2009-2010 AE Senior Thesis

Technical Report I

Structural Concepts and Existing Structural Conditions of University Medical Center at Princeton

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Executive Summary

The structural system of the New Hospital at the University Medical Center at Princeton is basic steel frame construction with a composite floor system and concrete spread footings. Gravity loads are received by the composite beams and composite girders and eventually delivered to the foundation through the columns. Lateral loads are transferred to brace and moment frames through a rigid composite floor diaphragm. The footings underneath the brace frames require mini-piles driven into bedrock in order to properly resist the tension force in the frame. The footings underneath the moment frames are large enough to resist tension forces as well as overturning moments.

Spot checks of gravity elements (beam, girder, column, etc.) have shown that member sizes are well within the limitations set forth in the International Building Code. However, lateral forces have a significant impact on member design. That particular analysis is outside the scope of this report but will be thoroughly evaluated at a later time.

Introduction

The University Medical Center at Princeton is a new state-of-the-art medical facility currently under construction in Plainsboro, NJ. The project consists of a Central Utility Plant, a Diagnostic and Treatment Center (D&T) and a New Hospital. The site already has an existing building (Building #2) and it will be connected to the north side of the New Hospital as part of the project. The Medical Office Building (MOB) is only proposed at this time. The 800,000 square foot complex is set to be complete by the summer of 2010.

The scope of this thesis project will be limited to structural analysis and re-design of the New Hospital. (Figure 1) This is the tallest portion of the complex at 92'-0" from grade to roof with a 14'-0" metal panel system above for a total height of 106'-0" above grade.

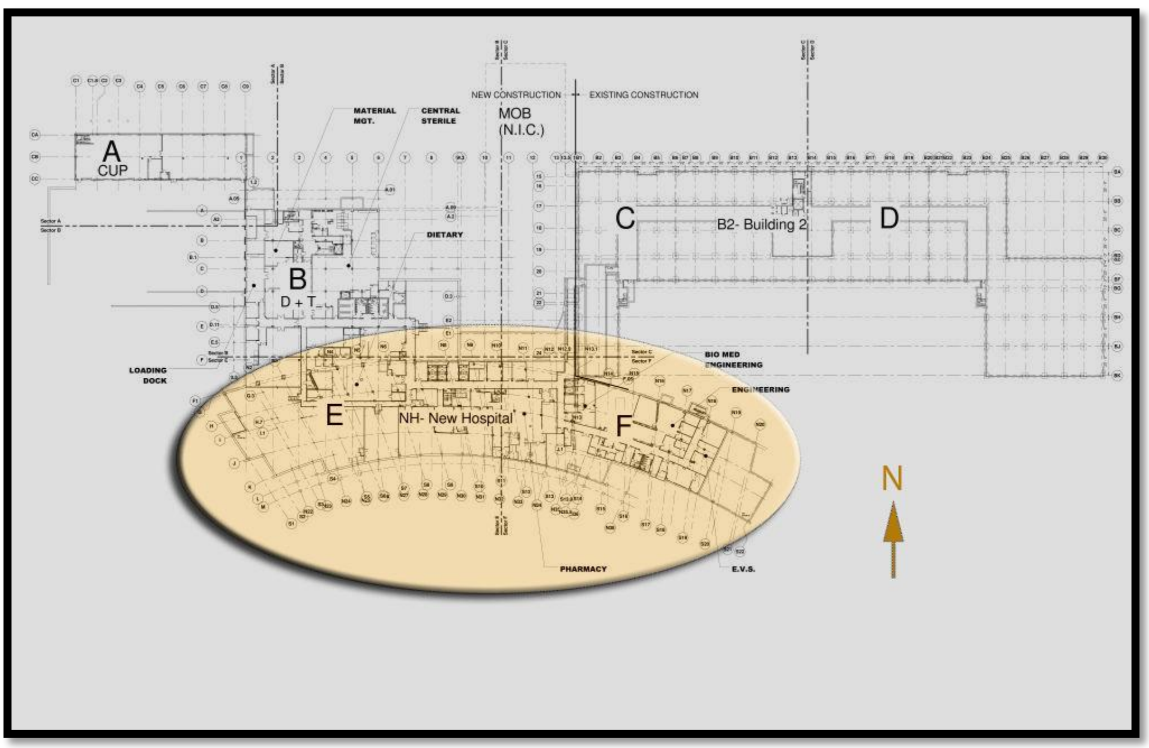


Figure 1: Overall Plan University Medical Center at Princeton

Structural System Overview

The structural system of the New Hospital at the University Medical Center was designed by O'Donnell & Naccarato Structural Engineers using a Load Resistance Factor Design approach. It is a structural steel building with a composite floor diaphragm. Braced frames run in both directions and there are two long moment frames spanning the entire length of the building on both the south and north facades. (Figure 12) Both the braced and moment frames are the building's main resistance to lateral load. Due to the great length of the building in the west-east direction, an expansion joint was placed at a distance from the western façade roughly equal to 2/3 of the total building length. This effectively splits the building into two different structures which behave on their own.

Foundation

Concrete piers with sizes anywhere from 18" x 18" to 48" x 78" are attached to the base of the steel columns and transmit vertical load from the superstructure to the concrete spread footings. The size of these footings varies from as small as 3'-0" x 3'-0" x 14" to as large as 21' x 21' x 50".

All footings supporting braced frame columns have mini-piles attached at their base in order to help with the high tension forces resulting from lateral loading. These piles extend to decomposed bedrock (8'-30' deep) and provide a tensile capacity of up to 150 kips. The top of all exterior footings are at a minimum depth of 42" below grade.

The floor at the base level is concrete slab-on-grade with thicknesses from 4"-12".

Huge concrete retaining walls with footings up to 17'-0" wide trace the perimeter of the foundation system.

Superstructure

The structural steel provides both gravity and lateral load resistance for the building. Columns are typically W14 while beams and girders range from W12-W27 shapes. Rectangular HSS shapes are used for the diagonal members in the braced frames and round HSS columns support the massive glass façade on the south face of the hospital. The HSS columns are intentionally exposed for architectural purposes. The floor layout is uniform and has a typical bay size of 30' x 30'. (Figure 2)

Figure 2: Typical bay size (30' x 30')

The floor system spanning over the main area of the building is composite construction. Typically, the concrete slab is 3-1/4" lightweight concrete poured over a 3" composite metal deck. In certain mechanical and roof areas, the floor system switches to a 6-1/2" normal weight concrete due to higher loads in those areas.

The composite floor is considered to act as a rigid diaphragm and therefore able to transmit lateral forces from the façade to the braced frames. There are six braced frames in the N-S direction for each wing of the hospital. In the W-E direction, there are four braced frames and two long moment frames on the north and south sides of the building. All of these frames contribute to the lateral force resisting system.

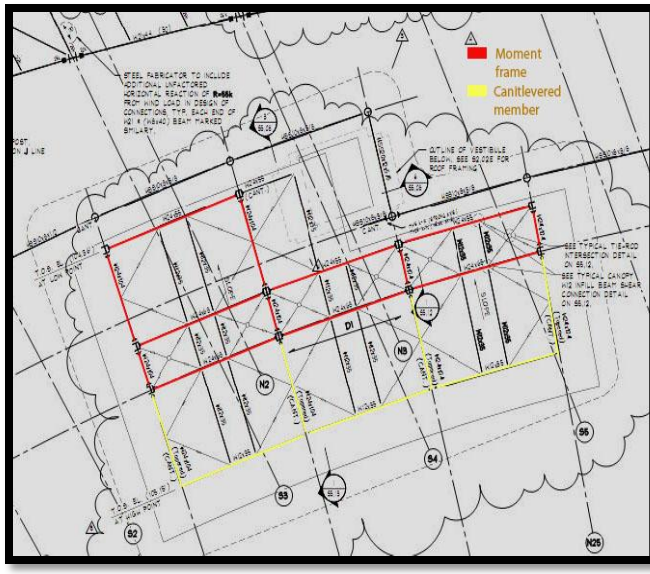


Figure 3: Canopy framing plan

The main entrance to the New Hospital is located on the south façade. Above this entrance is a canopy which is at the second floor elevation at the western and eastern ends and extends to the third floor elevation in the middle. The end canopies are separate from the main structure and are designed to resist gravity and lateral loading. (Figure 3 above) The end of the canopy cantilevers out approximately 16'-0". (Figure 4 below)

Four braced frames (two for each structure) run parallel with the expansion joint and these frames transmit load into a 6'-0" mat foundation, the only mat foundation in the entire substructure.

While the southern façade is entirely glass curtain wall, the other three facades do utilize masonry materials. These are non-load bearing walls but the masonry is designed to carry its own weight and to transmit lateral forces to the floor diaphragm.

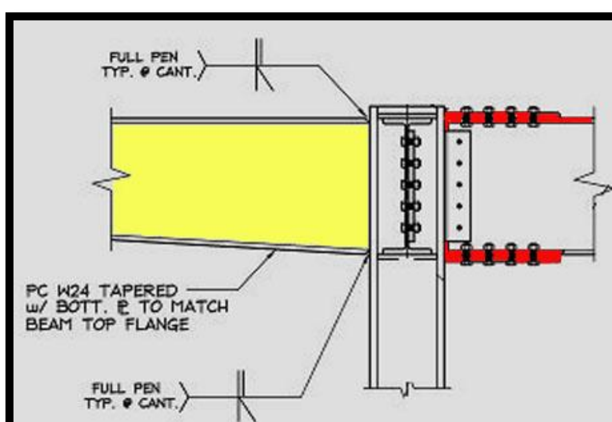


Figure 4: Canopy framing section @ cantilever

Materials

Concrete	
Footings	f'c = 3000 psi
Retaining walls	f'c = 3000 psi
Foundation walls	f'c = 3000 psi
Piers	Min. of f'c = 3000 psi
Slab on grade	f'c = 3500 psi
Slab on metal deck	f'c = 4000 psi
Lightweight concrete	f'c = 3500 psi
Structural Steel	
Wide Flange Shapes	ASTM A992
Rectangular/Square HSS Shapes	ASTM A500 Grade B
Steel Pipe Sections	ASTM A501 or ASTM A53, Type E or S, Grade B
Angles	ASTM A36
Plates	ASTM A36
¾" Bolts	A325 or A490
Anchor Rods	ASTM F1554 Grade 55
Welding Electrode	E70XX
Reinforcement	
Reinforcing bars	ASTM A615 Grade 60
Welded Wire Fabric	ASTM A185
Decking	
Roof deck	1-1/2" Galvanized Type B Metal Deck, 22 Ga.
Floor deck	3" LOK-Floor Composite Metal Deck, 20 or 18 Ga.
¾" Shear Studs	ASTM A108
Masonry	
Solid Units	ASTM C90, f'c = 1900 psi
Hollow Units	ASTM C90, f'c = 1900 psi
Ivany Units	f'c = 3000 psi
Grout	f'c = 3000 psi
Brick	ASTM C216 Grade SW, f'c = 3000 psi

Figure 5: Structural materials and material strengths

Applicable Codes

All codes used in the structural design of the New Hospital are listed below.

Model Codes:

New Jersey Uniform Construction Code (NJUCC, NJAC 5:23)

2006 *International Building Code (New Jersey Edition)*

Design Codes:

ACI 318-08 Building Code Requirements for Structural Concrete

AISC Steel Construction Manual

Structural Standards:

ASCE7-05

Design Loads

Live loads were obtained from ASCE7-05 and are considered to be the absolute minimum design loads allowed for a hospital. (Figure 6) However, because this facility is a hospital it is likely the designer used higher live load values in order to have a safer design. Most of the dead loads are assumed based upon standard industry practice. (Figure 7) For a preliminary analysis such as this, these assumptions are practical. The weight of lightweight and normal weight concrete was calculated and is considered to be accurate. This calculation can be found in Appendix C.

Live Loads	
First Floor Corridors	100 psf
Lobbies	100 psf
Corridors above First Floor	80 psf
Patient Rooms	40 psf
Operating Rooms	60 psf
Roof	20 psf
Penthouse Floor	100 psf
Offices	50 psf
Stairs	100 psf

Figure 6: Live loads per ASCE7-05

Dead Loads	
Superimposed	
Partitions	20 psf
MEP	8 psf
Ceiling	5 psf
Total	33 psf
Typical Floor	
3" metal deck	3 psf
3-1/4" LW concrete	48 psf
Allowance for steel framing	5 psf
Total	56 psf
Mechanical Roof	
3" metal deck	3 psf
6-1/2" NW concrete	100 psf
Allowance for steel framing	7 psf
Total	110 psf
Hospital Roof	
3" metal deck	3 psf
6-1/2" NW concrete	100 psf
Allowance for steel framing	6 psf
MEP	20 psf
Total	129 psf
Walls	
Curtain wall	25 psf

Figure 7: Assumed dead loads

Gravity System

Vertical loads are transmitted directly to the composite beams (12-16" deep). These beams carry the load to the composite girders (18-30" deep) which then transfer the force to nearby columns through a partially restrained (PR) moment connection. (Figure 8) This type of connection allows for rotation of the beam under gravity loads. Therefore, no moments from gravity loading are restrained by this connection and the beam is designed as simply supported.

