ECMC Skilled Nursing Facility

Architectural Engineering Senior Thesis 2011

Technical Report 1
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AE 481W

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Executive Summary

The existing conditions and structural concepts found in this report describe the physical conditions for the structure and relative design concepts of the ECMC Skilled Nursing Facility. All of the main structural elements were examined such that an overview could be concluded on how each structural component works together.

Existing drawings, specifications, and geotechnical reports were provided by Cannon Design, the lead designer of the project. These items design values were compared to the more recent codes and standards. Calculations were made on typical structural elements to help clarify the thesis design analysis performed on the ECMC Skilled Nursing Facility. In the event that direct design information was not presented, an educated assumption was made based on current knowledge and consultant clarification.

Calculations were performed according to ASCE 7-10 and IBC 2006 to obtain gravity and lateral loads. The loads included in this analysis are dead, live, snow, seismic, and wind loads. These calculations are compared to design loads provided by Cannon Design, who used ASCE 7-02 and the NYC Building Code of 2007. Thesis calculations produced base shears caused by wind load slightly greater than the original design base shear. This change is possibly due to the fact that the Importance factor is no longer included in the computation of velocity pressure q_z. The larger increase in basic wind speeds between the ASCE 7-02 and ASCE 7-10 codes also offer a more conservative approach to wind calculation.

A seismic analysis was performed on the structure and due to its radial geometry and layout of concentric brace frames throughout the structure, it was assumed that the building experienced similar seismic shear forces in both the N/S and E/W directions. The seismic loads calculated in this report were roughly half of what was calculated by Cannon Design. This large difference in base shear is possibly due to a miscalculation of C_s. The C_s calculated turned out to be very low when compared to Cannon Design's value of C_s. Another possibility for getting a low C_s value could be due to a poor assumption of the building's natural period. The calculation involved used an approximate building period, which may not be an accurate representation of the period which can be affected by the orientation of lateral systems throughout the building. These seem to be the main cause for the large difference in base shear.

Executive Summary (cont.)

Upon completion of these analyses, spot checks were performed to verify the validity of gravity loads on the structure. These spot checks may differ because of differing assumptions of live load. The calculations used a live load in corridors that was half of the value chosen by Cannon Design. The original document's live load may be higher because of a conservative decision. Another reason values may differ is the fact that most engineers checking these calculations check the system as a whole, allowing loads to be distributed and interact with other structural system components. The calculations in this report are only incorporating interactions from the individual member and not accounting for any carry over moments or shears from other structural components, such as the floor slabs.

Introduction

The new ECMC Skilled Nursing Facility serves as a long term medical care center for citizens found throughout the region. The building is located on the ECMC campus found at 462 Grider Street in Buffalo, NY. This site was chosen to bring residents closer to their families living in the heart of Buffalo. As you can see here in Figure 1, the site sits right off the Kensington Expressway, providing ease of access to commuters visiting the ECMC Skilled Nursing Facility. Since the Erie County Medical Center is found within close proximity of the new building, residents can receive fast and effective care in an event of emergency.

Figure 1: Aerial view of ECMC Skilled Nursing Facility site shown in white. Photo courtesy of Bing Maps.

The new facility is the largest of four new structures being built on the ECMC campus located in central Buffalo, NY. The new campus will also contain a new Renal Dialysis Center, Bone Center, and parking garage. Each of the three new facilities will be connected to the main medical center via an axial corridor, which provides enclosed access to emergency rooms, operation rooms, and other facilities found within the Erie County Medical Center.

Architectural Overview

The new Erie County Medical Center Skilled Nursing Facility is a five-story 296,489 square-foot building offering long-term medical care for citizens in the region. The facility consists of an eight-wing design with a central core. The main entrance to the building is located to the east and is sheltered from the elements by a large porte-cochere. There is a penthouse level that contains the facility's mechanical and HVAC units. Each floor features one garden terrace, providing an outdoor space accessible to both residents and staff. The exterior of the building is clad in brick, stone veneers, composite metal panels, and spandrel glass curtain wall system.

Figure 2: Exterior view of stacked garden terraces, green wall, and the building's vertical and horizontal shading panels. Rendering courtesy of Cannon Design.

