TECHNICAL REPORT 1

Structural Concepts / Structural Existing Conditions

Penn State Hershey Medical Center Children's Hospital

Hershey, Pennsylvania

Matthew V Vandersall

The Pennsylvania State University

Architectural Engineering

Structural Option

Adviser: Dr. Richard Behr

October 7, 2010

Table of Contents

Executive Summary

The objective of Technical Report 1 is to investigate the structural system for the Penn State Hershey Medical Center Children's Hospital. To achieve this objective, this report will focus on the following:

- Exploring the structural concepts and conditions of the structural design
- Computing all required loads including wind, seismic, and snow for the existing systems
- Verifying typical framing elements in gravity load areas with hand calculations

An introduction to the structural systems is provided to summarize some of the existing conditions and structural concepts. These conditions are subdivided into separate sections to explore the foundation, floor, roof, and lateral systems. A list of building codes and materials used in the design is also provided for reference in the analysis that follows.

Using the calculating procedures listed in ASCE 7-10, the loadings due to wind, seismic, and snow forces were determined for the structure. Loading diagrams included within this report show that the predominantly controlling force is wind pressure striking the North and South faces of the structure. This information will be used in future reports to analyze the story drift of the lateral system.

Spot checks were performed to confirm that the structure was adequately designed under gravity loads. The structural components that were considered include a composite beam design, a girder design, and a column design. These members were determined to have been properly designed and to have met all strength and serviceability requirements. All hand calculations that were performed for this report are included within the appendix.

The investigation of this report shows that structural concepts and conditions of the Children's Hospital are sufficiently designed. The wind pressures along with the gravity loads are determined to be the overall design factors. Future reports will include a more intensive analysis for the lateral system which would provide additional information on the response of the structure.

Building Overview

The new Penn State Hershey Medical Center Children's Hospital is located at 500 University Drive in Hershey, Pennsylvania. The Children's Hospital is an expansion project on the existing Cancer Institute and Main Hospital. The overall project plan calls for a five story, 263,556 square-foot addition which will contain a number of operating rooms, offices, and patient rooms specializing in pediatric care. The exterior of the building utilizes spandrel glass and an aluminum curtain wall system. The main curve of the façade helps to tie the building into the existing curve along the Cancer Institute. A vegetated roof garden will be situated on the third level above the existing Cancer Institute. See Figure 1 for a site plan of the Children's Hospital.

The dates of construction for the Children's Hospital are scheduled for March 2010 to August 2012. The drawing specifications for the Children's Hospital note that an additional two floors of occupancy are intended for a later date. The range of this thesis project will be limited to the structural analysis of the Children's Hospital.

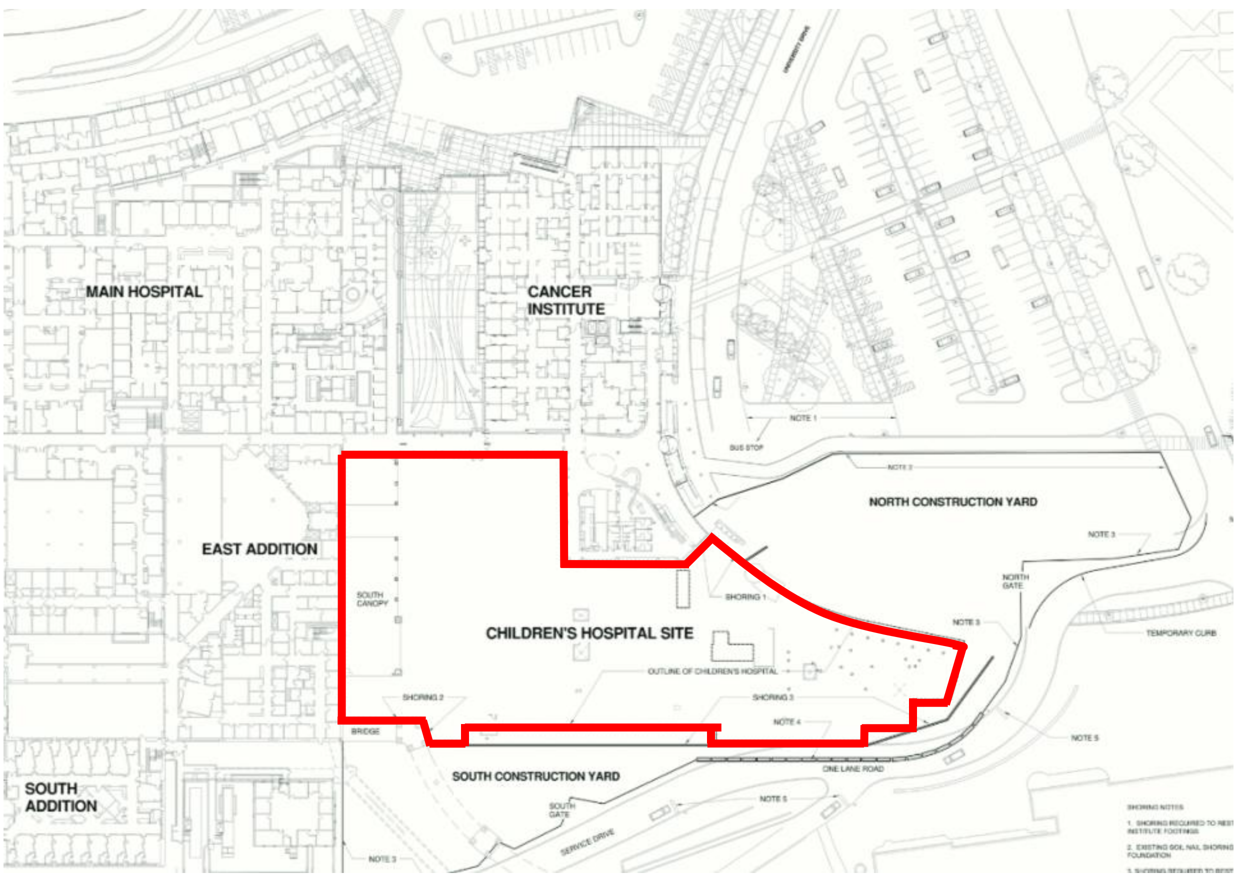


Figure 1 – Site Plan

Introduction to Structural System

The primary structural system comprises of structural steel framing integrated with a composite floor system. The composite floor consists of metal decking with normal weight concrete topping. Shear studs are welded to the supporting beam and embedded into the slab allowing interaction between the two elements. Transfer girders help to transmit the gravity loads from the beams to the columns. All of the columns consist of W14 members which allow for easier constructability. The lateral force resisting system consists of moment connected frames along the East-West direction while diagonal bracing members assist in North-South bracing.

Foundation

Due to the potential for excessive settlement, micropiles were utilized as recommended in the Geotechnical Report provided by CMT Laboratories. Micropiles consist of a casing that is injected with grout to create a friction bond within the bond zone. The piles that are used in the design are specified for a compression load of 280kips and a tension capacity of 170 kips. There are over 600 micropiles that were used in the foundation of the structure. See Figure 2 for a detail section of a typical micropile.

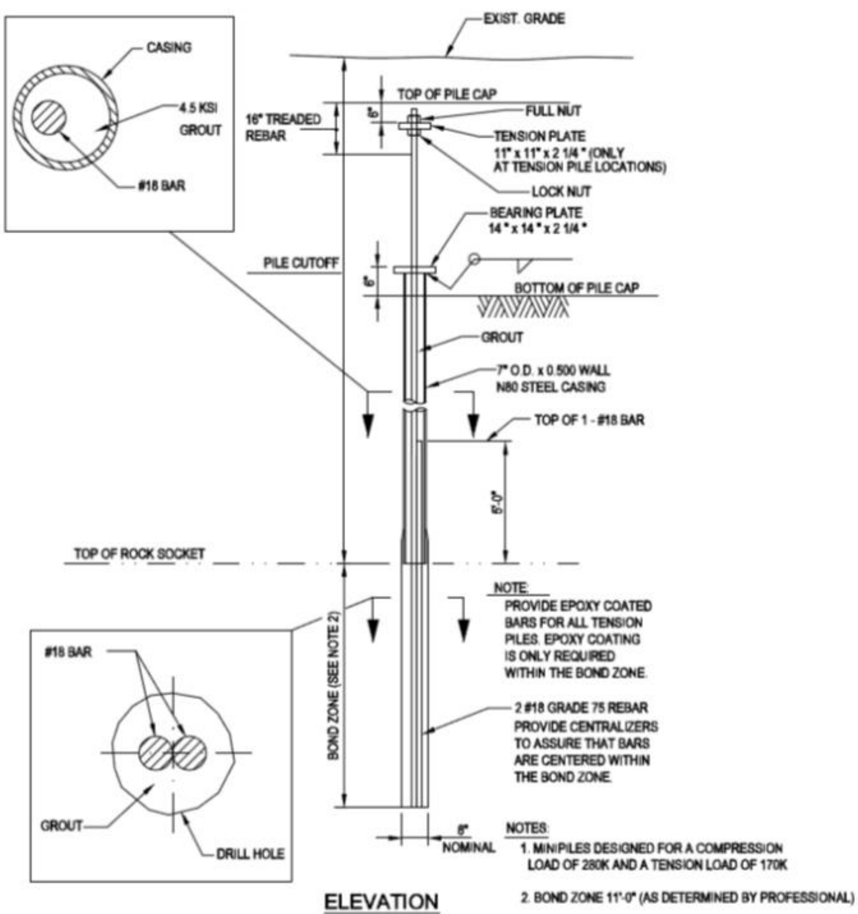


Figure 2 - Micropile Detail

The micropiles are grouped into various sizes of pile caps ranging from 3'0" x 3'0" to 10'0" x 15'0" with a depth ranging from 3' 6" to 6' 0". An example of a typical pile cap can be seen in Figure 3. Typical strut beams of 1' 6" wide by 2' 8" deep span between all pile caps to provide resistance to lateral column base movement. See "Figure 4 – Typ. Strut Beam" below.

