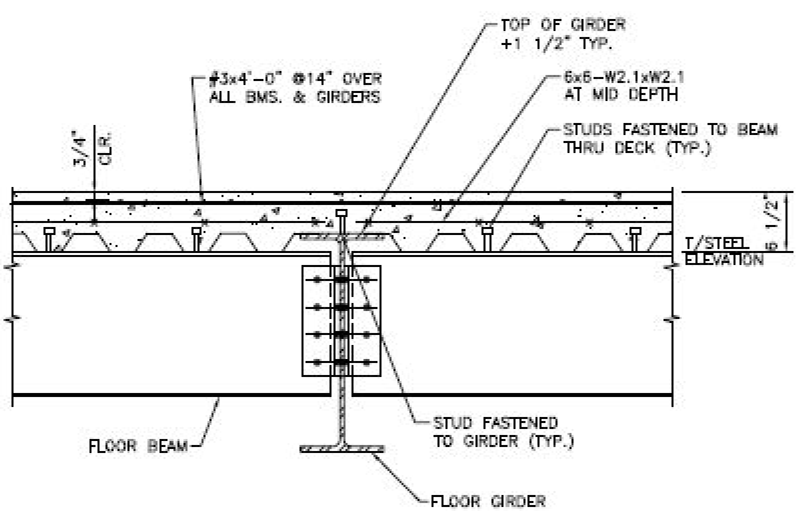


Figure 2: *Detail of Composite Floor System*

Columns

The columns used in St. Vincent Mercy Medical Center Heart Pavilion range from W10x119's to W12x210's, depending on their location within the building. While these sizes may seem large based purely on gravity, each column must resist induced moment since all columns are part of a moment connection. Pipe columns are used to support the roof for the main entrance lobby and the emergency vestibule canopy. All of the main building columns are spliced at the 2nd-3rd floor. Base plates range in thickness from 1" to 2 ¼" depending on which columns they are supporting. Each base plate utilizes a standard 4 bolt connection using either ¾" A325 or 1 ¼" A325 bolts.

Lateral System

At the time of design, braced frames were thought to be architecturally incompatible with this floor plan. As a result, steel moment frames were used for the lateral load resisting system at every column in both directions, as indicated in red in Figure 3. The moment frames are connected in two different fashions as seen in Figures 4 and 5 below. The beam to column web moment connection is comprised of flange plates that are fillet welded to the column web and flange. The beam flanges are full-penetration welded to these plates. The beam to column flange moment connection utilizes double angles connecting the beam to the column flange, where the column flange is then full penetration welded to the beam flange.

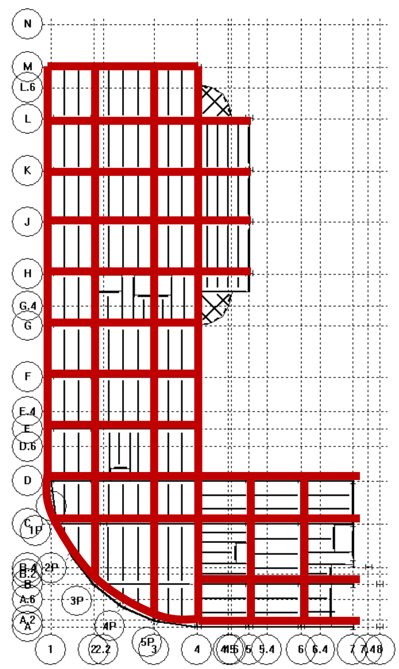


Figure 3: *Typical Floor Plan Indicating Lateral System*

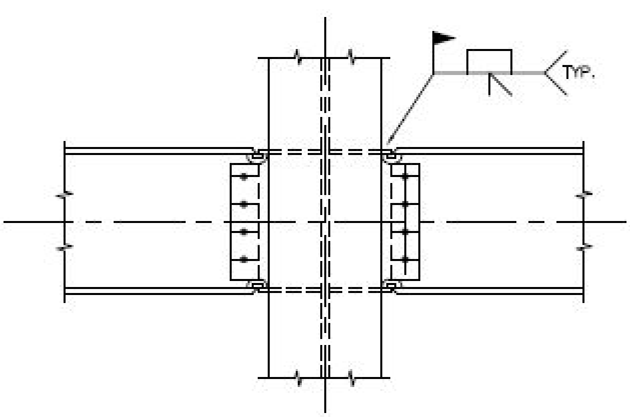


Figure 4: *Beam to Column Web Connection*

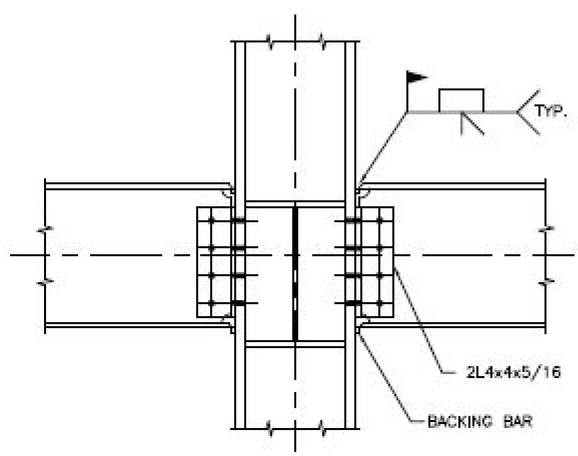


Figure 5: *Beam to Column Flange Connection*

CODE AND DESIGN REQUIREMENTS

Various references were used by the engineer of record in order to carry out the structural design of St. Vincent Mercy Medical Center Heart Pavilion:

- The 2002 International Building Code as amended by the State of Ohio
- The Building Code Requirements for Structural Concrete (ACI 318-02), American Concrete Institute
- Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings —Load and Resistance Factor Design, Third Edition, American Institute of Steel Construction
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-02), American Society of Civil Engineers

Deflection Criteria

Floor Deflection:

L/240 Total Load

L/360 Live Load

L/600 Curtain Wall Load

L/1666 Impact Load on Elevator Support Beams

Lateral Deflection:

H/500 Total Allowable Wind Drift

H/400 Total Story Wind Drift

$0.015h_{sx}$ Total Allowable Seismic Drift

MATERIALS

Multiple materials were used for the construction of St. Vincent Mercy Medical Center Heart Pavilion. The details of these materials are listed as follows:

Concrete

Foundations	f'_c = 3000 psi
Walls	f'_c = 3000 psi
Slabs	f'_c = 3500 psi
Grade Beams	f'_c = 4000 psi

Reinforcing Steel

Reinforcing Bar	A.S.T.M. A-615 GRADE 60
Tie Wire	A.S.T.M. A-82
Welded Wire Fabric	A.S.T.M. A-185

Structural Steel

Wide Flange	A.S.T.M. A992
Angle, Plate, Channel	A.S.T.M. A36
Connection Bolts	A.S.T.M. A325
Anchor Bolts	A.S.T.M. A307 OR A36
Square/Rectangle (HSS)	A.S.T.M. A500, GRADE B
Round (HSS)	A.S.T.M. A500, GRADE B

Metal Deck and Shear Studs

Composite Floor	2” 20. GA.
Roof Deck	1 ½” 22 GA.
Shear Studs	¾” x 5 ½”

GRAVITY LOADS

Loading conditions are a very important consideration for the design of any structure. The dead load conditions assumed by the engineer of record at the time of design and live load conditions obtained from ASCE 7-02 are provided for reference:

Dead Loads (Assumed Construction Dead Loads)

Concrete	150 PCF
Steel	490 PCF
Partitions	20 PSF
MEP	10 PSF
Windows & Framing	10 PSF
Finishes & Miscellaneous	5 PSF
Roof	20 PSF

Live Loads (Obtained from ASCE 7-05)

First Floor Corridors	100 PSF
Lobbies	100 PSF
Loading Dock	100 PSF
Penthouse Floor	100 PSF
Corridors above First Floor	80 PSF
Patient Rooms	60 PSF
Operating rooms	60 PSF
Bridge Floor	60 PSF
Roof	20 PSF

LATERAL LOADS

The following section addresses wind and seismic analysis using ASCE 7-05. For a detailed summary, please refer to Appendix B and C. Figure 7 below shows simplified assumptions made within this preliminary analysis of lateral loading.

Wind Analysis

Design pressures were found using the analytical method described in section 6.5 of ASCE 7-05. Please refer to appendix B for constants and equations used for the execution of this procedure. A few assumptions were made for the calculation of B (horizontal dimension of building measured normal to direction of wind) and L (horizontal dimension of building measured parallel to direction of wind). First, the protruding loading dock and pedestrian bridge were neglected for their contribution to wind loads. Second, the curtain wall on the north side of the building is taken to be a rectangular shape. In addition, wind effects on the roof and canopy entrance were neglected. These assumptions were made in order to make this analysis a simpler procedure. A more detailed and accurate analysis of lateral loads will be studied in a future technical report.

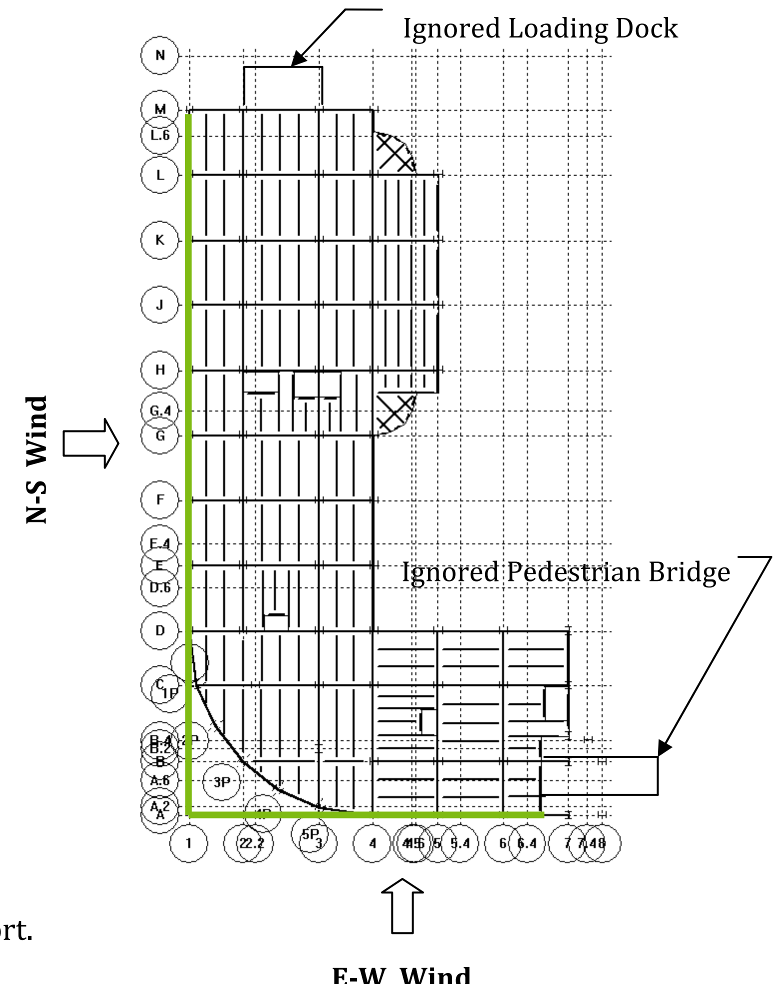


Figure 7: *Typical Floor Plan*

The approximate fundamental frequency of the building was determined using the commentary within ASCE 7-05. It was determined that the building is flexible in nature. This conclusion makes sense, as moment frames are naturally more flexible than braced frames. Due to some inconsistency in floor to floor heights, the pressure distribution is not a perfect curve. However, linear progression is seen by the wind design tables and the pressure diagrams located following pages. Loading diagrams for both directions are also provided for reference as seen in Figures 12 and 13.

Wind Design Parameters

The following tables are provided for a summary of wind design pressures and loads found per ASCE 7-05.