It is important to note that while this connection does not deliver moment to the column under gravity load, it does behave as a fully restrained (FR) moment connection when subjected to lateral loading. This will be discussed later in the report.

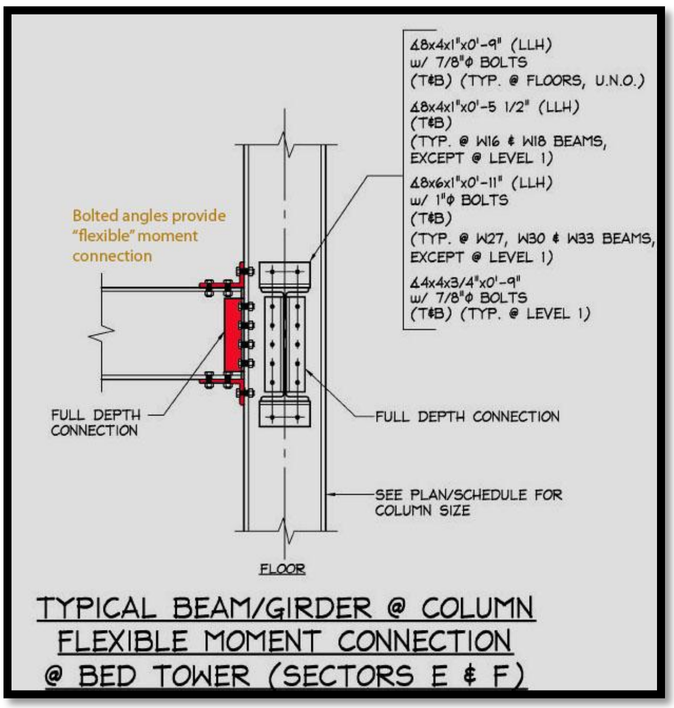


Figure 8: Typical PR Moment Connection

The PR moment connections only exist on the north and south ends of the hospital. Typical shear connections are located at all other beam/column intersections. This means that the beams and girders are handling all of the moment resulting from dead and live loading.

The typical column size for the New Hospital is a W14. There are six floors in the building so columns are spliced at the third and fifth levels. (Figure 9) Most of the splices involve columns with the same depth. This makes the erection much simpler even though certain upper level columns may be overdesigned.

The vertical force from the girder is transferred through the column and into a concrete pier which sits directly above the spread footing. (Figure 10) The larger pier sizes are typically found above footings supporting two columns. (Figure 11)

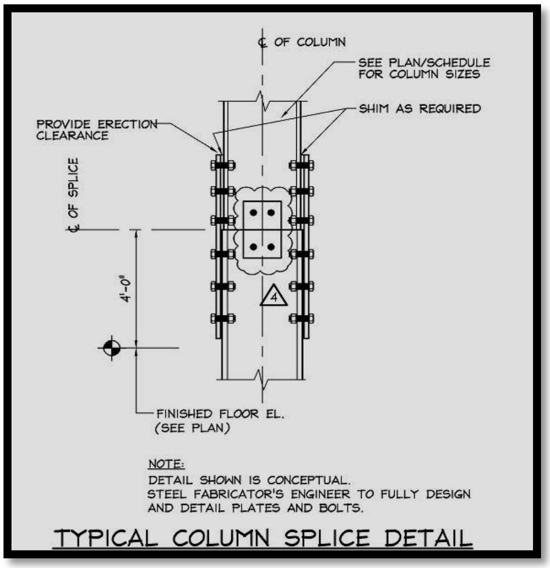


Figure 9: Typical column splice detail

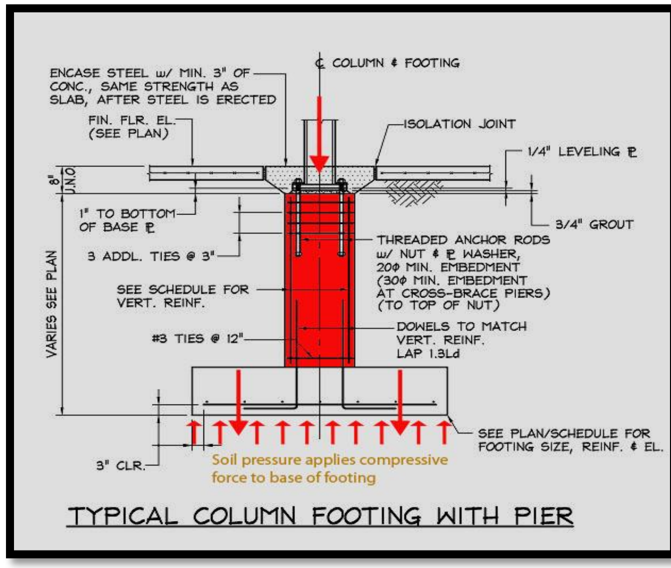


Figure 10: Typical concrete pier detail

The footings accept the vertical force from the piers and take it to the soil. French & Parrello Associates performed the soil analysis on the site and reported the site soil to be a mixture of fine sand, silt, clay, and various amounts of gravel. This composition is rated as a medium to stiff soil. Highly decomposed bedrock was found to be scattered throughout the site at a depth of 8'-30' but predominantly sloping from northwest to southeast. As a result, the spread footings (placed on the stiff soil) have an allowable bearing capacity of 4000 psf. Any footing placed on the decomposed bedrock has an allowable bearing capacity of 8000 psf.

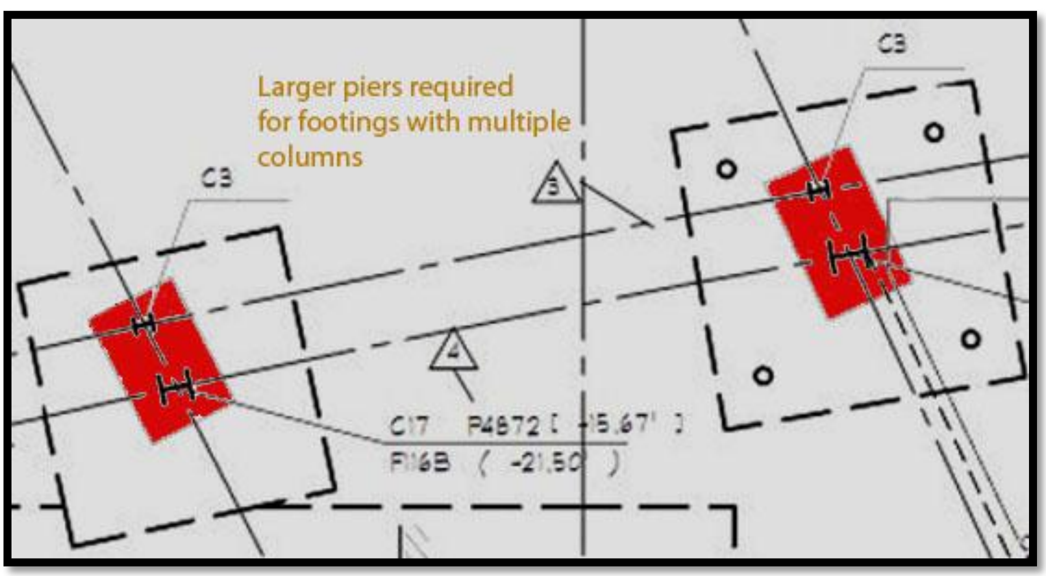


Figure 11: Concrete piers supporting multiple columns

Lateral System

The primary components of the lateral force resisting system in the New Hospital are braced and moment frames. (Figure 12) Expansion joints are located between the D&T building and the New Hospital and within the New Hospital itself at about 2/3 the length of the building from the west façade.

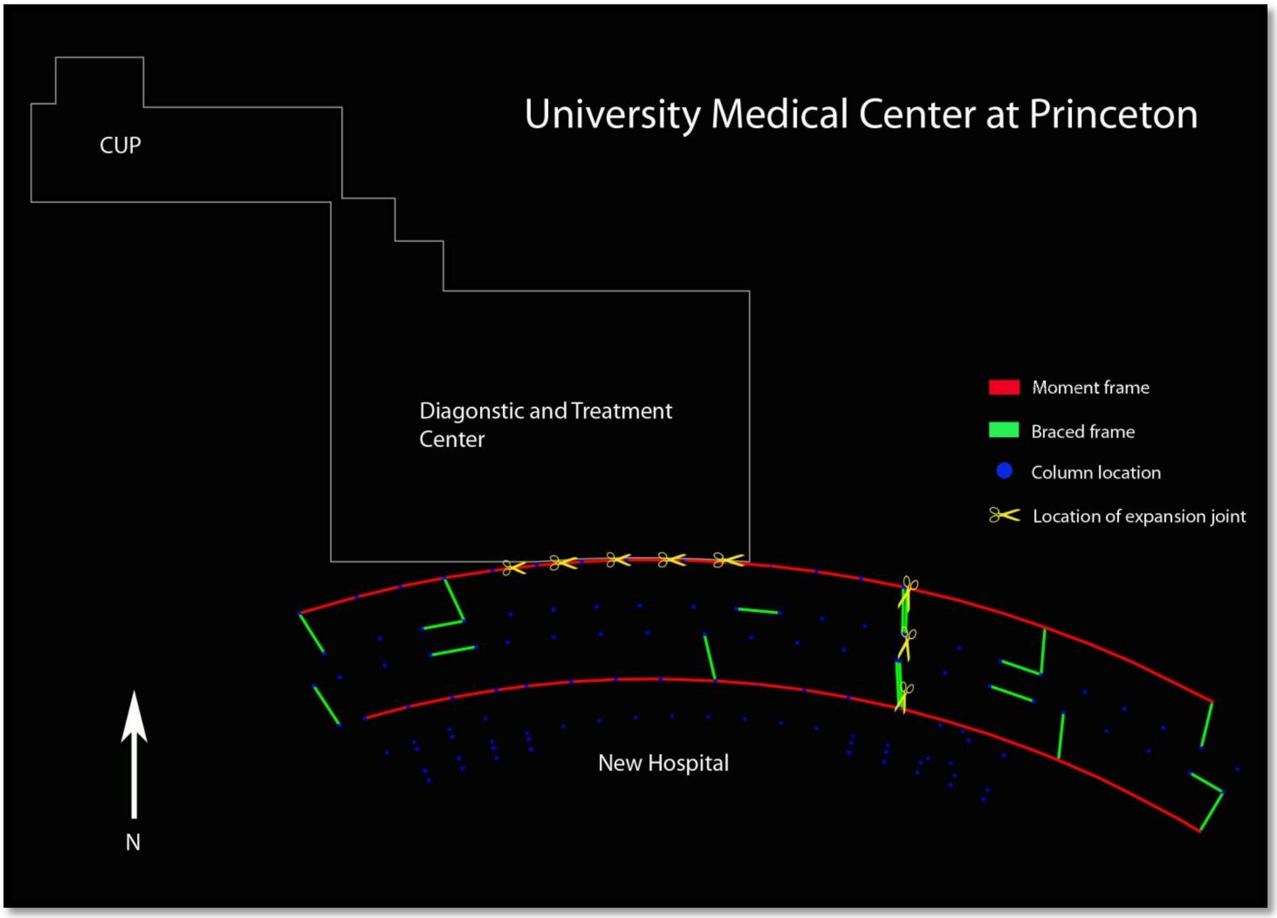


Figure 12: Lateral Force Resisting System for New Hospital

On the western wing of the facility, there are six braced frames running in the N-S direction. In the W-E direction, there are four braced frames and two long moment frames. The eastern wing has a similar layout with six braced frames in the N-S and four in the W-E as well as two moment frames in the W-E.

The two primary lateral forces exerted on this building will result from wind pressure and seismic activity. When wind strikes the façade of the New Hospital, the force created from the pressure distribution on the wall is transmitted to the floor diaphragm through the bent plate connection. (Figure 13) The floor is considered to be rigid which implies that it can deliver forces to joining members without experiencing any lateral deformation. The composite floor acts as a "collector" of forces and distributes its "collection" to the braced frames.