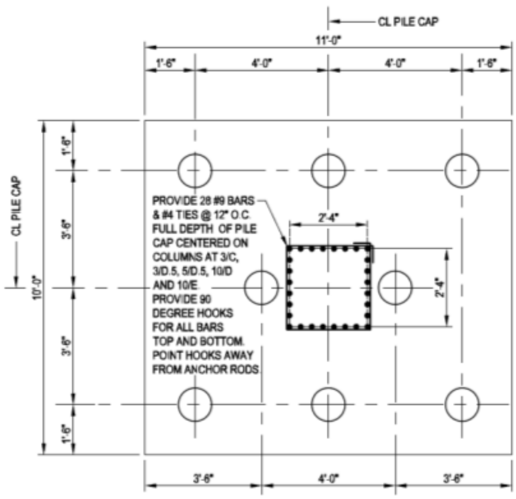


Figure 3 - P8 Pile Cap Plan

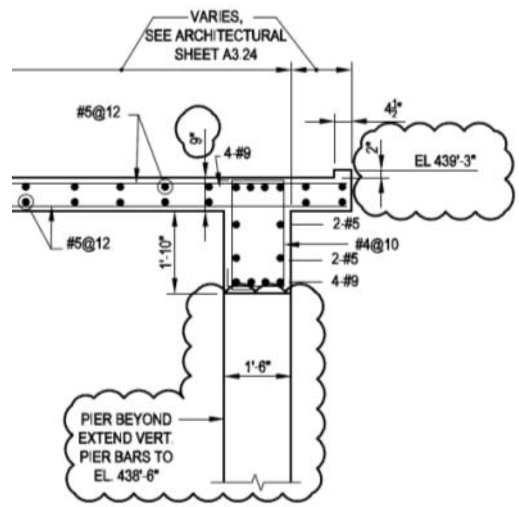


Figure 4 - Typ. Strut Beam

The floor at the ground level is a 5" concrete slab while in heavier load areas such as elevator pits and mechanical rooms a slab thickness of 6" is used. Below is an overview of the West End foundation plan.

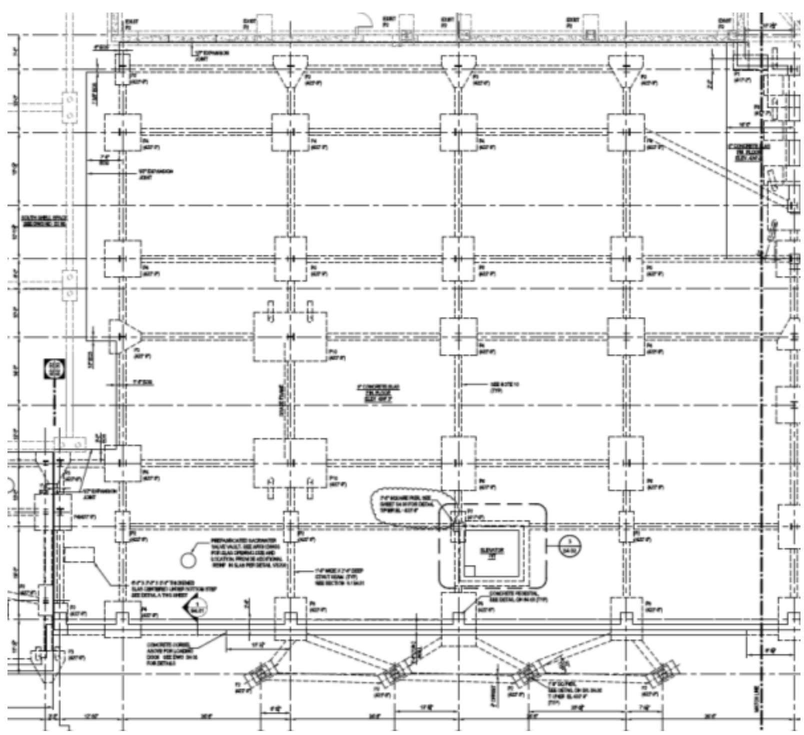


Figure 5 - West End Foundation Plan

Floor System

The typical floor slab throughout all five stories consists of a composite floor system denoted on structural drawings as S1 TYP. This slab type is comprised of a 2" deep, 20-gage composite metal deck with a 4 ½" topping thickness. The reinforcement within the slab is 6x6 W2.1xW2.1 Welded Wire Fabric. The only change in slab thickness occurs at an area on Level 2 marked as having a slab type of S2 TYP (see Figure 6). Here, a 6" concrete slab sits on a 2" deep, 20 gage composite deck with 6x6 W2.9xW2.9 Welded Wire Fabric. The main reason behind increasing the slab thickness in this area is to account for a future MRI space where the live load is considered to be 215 PSF. All floor slabs are connected to wide flange beams using ¾" diameter shear studs where the number of studs is listed on each beam in the framing plans. The typical span for a wide flange beam is 34' 6".

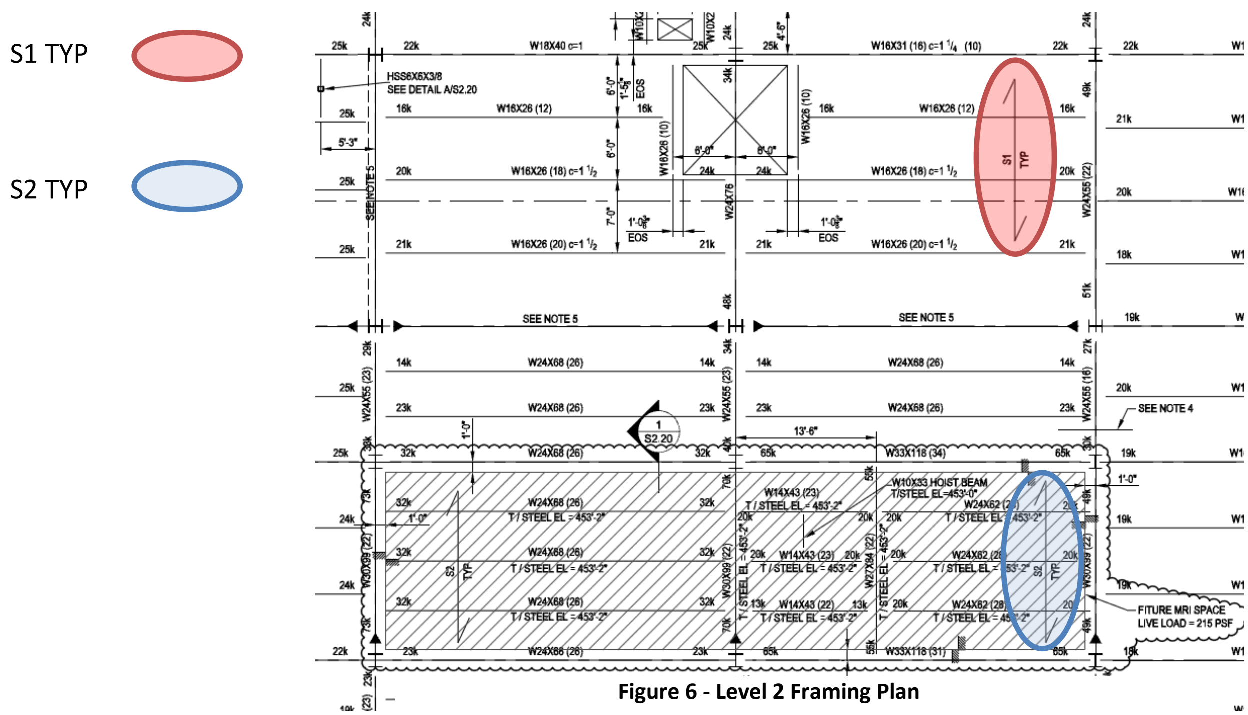


Figure 6 - Level 2 Framing Plan

Roof System

The roof system for the Children's Hospital utilizes the same construction as the S1 TYP floor designation. Future plans call for an additional two stories of occupiable space to be constructed above the current roof level. Figure 7 shows how the columns for the future sixth floor are to be attached to the existing columns. The roofing material consists of a multiple-ply built-up roofing membrane on top of insulation. Surrounding the roof is an 8" thick parapet wall that rises 1' 4" above the top of the composite slab.

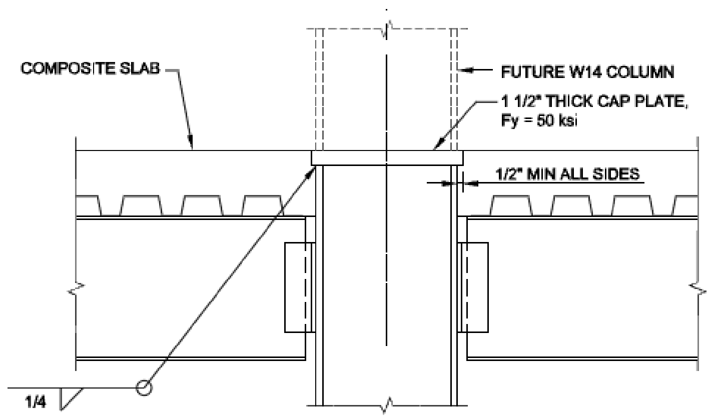


Figure 7 - Top of Column at Future Sixth Floor

Lateral System

The main lateral force resisting system is composed of several moment frames located at the interior of the floor plan. These moment frames run in the East-West direction along the floor plan and are represented in Figure 8 with red. The purpose in placing the moment frames in these locations is to allow for a consistent and open floor space which is important for the functionality of a hospital. Running perpendicular to the moment frames are diagonally braced frames which are represented with blue in Figure 8. The locations of these braced frames are set in locations where space requirements are not as significant such as partitions to the elevator banks.