Floor Height (ft)	Level	Total Height (ft)	K_z	q_z	Wind Pressures (psf)					
					N-S Windward	N-S Leeward	N-S Side Wall	E-W Windward	E-W Leeward	E-W Side Wall
14.40	Roof	57.40	0.84	17.09	13.89	-9.83	-12.54	14.31	-7.54	-12.91
14.00	3	43.00	0.78	15.74	13.03	-9.83	-12.54	13.42	-7.54	-12.91
14.00	2	29.00	0.69	14.06	11.97	-9.83	-12.54	12.32	-7.54	-12.91
15.00	1	15.00	0.57	11.65	10.44	-9.83	-12.54	10.73	-7.54	-12.91

Figure 8: *Distribution of Windward and Leeward Pressures*

Level	Wind Design					
	Load (k)		Shear (k)		Moment (ft-k)	
	N-S	E-W	N-S	E-W	N-S	E-W
Roof	57	28	0	0	3284	1580
3	111	53	57	28	4764	2287
2	105	50	168	81	3037	1450
1	102	48	273	131	1533	726
Total	375	179	375	179	12618	6043

Figure 9: *Total Base Shear from Windward and Leeward Pressures*

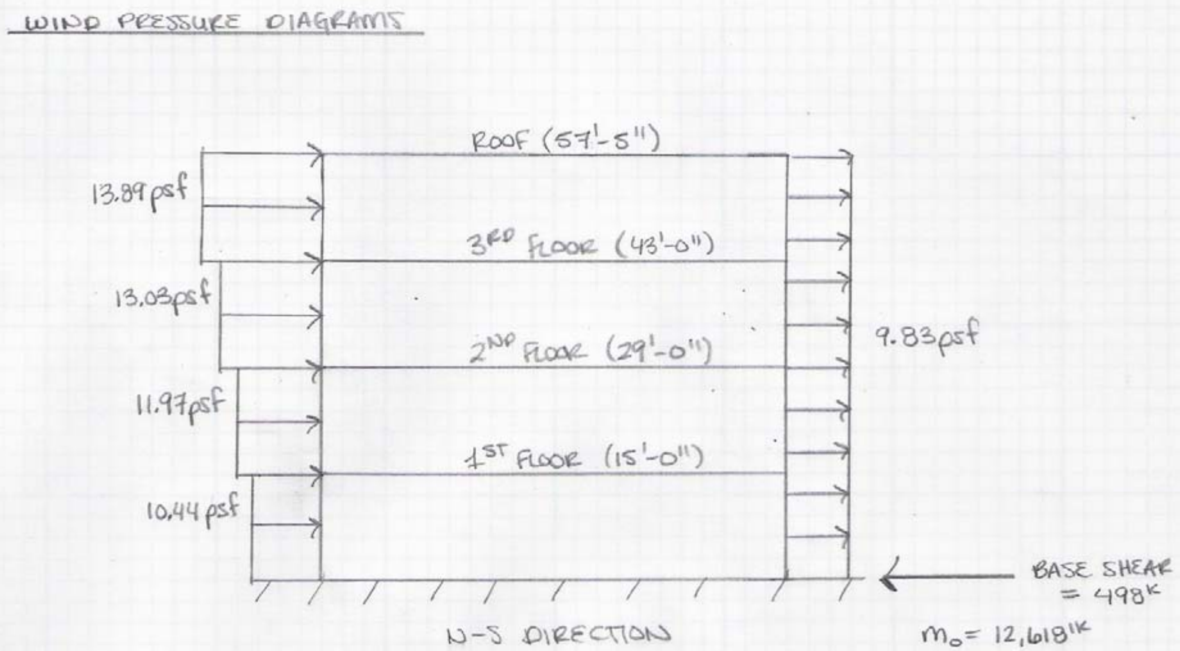


Figure 10: *Pressure Distribution along N-S Face*

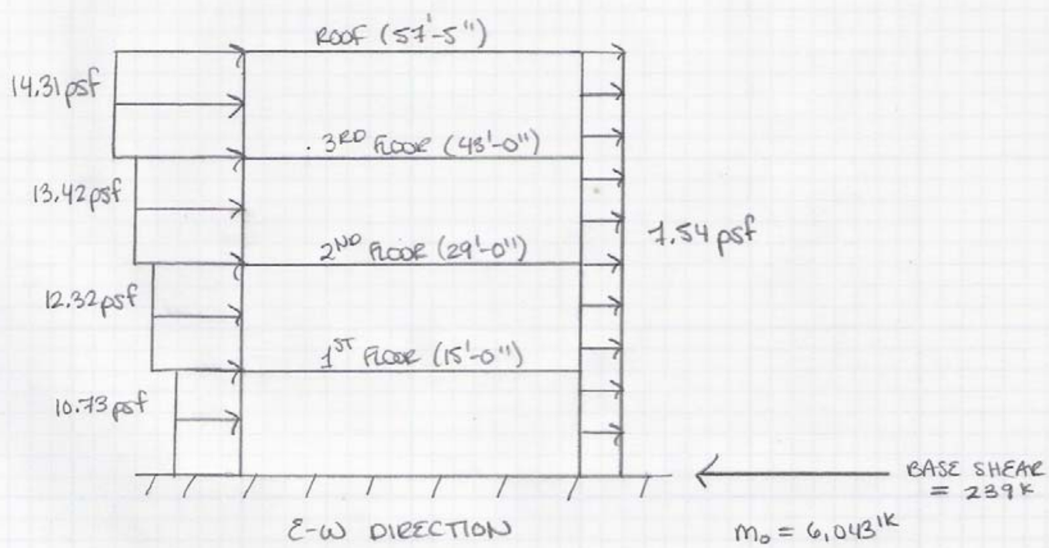


Figure 11: *Pressure Distribution along E-W Face*

WIND STORY LOAD DIAGRAM

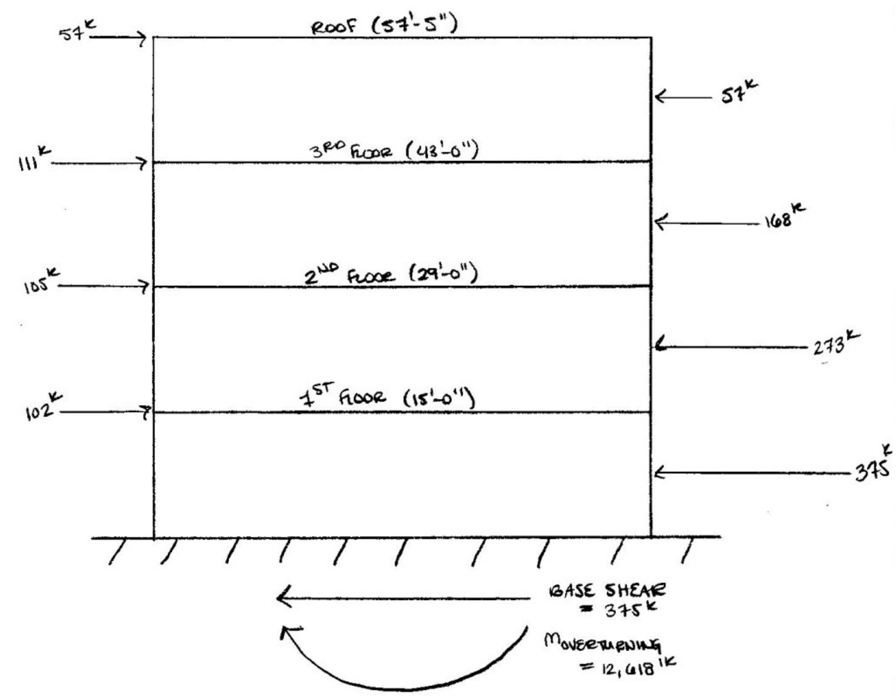


Figure 12: *Load Distribution along N-S Face*

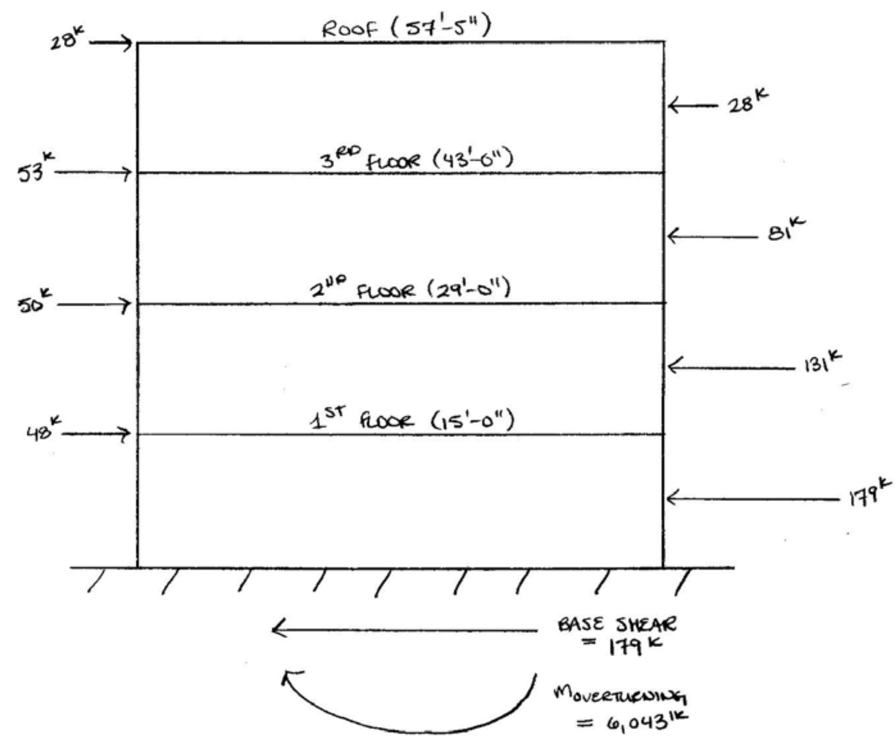


Figure 13: *Load Distribution along E-W Face*

Seismic Analysis

Seismic loads were analyzed using chapters 11 and 12 of ASCE 7-05. Please refer to Appendix C for detailed calculations used to obtain building weight as well as base shear and overturning moment distribution for each floor as seen in Figure 14 below. According to the engineer of record, seismic analysis was found to control this design.

Base Shear and Overturning Moment Distribution							
Story	h_x (ft)	Story Weight (k)	$h_x^k W_x$	C_{vx}	$F_x = C_{vx}V$	V_x (k)	M_x (ft-k)
Roof	57.4	1132	100432	0.219	241	241	13817
3	43	2824	181955	0.396	436	677	29103
2	29	2751	114571	0.250	275	951	27591
1	15	3100	62203	0.135	149	1100	16507
Main	0	2236	0	0.000	0	1100	0
Total	57.4	12043	459162	1.000	**1100**		**87017**
Base Shear =	1100	k					

Figure 14: *Base Shear and Overturning Moment Distribution*

The base shear value for this building seems extremely high at first glance, however, the nature of the soil within the site had a significant impact on the determination of this value. Based on field and laboratory test data within the geotechnical report for the site, it was determined that more than 10 feet of soils located 12 to 40 feet below existing grade has an un-drained shear strength of less than 500 psf. As a result, the site is characterized by the Ohio Building Code as Seismic Site Class E, "Soft Soil Profile". This means that the soil is very weak and cannot take great shear force. If the soil was classified as Seismic Site Class B, the base shear would be reduced by approximately 60%. Without considering torsion effects, this reduction leads to a wind-controlled design.

Due to the fact that the soil is very soft in nature, seismic forces were found to control the design of this building without considering torsion effects. Future technical reports will revisit this topic, taking torsion effects into account, in an effort to optimize the structural system.

SPOT CHECKS

A typical bay on the second floor was analyzed in order to confirm the engineer of record's design methods. Please refer to Appendix E for detailed calculations of the following descriptions.

The first spot check performed was an evaluation of a composite beam within an interior bay. The calculations show that the typical W16x26 beam can carry the bending moment created by placing the concrete during construction. Once the concrete is placed and the two materials are working together as a composite system, the moment capacity is increased and the system can then carry the factored moment resulting from applied dead and live loads.

Next, a girder was examined to ensure that the member can transfer the loads from the composite beams to the columns. It was confirmed within the calculations that a W24x55, the typical member chosen, can carry the induced moment created by the beams framing in on both sides.

Dead loads applied to the columns were computed using the floor weights from the seismic calculations, taking into account the influence area. A summary of the accumulated load on the column at each floor is located in Appendix E. Live loads were applied in accordance with ASCE 7-05. It was assumed that the effective length, KL, of each column was equal to the floor to floor height of the particular column. After performing the compression check for the column on the fourth floor using the flexural buckling equations in Chapter E of the AISC Steel Manual, the Available Strength in Axial Compression Table, Table 4-1, was used as it is based upon the same method.

Upon completion of these calculations, it was concluded that the capacity of the structure will carry the loads applied.

CONCLUSION

Technical Report I examines existing structural conditions of St. Vincent Mercy Medical Center Heart Pavilion in an attempt to better understand the design decisions made at the onset of the project. A detailed discussion of the structural system and floor framing plans are included within this report for a better understanding of how the structure works.