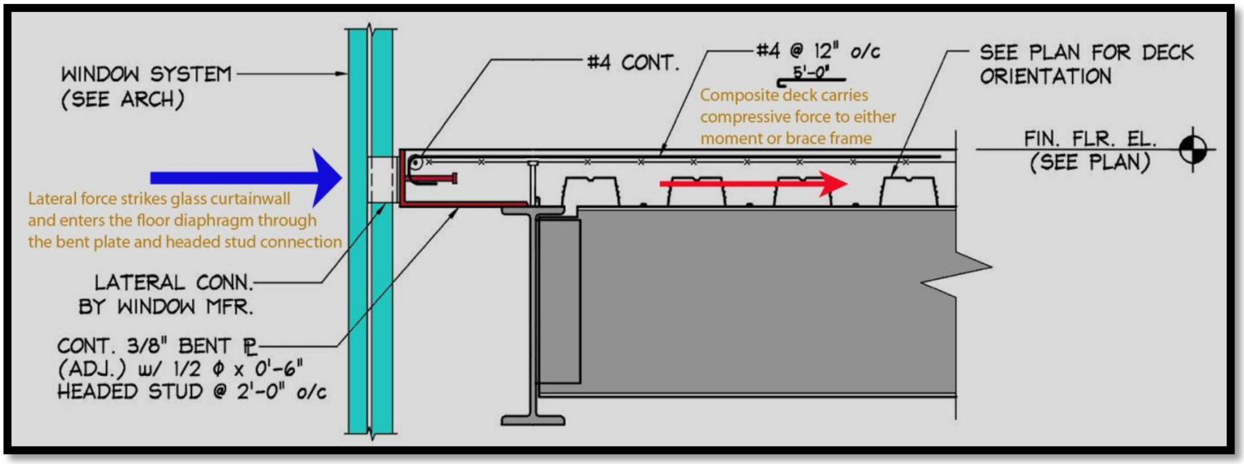


Figure 13: Lateral load acting through composite slab diaphragm

Once the force reaches the braced frame, it is transferred downward by the diagonal HSS members. These members are under compression and transfer that force to the column on the leeward side of the wind direction. That column then takes the compressive force down through the pier and into the foundation where it is ultimately resisted by the soil underneath the footing. (Figure 14)

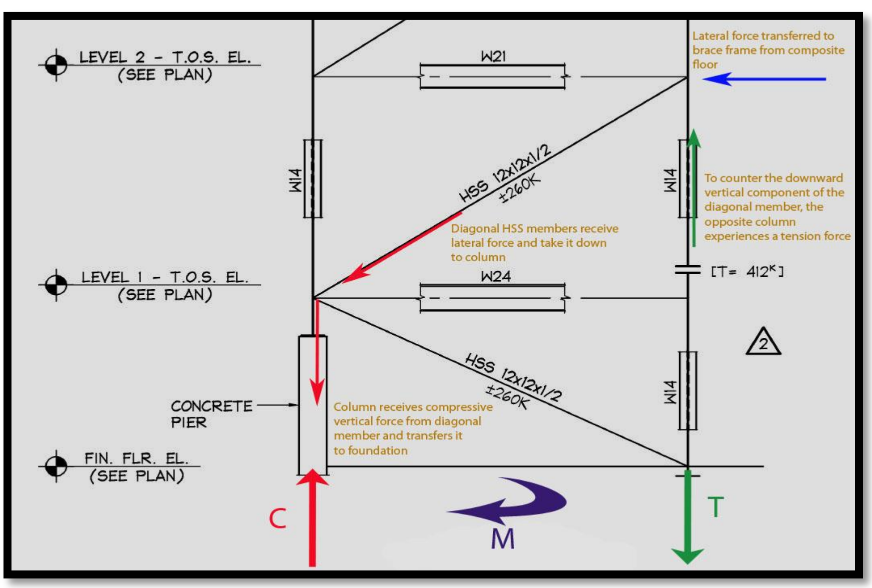


Figure 14: Lateral load acting through braced frame

On the windward side, the opposite is true. Due to force equilibrium at the frame joints, the windward columns of the braced frame experience a tension force. This force essentially tries to pull the column out of the ground. The foundation needs to be large enough to resist this motion. For this building, the foundations were not able to be designed to resist the tension force. Instead, mini-piles were placed underneath the spread footings and were driven down into the decomposed bedrock. (Figure 16) This design provided the foundation with the needed capacity to resist the tension force.

The final result can be seen in Figure 14 and Figure 15. The lateral force wants to overturn the frame in a counter-clockwise direction. The soil on the leeward side pushes up against the compressive force from the footing and the mini-piles pull down against the tension force on the windward side. This forms a force couple which creates a moment large enough to resist the rotation.

The north and south side moment frames also participate in the dissipation of lateral forces. These frames handle moment at the framing connections rather than using diagonal members to transmit the lateral force into an axial force in the column.

As stated earlier, the double angle connection that exists in the moment frames of the New Hospital is a partially restrained (PR) moment connection. Theoretically, a fully restrained moment connection handles all of the moment without any rotation (fixed end support). On the flip side, a simple shear connection is allowed to rotate and therefore cannot resist any moment (pinned support). A PR moment connection falls somewhere in-between the two.

Under gravity loads the PR moment connection provides no restraint against rotation thus behaving as a simple shear connection. (Figure 17) But under lateral loading, the connection provides rotational resistance and induces a moment on the column.

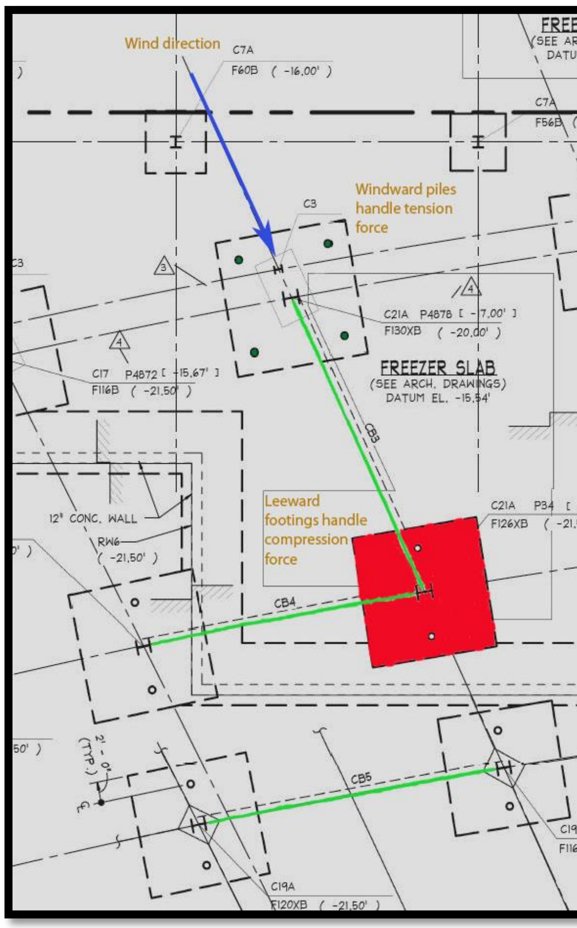


Figure 15: Braced frame foundation

Essentially, this turns moment frame columns into beam-columns because they are resisting moment (beam) and axial (column) forces. These forces are transmitted directly to the foundation through the concrete pier below the base of the column. Since there is no force couple like there is in the braced frame, the footings have to be sized to properly handle the compressive force as well as the overturning moment. For this structure, it was determined that the footings were capable of being upsized in order to handle the moment without having mini-piles attached to the bedrock.

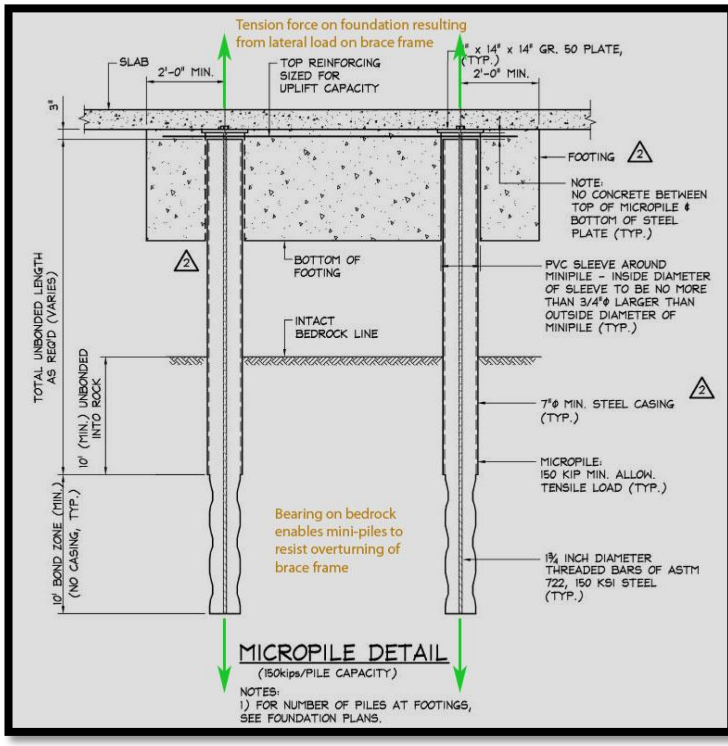


Figure 16: Mini-pile detail

In the N-S direction, lateral loads are handled only by braced frames (6 in the west wing and 6 in the east wing). In the W-E direction, there are braced and moment frames which handle the lateral load. In order to determine the percentage of force distributed to each frame, the relative stiffness of the two moment frames must be calculated. This calculation is outside the scope of this report but it can be assumed that the braced frame is twice as stiff as the moment frame per unit length. This results in more load taken by the braced frame on a per length basis. However, the moment frames are significantly longer than the braced frames so it could be that both frames resist the same magnitude of load. This assumption will be used as a launching point for a detailed analysis of the lateral force resisting system in the next technical assignment.

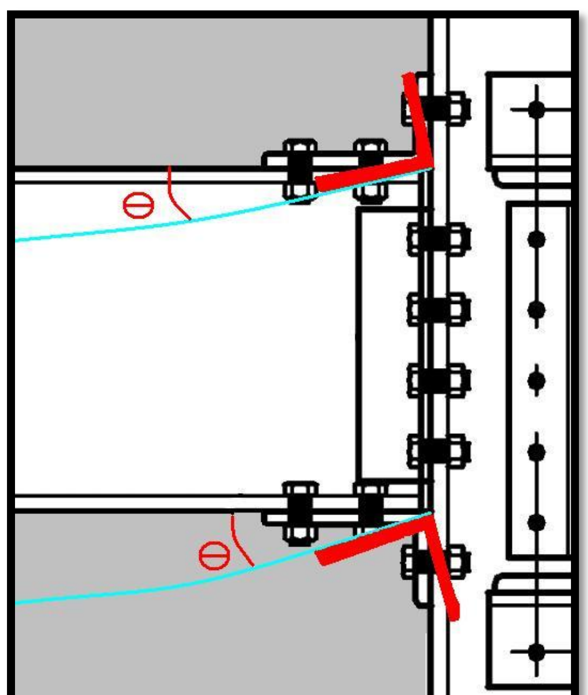


Figure 17: PR moment connection under gravity load

Wind Load Determination

The footprint of the New Hospital is curved concavely to the south which requires a complicated analysis to adequetely determine the wind pressure on the building facade. For this report, the building footprint is assumed to be rectangluar with an W-E dimension of 600' and a N-S dimension of 138'. (Figure 18) Since this is purely a determination of basic wind pressures, it is also assumed that the building behaves as one structure instead of two as stated earlier. This assumption is made to simplify the calculation as well as the results. Other assumptions state that the building is not subject to:

- Across wind loading
- Vortex shedding
- Galloping or fluttering due to instability
- Channeling or buffeting effects due to the site

In future reports, all of these assumptions will need to be addressed, especially the torsional effect of the wind pressure due to the curved façade.

Based upon these assumptions, the wind load pressures were determined using Method 2 from ASCE7-05. Coefficients for pressure determination are obtained from tables, charts, and graphs in the code. (Figure 19) These values are independent and are solely determined by site and building characteristics.

Wind variables are listed in Figure 20. These values are determined from equations in the ASCE code and are dependent upon values of the coefficients listed in Figure 19.