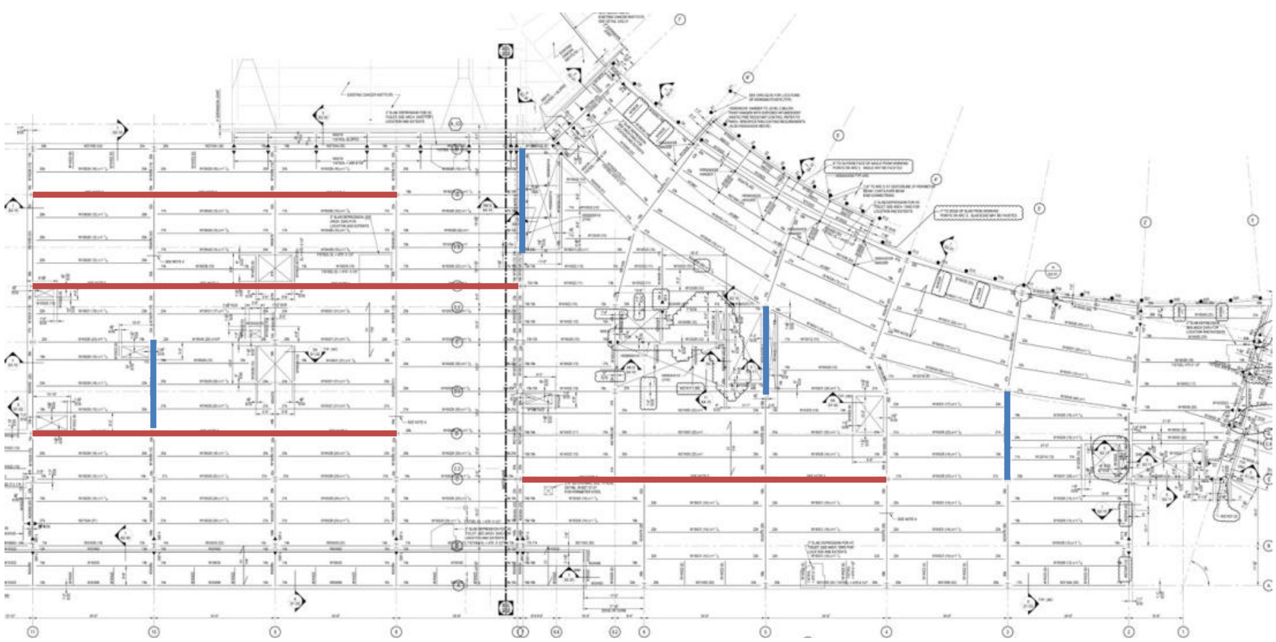


Figure 8 - Framing Plan

Elevations of the typical moment frame can be seen in Figure 9. The main lateral members used in the moment frame system are wide flange sections, primarily W24x229 and W24x176 while the columns are W14x342 and W14x283. An elevation of a braced frame used in the structure is shown in Figure 10 which is comprised of W10x112 and W10x88 bracing members.

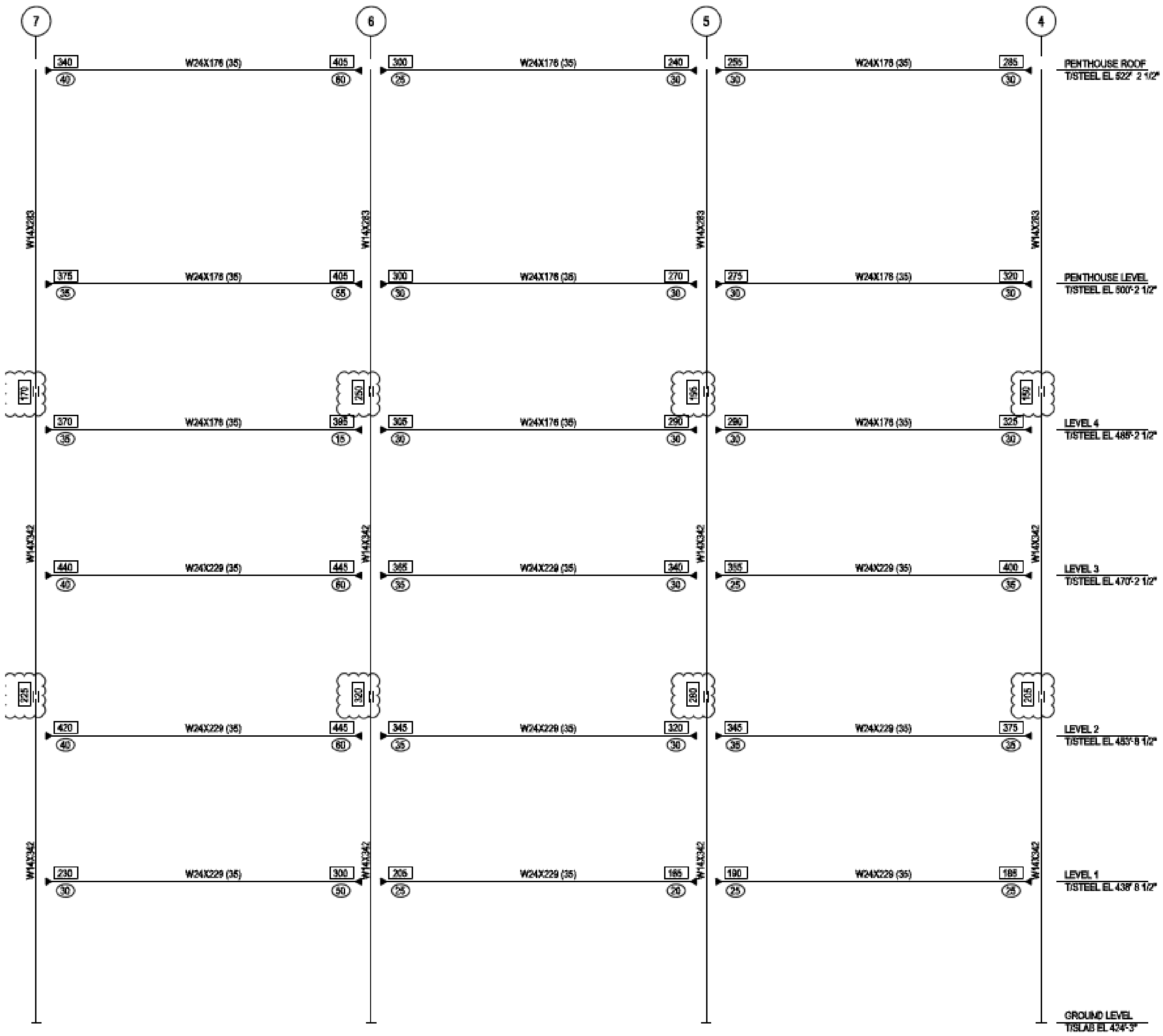


Figure 9 - Elevation: Moment Frame

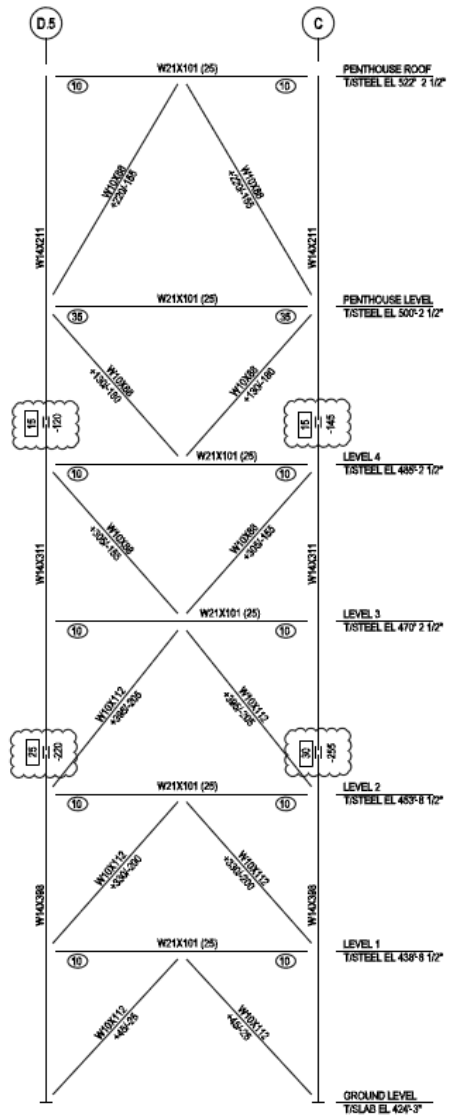


Figure 10 - Elevation: Braced Frame

Conclusions on Structural System

The structural system for the Children's Hospital allows for optimal use of space and provides room for future expansion when the need arises. The importance of using a composite floor system is that it allows for smaller framing members to be used. By using smaller members, the floor to floor height can be increased. Another benefit of using a composite floor system is that it assists in providing additional lateral resistance by creating a stiffer structure. This along with the moment frames allow for larger spaces that are necessary for daily operations of the Children's Hospital.

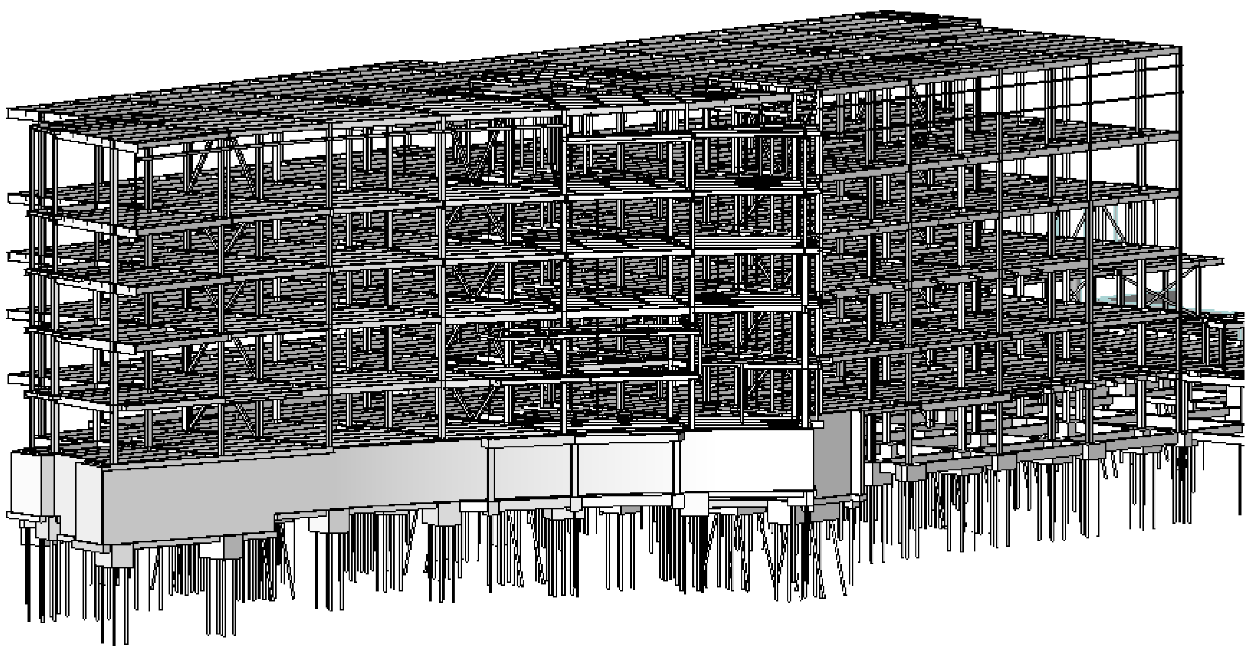


Figure 11 - Framing Render

Building Codes

The building codes used by the structural engineer in the design of the structural system as listed in the specifications are listed as the following:

"International Building Code, 2006 Edition"

SEI/ASCE 7-05, Third Edition – "Minimum Design Loads for Buildings and Other Structures"

AISC – "Manual of Steel Construction – Load and Resistance Factor Design"