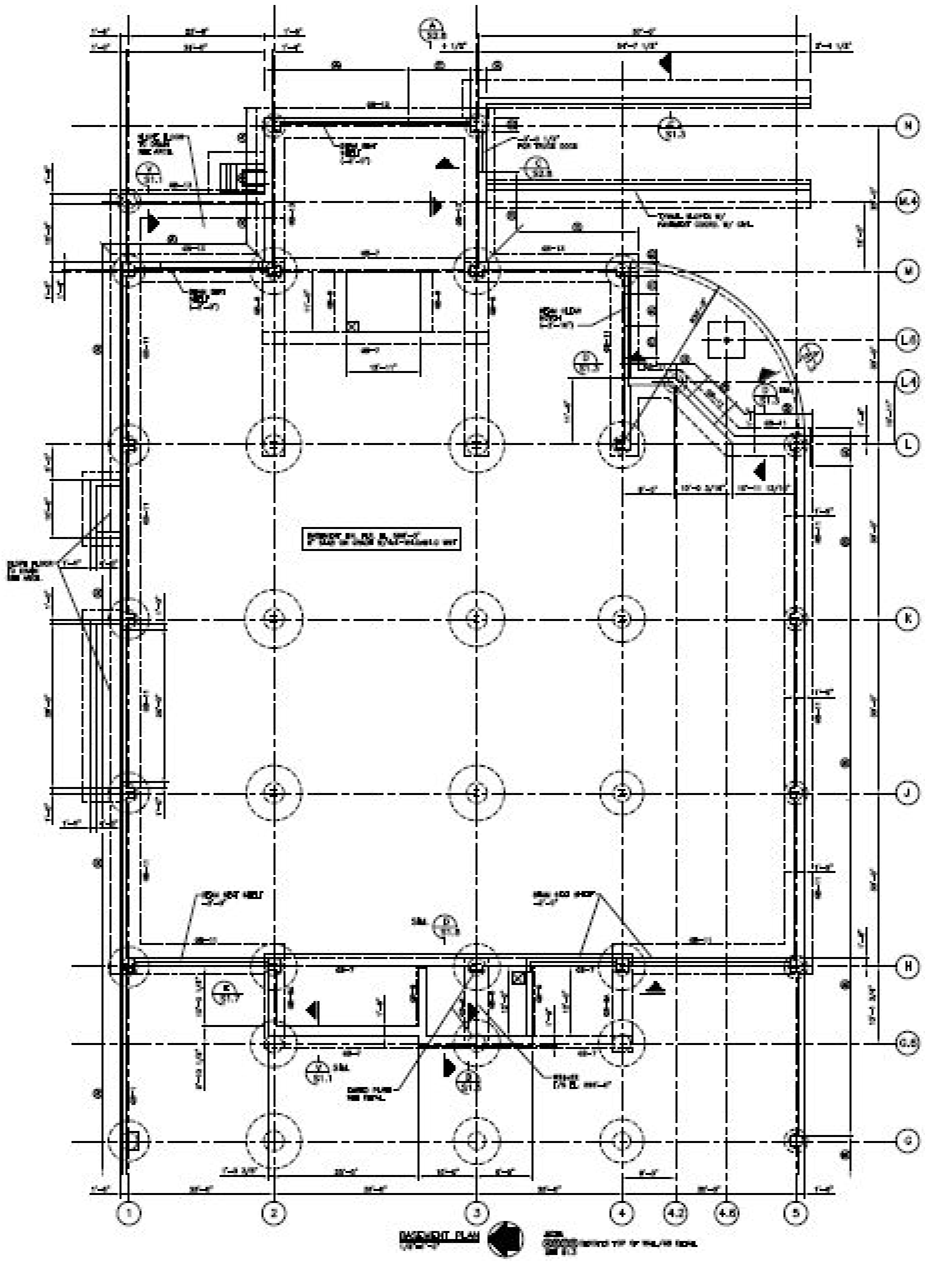
Several calculations were done on individual members in order to verify the structural engineer's design. A typical bay within the building was solely analyzed for exposure to gravity loads. Within this typical bay, the moment capacity of an infill beam was checked due to placement of the concrete during construction. After confirming that the beam could take the applied moment, the composite system was checked for moment capacity due to live and dead loads applied to the floor. The composite system was then checked for live load deflection and it was concluded that the floor system meets serviceability criteria. A spot check was performed on a girder to ensure sufficient load transfer from the beams to columns. Results from the spot concluded that the girder is adequate to transfer the design forces. Finally, column spot checks were executed. These checks were done using the flexural buckling equations found in Chapter E of the AISC Steel Manual. Calculations indicated that the columns were not designed on gravity alone, leading one to believe that they were sized to handle the moments generated by lateral loading.

In an effort to further understand the structure, as well as the reason large column sizes were chosen, an analysis of wind and seismic forces was prepared. Upon comparing the base shear values obtained from wind and seismic calculations, it was determined that seismic forces control the design of this structure without considering torsion effects. The main lateral force resisting system is made up of steel moment frames at every column within the building, meaning that every column is required to resist a significant amount of moment due to lateral forces. This gives justification for the column sizes chosen by the engineer of record and future technical reports will further verify the column sizes.

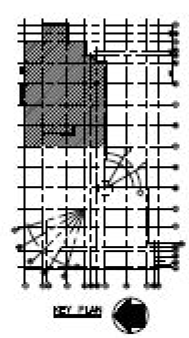
All design values used and procedures carried out were done in accordance with applicable codes. Please refer to the appendices for further review of detailed notes, figures, or tables regarding this matter. Questions should be directed to Kristen M. Lechner via email: kml5016@psu.edu.