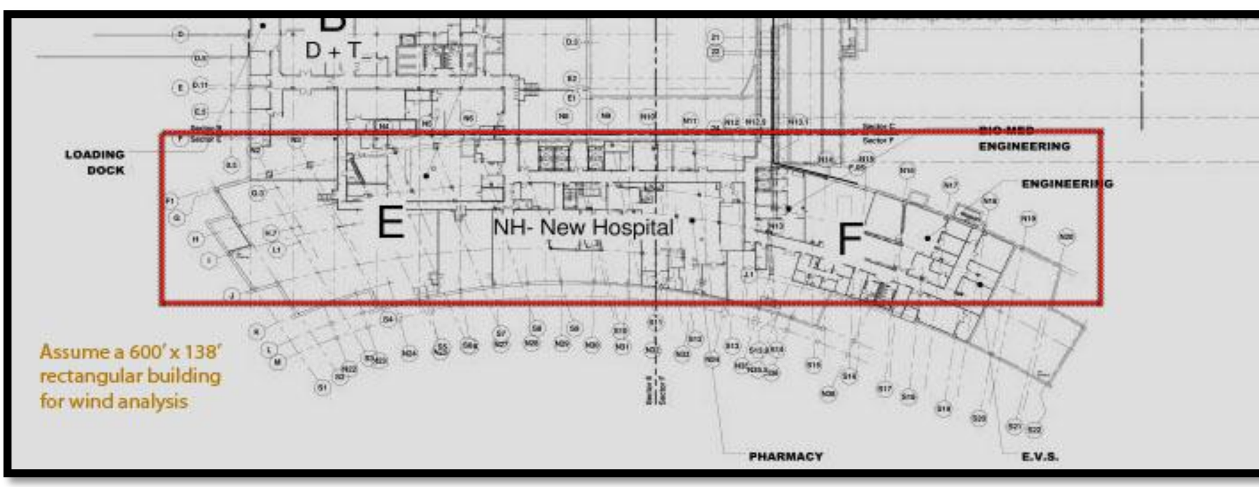


Figure 18: Simplified footprint for wind load calculation

<table>
<tr><th colspan="5">Wind Coefficients</th></tr>
<tr><th colspan="2">Title</th><th>Symbol</th><th>Value</th><th>Source</th></tr>
<tr><td colspan="2">Basic Wind Speed</td><td>V</td><td>95 mph</td><td>Figure 6-1</td></tr>
<tr><td colspan="2">Directionality Factor</td><td>K_d</td><td>0.85</td><td>Table 6-4</td></tr>
<tr><td colspan="2">Importance Factor</td><td>I</td><td>1.15</td><td>Table 6-2</td></tr>
<tr><td colspan="2">Topographic Factor</td><td>K_{ZT}</td><td>1.0</td><td>ASCE7-05 Sec. 6.5.7.2</td></tr>
<tr><td colspan="2">Exposure Category</td><td colspan="3">B</td></tr>
<tr><td colspan="2">3 sec. gust speed power law exponent</td><td>a</td><td>7.0</td><td>Table 6-3</td></tr>
<tr><td colspan="2">Nominal Height of Atmospheric Boundary</td><td>Z_g</td><td>1200 ft</td><td>Table 6-3</td></tr>
<tr><td colspan="2">Turbulence Intensity Factor</td><td>c</td><td>0.3</td><td>Table 6-2</td></tr>
<tr><td colspan="2">Peak Factor for Background Response</td><td>g_Q</td><td>3.4</td><td>ASCE7-05 Sec. 6.5.8.2</td></tr>
<tr><td colspan="2">Peak Factor for Wind Response</td><td>g_V</td><td>3.4</td><td>ASCE7-05 Sec. 6.5.8.2</td></tr>
<tr><td colspan="2">Integral Length Scale Factor</td><td>l</td><td>320 ft</td><td>Table 6-2</td></tr>
<tr><td colspan="2">Ratio of Solid Area to Gross Area</td><td>e</td><td>0.33</td><td>Table 6-2</td></tr>
<tr><td colspan="2">Damping Ratio</td><td>b</td><td>0.01</td><td>ASCE7-05 pg. 294 Commentary</td></tr>
<tr><td colspan="2">Mean Hourly Wind Speed Factor</td><td>b'</td><td>0.45</td><td>Table 6-2</td></tr>
<tr><td colspan="2">Enclosure Classification</td><td colspan="3">Closed</td></tr>
<tr><td rowspan="3">External Wall Pressure Coefficient</td><td>Windward</td><td rowspan="3">C_P</td><td>0.8</td><td>Figure 6-6</td></tr>
<tr><td>Leeward</td><td>-0.5</td><td>Figure 6-6</td></tr>
<tr><td>Side</td><td>-0.2</td><td>Figure 6-6</td></tr>
<tr><td colspan="2" rowspan="2">Internal Wall Pressure Coefficient</td><td rowspan="2">GC_{pi}</td><td>0.18</td><td>Figure 6-5</td></tr>
<tr><td>-0.18</td><td>Figure 6-5</td></tr>
<tr><td rowspan="2">Combined Net Pressure Coefficient</td><td>Windward</td><td rowspan="2">GC_{PN}</td><td>1.5</td><td>ASCE7-05 Sec. 6.5.12.2.4</td></tr>
<tr><td>Leeward</td><td>-1.0</td><td>ASCE7-05 Sec. 6.5.12.2.4</td></tr>
</table>

Figure 19: Wind coefficients

Wind Variables			
Title	Symbol	Value	Source
Equivalent Roof Height	z'	55.5 ft	
Building Natural Frequency	n_1	0.53	C6-14
Peak Factor for Resonant Response	g_r	4.04	Eq. 6-4, Eq. 6-8
Turbulence Intensity	I_Z	0.275	Eq. 6-5
Integral Length Scale of Turbulence	L_Z	380.55	Eq. 6-6
Mean Hourly Wind Speed at Height z	V_Z	71.40	Eq. 6-14
Reduced Frequency	N_1	2.82	Eq. 6-12

Figure 20: Wind variables

The above values are used to calculate a gust factor for wind in both N-S and W-E directions. (Figure 21)

Wind Gust Factor Calculation							
North-South				West-East			
Symbol		Value	Source	Symbol		Value	Source
R_N		0.072	Eq. 6-11	R_N		0.072	Eq. 6-11
η	For R_h	3.65	ASCE7-05 Sec. 6.5.8.2	η	For R_h	3.65	ASCE7-05 Sec. 6.5.8.2
	For R_B	20.42			For R_B	4.71	
	For R_L	15.78			For R_L	68.59	
R_h		0.236	Eq. 6-13	R_h		0.236	Eq. 6-13
R_B		0.48	Eq. 6-13	R_B		0.190	Eq. 6-13
R_L		0.061	Eq. 6-13	R_L		0.145	Eq. 6-13
R		0.213	Eq. 6-10	R		0.416	Eq. 6-10
Q		0.719	Eq. 6-5	Q		0.822	Eq. 6-5
Gust Factor (G_f)		0.79	Eq. 6-8	Gust Factor (G_f)		0.902	Eq. 6-8
B		600 ft		B		138 ft	
L		138 ft		L		600 ft	

Figure 21: Gust Factor

With the gust factor determined, wind pressure and corresponding forces can be determined. Tabulated results can be found in Appendix B. Detailed wind load calculations can be found in Appendix D.

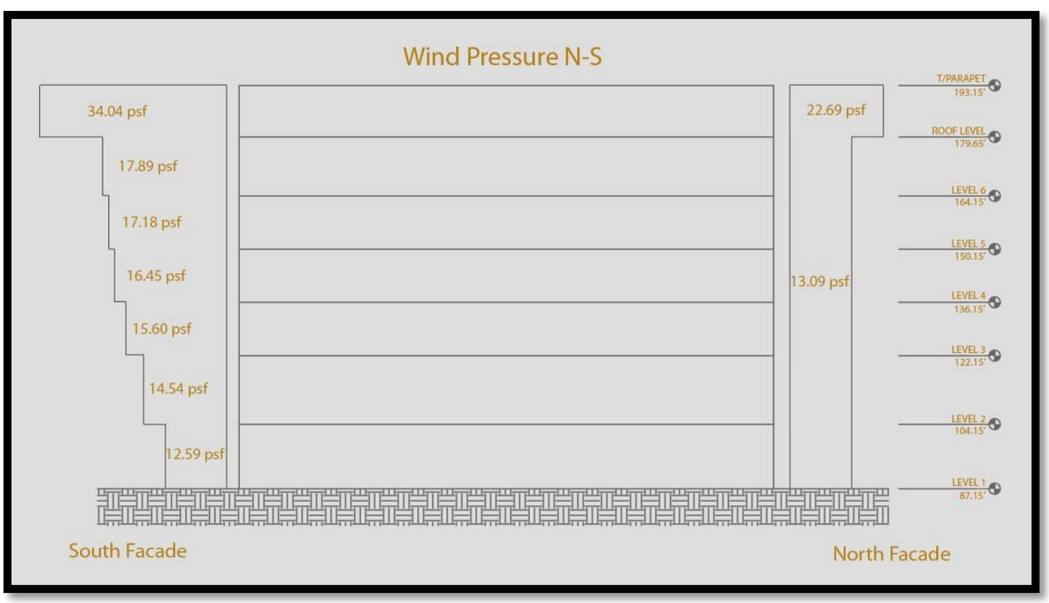


Figure 22: N-S Wind pressure diagram

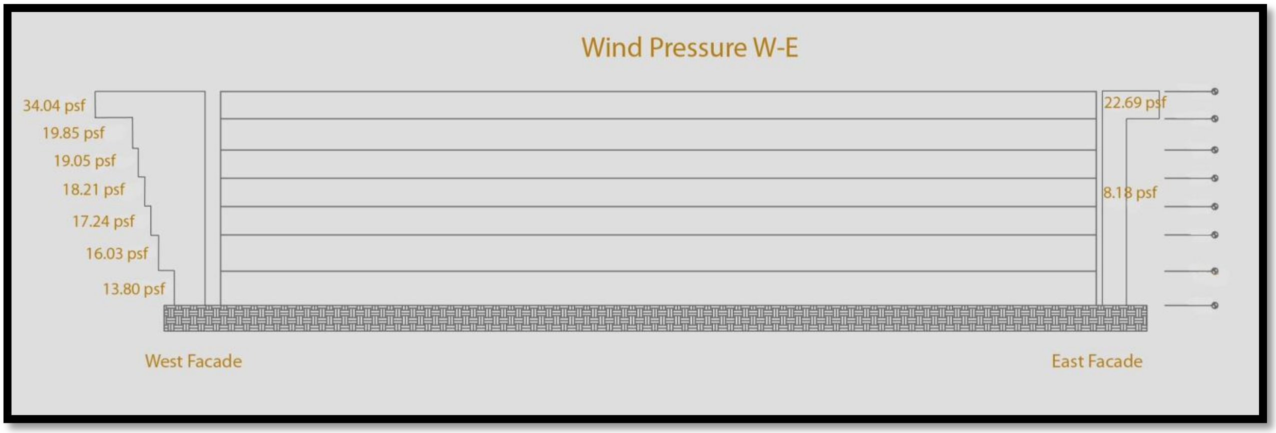


Figure 23: W-E Wind pressure diagram

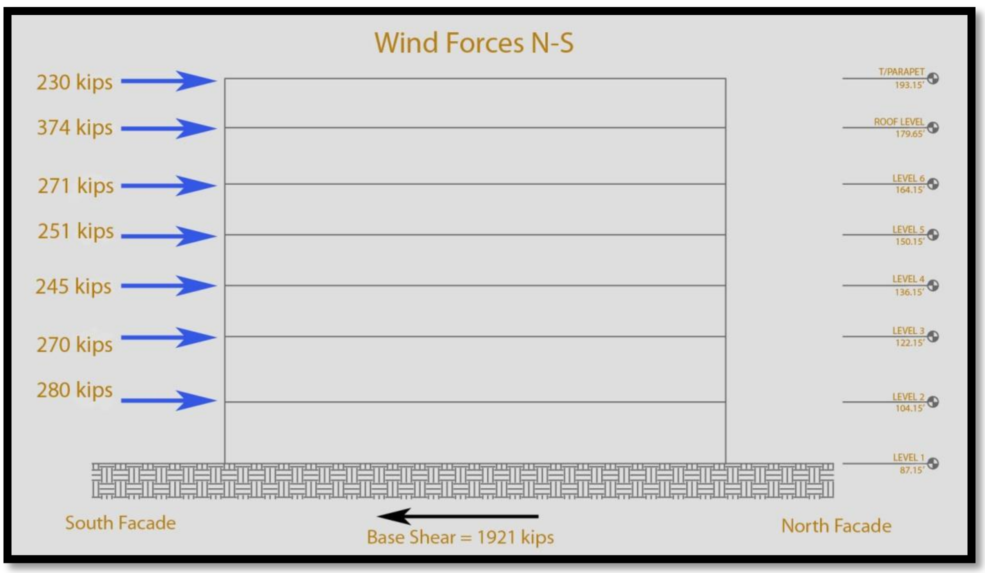


Figure 24: N-S Wind forces

Figure 25: W-E Wind forces

Seismic Load Determination

Based upon the geotechnical report provided by French & Parrello Associates, the site soils fall into Class D. From there, the seismic coefficients and variables can be obtained. (Figure 24, 25)

Seismic Coefficients			
Title	Symbol	Value	Source
Importance Factor	I	1.5	Table 11.5-1
Occupancy		IV	Table 11.6-2
Site Classification		D	Geotechnical Report
Spectral Response at Short Periods	S_S	0.3 %g	Figure 22-1
Spectral Response at 1 sec.	S_1	0.07 %g	Figure 22-2
Short Period Site Coefficient	F_A	1.56	Interpolate Table 11.4-1
Long Period Site Coefficient	F_V	2.4	Table 11.4-2
Response Modification	R	3.0	Table 12.2-1
Deflection Amplification	C_d	3.0	Table 12.2-1
Building Height	h	107 ft	Building Elevation
Long Period Transition Period	T_L	6.0	Figure 22-15
Fundamental Period	T	Unknown	
Approximate Period Parameter	C_t	0.02	Table 12.8-2
	x	0.75	Table 12.8-2

Figure 24: Seismic Coefficients

Seismic Variables			
Title	Symbol	Value	Source
Adjusted Spectral Response at Short Periods	S_{MS}	0.468	Eq. 11.4-1
Adjusted Spectral Response at 1 sec.	S_{M1}	0.168	Eq. 11.4-2
Design Spectral Response Acceleration at Short Periods	S_{DS}	0.312	Eq. 11.4-3
Design Spectral Response Acceleration at 1 sec.	S_{D1}	0.112	Eq. 11.-4
Seismic Response Coefficient	C_s	0.842	Eq. 12.8-3
Approximate Fundamental Period	T_A	0.665	Eq. 12.8-7

Figure 25: Seismic Variables

For clarity, the seismic coefficients and variables are separated. In order to obtain the seismic base shear, the weight of the building must first be determined. For this calculation, the New Hospital was divided along the expansion joint and considered as two separate structures. The weight of each is listed in Appendix A.