The facility also incorporates green building into many of its elegant features. The composite metal panels that run vertically and horizontally across each wing of the building, visible in Figure 2, provide solar shading along with architectural accent. A green wall is featured on each outdoor garden terrace, providing residence with a sense of nature and greenery. The ECMC Skilled Nursing Facility provides an eclectic, modern atmosphere and quality care for long-term care patients found within the Buffalo area.

Structural Overview

The ECMC Skilled Nursing Facility consists of 8 wings and a central core, with an overall building footprint of about 50,000 square feet. The building sits at a maximum height of 90′ above grade with a common floor to floor height of 13′-4″. The ECMC Skilled Nursing Facility mainly consists of steel framing with a 5″ concrete slab on grade on the ground floor. The Penthouse level contains 6.5″ thick normal weight concrete slab on metal deck. All other floors have a 5.25″ thick lightweight concrete on metal deck floor system. All concrete is cast-in-place.

Foundation System

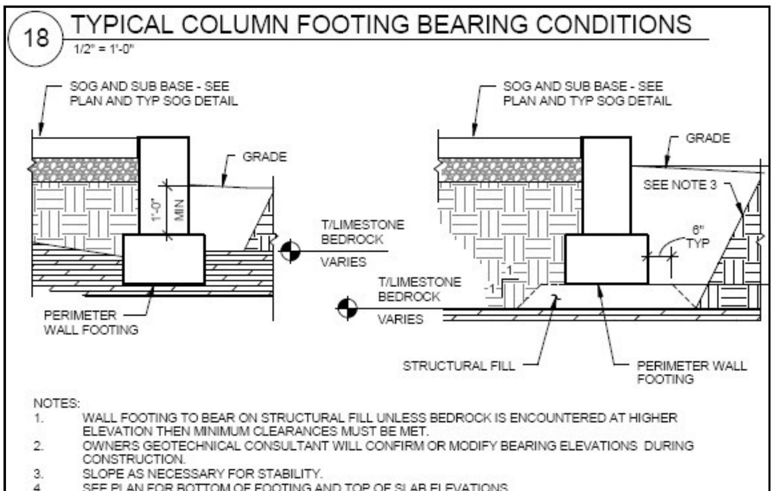


Figure 3: Footing bearing conditions. On bedrock (left detail), and on Structural Fill (right detail). Detail courtesy of Cannon Design.

The geotechnical report was conducted by Empire Geo Services, Inc. The study classified the soils using the Unified Soil Classification System, and found that the indigenous soils consisted mainly of reddish brown and brown sandy silt, sandy clayey silt, and silty sand. The ECMC Skilled Nursing Facility foundations sit primarily on limestone bedrock, although in some areas the foundation does sit on structural fill. Depths of limestone bedrock range from 2ft to 12ft. The building foundations of the ECMC Skilled Nursing Facility are comprised of spread footings and concrete piers with a maximum bearing capacity of 5,000 psf for footings on structural fill and 16,000 psf for footings on limestone bedrock. Concrete piers range in size from 22″ to 40″ square.

Floor System

The floor system on all floors except at the penthouse level consists of a 5.25" thick lightweight concrete floor slab on 2" - 20 gage metal decking, creating a one-way composite floor slab system. The concrete topping contains 24 pounds per cubic yard of blended fiber reinforcement. Steel decking is placed continuous over three or more spans except where framing does not permit. Shear studs are welded to the steel framing system in accordance to required specification. Refer to Figures 4 and 5 for composite system details.

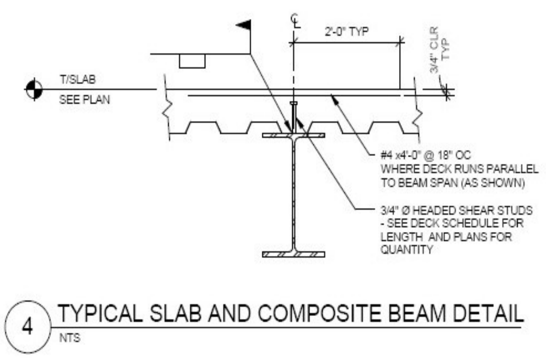


Figure 4: Composite deck system (parallel edge condition). Detail courtesy of Cannon Design.