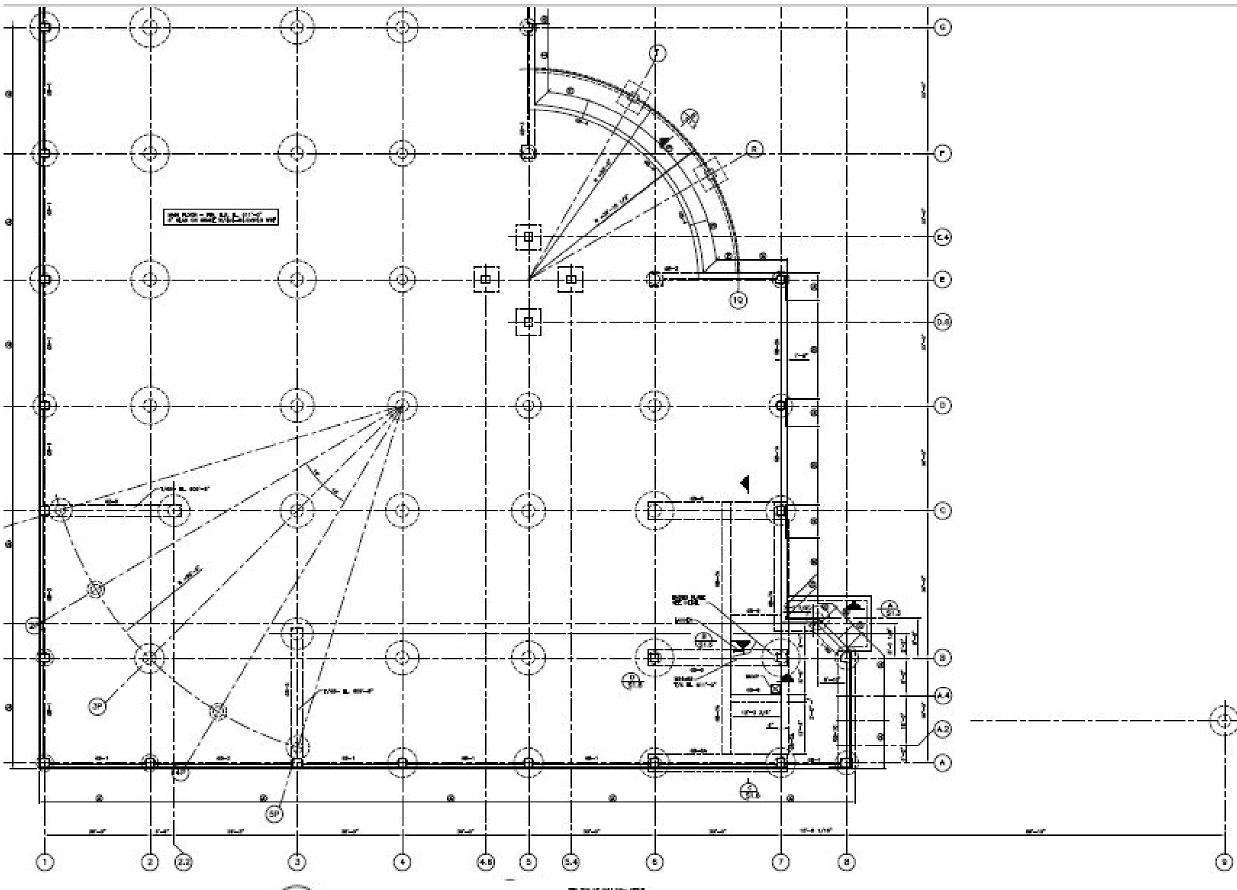
APPENDIX A: BUILDING LAYOUT

Photos courtesy of Ruby + Associates

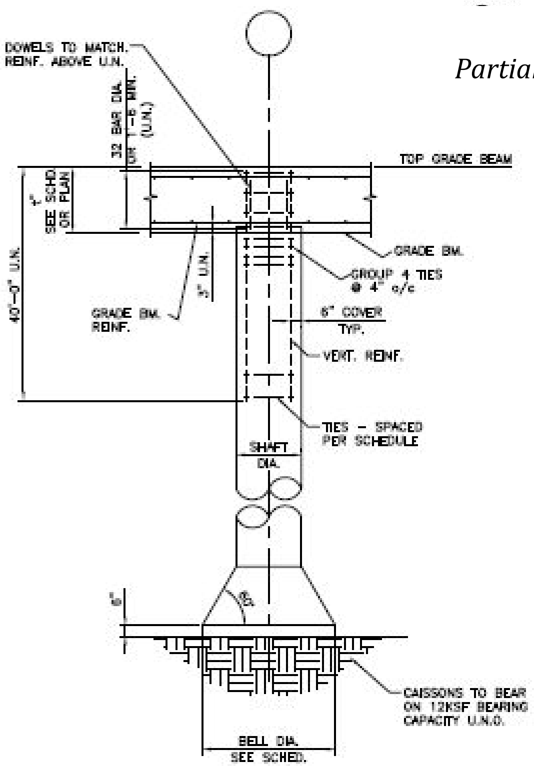


Partial Foundation Plan

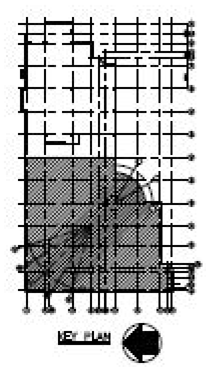


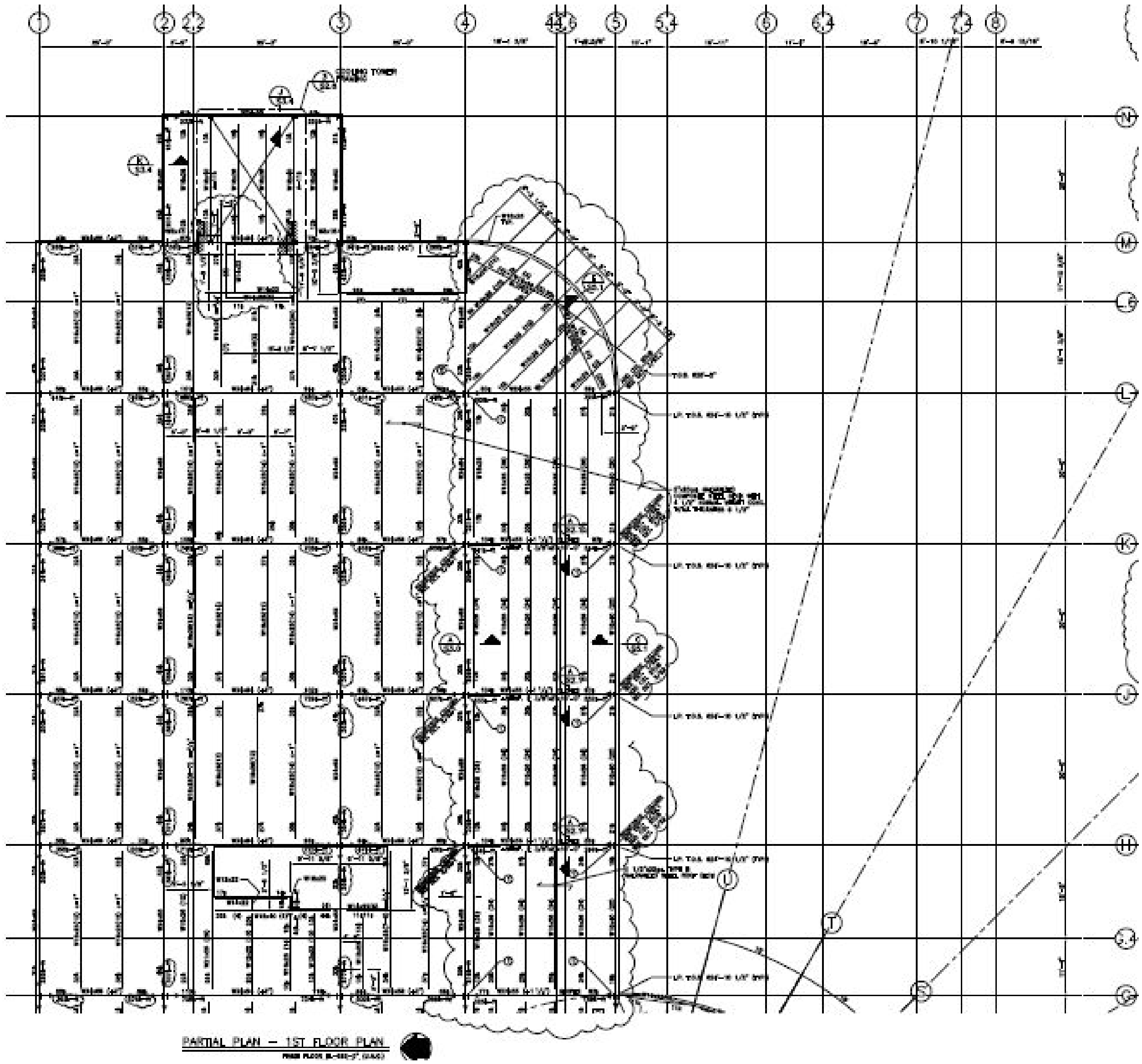


Partial Foundation Plan



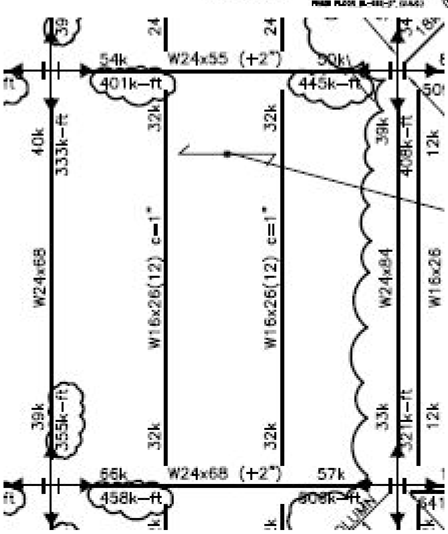
Caisson Detail at Interior Grade Beam



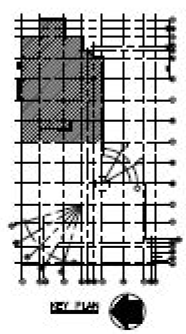


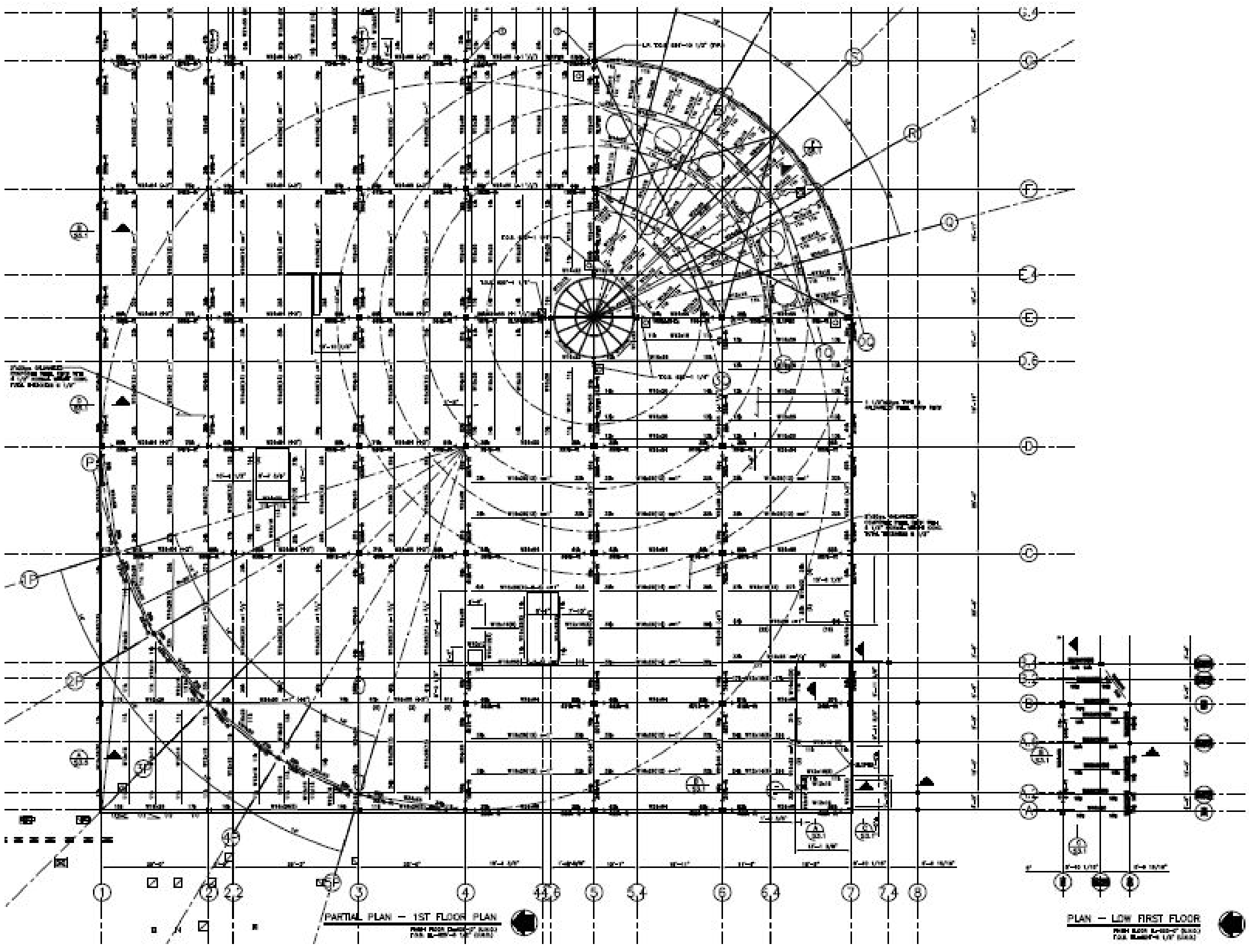
Partial First Floor Plan

(Also Typical for Main Floor)



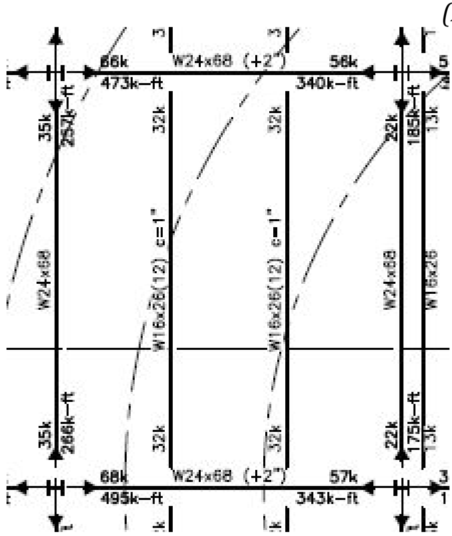
Enlarged Typical Bay



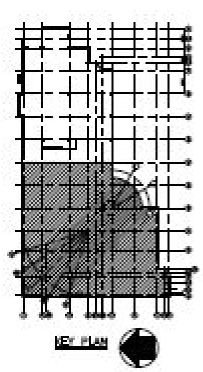


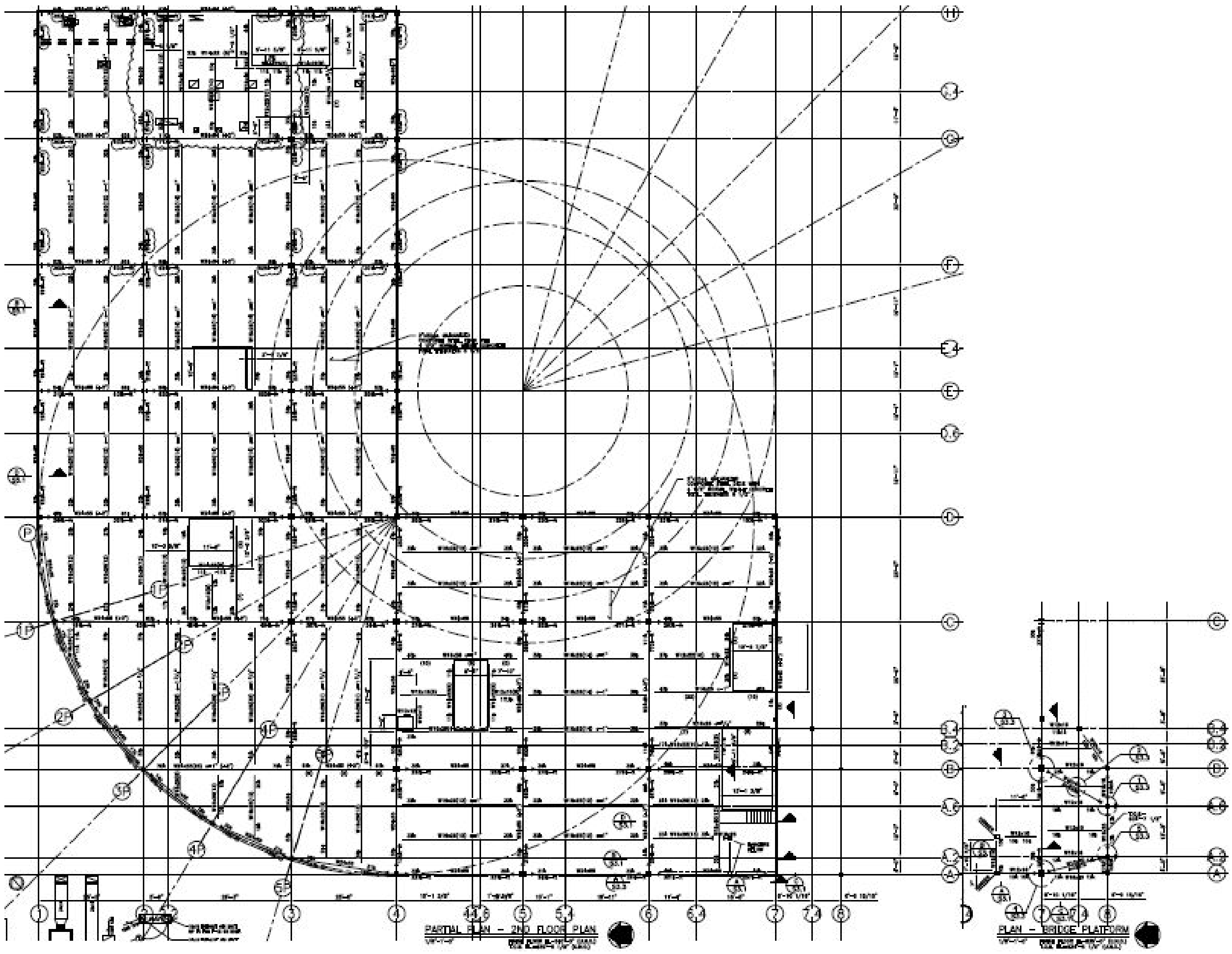
Partial First Floor Plan

(Also Typical for Main Floor)