It should be noted that this is a rough estimation of building weight.

Seismic Force on West Wing of New Hospital				
Level	Height (ft.)	Weight (k)	C_{VX}	F_x (k)
2	17	3552	.056	89.61
3	35	3587	.124	197.79
4	49	3427	.171	272.01
5	63	3427	.224	357.06
6	77	3414	.278	442.07
Roof	91	1501	.146	232.87
	Total Building Weight	18909 kips	Total Base Shear	1591 kips
			Designer Base Shear	980 kips

Figure 26: Seismic force per level of west side of hospital

Seismic Force on East Wing of New Hospital				
Level	Height (ft.)	Weight (k)	C_{VX}	F_x (k)
2	17	1754	.051	36.34
3	35	1770	.113	80.14
4	49	1695	.155	110.49
5	63	1695	.204	145.04
6	77	1691	.253	179.75
Roof	91	1251	.224	159.39
	Total Building Weight	9858 kips	Total Base Shear	711 kips
			Designer Base Shear	580 kips

Figure 27: Seismic force per level of east side of hospital

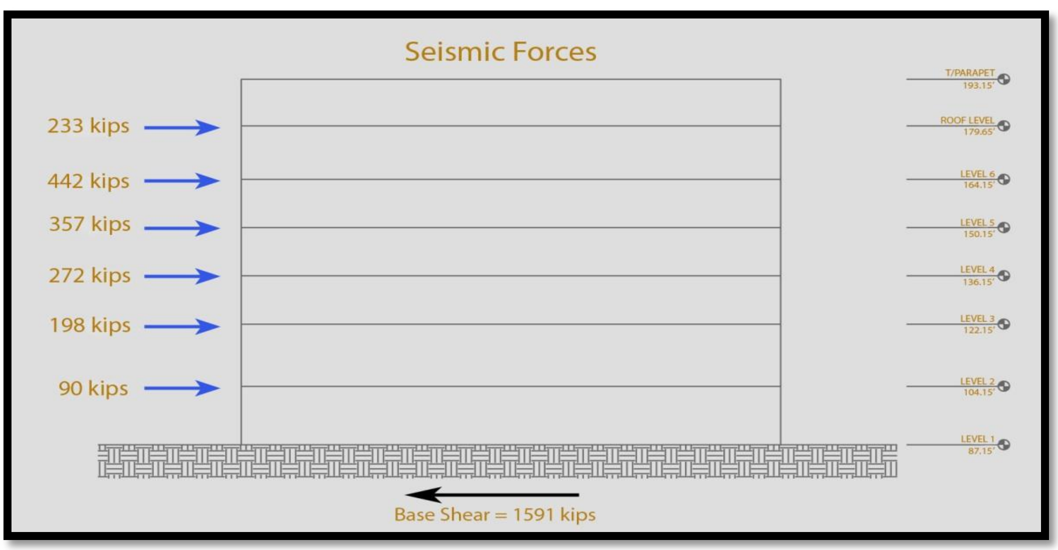


Figure 28: Seismic force per floor on west side of hospital

Figure 29: Seismic force per floor on east side of hospital

The values calculated for this report are well over the values obtained by the designer. Since the magnitude of the difference is significant, it is unlikely that the building weight calculation was done incorrectly. Another possible explanation would be the value used for T, the fundamental period of the structure. According to the ASCE code T_a, the approximate period, is acceptable to be used for the calculation of $C_{S.}$ The value used for T_a in this report is 0.0842. However, a value of T equal to C_u*T_a could be used and still be considered acceptable according to the code. If the designer applied this provision to the calculation, the base shear value equals 937 kips for the west wing of the New Hospital. This new value is close to the 980 kips determined in this report. Based upon this new result it seems that a different value for the fundamental period, T was used by the designer.. In any case this report used a conservative calculation. In future reports, this calculation will be refined. Detailed seismic load calculations can be found in Appendix E.

Snow Load Determination

The snow load coefficients are tabulated in Figure 30. The main issue with this calculation is the drift snow load due to the high parapet on the roof. The height of the snow drift at the parapet is listed in the table along with the design snow load, $p_{s.}$

A detailed calculation of snow load can be found in Appendix F.

Snow Coefficients				
Title		Symbol	Value	Source
Ground Snow Load		p_g	30 psf	Figure 7-1
Exposure Factor		C_E	1.0	Table 7-2
Thermal Factor		C_T	1.15	Table 7-3
Slope Factor		C_S	1.0	Figure 7-2
Importance Factor		I_S	1.2	Table 7-4
Snow Density		g	17.9 pcf	Eq. 7-3
Sloped Roof Snow Load		p_S	25.2 psf	Eq. 7-2
Height of Snow Drift at Parapet	N-S Direction	h_d	3.0 ft	Fig. 7-9
	E-W Direction		5.6 ft	Fig. 7-9
Exposure Category		B		

Figure 30: Snow load coefficients

Gravity Spot Checks

Spot checks were completed for a typical composite beam, a typical composite girder, and a steel column on the lower level. The beam and girder are supporting a patient room and corridor. The column supports the weight of the six stories above it as well as the roof. A column takedown was completed to determine the amount of gravity load on the column. (Figure 37)

These members were evaluated under gravity load only. However, lateral forces contribute significantly in the design of member sizes so it is expected that some of these members have much higher capacity than what is needed to support gravity loading. A more accurate spot check will be completed once the lateral loads on the building have been analyzed.

The detailed calculations for the spot checks can be found in Appendix G.

Column Takedown- Interior Column @ H-N6										
Column Below Level	Tributary Area	Live Load Influence Area	Live Load Reduction	Dead Load	Dead Load	Roof Live Load	Roof Live Load	Floor Live Load	Floor Live Load	Factored Column Load ($1.2D+1.6L+0.5L_R$)
				psf	k	psf	k	psf	k	
Roof	711	2842	1.00	129	91.7	20.0	14.2	0.0	0.0	132.8
6	1422	5684	0.45	56	131.5			80.0	39.8	228.6
5	2133	8526	0.41	56	171.4			80.0	77.4	336.6
4	2844	11368	0.40	56	211.2			80.0	114.4	443.6
3	3555	14210	0.40	56	251.0			80.0	151.4	550.5
2	4266	17052	0.40	56	290.8			80.0	188.4	657.4
1	4977	19894	0.40	56	330.6			80.0	225.3	764.4

Figure 37: Column load takedown @ H-N6

Summary

The steel frame of the New Hospital is the main structural system and handles both gravity and lateral loading. Composite beams and girders take gravity forces to the columns while braced frames within the structure and moment frames on the north and south faces resist lateral forces. Large spread footings are required to resist the overturning moments caused by lateral loads. The tension force in the braced frame is so large that mini piles are needed to help the foundation anchor the frame to the ground. These piles are driven down to decomposed bedrock which is anywhere from 8'-30' below grade.

Gravity and lateral loads on the building were calculated and did not quite match up with the values used by the designer. In the case of the gravity loads, it is assumed that the designer used higher loading than what is required by the code. In the case of seismic loads, it is assumed that the designer used a fundamental period value which is substantially greater than the approximate period used for this report. While the approximate period is more conservative, this calculation will be refined in a later report.

Using the calculated loads, spot checks were performed on gravity members. All designs are within the limitations of the codes used. A more detailed spot check will be performed once a through lateral analysis of the building was been completed.

Appendix A

Building Weight-New Hospital (West Wing)						
Total Area	38870	sq. ft.				
Stories	6					
Slabs						
	Typical	Mechanical	Roof	Superimposed		
Weight	56 psf	110 psf	129 psf	20 psf		
Area	38870 sf	771 sf	7410 sf	38870 sf		
Location	Floors 2-6	Roof	Roof	Floors 2-6		
Total	2177 Kips	68 Kips	911 Kips	777 Kips		
Columns						
	Floor 2	Floor 3	Floor 4	Floor 5	Floor 6	Roof
Height	17 ft	18 ft	14 ft	14 ft	14 ft	14 ft
Mean Weight	184.89	184.89	159.67	159.67	143.22	143.22
Quantity	55	55	55	55	55	55
Total	173 Kips	183 Kips	123 Kips	123 Kips	110 Kips	110 Kips
Facade (perimeter = 1000 ft)						
Height	17 ft	18 ft	14 ft	14 ft	14 ft	14 ft
Total	425 Kips	450 Kips	350 Kips	350 Kips	350 Kips	350 Kips
Floor Weight	3552 Kips	3587 Kips	3427 Kips	3427 Kips	3414 Kips	1501 Kips
Total Building Weight			**18909 Kips**			

Figure 31: Building weight (west wing)

Building Weight-New Hospital (East Wing)						
Total Area	19478	sq. ft.				
Stories	6					
Slabs						
	Typical	Mechanical	Roof	Superimposed		
Weight	56 psf	110 psf	129 psf	20 psf		
Area	19478 sf	771 sf	7410 sf	19478 sf		
Location	Floors 2-6	Roof	Roof	Floors 2-6		
Total	1091 Kips	85 Kips	956 Kips	390 Kips		
Columns						
	Floor 2	Floor 3	Floor 4	Floor 5	Floor 6	Roof
Height	17 ft	18 ft	14 ft	14 ft	14 ft	14 ft
Mean Weight	144.60	144.60	114.72	144.72	101.84	101.84
Quantity	25	25	25	25	25	25
Total	61 Kips	65 Kips	40 Kips	40 Kips	36 Kips	36 Kips
Façade (perimeter = 500 ft)						
Height	17 ft	18 ft	14 ft	14 ft	14 ft	14 ft
Total	213 Kips	225 Kips	175 Kips	175 Kips	175 Kips	175 Kips
Floor Weight	1754 Kips	1771 Kips	1696 Kips	1695 Kips	1691 Kips	1252 Kips
Total Building Weight			9858 Kips			

Figure 32: Building weight (east wing)

Appendix B

Wind Pressure on New Hospital (N-S Direction) B = 600 ft. L = 138 ft.							
Level	Height Above Ground	Story Height	K_z	q_z	p_z		Total Pressure
					Windward	Leeward	
	(ft)	(ft)			(psf)	(psf)	(psf)
1	0	0	0.00	0.00	0.00	0.00	0.00
2	17	17	0.60	13.45	12.59	-13.09	25.68
3	35	18	0.73	16.53	14.54	-13.09	27.63
4	49	14	0.81	18.20	15.60	-13.09	28.69
5	63	14	0.87	19.56	16.45	-13.09	29.54
6	77	14	0.92	20.71	17.18	-13.09	30.27
Roof	92.5	15.5	0.97	21.83	17.89	-13.09	30.98
Parapet	106	13.5	1.00	22.69	34.04	-22.69	56.73

Figure 33: Wind pressure (N-S)

Wind Pressure on New Hospital (W-E Direction) B = 138 ft. L = 600 ft.							
Level	Height Above Ground	Story Height	K_z	q_z	p_z		Total Pressure
					Windward	Leeward	
	(ft)	(ft)			(psf)	(psf)	(psf)
1	0	0	0.00	0.00	0.00	0.00	0.00
2	17	17	0.60	13.45	13.80	-8.18	21.99
3	35	18	0.73	16.53	16.03	-8.18	24.21
4	49	14	0.81	18.20	17.24	-8.18	25.42
5	63	14	0.87	19.56	18.21	-8.18	26.40
6	77	14	0.92	20.71	19.05	-8.18	27.23
Roof	92.5	15.5	0.97	21.83	19.85	-8.18	28.04
Parapet	106	15.5	1.00	22.69	34.04	-22.69	56.73

Figure 34: Wind pressure (W-E)