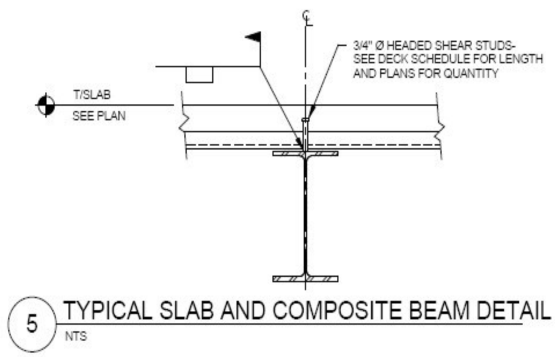


Figure 5: Composite deck system (perpendicular edge condition). Detail courtesy of Cannon Design.

Framing System

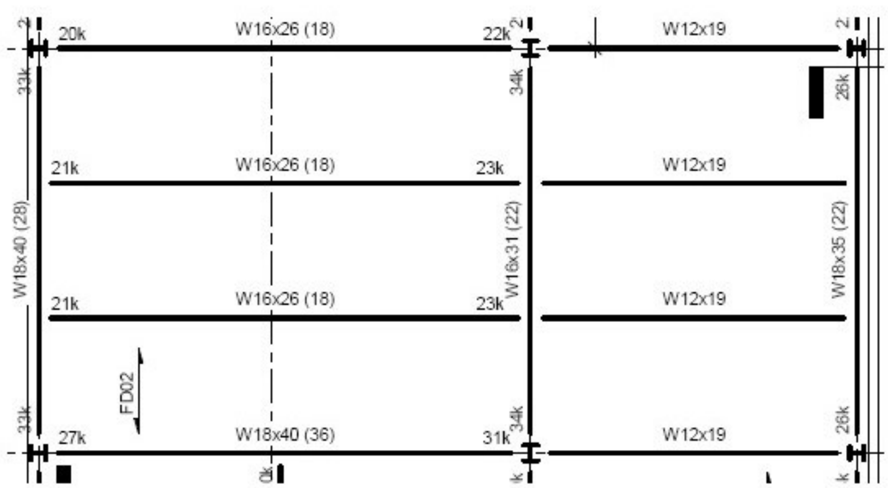


Figure 6: Typical bay layout for building wing. Detail courtesy of Cannon Design.

The structural framing system is primarily composed of W10 columns and W12 and W16 beams; however the girders vary in sizes ranging from W14 to W24, mainly depending on the size of the span and applied loads on the girder. Typical beam spacing varies from 6'-8"o.c. to 8'-8"o.c. Figure 6 shows a typical grid layout for a building wing. Columns are spliced at 4' above the 2nd and 4th floor levels, and typically span between 26'-8" and 33'-4".

Lateral System

The lateral resisting system consists of a concentrically brace frame system composed of shear connections with HSS cross bracing. Lateral HSS bracing is predominantly located at the end of each wing, and also found surrounding the central building core. Because of the radial shape of the building and symmetrical layout of the structure, the brace framing can oppose seismic and wind forces from any angle. The HSS bracing size is mainly HSS 6x6x3/8, but can increase in size up to HSS 7x7x1/2 in some ground floor areas for additional lateral strength. Figure 7 contains multiple details and an elevation of a typical brace frame for the ECMC Skilled Nursing Facility.