AISC 360-05 – "Specification for Structural Steel Buildings"

AISC 303-05 – "Code of Standard Practice for Steel Buildings and Bridges"

ACI 318-05 – "Building Code Requirements for Structural Concrete"

The building codes that will be referenced throughout the research, calculations, and findings of this report are as follows:

"International Building Code, 2009 Edition"

SEI/ASCE 7-10 – "Minimum Design Loads for Buildings and Other Structures"

AISC – Steel Construction Manual, 13th Edition

ACI 318-05 – "Building Code Requirements for Structural Concrete"

Materials

Structural Steel	
Wide Flanges	ASTM A992 Grade 50
Plates, Bars, and Angles	ASTM A36
HSS Rectangular Members	ASTM A500 Grade B
HSS Round Members	ASTM A500 Grade B
Anchor Rods	ASTM F1554 Grade 36
¾" High-Strength Bolts	ASTM A325-X
Welding Electrode	E70XX
Concrete	
Pile Caps	f'c = 4000 psi
Slab on Grade	f'c = 4000 psi
Foundation Walls	f'c = 4000 psi
Column Pedestals	f'c = 4000 psi
Strut Beams	f'c = 4000 psi
Note: all concrete is normal weight concrete (145 pcf)	
Reinforcement	
Reinforcing Bars	ASTM A615 Grade 60
Welded Wire Fabric	ASTM A185
Decking	
Floor Deck	2" Composite Metal Deck, 20 Ga.
Roof Deck	1 ½" Metal Roof Deck, 20 Ga.
¾" Shear Studs	ASTM A108
Masonry	
Grout (micropiles)	f'c = 4500 psi

Gravity and Lateral Loads

The following live loads were determined using ASCE 7-10 while most of the dead loads are assumed based on the industry standard. Where specific gravity loads could not be determined, estimation was made with basic research.

Dead and Live Loads

Dead Loads	
Normal Weight Concrete	145 pcf
Structural Steel	490 pcf
2" Deep Metal Deck	69 psf
Superimposed Dead Load	30 psf
Aluminum Cladding	0.75 psf
Note: Superimposed Dead Load includes MEP systems, ceiling weights, and finishes	
Live Loads	
Lobbies/Moveable Seat Areas	100 psf
Corridors (First Floor)	100 psf
Corridors (Above First Floor)	80 psf
Classrooms, Scientific Labs, Offices, Etc.	80 psf
Electrical and Mechanical Rooms	250 psf
Stairs and Landings	100 psf
Storage Areas: Light Storage	125 psf
Storage Areas: Heavy Storage	250 psf
Computer Rooms	100 psf
Courtyards	100 psf
Future MRI Space	215 psf

Wind Load Calculations and Diagrams

Wind load analysis is a critical factor in the structural design of the Children's Hospital. The wind forces were determined using ASCE 7-10 for Main Wind Force Resisting Systems (MWFRS). The structure was analyzed as a 352.3 ft by 131.3 ft rectangle with a building height of 85.5 ft to the top of the parapet. The wind pressures were calculated for each face and then distributed to each story level. The total base shear and overturning moment were subsequently calculated for the building. Further factors and hand calculations for the wind analysis can be found in Appendix A of this report.

The following pages provide various tables that were used in determining the wind forces:

Table 1 provides the basic wind factors defined by the site location and topography

Table 2 shows the gust effect factor since 0.85 for rigid buildings could not be assumed

Table 3 shows the calculated wind pressures on each face of the building.

Table 4 calculates the total base shear and overturning moment

Conclusion to Wind Load Analysis

From Figure 11, the total base shear was calculated to be 1525.61 kips for the North-South wind loading. The total base shear for the East-West wind loading was determined to be 519.4 kips in Figure 12. The large difference in base shear is attributed to the face of the building normal to each wind direction. Since the North and South facades have about three times larger surface area than the East and West faces, the wind pressure is expected to be larger on those faces. The wind data gathered from this analysis will be used in further thesis reports when analyzing the response of the existing lateral system and confirming of the design.

Table 1: General Requirements	
Occupancy Category	IV
Exposure Category	C
V (MPH)	120
$\mathbf{K_d}$	0.85
$\mathbf{K_{zt}}$	1.0
Enclosure Classification	Enclosed

Table 2: Gust Effect Factor		
	N-S	E-W
B (ft)	352.3	131.3
L (ft)	131.3	352.3
h (ft)	85.5	85.5
n_1	0.632	0.632
β (assumed 1%)	0.01	0.01
Structure ($\eta_1 < 1$ Hz)	Flexible	Flexible
g_Q	3.4	3.4
g_v	3.4	3.4
g_R	4.08	4.08
z	51.3	51.3
L_z	546.12	546.12
I_z	0.152	0.152
Q	0.804	0.860
V_z	122.43	122.43
N_1	2.82	2.82
R_n	0.0726	0.0726
η for R_h	2.03	2.03
R_h	0.373	0.373
η for R_B	8.37	3.12
R_B	0.11	0.27
η for R_L	10.44	28.01
R_L	0.09	0.04
R	0.418	0.632
$\mathbf{G_f}$	**0.902**	**0.988**

Table 3: Wind Pressure on N-S Face and E-W Face

	Level	Height (ft)	K_z	q_z	Wind Pressure	
					N-S (psf)	E-W (psf)
Windward	2	15	0.85	26.63	26.17	27.98
	3	31.5	0.99	31.02	29.34	31.45
	4	46.5	1.07	33.53	31.15	33.43
	Penthouse	61.5	1.14	35.72	32.73	35.16
	Roof	83.5	1.22	38.23	34.54	37.14
	T.O. Parapet	85.5	1.23	38.54	57.81	57.81
Leeward	2 to Roof	85.5	1.23	38.54	-24.32	-17.22
	T.O. Parapet	85.5	1.23	38.54	-38.54	-38.54

Table 4: Story Shear and Overturning Moment

	Total Pressure		Story Force		Story Shear		Overturning Moment	
	N-S (psf)	E-W (psf)	N-S (Kips)	E-W (Kips)	N-S (Kips)	E-W (Kips)	N-S (ft-kips)	E-W (ft-kips)
2	50.49	45.20	280.15	93.47	1525.61	519.40	50874.35	17477.91
3	53.66	48.67	297.73	100.64	1245.46	425.92	30324.28	10450.16
4	55.47	50.65	293.12	99.75	947.73	325.28	16108.33	5570.92
P.H.	57.05	52.38	371.83	127.23	654.61	225.53	6289.12	2187.91
Roof	58.86	54.36	248.84	85.65	282.78	98.30	67.89	25.30
T.O. Parapet	96.35	96.35	33.94	12.65	33.94	12.65	0.00	0.00

BASE SHEAR		BASE MOMENT	
N-S (Kips)	E-W (Kips)	N-S (ft-kips)	E-W (ft-kips)
1525.61	519.40	73758.55	25268.89

Figure 12 - North-South Wind Pressure and Base Shear

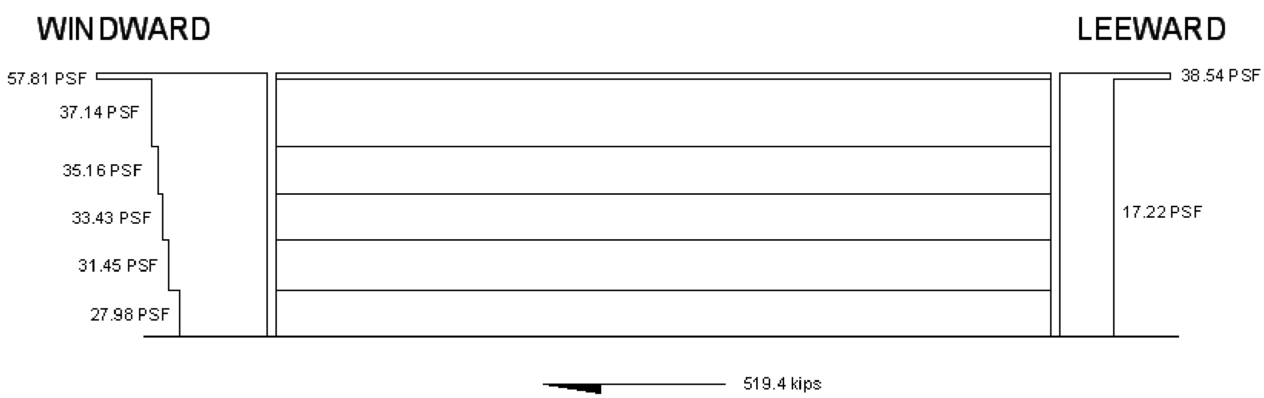


Figure 13 - East-West Wind Pressure and Base Shear

Seismic Load Calculations and Diagrams

Despite the site location being in an area of the country where the effects of earthquakes are minimal, it is still necessary to analyze the structure in terms of its seismic response. Seismic analysis was performed using ASCE 7-10 for seismic design criteria. To determine the base shear for the structure, the total weight for all floors above grade was calculated, see Appendix B. The weight was estimated to be around 25,350 kips. The base shear was calculated by finding the seismic response coefficient and multiplying that by the weight of the structure. The seismic response coefficient C_s was determined to be 4.6% which is comparable for a five story building. The calculations for determining the seismic response coefficient can be found in Appendix C.

Conclusion to Seismic Load Analysis

The base shear for the structure was determined to be 1166.1 kips. Table 5 and Figure 13 show how each level experiences a different percent of the base shear based on the weight of that floor in relation to the overall weight. Comparing the base shear under wind loads to the base shear under seismic loads, the wind loads were determined to be the controlling case. Since the site is located on the East Coast where predominantly wind controls, it is not surprising that this is the case. However, since the weight of the building was estimated based on a rough footprint area and assumed self-weights, a more accurate account for the weight will yield different results for the base shear.