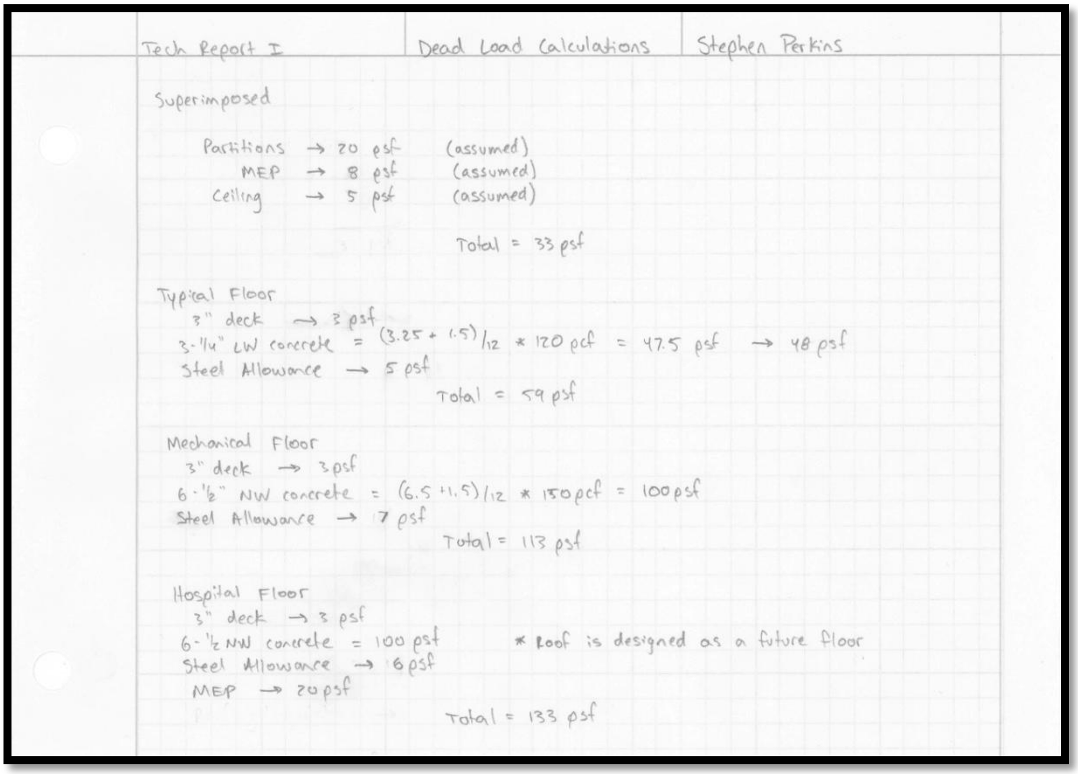
Wind Force on New Hospital (N-S Direction) B = 600' L= 138'								
Level	Height Above Ground	Story Height	Force		Shear		Moment	
			windward	total	windward	total	windward	total
	(ft)	(ft)	(k)	(k)	(k)	(k)	(ft-k)	(ft-k)
1	0	0	0	0	1457.36	1920.52	0	0
2	17	17	185.57	280.20	1457.36	1920.52	3155	4763
3	35	18	196.39	269.70	1271.80	1640.32	6874	9440
4	49	14	178.29	244.58	1075.40	1370.62	8736	11984
5	63	14	187.36	251.24	897.11	1126.04	11804	15828
6	77	14	203.48	271.21	709.75	874.80	15668	20883
Roof	92.5	15.5	276.50	373.82	506.27	603.59	25577	34579
Parapet	106	15.5	229.77	229.77	229.77	229.77	24355	24355
				Total:	1457.4	1920.5	96168	121832

Figure 35: Wind force (N-S)

Wind Force on New Hospital (W-E Direction) B = 138' L= 600'								
Level	Height Above Ground	Story Height	Force		Shear		Moment	
			windward	total	windward	total	windward	total
	(ft)	(ft)	(k)	(k)	(k)	(k)	(ft-k)	(ft-k)
1	0	0	0	0	1626.85	419.99	0	0
2	17	17	203.89	55.86	1626.85	419.99	3466	950
3	35	18	216.67	54.63	1422.95	364.12	7583	1912
4	49	14	197.15	50.06	1206.28	309.49	9661	2453
5	63	14	207.51	51.81	1009.13	259.44	13073	3264
6	77	14	225.66	56.29	801.62	207.63	17376	4335
Roof	92.5	15.5	312.15	90.66	575.96	151.34	28874	8386
Parapet	106	15.5	263.81	60.68	263.81	60.68	27964	6432
				Total:	1626.8	420.0	107997	27731

Figure 36: Wind force (W-E)

Appendix C



Tech Report I | Dead Load Calculations | Stephen Perkins

Superimposed

Partitions → 20 psf (assumed)
MEP → 8 psf (assumed)
Ceiling → 5 psf (assumed)

Total = 33 psf

Typical Floor
3" deck → 3 psf
3-1/4" LW concrete = (3.25 + 1.5)/12 * 120 pcf = 47.5 psf → 48 psf
Steel Allowance → 5 psf
Total = 59 psf

Mechanical Floor
3" deck → 3 psf
6-1/2" NW concrete = (6.5 + 1.5)/12 * 150 pcf = 100 psf
Steel Allowance → 7 psf
Total = 113 psf

Hospital Floor
3" deck → 3 psf
6-1/2 NW concrete = 100 psf
Steel Allowance → 6 psf
MEP → 20 psf
Total = 133 psf

* Roof is designed as a future floor

Figure 37: Detailed dead load calculations

Appendix D

Tech Report I | Design Wind Calculations | Stephen Perkins

ACI 6.5.1 Assume regular shape

Assume building is not subject to : across wind loading
vortex shedding
galloping or flutter instability
channeling effects or buffeting due to site

∴ use Method 2 (only considering wind acting on New Hospital)

$V = 95$ mph Figure 6-1 (Also from dwgs)
$K_d = 0.85$ Table 6-4
$I = 1.15$ Table 6-2
$K_{zt} = 1.0$
Exposure = B
$\alpha = 7.0$ Table 6-3
$z_g = 1200$ ft Table 6-3

@ 106', $K_z = 2.01\left(106/1200\right)^{2/7} = 1.00$

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

@20', $q_z = 0.00256(0.62)(1.0)(0.85)(95^2)(1.15) = 14.00$

Height above Ground, z (ft)	$K_z = 2.01\left(z/z_g\right)^{2/\alpha}$ Case II	q_z
0-15	0.57	12.87
20	0.62	14.00
25	0.66	14.91
30	0.70	15.81
40	0.76	17.16
50	0.81	18.29
60	0.85	19.20
70	0.89	20.10
80	0.93	21.00
90	0.96	21.68
100	0.99	22.36
106	1.00	22.58 = q_p

*Roof mean height = max height
↳ No slope is assumed

$z = 92.5'$ $h = 107'$ (includes 14.5' Parapet)
$\bar{z} = 0.6z = 55.5'$

$n_1 = 22.2/h^{0.8}$ C6-14
$= 22.2/107^{0.8}$
$= 0.53 < 1.0$ ∴ structure is considered flexible

$g_Q = 3.4$ 6.5.8.2
$g_v = 3.4$ 6.5.8.2
$g_R = \sqrt{2\ln(3600 n_1)} + 0.577/\sqrt{2\ln(3600 n_1)}$
$= \sqrt{2\ln(3600 \times 0.53)} + 0.577/\sqrt{2\ln(3600 \times 0.53)}$
$= 4.04$

$c = 0.3$ Table 6-2

$I_{\bar{z}} = c\left(33/\bar{z}\right)^{1/6}$
$= 0.3\left(33/55.5\right)^{1/6}$
$= 0.275$

$\ell = 320'$ Table 6-2

$\bar{\epsilon} = 1/3.0$ Table 6-2

$L_{\bar{z}} = \ell\left(\bar{z}/33\right)^{\bar{\epsilon}}$
$= 320\left(55.5/33\right)^{(1/3)} = 380.55$ ft

Figure 38: Detailed wind load calculation pg.1

Wind N-S

$B = 600$ ft $\quad L = 138$ ft

$$Q = \sqrt{1/\left(1+0.63\left(B+h/L_{\bar{z}}\right)^{0.63}\right)}$$
$$= \sqrt{1/1+0.63\left(360+106/380.55\right)^{0.63}}$$
$$= 0.763$$

$\bar{b} = 0.45$

$\bar{\alpha} = 1/4.0$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\bar{z}/33\right)^{\bar{\alpha}} V\left(88/60\right)$$
$$= 0.45\left(55.5/33\right)^{0.25} 95\left(88/60\right)$$
$$= 71.4$$

$$N_1 = n_1 L_{\bar{z}}/\bar{V}_{\bar{z}}$$
$$= 0.53(380.55)/71.4$$
$$= 2.82$$

$$R_N = 7.47N_1/\left(1+10.3N_1\right)^{5/3}$$
$$= 7.47(2.82)/\left(1+10.3(2.82)\right)^{5/3}$$
$$= 0.072$$

$$\eta_{\text{for } R_h} = 4.6 n_1 h/\bar{V}_{\bar{z}}$$
$$= 4.6(0.53)(107)/71.4$$
$$= 3.65$$

$$\eta_{\text{for } R_B} = 4.6 n_1 B/\bar{V}_{\bar{z}}$$
$$= 4.6(0.53)(600)/71.4$$
$$= 20.49$$

$$\eta_{\text{for } R_L} = 15.4 n_1 L/\bar{V}_{\bar{z}}$$
$$= 15.4(0.53)(138)/71.4$$
$$= 15.78$$

$$R_h = 1/\eta - 1/2\eta^2\left(1-e^{-2\eta}\right)$$
$$= 1/3.65 - 1/2(3.65^2)\left(1-e^{-2(3.65)}\right)$$
$$= 0.236$$

$$R_B = 1/20.49 - 1/2(20.49^2)\left(1-e^{-2(20.49)}\right)$$
$$= 0.080$$

$$R_L = 1/15.78 - 1/2(15.78^2)\left(1-e^{-2(15.78)}\right)$$
$$= 0.061$$

$\beta = 0.01$ pg. 294

$$R = \sqrt{1/\beta \, R_N R_h R_B \left(0.53+0.47R_L\right)}$$
$$= \sqrt{1/0.01(0.072)(0.236)(0.080)(0.53+0.47(0.061))}$$
$$= 0.276$$

Wind E-W

$B = 138$ ft $\quad L = 600$ ft

$Q = 0.823$

Same procedure as N-S

$\eta_{\text{for } R_h} = 3.65$

$\eta_{\text{for } R_B} = 4.71$

$\eta_{\text{for } R_L} = 68.59$

$R_h = 0.236$

$R_B = 0.190$

$R_L = 0.014$

$R = 0.419$

Gust factors:

N-S

$$G_f = 0.925\left[\frac{1+1.7I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_r^2 R^2}}{1+1.7g_v I_{\bar{z}}}\right]$$
$$= 0.925\left[\frac{1+1.7(0.275)\sqrt{3.4^2(0.719^2)+4.04^2(0.276^2)}}{1+1.7(3.4)(0.275)}\right]$$
$$= 0.805$$

E-W

$$G_f = 0.925\left[\frac{1+1.7(0.275)\sqrt{3.4^2(0.822^2)+4.04^2(0.419^2)}}{1+1.7(3.4)(0.275)}\right]$$
$$= 0.903$$

Figure 39: Detailed wind load calculations pg.2

Parapet

$q_p = 22.36$ @ 100'
$= 22.58$ @ 106'

$GC_{pn} = +1.5$ windward

$GC_{pn} = -1.0$ leeward

$p_p = q_p GC_{pn}$ (windward)

$= 22.36(1.5) = 33.54$ @ 100' (windward)
$= 22.58(1.5) = 33.87$ @ 106'

$= 22.75(-1.0) = -22.75$ @ 100' (leeward)
$= 22.75(-1.0) = -22.75$ @ 106'

Pressure Coefficients

External Wall Pressure (windward) $C_p = 0.8$ Figure 6-6
(leeward N-S) $C_p = -0.5$ Figure 6-6
(leeward E-W) $C_p = -0.2$ Figure 6-6

Internal Pressure Coefficients $GC_{pi} = 0.18$
$GC_{pi} = -0.18$

N/S

windward $p_z = q_z G_f C_p - q_h (GC_{pi})$
$= q_z (0.79)(0.8) - 22.75(-0.18)$
$= 0.632 q_z + 4.095$

$p_{z@30'} = 0.632(15.8) + 4.095 = 14.09$ psf

leeward $p_h = q_h G_f C_p - q_h (GC_{pi})$
$= 22.75(0.79)(-0.5) - 22.75(0.18)$
$= -13.08$ psf