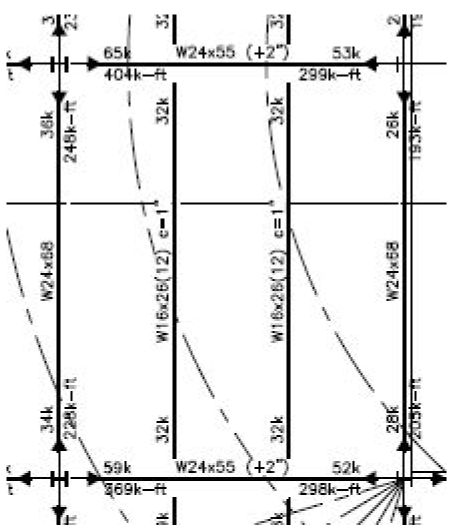


Enlarged Typical Bay

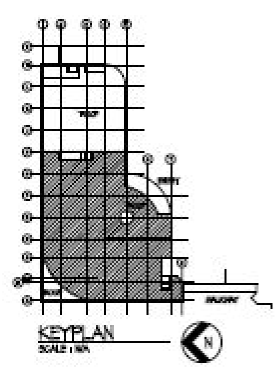


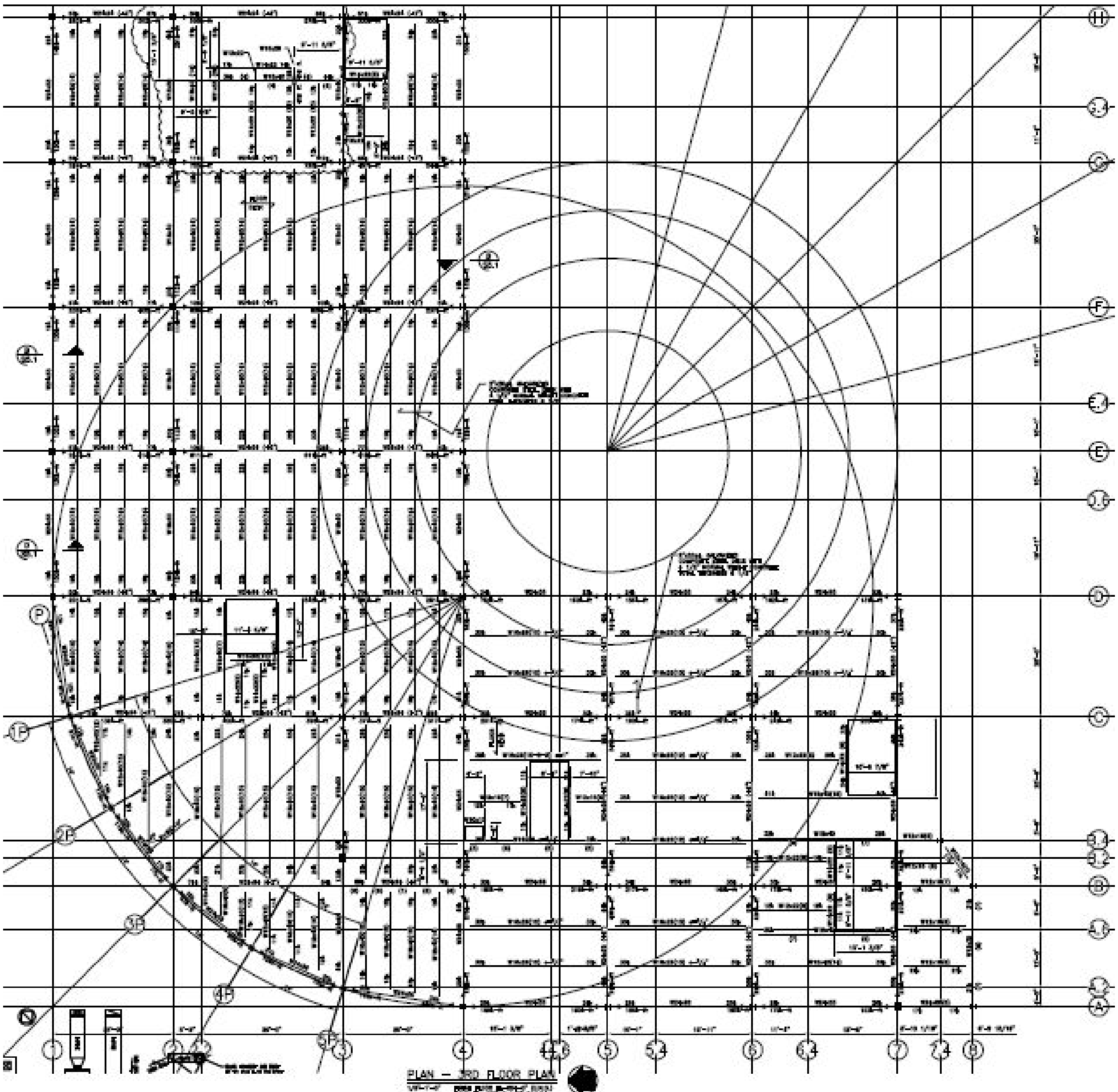


Second Floor Plan

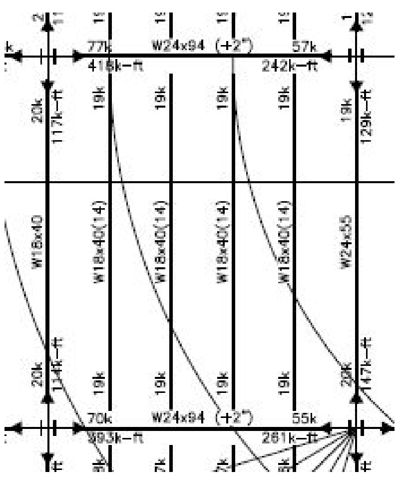


Enlarged Typical Bay

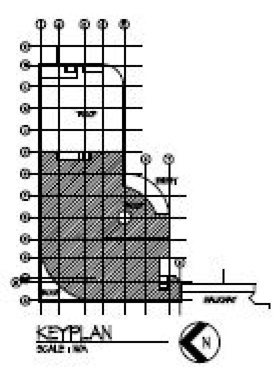


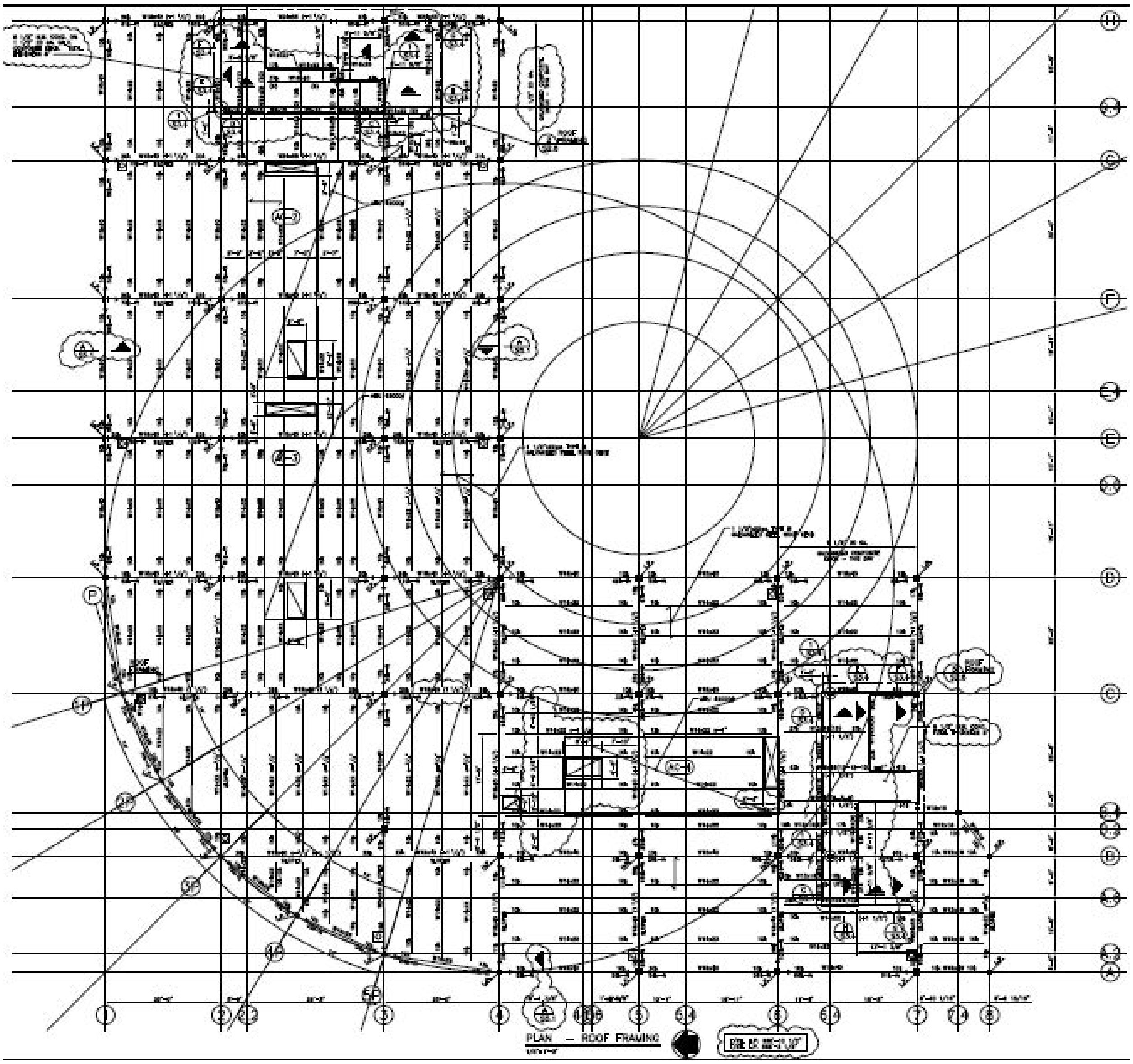


Third Floor Plan

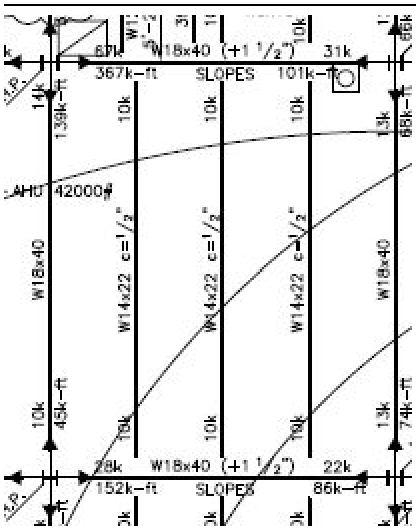


Enlarged Typical Bay

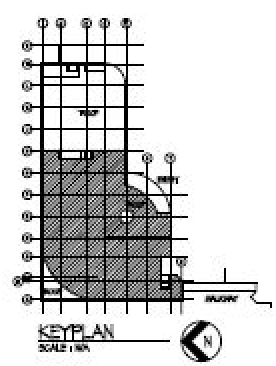




Roof Plan



Enlarged Typical Bay



APPENDIX B: WIND ANALYSIS

Photo courtesy of www.wbdg.org

Main Wind Force Resisting System

Building Information	
Number of Floors	4
Building Height (ft)	57.4
N-S Building Length (ft)	335
E-W Building Length (ft)	175
L/B in N-S Direction	1.91
L/B in E-W Direction	0.52

Building Location Factors	
Basic Wind Speed (V) mph	90
Exposure Category	B
Importance Factor (I)	1.15
Wind Directionality Factor (K_d)	0.85
Topographic Factor (K_{zt})	1.0

Variables to Obtain Gust Factor		
Variable	Wind Direction	
	N-S	E-W
n_1 (Hz)	0.869	0.869
Stiffness	Flexible	Flexible
B	335	175
L	175	335
h	57.4	57.4
g_q	3.4	3.4
g_v	3.4	3.4
g_r	4.16	4.16
z_{BAR}	34	34
ε_{BAR}	0.333	0.333
L_{BAR}	320	320
b_{BAR}	0.45	0.45
α_{BAR}	0.25	0.25
Iz_{BAR}	0.298	0.298
Lz_{BAR}	325	325
Q	0.765	0.814
Vz_{BAR}	60.0	60.0
N_1	4.7	4.7
n_h	3.82	3.82
n_B	22.32	11.66
n_L	39.03	74.71
R_h	0.227	0.227
R_B	0.044	0.082
R_L	0.025	0.013
R_n	0.0528	0.0528
R	0.0755	0.1028
G_f	**0.791**	**0.822**

Main Wind Force Resisting System

Floor Height (ft)	Level	Total Height (ft)	K_z	q_z	Wind Pressures (psf)					
					N-S Windward	N-S Leeward	N-S Side Wall	E-W Windward	E-W Leeward	E-W Side Wall
14.40	Roof	57.40	0.84	17.09	13.89	-9.83	-12.54	14.31	-7.54	-12.91
14.00	3	43.00	0.78	15.74	13.03	-9.83	-12.54	13.42	-7.54	-12.91
14.00	2	29.00	0.69	14.06	11.97	-9.83	-12.54	12.32	-7.54	-12.91
15.00	1	15.00	0.57	11.65	10.44	-9.83	-12.54	10.73	-7.54	-12.91

Distribution of Windward and Leeward Pressures

Level	Wind Design					
	Load (k)		Shear (k)		Moment (ft-k)	
	N-S	E-W	N-S	E-W	N-S	E-W
Roof	57	28	0	0	3284	1580
3	111	53	57	28	4764	2287
2	105	50	168	81	3037	1450
1	102	48	273	131	1533	726
Total	375	179	498	239	12618	6043

Total Base Shear from Windward and Leeward Pressures

WIND DESIGN

USING ANALYTICAL PROCEDURE —

$V = 90$ MPH (FIGURE 6-1)

$K_d = 0.85$ (TABLE 6-4)

$I = 1.15$ (TABLE 6-1)

EXPOSURE B (REFERENCE § 6.5.6)

$K_{zt} = 1.0$ (FIGURE 6-4)

VELOCITY PRESSURE EXPOSURE COEFFICIENT, K_h (TABLE 6-3)

BUILDING HEIGHT = 57'5"

HEIGHT	K_h
50'	0.81
57.4'	0.84
60'	0.85

VELOCITY PRESSURE (q_p)

$$q_p = 0.00256\, K_h K_{zt} K_d V^2 I$$

$$q_p = 0.00256(0.84)(1.0)(0.85)(90)^2(1.15) = 17.0 \text{ psf}$$

COMBINED NET PRESSURE COEFFICIENT (GC_{pn}) (REFERENCE § 6.5.12.2.4)

$GC_{pn} = 1.5$ (WINDWARD)

$GC_{pn} = -1.0$ (LEEWARD)

COMBINED NET DESIGN PRESSURE ON PARAPET, p_p

$$p_p = q_p\, GC_{pn}$$

$p_p = 17.0(1.5) = 25.5$ psf (WINDWARD)

$p_p = 17.0(-1.0) = -17.0$ psf (LEEWARD)