Table 5: Base Shear and Overturning Moment							
Level	**Height h_x (ft)**	**Story Weight w_x (kips)**	**w_x*h_x^k**	**C_{vx}**	**Lateral Force F_i**	**Story Shear V_x (kips)**	**Moment M_x (ft-k)**
2	15	4576.23	127967.62	0.048	55.45	1110.65	831.77
3	31.5	4539.24	316158.69	0.117	137.00	973.65	4315.45
4	46.5	4437.05	498955.09	0.185	216.21	757.44	10053.68
Penthouse	61.5	4588.29	727724.93	0.270	315.34	442.10	19393.37
Roof	83.5	4416.09	1020263.34	0.379	442.10	0.00	36915.57
Total		**22556.9**	**2691069.66**		**1166.10**	**1166.10**	**71509.85**

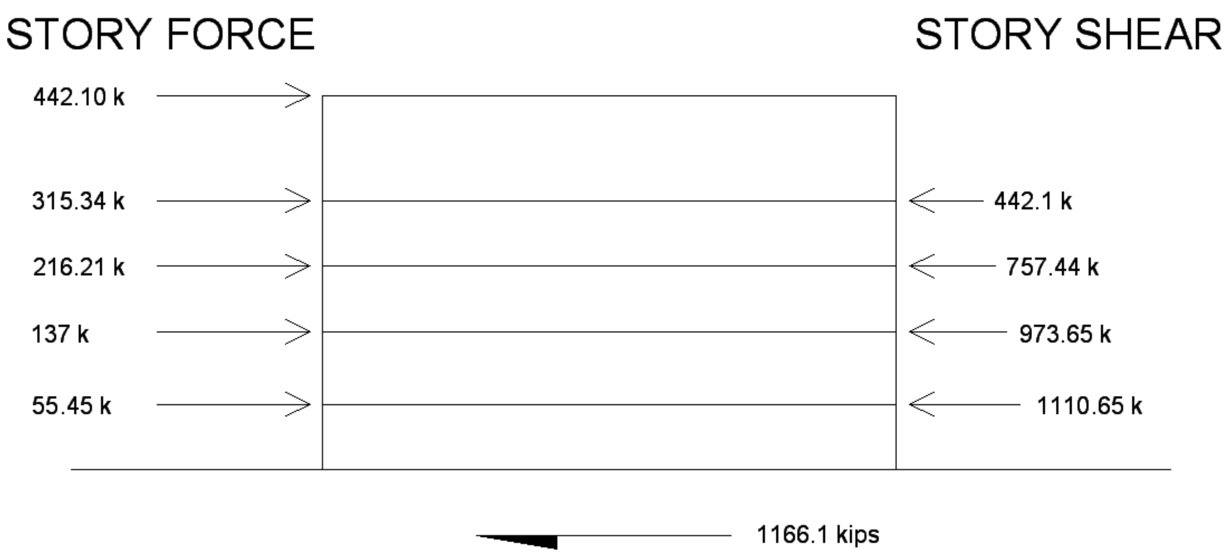


Figure 14 - Seismic Load Diagram

Snow Load Calculations

A snow load analysis was determined to provide an adequate estimate for the roof loading under seasonal conditions. Using ASCE 7-10 Chapter 7, the ground snow load was found to be 30 psf for the site. After considering the exposure of the roof, the importance factor, and thermal factor, a roof snow load of 20.79 psf was determined. Since there is a parapet that runs along the perimeter of the roof, a separate drift snow load was calculated. The maximum intensity of the drift surcharge load was calculated to be 53.34 psf increasing linearly from a distance of 16 ft away from the parapet. The calculations and a diagram showing the distribution of these snow loads along the roof level can be found in Appendix D

Spot-Checks of Typical Framing Elements

Composite Metal Deck Analysis

The typical composite slab used throughout the structural plans is a 2" deep, 20-gage composite metal deck with a 4 ½" topping thickness as noted in the "Floor System" section of this report. The metal decking of the composite slabs span perpendicular to the direction of the beams. A section of the slab was analyzed to check the adequacy of the selection to use this specific composite floor system. Figure 14 shows the slab area that was considered for the spot checks.

From the "Vulcraft Deck Catalog," a maximum unshored clear span for 3 or more spans utilizing the same composite system is 8' 4" which is greater than the given tributary width of 6' 4". Similarly, the catalog specifies a maximum superimposed live load of 385 psf which is greater than the 110 psf loading that was determined for that section. Therefore, it is apparent that the use of this composite floor system is adequate for this section. For more details and hand calculations, see Appendix E.

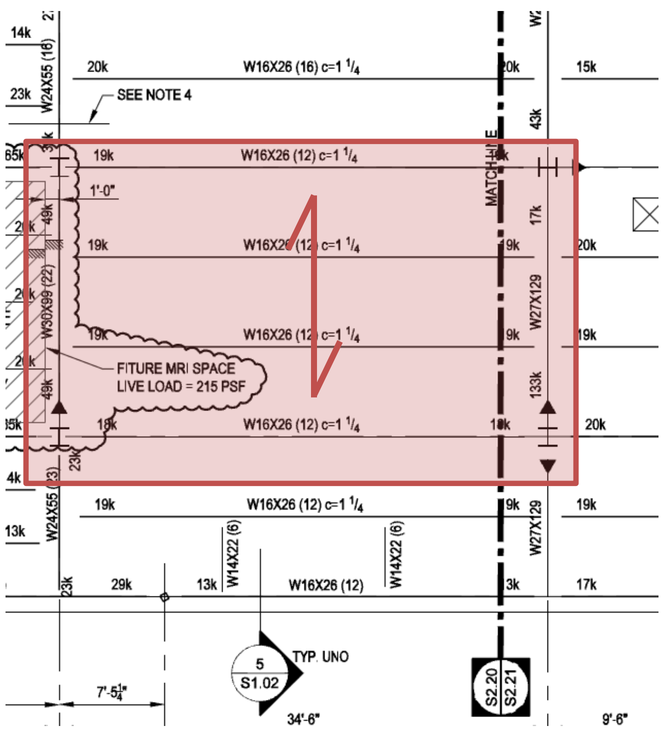


Figure 15 - Composite Slab Spot Check

Typical W16x26 Beam Analysis

Figure 15 shows the beam that was taken to be a typical representation of all the beams within the section that was checked. Since it is a composite floor system, shear studs allow interaction between the slab and the supporting member. For this composite beam, the drawings specified that 12 shear studs would be needed to provide adequate shear. As a result it was necessary to check the strength and serviceability requirements for the composite beam.

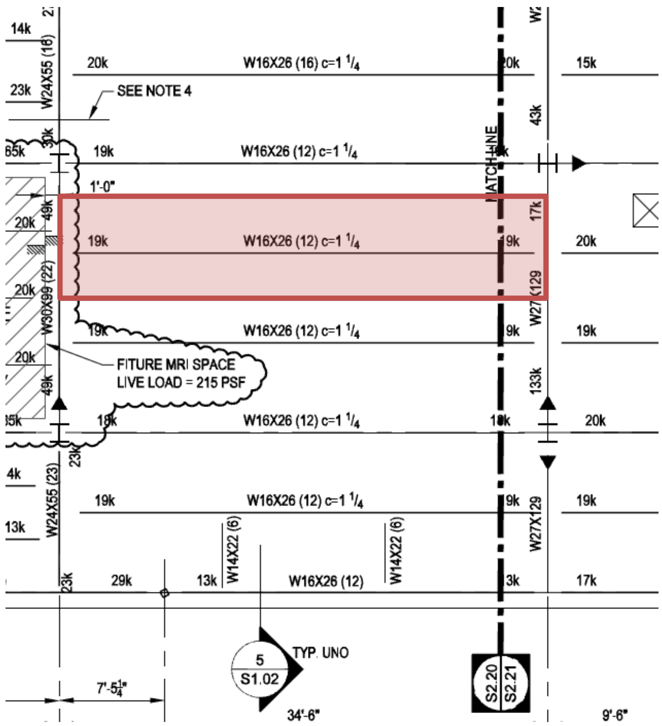


Figure 16 - Typical W16x26 Check

Hand calculations for the spot check of the typical composite beam are shown in Appendix E. The number of shear studs used in conjunction with the composite beam was confirmed to be 12 – ¾" diameter shear studs. The composite beam was determined to be adequate under strength requirements, having a moment capacity of 252 ft-kips while the moment due to service loads was calculated to be 233.6 ft-kips. While checking the deflection, the wet concrete deflection was found to exceed the allowable limit of $l/240$. To correct this issue, a camber of 1 ¼" was used to allow the composite beam to meet all serviceability requirements as specified on the structural drawings.

Girder Analysis

The analysis of the girder was determined necessary since the purpose is to transfer the floor loadings to the columns. The girder in this section was designated a W27x129. One of the complications that arose while spot checking this specific girder is that one end was moment connected as can be seen in Figure 16. In the analysis, a fixed-pinned beam was considered with two point loads at the locations where the W16x26 composite beams frame in.

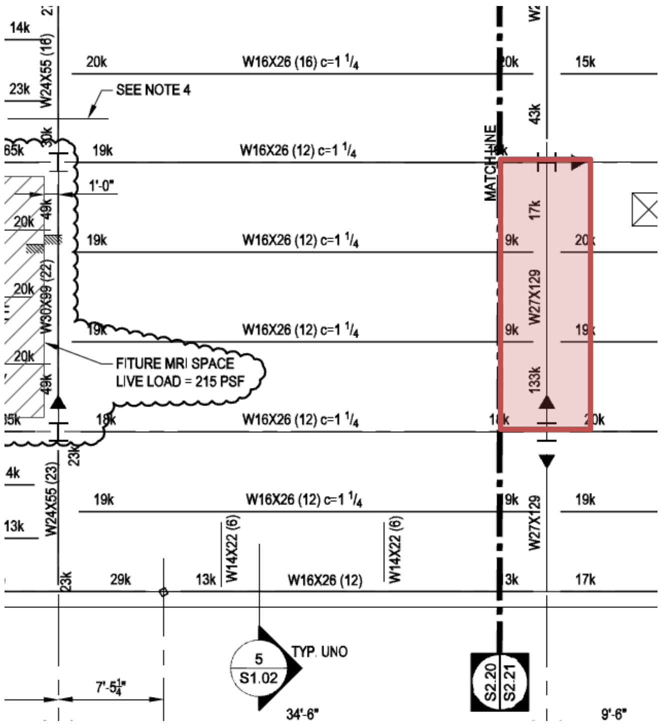


Figure 17 - W27x129 Girder Check

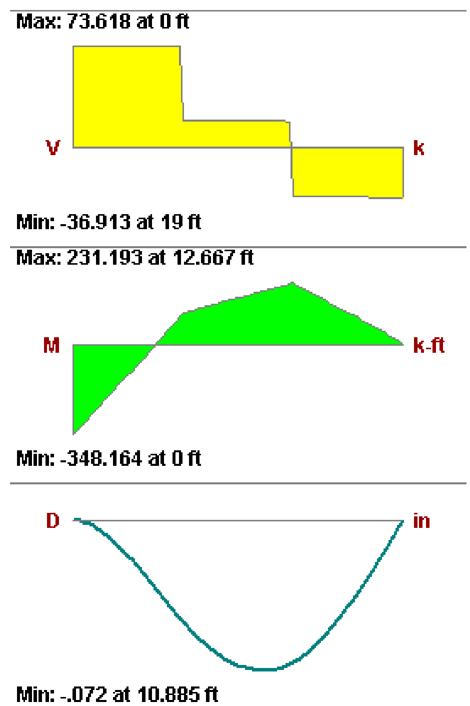


Figure 18 - RISA Analysis

Since the fixed-pinned condition is a statically indeterminate beam, for efficiency reasons, RISA-2D was used to obtain the maximum moment and deflection for the beam under the given service loads. The moment due to the loading was 348.2 ft-kips while the maximum allowable bending moment at an unbraced length of 19 ft was 1090 ft-kips. Similarly, the live load deflection was 0.072 in while the serviceability limit was 0.633 in. The large difference between the nominal and design values are most likely attributed to the moment connections on the member. The large capacity in the member is to aid the structure in resisting lateral forces rather than gravity loads.