E-W

windward $p_z = 0.902(0.8) q_z - 22.75(-0.18)$
$= 0.72 q_z + 4.09$

$p_{z@60'} = 0.72(19.29) + 4.09$
$= 17.97$ psf

leeward $p_h = 22.75(0.902)(-0.2) - 22.75(0.18)$
$= -8.20$ psf

Figure 40: Detailed wind load calculations pg.3

Appendix E

Tech Report I | Design Seismic Calculations | Stephen Perkins

Site Classification

soil on site : Discontinuous layers of course to fine sand, silt and clay, and coarse to fine gravel.
Bedrock does exist on site (8 - 30' below grade)

soils classified as stiff to hard on geotech report

∴ For seismic considerations, use site class D

$S_s = 0.3\ \%\ g$ Figure 22-1

$S_1 = 0.07\ \%\ g$ Figure 22-2

$F_a = 1.56$ Interpolate Table 11.4-1

	$S_s \le 0.25$	$S_s = 0.5$
D	1.6	1.4

⟹

$1.6 - 1.4 = 0.2$
$0.5 - 0.25 = 0.25$
$0.3 - 0.25 = 0.05$

$0.25/0.05 = 5$
Divide $0.2/5 = 0.04$

$\therefore F_a = 1.6 - 0.04 = 1.56$

$F_v = 2.4$ Table 11.4-2

$S_{MS} = F_a S_s = 1.56(0.3) = 0.468$ Eq. 11.4-1

$S_{M1} = F_v S_1 = 2.4(0.07) = 0.168$ Eq. 11.4-2

$S_{DS} = 2/3\ S_{MS} = 2/3(0.468) = 0.312$ Eq. 11.4-3

$S_{D1} = 2/3\ S_{M1} = 2/3(0.168) = 0.112$ Eq. 11-4-4

Occupancy = IV
Since $S_{D1} = 0.112$, Seismic Design Category = C Table 11.6-2

$I = 1.50$ Table 11.5-1

$T_L = 6.0$ s Figure 22-15

Seismic Force - Resisting Systems
→ ordinary braced frames, steel

$R = 3.0$ Table 12.2-1
$C_d = 3.0$

* Assume rigid diaphragm (composite deck w/ concrete slab)
↳ use Equivalent Force method

Figure 41: Detailed seismic load calculations pg.1

$C_s = S_{DS}/(R/I) = 0.312/(3/1.5)$ Eq. 12.8-2
$= 0.156$

$C_t = 0.02$ Table 12.8-2
$x = 0.75$

$T_a = C_t h_n^x = 0.02(107')^{0.75}$ Eq. 12.8-7
$= 0.665 < T_L = 6.0\ s$

$\therefore C_s \leq S_{D1}/T(R/I)$ Eq. 12.8-3
$= 0.112/0.665(3/1.5)$
$= 0.0842 < C_s = 0.156$
$\therefore C_s = 0.0842 > 0.01$ ✓ ok!

$C_u = 1.7$ Table 12.8-1

$W = 18{,}909$ kips See Bldg. weight calculation for West Wing of New Hospital

$V = C_s W = 0.0842(18{,}909)$ Eq. 12.8-1
$= 1592^k$

$F_x = C_{vx} V$ Eq. 12.8-11

$C_{vx} = \dfrac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$ Eq. 12.8-12

k: $0.665 - 0.5 = 0.165$
Every 0.5 increase = 0.25 increase in k value.
$\therefore k = (0.165/0.5)0.25 + 1$
$= 1 + 0.0825$
$= 1.0825$

Dead Load by Floor:
2 ⇒ 3552^k @ 17'
3 ⇒ 3587^k @ 35'
4 ⇒ 3427^k @ 49'
5 ⇒ 3427^k @ 63'
6 ⇒ 3414^k @ 77'
Roof ⇒ 1501^k @ 91'

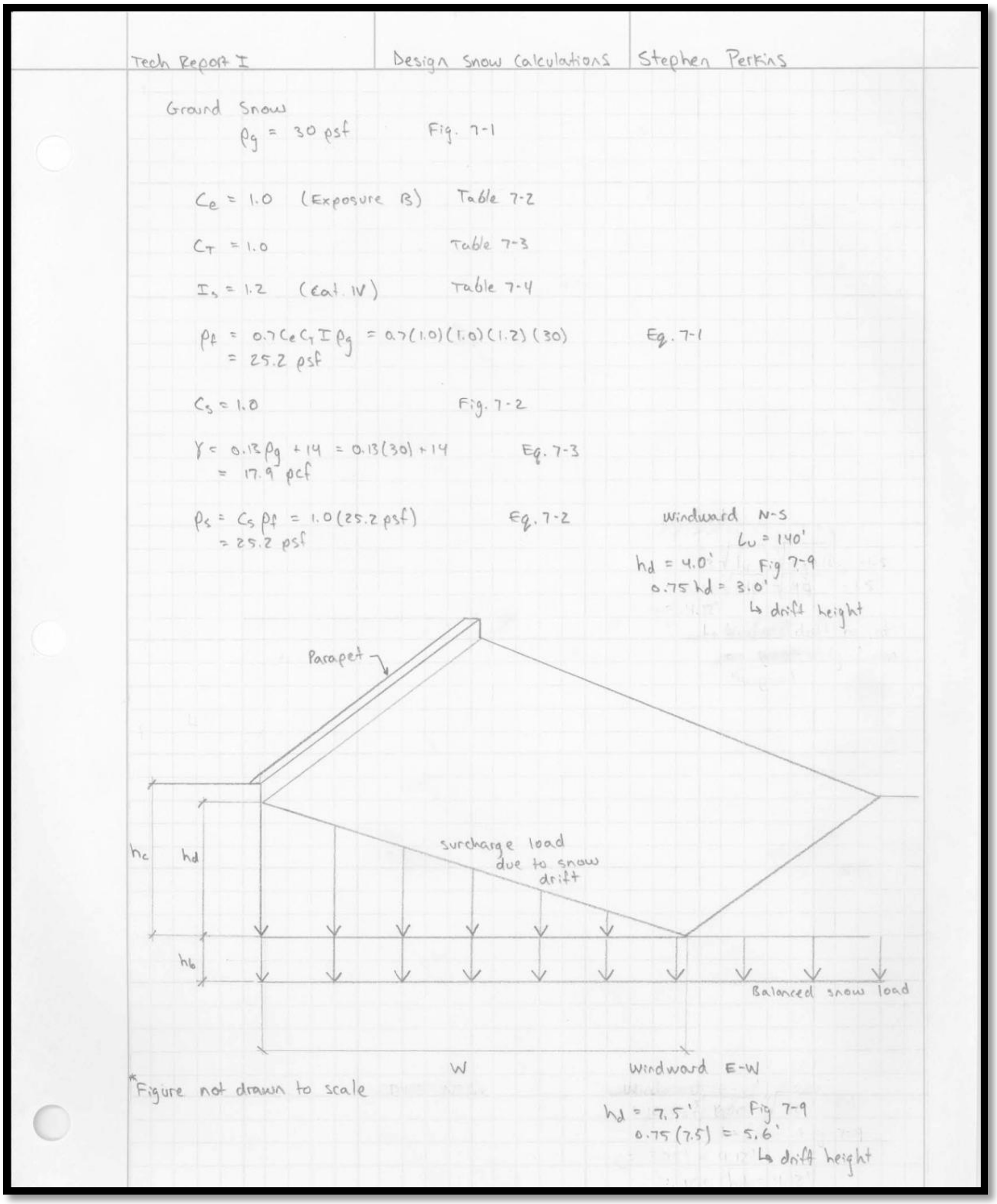
$\sum_{i=1}^{n} w_i h_i^k = 3552(17)^{1.08} + 3587(35)^{1.08} + 3427(49)^{1.08} + 3427(63)^{1.08} + 3414(77)^{1.08} + 1501(91)^{1.08}$
$= 1340663.2$

$C_{v_2} = 0.056$ $F_{x_2} = 0.056(1592) = 89.2^k$
$C_{v_3} = 0.124$ $F_{x_3} = 0.124(1592) = 197.4^k$
$C_{v_4} = 0.171$ $F_{x_4} = 0.171(1592) = 272.2^k$
$C_{v_5} = 0.224$ $F_{x_5} = 0.224(1592) = 356.6^k$
$C_{v_6} = 0.278$ $F_{x_6} = 0.278(1592) = 442.6^k$
$C_{v_R} = 0.146$ $F_{x_R} = 0.146(1592) = 232.4^k$

Sum = 1.0 ✓

Figure 42: Detailed seismic load calculations pg.2

Appendix F



Tech Report I | Design Snow Calculations | Stephen Perkins

Ground Snow

$p_g = 30$ psf   Fig. 7-1

$C_e = 1.0$ (Exposure B)   Table 7-2

$C_T = 1.0$ Table 7-3

$I_s = 1.2$ (Cat. IV)   Table 7-4

$p_f = 0.7 C_e C_t I p_g = 0.7(1.0)(1.0)(1.2)(30)$
$= 25.2$ psf   Eq. 7-1

$C_s = 1.0$ Fig. 7-2

$\gamma = 0.13 p_g + 14 = 0.13(30) + 14$
$= 17.9$ pcf   Eq. 7-3

$p_s = C_s p_f = 1.0(25.2 \text{ psf})$
$= 25.2$ psf   Eq. 7-2

Windward N-S
$l_u = 140'$
$h_d = 4.0'$ Fig 7-9
$0.75 h_d = 3.0'$
↳ drift height

*Figure not drawn to scale

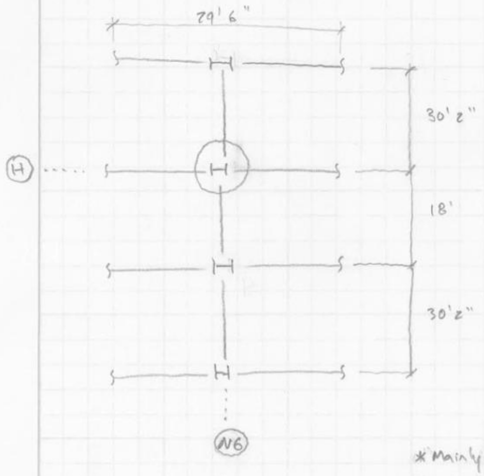
Windward E-W
$h_d = 7.5'$ Fig 7-9
$0.75(7.5) = 5.6'$
↳ drift height

Figure 43: Detailed snow load calculations

Appendix G

Tech Report I | Column Load Takedown | Stephen Perkins

Interior Column @ (H - N6)



Influence Area - Below Level 6

$A_I = [2(29.5) * (30.17 + 18)] * 2$
$= 5684\ ft^2$

Live Load reduction

$L = L_o (0.25 + 15/\sqrt{K_{LL}A_T})$
$= L_o (0.25 + 15/\sqrt{5684})$
$= 0.45 L_o$

* Assume no reduction for roof
* A_I is same at each level

Tributary Area - Below level 6

$A_T = 29.5 * [(30.17/2) + (18/2)]$
$= 711\ ft^2$

Floor live load
* Mainly supporting corridor => 80 psf

Dead Load - Below Level 6

Hospital Roof = 129 psf
$A_T = 711\ ft^2$
load = 129 * 711 = 91.7^k

Typical Floor = 56 psf
$A_T = 711\ ft^2$
Load = 91.7 + 56(711) = 132^k

Live Load - Below Level 6

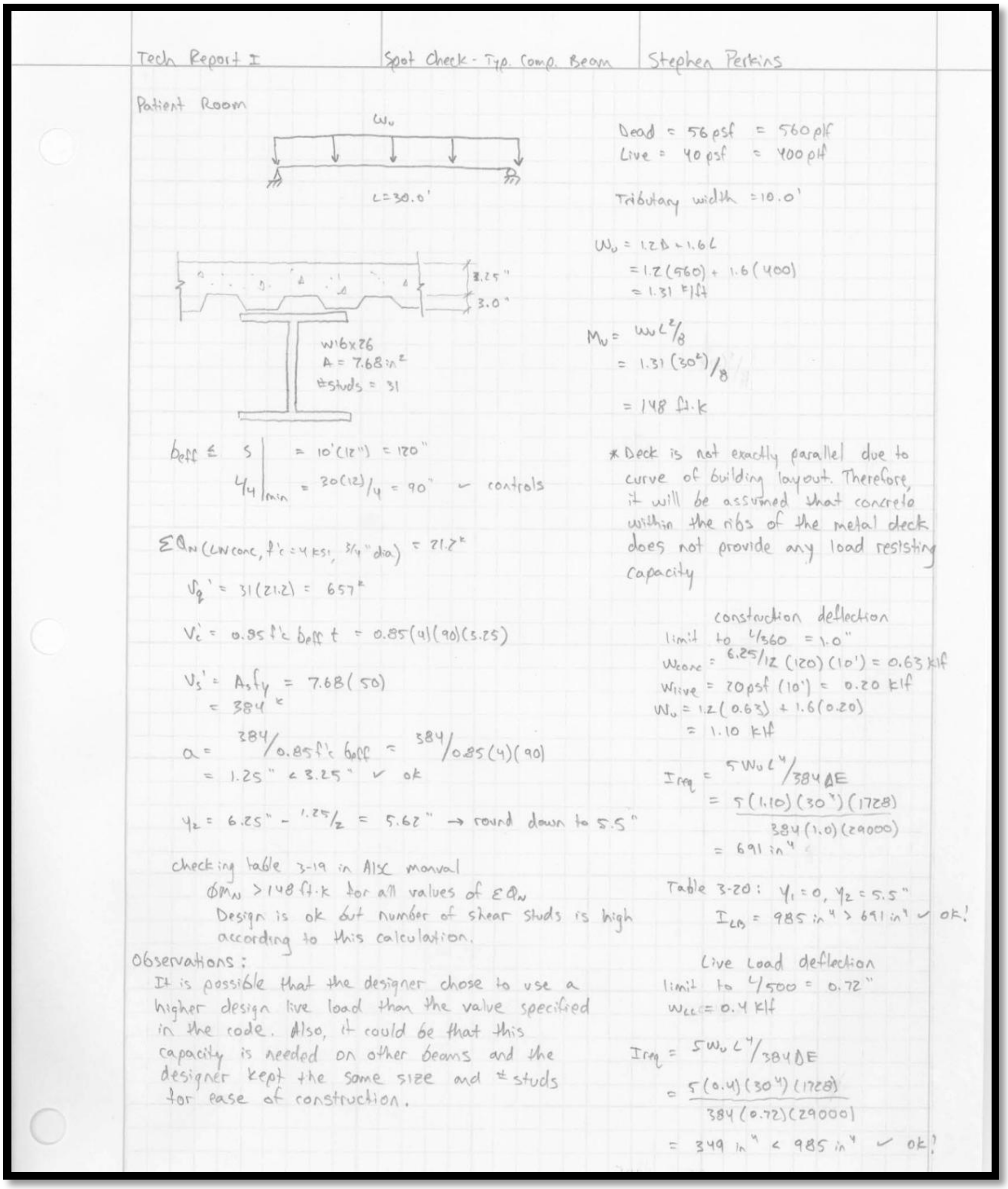
Roof Live = 20 psf
$A_T = 711\ ft^2$
Load = 711(20) = 14.2^k