FORCES ON PARAPETS

HT. OF PARAPET = 4'-1½"

$F = 25.5 \text{ psf}(4.125') = 105.2$ plf (WINDWARD)

$F = +17.0 \text{ psf}(4.125') = 70.1$ plf (LEEWARD)

APPROXIMATE FUNDAMENTAL FREQUENCY, n_1 (REFERENCE C6.5.8 IN COMMENTARY OF ASCE 7-05)

FOR STEEL MOMENT RESISTING FRAMES,

$$n_1 = \frac{22.1}{H^{0.8}} = \frac{22.1}{(57.4)^{0.8}} = 0.869 \text{ Hz} < 1.0$$

∴ FLEXIBLE STRUCTURE

OBTAIN GUST EFFECT FACTOR —

$g_a = g_v = 3.4$

$$g_r = \sqrt{2\ln(3600\eta_1)} + \frac{0.577}{\sqrt{2\ln(3600\eta_1)}}$$

$$g_r = \sqrt{2\ln(3600(0.869))} + \frac{0.577}{\sqrt{2\ln(3600(0.869))}} = 4.156$$

$\bar{z} = 0.6h = 0.6(57.4') = 34.4'$; $z_{min} = 30'$

$34.4' > 30'$ $\therefore$ OK

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.3\left(\frac{33}{34.4}\right)^{1/6} = 0.298$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{34.4}{33}\right)^{1/3} = 324.5$$

$$Q = \sqrt{\frac{1}{\left(1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}\right)}}$$

$Q = 0.765$ FOR N-S $(B = 335')$

$Q = 0.814$ FOR E-W $(B = 175')$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = 0.45\left(\frac{34.4}{33}\right)^{1/4}(90)\left(\frac{88}{60}\right) = 60.0$$

$$N_1 = \frac{\eta_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{0.869(324.5)}{60.0} = 4.70$$

$$R_n = \frac{7.47N_1}{(1+10.3N_1)^{5/3}} = \frac{7.47(4.70)}{(1+10.3(4.7))^{5/3}} = 0.053$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta}) = \frac{1}{3.82} - \frac{1}{2(3.82)^2}(1-e^{-2(3.82)}) = 0.228$$

$$\eta = 4.6\eta_1 h/\bar{V}_{\bar{z}} = 4.6(0.869)(57.4)/60.0 = 3.82$$

$$R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta})$$

$$\eta = 4.6\eta_1 B/\bar{V}_{\bar{z}} = 4.6(0.869)(335)/60 = 22.3 \text{ [N-S DIR.]}$$

$$= 4.6(0.869)(175)/60 = 11.7 \text{ [E-W DIR.]}$$

$$R_B = \frac{1}{22.3} - \frac{1}{2(22.3)^2}(1-e^{-2(22.3)}) = 0.044 \text{ [N-S DIR.]}$$

$$R_B = \frac{1}{11.7} - \frac{1}{2(11.7)^2}(1-e^{-2(11.7)}) = 0.082 \text{ [E-W DIR.]}$$

$$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right)$$

$$\eta = 15.4\,n_1\,L/\bar{V}_{\bar{z}} = 15.4(0.869)(175)/60 = 39.0 \quad [N\text{-}S]$$

$$= 15.4(0.869)(335)/60 = 74.7 \quad [E\text{-}W]$$

$$R_L = \frac{1}{39} - \frac{1}{2(39)^2}\left(1-e^{-2(39)}\right) = 0.025 \quad [N\text{-}S]$$

$$R_L = \frac{1}{74.7} - \frac{1}{2(74.7)}\left(1-e^{-2(74.7)}\right) = 0.013 \quad [E\text{-}W]$$

$$R = \sqrt{\frac{1}{\beta}(R_n R_h R_B)(0.53+0.47R_L)} \quad \text{WHERE } \beta = 0.05$$

$$R = \sqrt{\frac{1}{0.05}\left[(0.053)(0.228)(0.044)(0.53+0.47(0.025))\right]} = 0.076 \quad [N\text{-}S]$$

$$R = \sqrt{\frac{1}{0.05}\left[(0.053)(0.228)(0.082)(0.53+0.47(0.013))\right]} = 0.103 \quad [E\text{-}W]$$

$$G_f = 0.925\left(\frac{1+1.7\,I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_r R^2}}{1+1.7\,g_v I_{\bar{z}}}\right)$$

$$G_f = 0.925\left(\frac{1+1.7(0.298)\sqrt{3.4^2(0.765)^2+4.156^2(0.076)^2}}{1+1.7(3.4)(0.298)}\right) = 0.791 \quad [N\text{-}S]$$

$$G_f = 0.925\left(\frac{1+1.7(0.298)\sqrt{3.4^2(0.814)^2+4.156^2(0.103)^2}}{1+1.7(3.4)(0.298)}\right) = 0.822 \quad [E\text{-}W]$$

VELOCITY PRESSURE, q_z

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

$$q_z = 0.00256(0.84)(1.0)(0.85)(90)^2(1.15) = 17.0 \text{ psf}$$

FOR WIND IN E-W DIRECTION (REFERENCE FIGURE 6-6)

WINDWARD WALL — $C_p = 0.8$

LEEWARD WALL — $L/B = 335/175 = 1.914$

$C_p = -0.318$ (BY INTERPOLATION)

SIDE WALL — $C_p = -0.7$

FOR WIND IN N-S DIRECTION (REFERENCE FIGURE 6-6)

WINDWARD WALL — $C_p = 0.8$

LEEWARD WALL — $L/B = 175/335 = 0.522 \quad \therefore C_p = -0.5$

SIDE WALL — $C_p = -0.7$

NOT INCLUDING UPLIFT ON ROOF SINCE ROOF FRAMING MADE UP OF W-SHAPES

$q_i = q_h = q_z$ FOR TOP OF BLDG $= 17.0$ psf

INTERNAL PRESSURE COEFFICIENT

$GC_{pi} = \pm 0.18$

DESIGN WIND PRESSURES — $p_z + p_h$ (EQ. 6-17)

WINDWARD WALLS:

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

$$p_z = (0.791)(0.8) q_z \pm 17.0(0.18) = (0.633 q_z \pm 3.06)\text{ psf} \quad [N-S]$$

$$p_z = (0.822)(0.8) q_z \pm 17.0(0.18) = (0.658 q_z \pm 3.06)\text{ psf} \quad [E-W]$$

LEEWARD WALLS + SIDE WALLS:

$$p_z = q_h G C_p - q_h (GC_{pi})$$

$$p_z = (17.0)(0.791) C_p \pm 17.0(0.18) = (13.4 C_p \pm 3.06)\text{ psf} \quad [N-S]$$

$$p_z = (17.0)(0.822) C_p \pm 17.0(0.18) = (14.0 C_p \pm 3.06)\text{ psf} \quad [E-W]$$

-End of Section-

APPENDIX C: SEISMIC ANALYSIS

Photo courtesy of
www.science.howstuffworks.com

Seismic Force Resisting System

Occupancy Category	IV
Importance Factor (I)	1.5
S_s	0.170
S_1	0.056
Site Class	E
Total Building Height (ft)	57.4
T_a	0.715
T_L	12
Frequency (Hz)	1.40
Structural Behavior	Rigid Diaphragm
Total Weight (k)	12043

S_{ms}	0.425
S_{m1}	0.196
S_{ds}	0.283
S_{d1}	0.131
SDC	B
R	3.0
C_s	0.091
k	1.11
Base Shear (k)	1100

Base Shear and Overturning Moment Distribution

Story	h_x (ft)	Story Weight (k)	$h_x^k W_x$	C_{vx}	$F_x = C_{vx}V$	V_x (k)	M_x (ft-k)
Roof	57.4	1132	100432	0.219	241	241	13817
3	43	2824	181955	0.396	436	677	29103
2	29	2751	114571	0.250	275	951	27591
1	15	3100	62203	0.135	149	1100	16507
Main	0	2236	0	0.000	0	1100	0
Total	57.4	12043	459162	1.000	**1100**		**87017**
Base Shear =	1100	k					

Seismic Force Resisting System: Floor Weights

Main Floor					
	Approx. Area =		47,410	SF	
	Floor to Floor Ht. =		15	ft	
Walls:			**Superimposed:**		
Perimeter =	1220	ft	Partitions =	20	psf
Height =	8	ft	MEP =	10	psf
Unit Wt. =	20	psf	Finishes =	5	psf
Weight =	**183**	**k**	**Weight =**	**1659**	**k**
Slab:					
		Thk. =	4.5	in	
		Unit Wt. =	150	pcf	
-Do not include weight of slab on main floor-					
Columns:					
Shape	Quantity	Weight (lb/ft)	Column Height (ft)	Total Weight (k)	
W10x112	7	112	7.5	5.88	
W12x40	11	40	7.5	3.30	
W12x96	11	96	7.5	7.92	
W12x106	2	106	7.5	1.59	
W12x120	11	120	7.5	9.90	
W12x136	6	136	7.5	6.12	
W12x152	3	152	7.5	3.42	
W12x170	26	170	7.5	33.15	
W12x210	6	210	7.5	9.45	
			Weight =	**81**	**k**
Beams:					
Shape	Quantity	Weight (lb/ft)	Beam Length (ft)	Total Weight (k)	
W12x16	17	16	19	5.17	
W12x22	1	22	18	0.40	
W14x22	2	22	24	1.06	
W16x26	12	26	8	2.50	
W16x26	7	26	25	4.55	
W16x26	119	26	30	92.82	
W16x36	1	36	10	0.36	
W18x40	2	40	30	2.40	
W24x55	1	55	11	0.61	
W24x55	4	55	25	5.50	
W24x55	12	55	30	19.80	
W24x55	1	55	35	1.93	
W24x68	21	68	25	35.70	
W24x68	3	68	35	7.14	
W24x68	34	68	30	69.36	
W24x76	1	76	35	2.66	
W24x84	5	84	25	10.50	
W24x84	6	84	30	15.12	
W24x84	8	84	35	23.52	
W24x94	3	94	30	8.46	
W24x94	1	94	35	3.29	
			Weight =	**313**	**k**
	Main Floor Weight =	**2236**	**k OR**	**47.2**	**psf**

Seismic Force Resisting System: Floor Weights

Floor 1					
	Approx. Area =		25,120	SF	
	Floor to Floor Ht. =		14	ft	
Walls:			Superimposed:		
Perimeter =	1200	ft	Partitions =	20	psf
Height =	14	ft	MEP =	10	psf
Unit Wt. =	20	psf	Finishes =	5	psf
Weight =	**336**	**k**	**Weight =**	**879**	**k**
Slab:					
		Thk. =	4.5	in	
		Unit Wt. =	150	pcf	
		Weight =	**1413**	**k**	
Columns:					
Shape	Quantity	Weight (lb/ft)	Column Height (ft)	Total Weight (k)	
W10x112	7	112	14	10.98	
W12x40	6	40	14	3.36	
W12x96	11	96	14	14.78	
W12x106	2	106	14	2.97	
W12x120	11	120	14	18.48	
W12x136	6	136	14	11.42	
W12x152	3	152	14	6.38	
W12x170	26	170	14	61.88	
W12x210	6	210	14	17.64	
			Weight =	**148**	**k**
Beams:					
Shape	Quantity	Weight (lb/ft)	Beam Length (ft)	Total Weight (k)	
W12x16	12	16	19	3.65	
W14x22	2	22	19.5	0.86	
W16x26	15	26	25	9.75	
W16x26	112	26	30	87.36	
W18x40	2	40	25	2.00	
W18x40	4	40	30	4.80	
W18x50	2	50	25	2.50	
W24x55	1	55	11	0.61	
W24x55	4	55	25	5.50	
W24x55	14	55	30	23.10	
W24x68	21	68	25	35.70	
W24x68	31	68	30	63.24	
W24x68	3	68	35	7.14	
W24x76	1	76	35	2.66	
W24x84	6	84	25	12.60	
W24x84	7	84	30	17.64	
W24x84	10	84	35	29.40	
W24x94	3	94	30	8.46	
W24x94	2	94	35	6.58	
			Weight =	**324**	**k**
	1st Floor Weight =	**3100**	**k OR**	**123**	**psf**