Column Analysis

The column that is highlighted in Figure 18 is a W14x342 that is located on the second level supporting the total weight of the floors above it. The floor to floor height for level two is 16.5 ft. Table 6 shows the weight contribution of each floor above level two based on various load cases. The maximum load case was taken for each floor and then summed to obtain the total load P_u on top of the column.

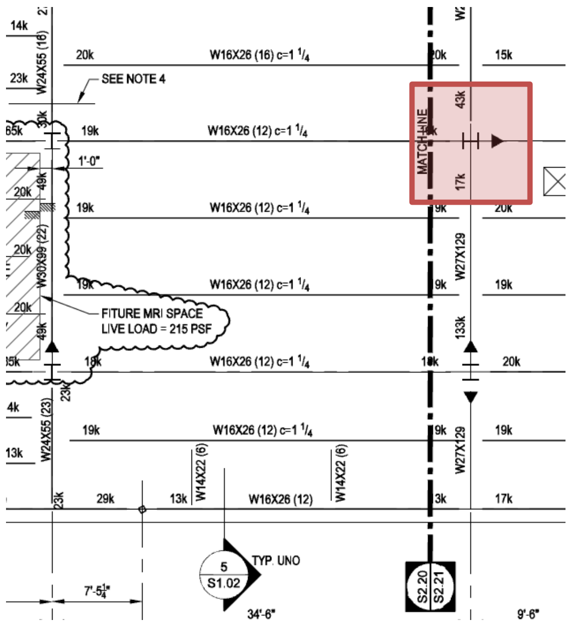


Figure 19 - W14x342 Column Check

Table 6 – Load on Level 2 Column											
Floor	**Tributary Area (ft^2)**	**DL (psf)**	**LL (psf)**	**SDL (psf)**	**Total DL (psf)**	**Snow (psf)**	**Column Weight (lbs)**	**1.4D**	**1.2D + 1.6L**	**1.2D +1.6S +0.5L**	**Total Weight (k)**
Roof	759	76.85	80	30	106.85	20.8	0	113.54	97.45	122.62	122.62
P.H.	759	76.85	80	30	106.85	0	6226	122.26	104.92	104.83	244.87
4	759	76.85	80	30	106.85	0	4245	119.48	102.54	102.45	364.36
3	759	76.85	80	30	106.85	0	5130	120.72	103.60	103.51	485.08

From Table 6, the axial load on the column was determined to be 485 kips. The axial strength capacity for a W14x342 with an effective length of 16.5 ft is 3840 kips. This large difference, similarly with the design of the girder, can be attributed to the increased strength required to act with the moment frame to transfer lateral forces into the foundation of the structure. Another possibility for such a high strength member is to support the weight of the additional floors that will be added at a later time.

Evaluations and Summary

A summary of the structural concepts and existing conditions of the Penn State Hershey Medical Center Children's Hospital can be found on page 9 of this report.

After determining the resulting base shear due to both wind and seismic responses, it was determined that the wind loading controlled in the design of the structure. A base shear of 1525.61 kips was determined for the North – South controlling lateral force. This can be verified by inspection since the length of the North and South facades is about 352 ft allowing for a larger surface area. Another factor that wind would be the controlling factor is that the site location is on the East coast where variable wind speeds are more common than seismic activity.

After performing spot checks on certain members of the structure, it was determined that components such as girders and columns were oversized. While this seems like a concern, numerous factors play into the structural designer's selection of these members. Since a detailed lateral analysis of the frame system has not yet been performed, the member sizes should account for additional wind forces.

Technical Report 2 is to follow which will focus on the pros and cons of alternative floor systems.

APPENDIX

Appendix A: Wind Calculations

MATT VANDERSALL | TECH REPORT #1 | WIND ANALYSIS | 1/3

GENERAL REQUIREMENTS

OCCUPANCY CATEGORY III
BASIC WIND SPEED, V = 120 MPH
WIND DIRECTIONALITY FACTOR $K_d = 0.85$
EXPOSURE CATEGORY = C
TOPOGRAPHIC FACTOR, $K_{zt} = 1.0$
GUST EFFECT FACTOR — CANNOT ASSUME

26.9.2.1 — LIMITATIONS FOR APPROXIMATE NATURAL FREQUENCY

1. BUILDING HEIGHT = 85.5' < 300' ∴ OK
2. BUILDING HEIGHT = 85.5' < 4 L_{eff} ?

$$L_{eff} = \frac{\sum_{i=1}^{n} h_i L_i}{\sum_{i=1}^{n} h_i}$$

WHERE h_i = HEIGHT ABOVE GRADE LEVEL i
L_i = BUILDING LENGTH AT LEVEL i PARALLEL TO THE WIND DIRECTION

FOR N-S DIRECTION

$$L_{eff} = \frac{(85.5\text{ FT})(131.3\text{ FT})}{85.5\text{ FT}} = 131.3\text{ FT}$$

$4(131.3\text{ FT}) = 525.2\text{ FT} > 85.5\text{ FT}$ ∴ OK

FOR E-W DIRECTION

$L_{eff} = 352.3\text{ FT}$

$4(352.3\text{ FT}) = 1409.2\text{ FT} > 85.5\text{ FT}$ ∴ OK

26.9.3 — APPROXIMATE NATURAL FREQUENCY

FOR STRUCTURAL STEEL MOMENT RESISTING FRAME BUILDING:

$n_a = 22.2 / h^{0.8}$ WHERE h = MEAN ROOF HEIGHT (FT)

$n_a = 22.2/(85.5\text{ FT})^{0.8}$

$n_a = 0.632 < 1.0\text{ Hz} \longrightarrow$ FLEXIBLE

26.9.5 — FLEXIBLE BUILDINGS

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

FOR N-S WIND:

$g_Q = g_v = 3.4$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}} = 4.08$$

$n_1 = n_a = 0.632$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)} = 0.418$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(2.82)}{(1 + 10.3(2.82))^{5/3}} = 0.0726$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{(0.632)(546.12)}{122.43} = 2.82$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 500\left(\frac{0.6(85.5')}{33}\right)^{1/5} = 546.12$$

MATT VANDERSALL | TECH REPORT #1 | WIND ANALYSIS | 2/3

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)V$$
$$= 0.65\left(\frac{0.6(85.5)}{33}\right)^{1/6.5}\left(\frac{88}{60}\right)(120) = 122.43$$

R_h: $\eta = \dfrac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \dfrac{4.6(0.632)(85.5)}{122.43} = 2.03 > 0$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) = \frac{1}{2.03} - \frac{1}{2(2.03)^2}\left(1 - e^{-2(2.03)}\right)$$

$R_h = 0.373$

R_B: $\eta = \dfrac{4.6 n_1 B}{\bar{V}_{\bar{z}}} = \dfrac{4.6(0.632)(352.3)}{122.43} = 8.37 > 0$

$$R_B = \frac{1}{8.37} - \frac{1}{2(8.37)^2}\left(1 - e^{-2(8.37)}\right) = 0.11$$

R_L: $\eta = \dfrac{15.4 n_1 L}{\bar{V}_{\bar{z}}} = \dfrac{15.4(0.632)(131.3)}{122.43} = 10.44 > 0$

$$R_L = \frac{1}{10.44} - \frac{1}{2(10.44)^2}\left(1 - e^{-2(10.44)}\right) = 0.089$$

ASSUME $\beta = 0.01$ FOR STEEL BUILDINGS

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = 0.804$$

$$I_{\bar{z}} = c\left(\frac{10}{\bar{z}}\right)^{1/6} = 0.2\left(\frac{10}{0.6(85.5)}\right)^{1/6} = 0.152$$

$\therefore$ FOR N-S WIND, $G_f = 0.899$

(NOTE: SEE SPREADSHEET FOR E-W WIND GUST FACTOR)

ENCLOSURE CLASSIFICATION: ENCLOSED

INTERNAL PRESSURE COEFFICIENT, $GC_{pi} = \pm 0.18$

VELOCITY PRESSURE EXPOSURE COEFFICIENT, K_z OR K_h: SEE SPREADSHEET

VELOCITY PRESSURE q_z OR q_h: $q_z = 0.00256\, K_z\, K_{zt}\, K_d\, V^2$ (lb/ft^2)

EXTERNAL PRESSURE COEFFICIENT, C_p OR C_N

N-S WIND: WALL —

SURFACE	L/B	C_p	USE WITH
WINDWARD	ALL	0.8	q_z
LEEWARD	0.37	-0.5	q_h
SIDE	ALL	-0.7	q_h

E-W WIND: WALL —

SURFACE	L/B	C_p	USE WITH
WINDWARD	ALL	0.8	q_z
LEEWARD	2.68	-0.27	q_h
SIDE	ALL	-0.7	q_h

MATT VANDERSALL | TECH REPORT #1 | WIND ANALYSIS | 3/3

WIND PRESSURE FOR ENCLOSED FLEXIBLE BUILDING:

$$p = qG_fC_p - q_i(GC_{pi}) \quad (lb/ft^2)$$

FOR PARAPET:

$$P_p = q_p(GC_{pn}) \quad (lb/ft^2) \qquad GC_{pn} = +1.5, \; -1.0$$

WINDWARD $\longrightarrow P_p = 38.54(1.5) = 57.81$ psf

LEEWARD $\longrightarrow P_p = 38.54(-1.0) = -38.54$ psf

DESIGN WIND PRESSURES:

N-S WIND —

WINDWARD, $p = q_z(0.902)(0.8) - (38.54)(\pm 0.18)$

$p = 0.722\, q_z + 6.94$

ADD $P_p = 57.81$ psf TO PARAPET

LEEWARD, $p = (38.54)(0.902)(-0.5) - (38.54)(\pm 0.18)$

$p = -24.32$ psf

ADD $P_p = -38.54$ psf TO PARAPET

E-W WIND —

WINDWARD, $p = q_z(0.788)(0.8) - (38.54)(\pm 0.18)$

$p = 0.79\, q_z + 6.94$

ADD $P_p = 57.81$ psf TO PARAPET

LEEWARD, $p = (38.54)(0.788)(-0.27) - (38.54)(\pm 0.18)$

$p = -17.22$ psf

ADD $P_p = -38.54$ psf TO PARAPET

Appendix B: Total Weight

MATT VANDERSALL | TECH REPORT #1 | TOTAL WEIGHT | 1/1

THE WEIGHT OF THE STRUCTURE WILL INCLUDE:
- FRAMING MEMBERS
- COLUMNS
- COMPOSITE FLOOR SLAB LOAD
- COLLATERAL LOAD — CEILINGS, INSULATION, HVAC DUCTS, L/E, FIRE PROTECTION, ETC.
- FACADE

FRAMING MEMBERS (BEAMS AND GIRDERS)
- WEIGHTS WERE CALCULATED FROM REVIT STRUCTURE MODEL

FIRST FLOOR = 0 ← SINCE AT GROUND LEVEL
SECOND FLOOR = 473.68 K
THIRD FLOOR = 427.19 K
FOURTH FLOOR = 347.16 K
PENTHOUSE = 441.68 K
PENTHOUSE ROOF = 391.01 K

COLUMNS
- TOTAL COLUMN WEIGHT CALCULATED FROM MODEL

TOTAL WEIGHT = (77.42 K/FT)/(5 FLOORS) = 15.484 K/FT PER FLOOR
FIRST FLOOR = (15.484 K/FT)(15 FT) = 233.26 K
SECOND FLOOR = (15.484 K/FT)(16.5 FT) = 255.49 K
THIRD FLOOR = 232.26 K
FOURTH FLOOR = 232.26 K
PENT HOUSE = (15.484 K/FT)(22 FT) = 340.65 K
PENT HOUSE ROOF = 0

SLAB WEIGHT
- FROM VULCRAFT CAT. – 2" DEEP 20 GAGE w/ 4 1/2" SLAB → 69 PSF
- ALL FLOORS ASSUMED TO HAVE SAME AREA = 38856.99 FT^2

SLAB WEIGHT FOR FLOORS 2 – P.H. ROOF = $(38856.99\ FT^2)(69\ PSF) = 2681.13\ K$

COLLATERAL LOAD
- FROM DRAWINGS, COLLATERAL LOAD = 30 PSF

LOAD ON FLOORS 2 – P.H. ROOF = $(38856.99\ FT^2)(30\ PSF) = 1165.7\ K$

FACADE
- ALUMINUM CLADDING FOR FACADE = 0.75 PSF

PERIMETER WEIGHT OF FACADE = (0.75 PSF)(960 FT) = 720 lb/FT
FIRST FLOOR = (720 lb/FT)(15 FT) = 10.8 K
SECOND FLOOR = (720 lb/FT)(16.5 FT) = 11.88 K
THIRD FLOOR = 10.8 K
FOURTH FLOOR = 10.8 K
PENT HOUSE = (720 lb/FT)(22 FT) = 15.84 K
PENTHOUSE ROOF = 0

$\boxed{\text{TOTAL BUILDING WEIGHT} = 25350\ K}$

Appendix C: Seismic Calculations

MATT VANDERSALL | TECH REPORT #1 | SEISMIC ANALYSIS | 1/1

SEISMIC USE GROUP : IV
SITE CLASS : D

SPECTRAL RESPONSE ACCEL. SHORT, $S_s = 0.207g$ (USGS)
SPECTRAL RESPONSE ACCEL. LONG, $S_1 = 0.055g$ (USGS)
SITE COEFFICIENT, $F_a = 1.6$
SITE COEFFICIENT, $F_v = 2.4$

SOIL MODIFIED ACCEL, $S_{ms} = F_a S_s$
$S_{ms} = (1.6)(0.207) = 0.3312g$
SOIL MODIFIED ACCEL, $S_{m1} = F_v S_1$
$S_{m1} = (2.4)(0.055) = 0.132g$
DESIGN SPECTRAL RESPONSE, SHORT, $S_{DS} = 2/3\, S_{ms}$
$S_{DS} = 2/3\,(0.3312) = 0.221g$
DESIGN SPECTRAL RESPONSE, 1SEC., $S_{D1} = 2/3\, S_{m1}$
$S_{D1} = 2/3\,(0.132) = 0.088g$

RESPONSE MODIFICATION FACTOR, $R = 3$ (TABLE 12.2-1: STEEL AND CONCRETE COMPOSITE ORDINARY MOMENT FRAMES)

IMPORTANCE FACTOR, $I_e = 1.50$
SEISMIC DESIGN CATEGORY : C (TABLE 11.6-1 FOR RISK CATEGORY IV :
$0.167 \le S_{DS} < 0.33 \longrightarrow C$
$0.067 \le S_{D1} < 0.133 \longrightarrow C$)

APPROX. PERIOD PARAMETER : $\left.\begin{array}{l} C_T = 0.02 \\ x = 0.75 \end{array}\right\}$ (TABLE 12.8-2)

STRUCTURAL HEIGHT : $h_n = 85.5$ FT

APPROX. FUNDAMENTAL PERIOD : $T_a = C_T h_n^x$
$T_a = 0.02(85.5)^{0.75} = 0.562$
COEFFICIENT FOR UPPER LIMIT ON CALCULATED PERIOD : $C_u = 1.7$ (TABLE 12.8-1)
FUNDAMENTAL PERIOD : $T = C_u T_a$
$T = (1.7)(0.562) = 0.955$

CHECK LONG-PERIOD TRANSITION PERIOD : $T_L = 6$ sec.
SEISMIC RESPONSE COEFFICIENT :

$$C_s = \min \left| \begin{array}{l} S_{DS}/(R/I) = 0.221/(3/1.5) = 0.111 \\ \text{FOR } T \le T_L:\ S_{D1}/[T \cdot R/I] = 0.088/[0.955 \cdot 3/1.5] = \boxed{0.046} \end{array} \right.$$

(NOTE: C_s SHALL NOT BE LESS THAN 0.01)

BASE SHEAR : $V_b = C_s \cdot W = (0.046)(25350\ k)$
$\boxed{V_b = 1166.1\ k}$

STRUCTURAL PERIOD EXPONENT : $0.5s < T = 0.955s < 2.5s \longrightarrow$ INTERPOLATE FOR k
$k = 1.23$

Appendix D: Snow Calculations

MATT VANDERSALL	TECH REPORT #1	SNOW ANALYSIS	1/1

ASCE 7-10 : 7.3 – FLAT ROOF SNOW LOADS, p_f

$$p_f = 0.7 C_e C_t I_s p_g$$

$C_e = 0.9$ FOR TERRAIN CATEGORY C AND FULLY EXPOSED ROOF

$C_t = 1.0$

$I_s = 1.1$ FOR RISK CATEGORY III

$p_g = 30$ PSF

$$p_f = 0.7(0.9)(1.0)(1.1)(30\text{ PSF}) = 20.79\text{ PSF}$$

DRIFT SNOW LOAD – ACCUMULATES AGAINST PARAPET

$h_d = 0.75[0.43(\ell_u)^{1/3}(p_g+10)^{1/4} - 1.5]$ WHERE $\ell_u = 129.3$ FT

$= 0.75[0.43(129.3\text{ FT})^{1/3}(30+10)^{1/4} - 1.5]$

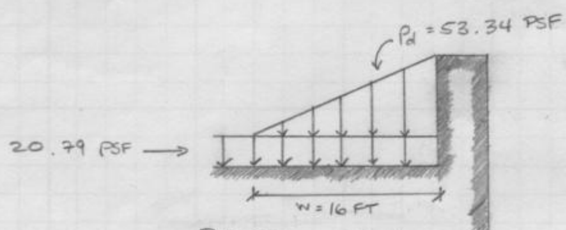
$= 2.98\text{ FT} > h_c = 2\text{ FT}$

$\therefore\ w = 4h_d^2/h_c = 4(2.98\text{ FT})^2/2\text{FT} = 17.8\text{ FT}$

W SHALL NOT EXCEED $8h_c = 8(2\text{FT}) = 16\text{ FT}$

$\gamma = 0.13p_g + 14 = 0.13(30\text{ PSF}) + 14 = 17.9$ PCF

MAX INTENSITY OF DRIFT SURCHARGE LOAD : $p_d = h_d\gamma = (2.98\text{ FT})(17.9\text{ PCF}) = 53.34$ PSF



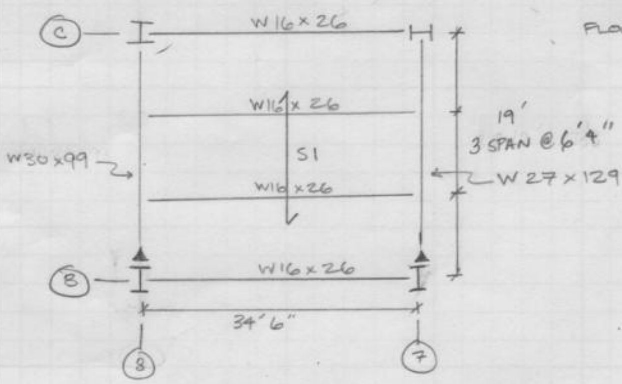
PARAPET ELEVATION

Appendix E: Spot Checks

MATT VANDERSALL	TECH REPORT #1	SPOT CHECKS	1/5

① COMPOSITE METAL DECK: LEVEL 2

• SLAB TYPE S1 → DECK DEPTH: 2"
GAGE: 20
TOPPING THICKNESS: 4½"
f'_c = 4000 psi, WEIGHT = 145 PCF



FLOOR LOADING:
S.D.L = 30 PSF
L.L. = 80 PSF
TOTAL LOAD = 110 PSF

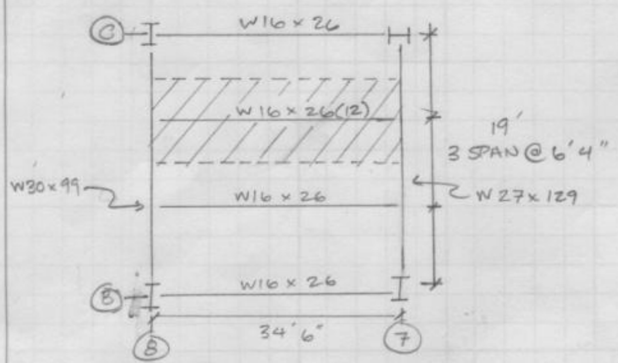
FROM VULCRAFT DECK CATALOG — 2VL120, 6½" TOTAL SLAB DEPTH

SDI MAX UNSHORED CLEAR SPAN
6½" SLAB, 3 OR MORE SPANS
↳ 8'4" > 6'4" ∴ OK

SUPERIMPOSED LIVE LOAD
6½" SLAB, 6'6" CLEAR SPAN
↳ 385 PSF >> 110 PSF ∴ OK

MATT VANDERSALL | TECH REPORT #1 | SPOT CHECKS | 2/5

(2) TYPICAL BEAM CHECK : LEVEL 2



W16 x 26
DECK / TOPPING = 2" DECK + 4 1/2" TOPPING
12 - 3/4" ϕ SHEAR STUDS
BEAM CAMBER = 1 1/4"