Corridors = 80 psf
$A_T = 711\ ft^2$
Load = 711(80) = 56.9^k
Reduced = 0.45(56.9) = 25.6^k
Total = $25.6^k + 14.2^k = 39.6^k$

Factored Column Load - Below Level 6
$(1.2D + 1.6L + 0.5L_R)$

$1.2(131.5) + 1.6(39.8) + 0.5(14.2) = 228.6^k$

Figure 44: Detailed column load takedown calculations



Tech Report I | Spot Check - Typ. Comp. Beam | Stephen Perkins

Patient Room

Dead = 56 psf = 560 plf
Live = 40 psf = 400 plf

Tributary width = 10.0'

$w_u = 1.2D + 1.6L$
$= 1.2(560) + 1.6(400)$
$= 1.31\ ^k/ft$

$M_u = w_u L^2/8$
$= 1.31(30^2)/8$
$= 148$ ft·k

$b_{eff} \le$ s $= 10'(12") = 120"$
$L/4 \,|_{min} = 30(12)/4 = 90"$ ✓ controls

* Deck is not exactly parallel due to curve of building layout. Therefore, it will be assumed that concrete within the ribs of the metal deck does not provide any load resisting capacity

ΣQ_N (LW conc, $f'_c = 4$ ksi, 3/4" dia) $= 21.2^k$

$V_q' = 31(21.2) = 657^k$

$V_c' = 0.85 f'_c b_{eff} t = 0.85(4)(90)(3.25)$

$V_s' = A_s f_y = 7.68(50)$
$= 384^k$

$a = 384/0.85 f'_c b_{eff} = 384/0.85(4)(90)$
$= 1.25" < 3.25"$ ✓ ok

$y_2 = 6.25" - 1.25/2 = 5.62"$ → round down to 5.5"

checking table 3-19 in AISC manual
$\phi M_N > 148$ ft·k for all values of ΣQ_N
Design is ok but number of shear studs is high according to this calculation.

construction deflection
limit to $L/360 = 1.0"$
$w_{conc} = 6.25/12\,(120)(10') = 0.63$ klf
$w_{live} = 20$ psf $(10') = 0.20$ klf
$w_u = 1.2(0.63) + 1.6(0.20)$
$= 1.10$ klf

$I_{req} = 5 w_u L^4 / 384 \Delta E$
$= \dfrac{5(1.10)(30^4)(1728)}{384(1.0)(29000)}$
$= 691$ in⁴

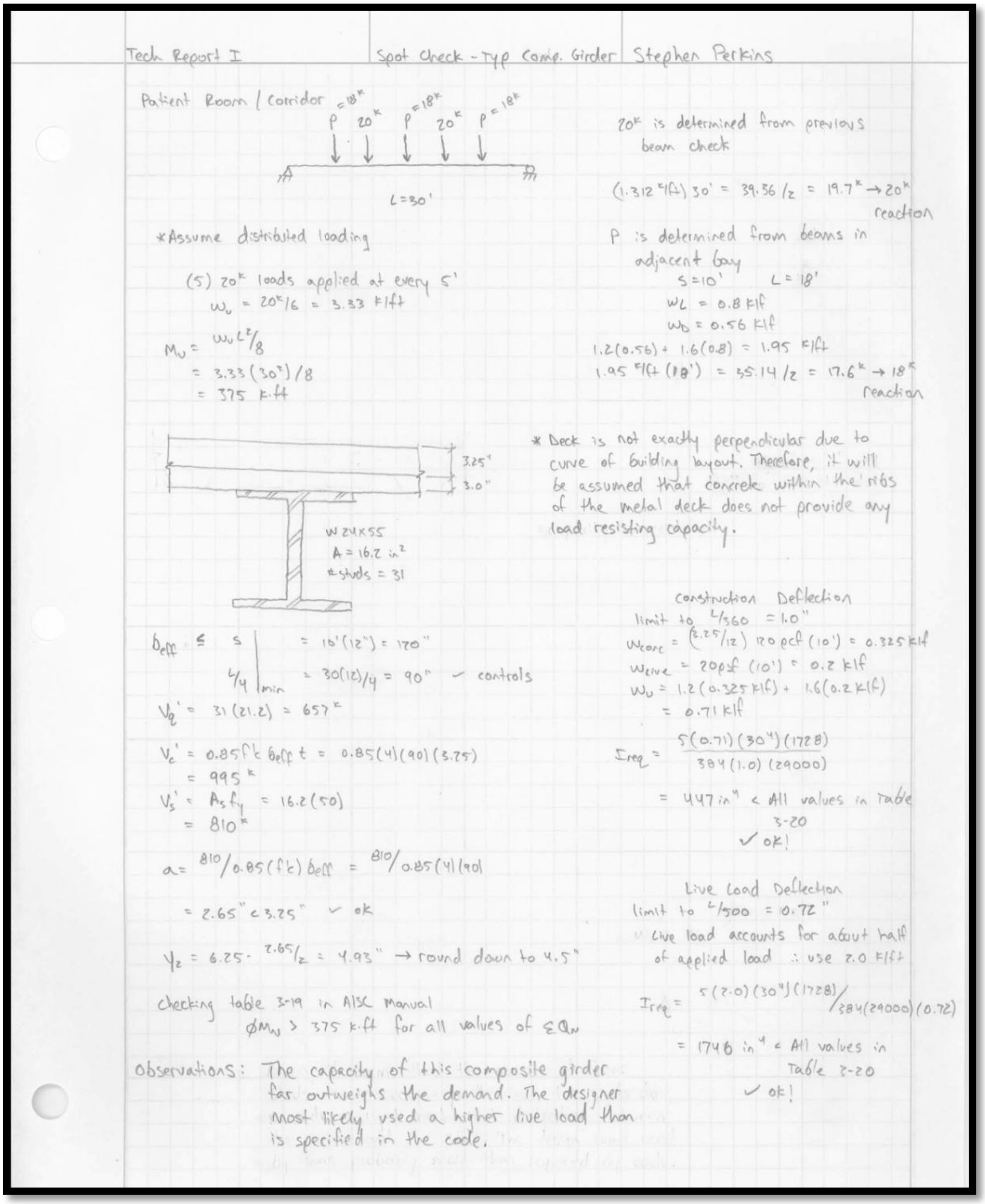
Table 3-20: $Y_1 = 0$, $Y_2 = 5.5"$
$I_{LB} = 985$ in⁴ > 691 in⁴ ✓ ok!

Observations:
It is possible that the designer chose to use a higher design live load than the value specified in the code. Also, it could be that this capacity is needed on other beams and the designer kept the same size and # studs for ease of construction.

Live Load deflection
limit to $L/500 = 0.72"$
$w_{LL} = 0.4$ klf

$I_{req} = 5 w_u L^4 / 384 \Delta E$
$= \dfrac{5(0.4)(30^4)(1728)}{384(0.72)(29000)}$
$= 349$ in⁴ < 985 in⁴ ✓ ok!

Figure 45: Composite beam spot check


Tech Report I | Spot Check - Typ Comp. Girder | Stephen Perkins

Patient Room / Corridor

$P = 18^k$ 20^k $P = 18^k$ 20^k $P = 18^k$

$L = 30'$

20^k is determined from previous beam check

$(1.312\ k/ft)\ 30' = 39.36/2 = 19.7^k \rightarrow 20^k$ reaction

*Assume distributed loading

P is determined from beams in adjacent bay

$S = 10'$ $L = 18'$

$w_L = 0.8$ klf

$w_D = 0.56$ klf

(5) 20^k loads applied at every 5'

$w_u = 20^k/6 = 3.33$ k/ft

$1.2(0.56) + 1.6(0.8) = 1.95$ k/ft

$1.95\ k/ft\ (18') = 35.14/2 = 17.6^k \rightarrow 18^k$ reaction

$M_u = w_u L^2/8$
$= 3.33(30^2)/8$
$= 375$ k.ft

3.25"
3.0"
W24x55
$A = 16.2\ in^2$
studs = 31

* Deck is not exactly perpendicular due to curve of building layout. Therefore, it will be assumed that concrete within the ribs of the metal deck does not provide any load resisting capacity.

$b_{eff} \leq$ s $= 10'(12") = 120"$
$L/4$ min $= 30(12)/4 = 90"$ ✓ controls

$V_Q' = 31(21.2) = 657^k$

$V_c' = 0.85 f'_c b_{eff} t = 0.85(4)(90)(3.25)$
$= 995^k$

$V_s' = A_s f_y = 16.2(50)$
$= 810^k$

$a = 810/0.85(f'_c) b_{eff} = 810/0.85(4)(90)$
$= 2.65" < 3.25"$ ✓ ok

$Y_2 = 6.25 - 2.65/2 = 4.93" \rightarrow$ round down to 4.5"

Checking table 3-19 in AISC Manual
$\phi M_n > 375$ k.ft for all values of ΣQ_n

Construction Deflection
limit to $L/360 = 1.0"$
$w_{conc} = (3.25/12)\ 120\ pcf\ (10') = 0.325$ klf
$w_{live} = 20psf\ (10') = 0.2$ klf
$w_u = 1.2(0.325\ klf) + 1.6(0.2\ klf)$
$= 0.71$ klf

$I_{req} = \frac{5(0.71)(30^4)(1728)}{384(1.0)(29000)}$
$= 447\ in^4 <$ All values in Table 3-20
✓ ok!

Live Load Deflection
limit to $L/500 = 0.72"$
Live load accounts for about half of applied load ∴ use 2.0 k/ft

$I_{req} = 5(2.0)(30^4)(1728)/384(29000)(0.72)$
$= 1746\ in^4 <$ All values in Table 3-20
✓ ok!

Observations: The capacity of this composite girder far outweighs the demand. The designer most likely used a higher live load than is specified in the code.

Figure 46: Composite girder spot check

Tech Report I | Spot check – Col@(H-N6) | Stephen Perkins

Evaluate @ Lower Level (column below Level 2)
* spliced @ level one and three

$P_u = 765^k$ (see col. takedown)

W14x145

$I_y = 677 \text{ in}^4$
$r_y = 3.98 \text{ in}$

$L = 15.0'$
$A = 42.7 \text{ in}^2$

$\frac{KL}{r_y} = \frac{15(12)}{3.98} = 45.22$

$4.71\sqrt{E/f_y} = 4.71\sqrt{29000/50} = 113 > 45.22$

∴ inelastic behavior

*Assume pin-pin K=1.0
Bracing occurring at floor → weak axis will control

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

$F_e = \pi^2 E / (KL/r)^2$
$= \pi^2(29000)/45.22^2$
$= 140 \text{ ksi}$

$F_{cr} = \left[0.658^{(50/140)}\right] 50$
$= 43 \text{ ksi}$

$P_{cr} = F_{cr} \cdot A = 43(42.7)$
$= 1838^k$

$\phi P_n = 0.9(1838)$
$= 1654^k \gg 765^k$

Observations: Based upon this spot check, it appears that the column is overdesigned. However, the New Hospital is actually designed for a future expansion of 4 extra floors.

If those theoretical floors are included, P_u increases to roughly 1200^k.

This was only a check of the column against gravity loads. It can be assumed that there are induced moments on this column which would account for the extra capacity. Since all girder-to-column connections are moment connections, this assumption makes sense.

Figure 47: Gravity column spot check