Seismic Force Resisting System: Floor Weights

Floor 2					
	Approx. Area =		25,120	SF	
	Floor to Floor Ht. =		14	ft	
Walls:			**Superimposed:**		
Perimeter =	755	ft	Partitions =	20	psf
Height =	14	ft	MEP =	10	psf
Unit Wt. =	20	psf	Finishes =	5	psf
Weight=	**211**	**k**	**Weight=**	**879**	**k**
Slab:					
		Thk. =	4.5	in	
		Unit Wt. =	150	pcf	
		Weight=	**1413**	**k**	
Columns:					
Shape	Quantity	Weight (lb/ft)	Column Height (ft)	Total Weight (k)	
W10x112	7	112	14	10.98	
W12x40	3	40	14	1.68	
W12x96	1	96	14	1.34	
W12x120	5	120	14	8.40	
W12x136	3	136	14	5.71	
W12x152	3	152	14	6.38	
W12x170	18	170	14	42.84	
W12x210	6	210	14	17.64	
			Weight =	**95**	**k**
Beams:					
Shape	Quantity	Weight (lb/ft)	Beam Length (ft)	Total Weight (k)	
W12x22	3	22	25	1.65	
W14x22	2	22	19.5	0.86	
W16x26	12	26	25	7.80	
W16x26	44	26	30	34.32	
W18x40	2	40	25	2.00	
W18x40	1	40	30	1.20	
W24x55	14	55	25	19.25	
W24x55	5	55	30	8.25	
W24x68	6	68	25	10.20	
W24x68	26	68	30	53.04	
W24x84	3	84	25	6.30	
W24x84	3	84	30	7.56	
			Weight =	**152**	**k**
	2nd Floor Weight =	**2751**	**k OR**	**110**	**psf**

Seismic Force Resisting System: Floor Weights

Floor 3					
	Approx. Area =		25,120	SF	
	Floor to Floor Ht. =		14.4	ft	
	Walls:		**Superimposed:**		
Perimeter =	755	ft	Partitions =	20	psf
Height =	14.4	ft	MEP =	10	psf
Unit Wt. =	20	psf	Finishes =	5	psf
Weight=	**217**	**k**	**Weight=**	**879**	**k**
		Slab:			
		Thk. =	4.5	in	
		Unit Wt. =	150	pcf	
		Weight=	**1413**	**k**	
		Columns:			
Shape	Quantity	Weight (lb/ft)	Column Height (ft)	Total Weight (k)	
W10x112	7	112	14.4	11.29	
W12x40	3	40	14.4	1.73	
W12x96	1	96	14.4	1.38	
W12x120	5	120	14.4	8.64	
W12x136	3	136	14.4	5.88	
W12x152	3	152	14.4	6.57	
W12x170	18	170	14.4	44.06	
W12x210	6	210	14.4	18.14	
			Weight =	**98**	**k**
		Beams:			
Shape	Quantity	Weight (lb/ft)	Beam Length (ft)	Total Weight (k)	
W12x16	4	16	15	0.96	
W12x22	2	22	25	1.10	
W12x35	1	35	25	0.88	
W14x22	1	22	19.5	0.43	
W16x26	15	26	25	9.75	
W18x40	79	40	30	94.80	
W18x40	12	40	35	16.80	
W21x55	1	55	30	1.65	
W24x55	16	55	25	22.00	
W24x55	4	55	30	6.60	
W24x68	5	68	25	8.50	
W24x94	13	94	25	30.55	
W24x94	7	94	35	23.03	
			Weight =	**217**	**k**
	3rd Floor Weight =	**2824**	**k OR**	**112**	**psf**

Seismic Force Resisting System: Floor Weights

Roof					
Approx. Area =		25,120	SF	for beams	
Approx. Area =		47,410	SF	for material	
Approx. Area =		3,370	SF	for slabs supporting PH	
Slab:			**Superimposed:**		
Thk. =	6.5	in	MEP =	10	psf
Unit Wt. =	150	pcf	Rf. Mat'l =	10	psf
Weight=	**274**	**k**	**Weight=**	**725**	**k**
Beams:					
Shape	Quantity	Weight (lb/ft)	Beam Length (ft)	Total Weight (k)	
W6x15	1	15	35	0.53	
W12x16	2	16	15	0.48	
W14x22	8	22	19.5	3.43	
W14x22	18	22	25	9.90	
W14x22	73	22	30	48.18	
W16x26	5	26	25	3.25	
W18x40	12	40	25	12.00	
W18x40	27	40	30	32.40	
W18x40	6	40	35	8.40	
W18x50	1	50	30	1.50	
W21x55	1	55	30	1.65	
W24x55	2	55	25	2.75	
W24x55	3	55	30	4.95	
W24x55	1	55	35	1.93	
W24x68	1	68	25	1.70	
			Weight =	**133**	**k**
	Roof Weight =	**1132**	**k OR**	**45**	**psf**

SEISMIC DESIGN

OCCUPANCY CATEGORY : III

IMPORTANCE FACTOR : 1.5 ⟸ VALUE USED BY DESIGN ENGINEER

SITE CLASS : E

S_S : 0.170, S_1 : 0.056 } USING APPLET W/ LONGITUDINAL + LATITUDE COORDINATES OF SITE

R : 3.0

h_n : 57.4 ft

T_L : 12 [FIG. 22-15 ASCE 7-05]

C_t : 0.028, x : 0.8 } FOR STEEL MOMENT FRAMES - TABLE 12.8-2

USING TABLE 11.4-1 (ASCE 7-05)

$$S_{MS} = F_a S_S = (2.5)(0.170) = 0.425$$

USING TABLE 11.4-2 (ASCE 7-05)

$$S_{M1} = F_v S_1 = (3.5)(0.056) = 0.196$$

$$\left.\begin{aligned} S_{DS} &= \tfrac{2}{3} S_{MS} = \tfrac{2}{3}(0.425) = 0.283 \\ S_{D1} &= \tfrac{2}{3} S_{M1} = \tfrac{2}{3}(0.196) = 0.131 \end{aligned}\right\}$$

USING TABLE 11.6-1 + 11.6-2 (ASCE 7-05)
SDC = B

RESPONSE MODIFICATION FACTOR = R = 3.0

$$T_a = C_t h_n^x = 0.028\,(57.4)^{0.8} = 0.715$$

$$T_S = S_{D1}/S_{DS} = 0.131/0.283 = 0.463$$

$$0.8T_S = 0.8(0.463) = 0.370$$

$0.8T_S < T_a$ ∴ TABLE 11.6-1,2 GIVES VALUES FOR C_t + x ✓

T_L = 12 [FIG 22-15 ASCE 7-05]

$$C_S = \left|\begin{array}{l} \dfrac{S_{DS}}{(R/I)} = \dfrac{0.283}{(3/1.5)} = 0.1415 \\[2ex] \dfrac{S_{D1}}{(T \cdot R/I)} = \dfrac{0.131}{(0.715(3/1.5))} = 0.0916 \quad \geq 0.01 \\[2ex] \text{MIN} \; \dfrac{S_{D1} \cdot T_L}{(T^2 \cdot R/I)} = \dfrac{0.131\,(12)}{(0.715^2(3/1.5))} = 1.537 \end{array}\right.$$

$$C_S = 0.0916 \approx 0.092$$

$$f = 1/T = 1/0.715 = 1.40 > 1.0$$ ∴ RIGID DIAPHRAGM

SEE EXCEL SPREADSHEET FOR FLOOR WEIGHTS

MAIN FLOOR: 47,410 SF 47.2 psf
1ST FLOOR: 25,120 SF 123 psf
2ND FLOOR: 25,120 SF 110 psf
3RD FLOOR: 25,120 SF 112 psf
ROOF: 25,120 SF 45 psf

W_T = TOTAL BLDG. WT.

$W_T = 47{,}410(47.2) + 25{,}120(123) + 25{,}120(110) + 25{,}120(112) + 25{,}120(45)$

$W_T = 12{,}043{,}000 \text{ lbs} = 12{,}043^k$

$V = C_s W_T$

$V = 0.092(12043^k) = 1100^k$

*NOTE: BASE SHEAR VALUE IS HIGH DUE TO BEING IN SITE CLASS E AND USING AN IMPORTANCE FACTOR OF 1.5

-End of Section-

APPENDIX D: SNOW ANALYSIS

Photo courtesy of
www.springfieldcolorado.com

SNOW LOAD CALCULATION

FLAT ROOF SNOW LOADS PER 7.3 ASCE 7-05

$p_f = 0.7 C_e C_t I p_g$

$p_g = 20\,psf$

$C_e = 1.0$

$C_t = 1.0$

$I = 1.2$ [OCCUPANCY CATEGORY IV]

$p_f = 0.7(1.0)(1.0)(1.2)(20\,psf) = 16.8\,psf$

DRIFT CONSIDERATIONS — SEE DIAGRAM BELOW:

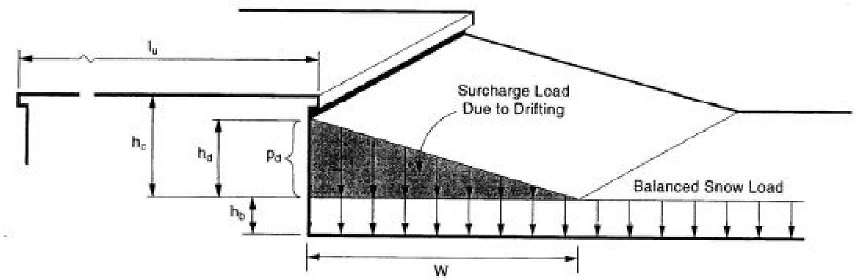


Figure 7-8 ***(ASCE 7-05)*****:** *Configuration of Snow Drifts on Lower Roofs*

LOOK AT DRIFT FOR PENTHOUSE WITH WORST CASE DIMENSION FROM EDGE OF ROOF ; WILL NOT CONSIDER DRIFT FOR LOWER FLOORS DUE TO 3½' HIGH PARAPET PREVENTING DRIFT FROM UPPER FLOORS

DIFF. IN ROOF HEIGHT = $h_c + b_b = 14'$

$\gamma = 0.13 p_g + 14 \leq 30\,pcf$ PER 7.7 ASCE 7-05

$\gamma = 0.13(20) + 14 = 16.6\,pcf < 30\,pcf$

LEEWARD DRIFTS:

$l_{UPPER\ RF} = 19'$ [LENGTH OF PENTHOUSE RF.]

$h_{d\,LEE} = 0.43 \sqrt[3]{l_{UPPER}}\ \sqrt[4]{p_g + 10} - 1.5$ PER FIGURE 7-9

$= 0.43 \sqrt[3]{(19)}\ \sqrt[4]{20+10} - 1.5 = 1.19'$

WINDWARD DRIFTS:

$l_{LOWER\ RF} = 175'$ [LENGTH FROM EDGE OF BLDG RF. TO PENTHOUSE]

$h_{d\,WIND} = 0.75(0.43) \sqrt[3]{l_{LOWER}}\ \sqrt[4]{p_g + 10} - 1.5$ PER FIGURE 7-9

$= 0.75(0.43) \sqrt[3]{175}\ \sqrt[4]{20+10} - 1.5 = 2.72'$

MAX $h_d = 2.72'$

$h_b = p_f/\gamma = 16.8/16.6 = 1.01'$

$h_c =$ DIFF IN RF HT $- h_b = 14' - 1.01' = 12.99'$

$$h_d = \min \begin{cases} h_{d\,wind} = 2.72' & \leftarrow \text{CONTROLS} \\ h_c = 12.99' \end{cases}$$

$$w = \min \begin{cases} 8h_c & = 8(12.99) = 104' \\ 4h_d \text{ IF MAX } h_{d\,wind/lee} \leq h_c & = 4(2.72) = 10.88' \quad \leftarrow \text{CONTROLS} \\ 4h_d^2/h_c \text{ ELSE} & \text{———} \end{cases}$$

$w = 10.88'$

$h_{DRIFT} + h_b = 2.92' + 1.01' = 3.73'$

$w = (h_{DRIFT} + h_b)(\gamma) = 3.73'(16.6) = 61.9$ psf @ HIGH END OF DRIFT

$$w = \begin{cases} p_f & \text{IF } w < l_{LOWER\ RF} \quad \leftarrow \quad 10.88' < 175' \quad \therefore \text{ THIS VALUE CONTROLS} \\ p_f + \frac{h_d}{w}(w - l_{LOWER\ RF}) & \text{ELSE} \end{cases}$$