DEAD LOAD — SLAB WT. = 69 PSF
S.D.L = 30 PSF
SELF-WT = 26 lb/FT

LIVE LOAD — 80 PSF < 100 PSF ∴ CHECK L.L. REDUCTION

LL REDUCTION: $K_{LL} A_T = (2)(6'4")(34'6") =$
$= 437 \text{ FT}^2 > 400$ ∴ CAN PERFORM L.L. REDUCTION

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

$$L = 80 \text{ PSF}\left(0.25 + \frac{15}{\sqrt{437}}\right) = \underline{77.4 \text{ PSF}}$$

TOTAL LOADS:
D.L. = (69 PSF)(6'4") + (30 PSF)(6'4") + 26 plf = 653 plf
L.L. = (77.4 PSF)(6'4") = 490.2 plf

CONSTRUCTION LOADS:
UNSHORED D.L. = (69 PSF)(6'4") + 26 plf = 463 plf
UNSHORED L.L. = (20 PSF)(6'4") = 126.67 plf

LOAD COMBINATION: 1.2D + 1.6L ⟶ ASSUME WILL CONTROL FOR GRAVITY ANALYSIS

$$w_u = 1.2(653 \text{ plf}) + 1.6(490.2 \text{ plf})$$
$$= 1.57 \text{ klf}$$

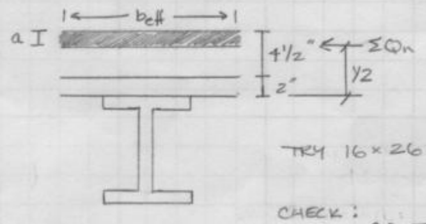
EFFECTIVE FLANGE WIDTH:

$$b_{eff} = \min \begin{cases} 2 \times \frac{SPAN}{8} = \frac{2(34'6")(\frac{12 \text{ IN}}{1 \text{ FT}})}{8} = 103.5 \text{ IN} \\ 2 \times \frac{1}{2}SPACING = 2\left[\frac{1}{2}(6'4")(12 \text{ IN/FT})\right] = 76 \text{ IN} \leftarrow \text{CONTROLS} \end{cases}$$

$$V_u = \frac{w_u l}{2} = \frac{(1.57 \text{ klf})(34.5 \text{ FT})}{2} = 27.1 \text{ K}$$

$$M_u = \frac{w_u l^2}{8} = \frac{(1.57 \text{ klf})(34.5 \text{ FT})^2}{8} = 233.6 \text{ FT}\cdot\text{K}$$

MATT VANDERSALL | TECH REPORT # 1 | SPOT CHECKS | 3/5



ASSUME $a \cong 1"$

$Y2 = t - a/2 = 6.5" - 0.5" = 6"$

$Q_n = 17.2\ k$ FOR 1 STUD/RIB, 4 KSI, NWC

TRY 16×26 — $\phi_b M_p = 166$ FT·K, PNA = 7

$\phi M_n = 252$ FT·K, $\Sigma Q_n = 96$ K

CHECK: $a = \dfrac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \dfrac{96\,K}{0.85(4\,KSI)(76\,IN)} = 0.37" < 1"$ ∴ OK

$Y2 = t - a/2 = 6.5" - \frac{0.37}{2} = 6.32" > 6"$ ∴ OK

NUMBER OF SHEAR STUDS: $\dfrac{96\,K}{17.2\,K/STUD} = 5.58 \longrightarrow$ 12 STUDS

CHECK UNSHORED STRENGTH:

$\omega_{DL} = 0.463$ KLF

$\omega_{LL} = 0.127$ KLF

LOAD COMBINATIONS: ① $\omega_u = 1.4\,\omega_{DL}$

$= 1.4(0.463\,KLF) = 0.648$ KLF

② $\omega_u = 1.2\,\omega_{DL} + 1.6\,\omega_{LL}$

$= 1.2(0.463\,KLF) + 1.6(0.127\,KLF) = 0.759$ KLF

$M_u = \dfrac{(0.759\,KLF)(36.5\,FT)^2}{8} = 126.4$ FT·K $< \phi_b M_p = 166$ FT·K ∴ OK

CHECK STRENGTH: $\phi M_n = 252$ FT·K > 233.6 FT·K ∴ OK

CHECK L.L. DEFLECTION:

$\omega_{LL} = (77.4\,PSF)(6'4") = 0.49$ KLF

$I_{LB} = 595\ IN^4$

CAMBER

$\Delta_{LL} = \dfrac{5 \omega l^4}{384 EI} = \dfrac{5(0.49\,KLF)(36.5\,FT)^4(1728)}{384(29000\,KSI)(595\,IN^4)} = 1.13\,IN - 1.25\,IN = -0.12 \longrightarrow 0$

$l/360 = (36.5\,FT)(12\,IN/FT)/360 = 1.21\,IN > 0$ ∴ OK

CHECK WET CONCRETE DEFLECTION:

$\omega_{WC} = (69\,PSF)(6'4") + 26\,PLF = 0.463$ KLF

$I_x = 301\ IN^4$

CAMBER

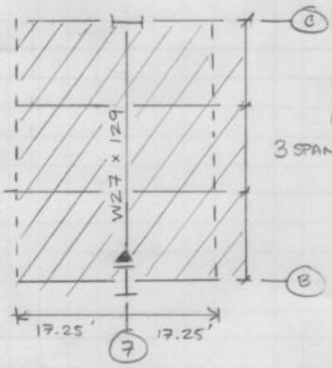
$\Delta_{WC} = \dfrac{5 \omega l^4}{384 EI} = \dfrac{5(0.463\,KLF)(36.5\,FT)^4(1728)}{384(29000\,KSI)(301\,IN^4)} = 2.12\,IN - 1.25\,IN = 0.87\,IN$

$\Delta_{WC\,MAX} = l/240 = (36.5\,FT)(12)/240 = 1.825\,IN > 0.87\,IN$ ∴ OK

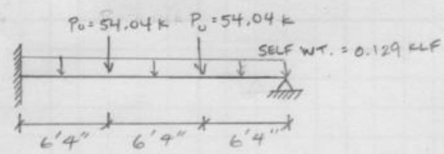
$\boxed{\text{USE W16×26 (12) C = 1 1/4"}}$

MATT VANDERSALL	TECH REPORT #1	SPOT CHECKS	4/5

③ GIRDER SPOT CHECK : LEVEL 2



GIRDER FBD:



TOTAL LOADS: DL = 0.653 KLF (17.25 FT)(2) = 22.5 k
LL = 0.49 KLF (17.25 FT)(2) = 16.9 k

GIRDER SELF WT.

CONSTRUCTION LOADS: D.L. = (0.463 KLF)(17.25 FT)(2) + (0.129 KLF)(19 FT) = 18.42 k
L.L. = (0.127 KLF)(17.25 FT)(2) = 4.38 k

$P_u = 1.2DL + 1.6LL$
$= 1.2(22.5k) + 1.6(16.9k) = 54.04$ k

FROM RISA ANALYSIS:

$M_u = 348.09$ FT.k

FROM STEEL MANUAL:

W27 x 129 FOR $l_u = 19$ FT $\rightarrow \phi M_n = 1090$ FT k > 348.09 FT k ∴ OK

LIVE LOAD DEFLECTION: $P_u = 16.9$ k

$\Delta_{LL} = 0.022$ IN

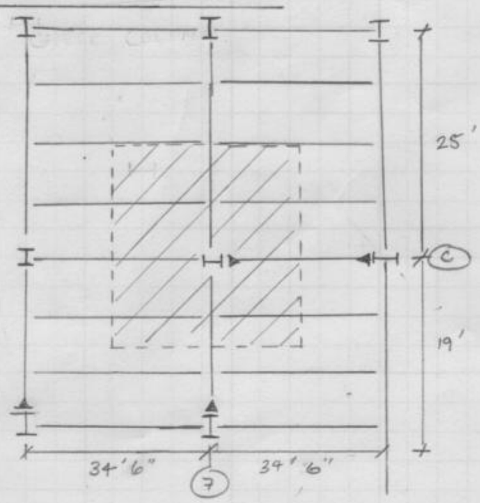
$l/360 = (19\text{ FT})(12\frac{\text{IN}}{\text{FT}})/360 = 0.633 > 0.022\text{IN}$ ∴ OK

$\boxed{\text{USE W27 x 129}}$

NOTE: NO SHEAR STUDS WERE INDICATED IN STRUCTURAL PLANS, THEREFORE GIRDER AND COMPOSITE FLOOR SLAB ACT INDEPENDENTLY

MATT VANDERSALL | TECH REPORT #1 | SPOT CHECKS | 5/5

(4) COLUMN SPOT CHECK : LEVEL 2



$A_t = \left(\frac{25' + 19'}{2}\right)\left(\frac{34'6" + 34'6"}{2}\right)$

$= 759\ FT^2$

LOAD AND AREA CALCULATIONS
SEE EXCEL

W 14 x 342

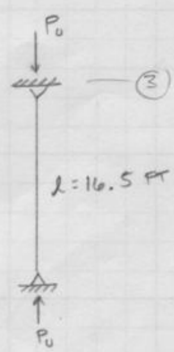
DEAD LOAD

SLAB = 69 PSF

BEAM = 26 PLF / 6'4" = 4.11 PSF

GIRDER = 129 PLF / 34'6" = 3.74 PSF

TOTAL D.L = 76.85 PSF



$P_u = 485.1^k$ FROM EXCEL

$k = 1.0$

AXIAL STRENGTH FOR W14 x 342 :

$\phi P_n = 3840\ K \gg 485.1^k \therefore$ OK

$\boxed{\text{USE W14 x 342}}$