$w = p_f = 16.8$ psf @ LOW END OF DRIFT

$w = 61.9$ psf @ HIGH END OF DRIFT

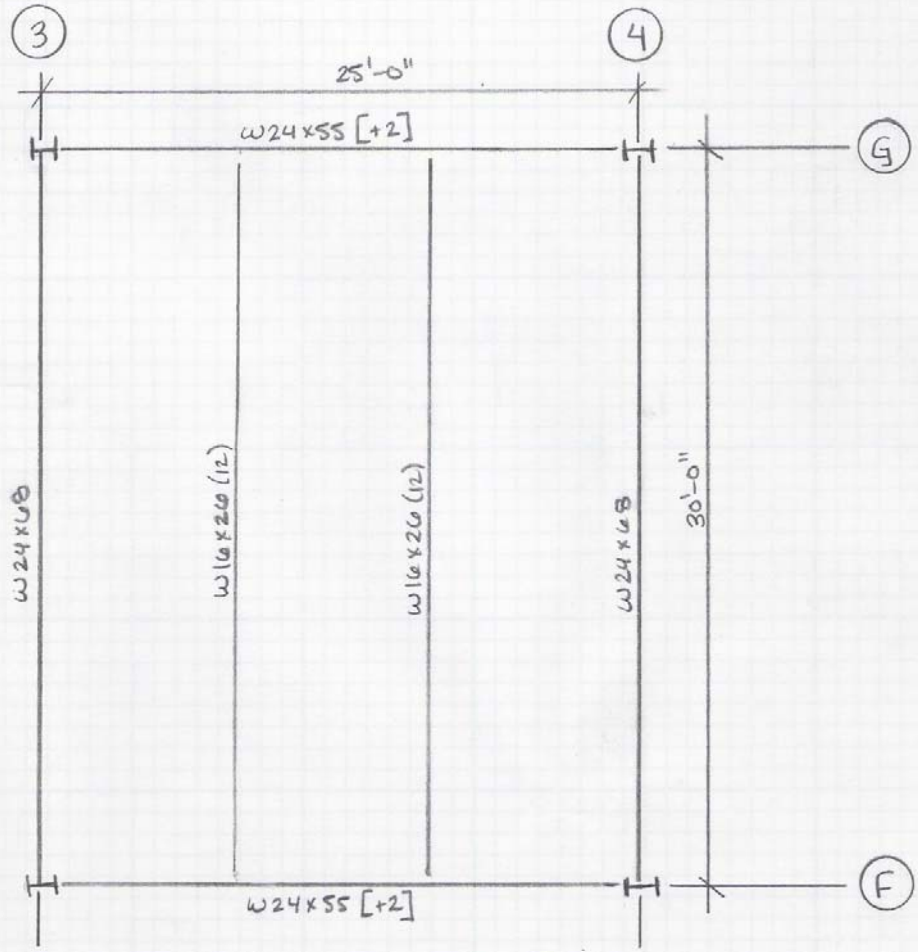
$w = 16.8$ psf @ LOW END OF DRIFT

-End of Section-

APPENDIX E: FLOOR SYSTEM AND MEMBER SPOT CHECKS

Photo courtesy of www.secapp.com

TYPICAL INTERIOR BAY : 2^{ND} FLOOR



LOADS:

LIVE: ASSUME 100 PSF

DEAD: 110 PSF [SEE APPENDIX C: SEISMIC ANALYSIS]

FLOOR SYSTEM:

$4\frac{1}{2}$" N.W.C.

2" x 20 GA DECK

LOAD COMBINATION:

1.2D + 1.6L

SPOT CHECK BEAM

FACTORED LOAD: $1.2D + 1.6L$

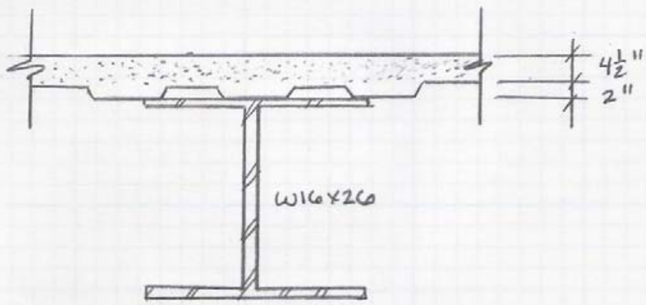
$w_u = 1.2(110\,psf) + 1.6(100\,psf) = 292\,psf$

TRIB. WIDTH = 8.33'

$w_u = 292\,psf\,(8.33')/1000 = 2.43\,klf$

$M_u = \dfrac{w_u l^2}{8} = \dfrac{(2.43\,klf)(30')^2}{8} = 273^{1k}$

$b_{eff} = \min \begin{cases} \text{SPACING} = 8.33' \times 12 = 100'' \\ \dfrac{\text{SPAN}}{4} = \dfrac{30(12)}{4} = 90'' \longleftarrow \text{CONTROLS} \end{cases}$



CHECK FOR DEFLECTION UNDER CONSTRUCTION LOADS:

$\Delta_{CONSTR.} = \dfrac{5 w_{conc} l^4}{384 EI}$

$w_{conc} = 150\,pcf\,(4.5''/12) = 56.3\,psf$

$w_{conc} = 56.3\,psf\,(8.33') = 469\,plf = 0.469\,klf$

$\Delta_{ALLOW} = l/360 = 30(12)/360 = 1''$

$I_{req} = \dfrac{5 w_{conc} l^4}{384 \Delta_{CONSTR} E} = \dfrac{5(0.469)(30)^4(1728)}{384(1)(29000)} = 295\,in^4$

$I_{W16x26} = 301\,in^4 > 295\,in^4$ ✓ OK

CHECK BENDING FOR CONSTRUCTION LOADING:

$w_{conc} = 0.469\,klf$

$w_{live} = 20\,psf(8.33') = 0.167\,klf$

$w_u = 1.2(0.469) + 1.6(0.167) = 0.83\,klf$

$M_u = w_u l^2/8 = 0.83(30)^2/8 = 93.4^{1k}$

$\phi M_{n\,W16x26} = 166^{1k} > 93.4^{1k}$

[COMPARE M_u WITH ϕM_n FOR W16x26 FROM Z_x TABLE B/C SYSTEM NOT COMPOSITE UNTIL CONSTRUCTION IS COMPLETE]

FROM TABLE 3-19:

ASSUME $\Sigma Q_n = 145^k$

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{145^k}{0.85(3.5)(90)} = 0.542''$$

$Y2 = 6.5'' - a/2 = 6.5'' - 0.542''/2 = 6.23''$ [ROUND ↓ TO 6'' TO BE CONSERVATIVE]

USING TABLE 3-19:

W16x26 $\quad Y2 = 6'' \quad \Sigma Q_n = 145^k$ @ PNA #6

$\phi M_n = 285^{'k} > M_u = 273^{'k}$

CHECK NUMBER OF SHEAR STUDS:

TABLE 3-21:

SHEAR STUD DIAM = 3/4'' ; 1 STUD/RIB
DECK PERPENDICULAR
$f'_c = 3000$ ksi (CONSERVATIVE)

} $Q_n = 17.2^k$

$$\# \text{ STUDS REQ'D} = \frac{\Sigma Q_n}{Q_n} \times 2 = \frac{145}{17.2} \times 2 = 16.9 \rightarrow 17 \text{ STUDS REQ'D}$$

STUDS PROVIDED = 30 [STUDS PLACED @ 12'' O.C. OVER LENGTH OF BM]

STUDS PROVIDED > # STUDS REQ'D ✓ OK

CHECK DEFLECTION:

TABLE 3-20:

$Y2 = 6'' \Rightarrow I_{LB} = 705 \text{ in}^4$

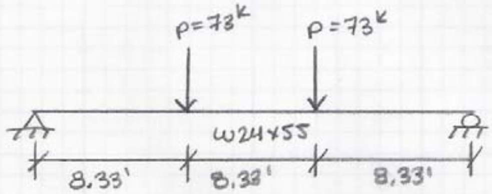
$$\Delta = \frac{5 w_u l^4}{384 E I_{LB}} = \frac{5(0.833)(30)^4}{384(29000)(705)} = 0.74''$$

$w_u = 100 \text{ psf}(8.33')/1000 = 0.833 \text{ klf}$

$\Delta_{ALLOW} = l/360 = 30(12)/360 = 1''$

$0.74'' < 1''$ ✓ OK

SPOT CHECK GIRDER



$w_{u,DL} = 1.2(110psf)(8.33')/1000 = 1.10 klf$

$P_{DL} = \frac{w_{u,DL}\,l}{2} = \frac{(1.10)(30)}{2} = 16.5^k$

$w_{u,LL} = 1.6(100psf)(8.33')/1000 = 1.33 klf$

$P_{LL} = \frac{w_{u,LL}\,l}{2} = \frac{(1.33)(30)}{2} = 20.0^k$

TOTAL P ON GIRDER = $(16.5^k + 20.0^k) \times 2 = 73^k$

BMS FRAME IN ON EACH SIDE

$M_{MAX} = P(a) = 73^k(8.33') = 608^{1k}$

ASSUME $Y2 = 4''$ $\Longrightarrow$ REQUIRING PNA # 7 (TABLE 3-19)

$\Sigma Q_n = 203^k$

$$b_{eff} = \min \begin{cases} \text{SPACING} = 30' = 360'' \\ \frac{\text{SPAN}}{4} = \frac{25}{4} = 6.25' = 75'' \leftarrow \text{CONTROLS} \end{cases}$$

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{203^k}{0.85(3.5)(75)} = 0.91''$$

GIRDERS PLACED 2" ABOVE BEAMS ∴ DEPTH = $4\frac{1}{2}''$ NOT $6\frac{1}{2}''$

$Y2 = 4\frac{1}{2} - a/2 = 4\frac{1}{2} - 0.91/2 = 4.04''$ [ROUND ↓ TO 4" TO BE CONSERVATIVE]

USING TABLE 3-19:

$Y2 = 4''$ $\Sigma Q_n = 203^k$ W24X55 PNA # 7

$\phi M_n = 705^{1k} > M_{MAX} = 608^{1k}$ ✓ OK

CHECK DEFLECTION:

$I_{LB} = 2160 in^4$ (TABLE 3-20)

$$\Delta = \frac{5 w_u l^4}{384 E I_{LB}} = \frac{5(0.833)(25)^4(1728)}{384(29000)(2160)} = 0.12''$$

$w_u = 100psf(8.33')/1000 = 0.833 klf$

$\Delta_{ALLOW} = l/360 = 25(12)/360 = 0.83''$

$0.12'' < 0.83''$ ✓ OK

Floor	Tributary Area (ft^2)	Dead Load (psf)	Live Load (psf)	Influence Area (ft^2)	Reduction Factor (>=0.4)	Live Load (k)	Dead Load (k)	Load Combination	Load at Floor (k)	Accumulated Load (k)
Roof	900	45	100	3600	-	90.0	40.6	1.2D + $0.5L_r$	93.7	93.7
3	900	112	100	3600	0.500	45.0	101.2	1.2D + 1.6L	193.4	287.1
2	900	110	100	3600	0.500	45.0	98.6	1.2D + 1.6L	190.3	477.4
1	900	123	100	3600	0.500	45.0	111.1	1.2D + 1.6L	205.3	682.6
Main	900	103	100	3600	0.500	45.0	92.7	1.2D + 1.6L	183.2	865.9

Accumulated Load on Columns

COLUMN SPOT CHECK : E3

SEE SPREADSHEET FOR LOADS

FLOOR 4 (ROOF) : $P_u = 93.7^k$

W12x170 ; $h = 14.5'$

$A_g = 50.0\ in^2$

$I_x = 1650\ in^4$ $\quad I_y = 517\ in^4$

$r_x = 5.74\ in$ $\quad r_y = 3.22\ in$

$\frac{KL}{r_x} = \frac{14.5(12)}{5.74} = 30.3$ $\quad \frac{KL}{r_y} = \frac{14.5(12)}{3.22} = 54.0$ ← CONTROLS

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

$54.0 < 113$ ∴ INELASTIC BEHAVIOR

$$F_{cr} = \left[0.658^{F_y/F_e}\right] F_y = \left[0.658^{50/98.1}\right](50) = 40.4\ ksi$$

$$F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2 (29000)}{(54)^2} = 98.1\ ksi$$

$$\phi P_n = \phi F_{cr} A_g = 0.9(40.4)(50.0) = 1818^k$$

$$P_u = 93.7^k \ll \phi P_n = 1818^k$$

CHECK W/ TABLE 4-22:

$\frac{KL}{r} = 54 \qquad \phi F_{cr} = 36.4 ksi$

$\phi F_{cr} = 0.9(40.4) = 36.4 ksi$ ✓ METHOD BY HAND CHECKS

CHECK W/ TABLE 4-1:

$KL = 14.5' \qquad W12 \times 170$

$\phi P_n = 1815^k \approx 1818^k$ ✓ METHOD BY HAND CHECKS

*NOTE: TABLE 4-1 SHALL BE USED FOR REMAINING COLUMN CHECKS AS IT IS BASED ON METHOD BY HAND SHOWN ABOVE.

COMMENTS: COLUMN SIZES ARE VERY LARGE WHILE CONSIDERING GRAVITY LOADS ALONE, HOWEVER, EVERY COLUMN IS PART OF A MOMENT CONNECTION HENCE RECEIVES LARGE INDUCED MOMENTS

FLOOR 3: $P_u = 287^k$

$W12 \times 170 \; ; \; h = 14' = KL$

TABLE 4-1:

$\phi P_n = 1840^k > P_u = 287^k$ ✓ OK

FLOOR 2: $P_u = 477^k$

$W12 \times 170 \; ; \; h = 14' = KL$

TABLE 4-1:

$\phi P_n = 1840^k > P_u = 477^k$ ✓ OK

FLOOR 1: $P_u = 683^k$

$W12 \times 170 \; ; \; h = 15' = KL$

TABLE 4-1:

$\phi P_n = 1790^k > P_u = 683^k$ ✓ OK

OBSERVATIONS:

COLUMNS ARE SO LARGE DUE TO THE MOMENT THEY MUST RESIST FROM LATERAL FORCES

-End of Section-