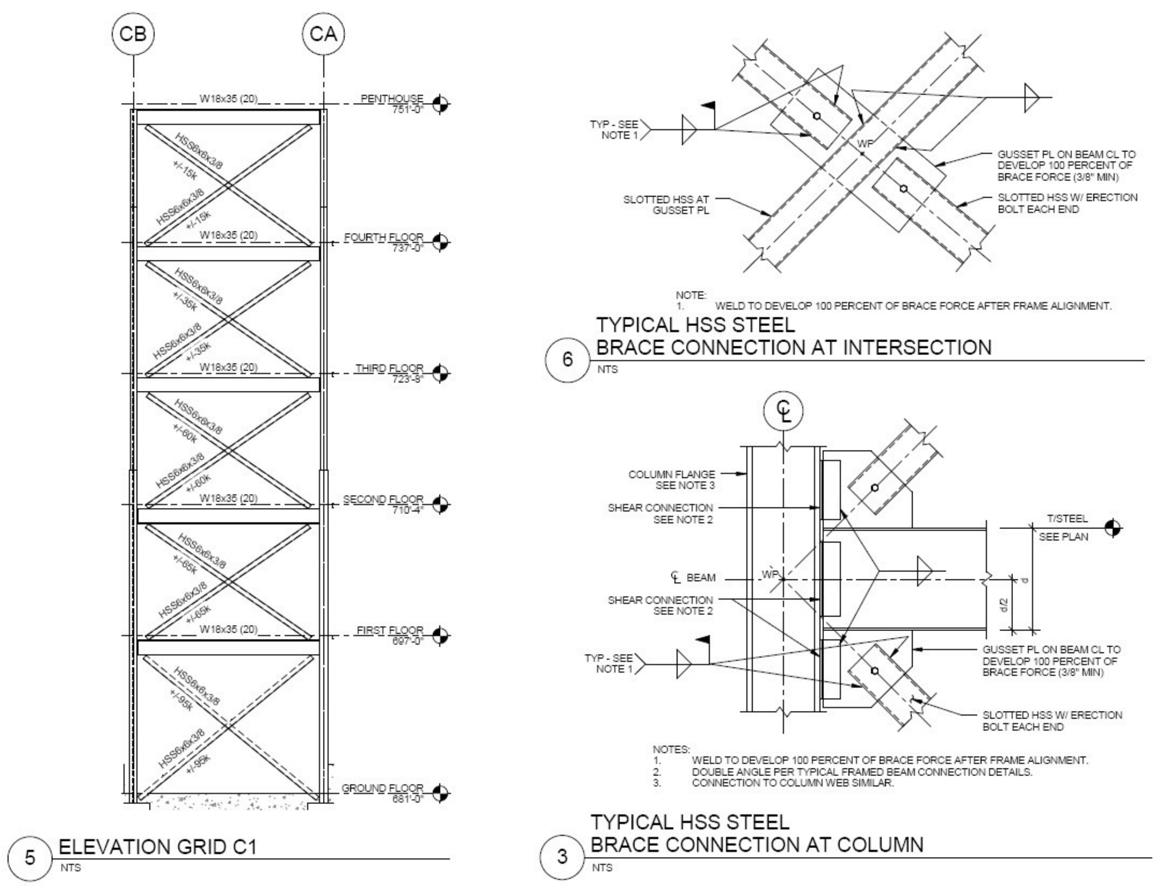


Figure 7: Typical lateral HSS brace frame (left). Typical HSS steel brace connection at intersection (upper right). Typical HSS steel brace connection at column (lower right). Details courtesy of Cannon Design.

Design Codes and Standards

Original Codes:

Design Codes:

- ACI 318-02, *Building Code Requirements for Structural Concrete*
- ACI 530-02, *Building Code Requirements for Masonry Structures*
- AISC LRFD - 3rd Edition, *Manual of Steel Construction: Load and Resistance Factor Design*
- AWS D1.1 - 00, *Structural Welding Code - Steel*

Model Code:

- NYS Building Code - 07, *Building Code of New York State 2007*

Structural Standard:

- ASCE 7-02, *Minimum Design Loads for Buildings and Other Structures*

Thesis Codes:

Design Codes:

- ACI 318-08, *Building Code Requirements for Structural Concrete*
- AISC Steel Construction Manual - 13th Edition (LRFD), *Load and Resistance Factor Design Specification for Structural Steel Buildings*

Model Code:

- IBC - 06, *2006 International Building Code*

Structural Standard:

- ASCE 7-10, *Minimum Design Loads for Buildings and Other Structures*

Material Properties

Materials

Structural Steel		
Wide Flange Shapes, WT Sections	ASTM A992	
Channels and Angles	ASTM A36	
Pipe	ASTM A53 Grade B	
Hollow Structural Sections (Rectangular and Round)	ASTM A500 Grade B	
Base Plates	ASTM A36 UNO	
All Other Steel Members	ASTM A36 UNO	
High Strength Bolts, Nuts, and Washers	ASTM A-325 / A-490 (Min. 3/4" Diameter)	
Anchor Rods	ASTM F1554	
Steel Shape Welding Electrode	E70XX	
Concrete	**F'c (psi)**	**Unit Weight (pcf)**
Footings	f'c = 3000psi	145
Foundation Walls	f'c = 4000psi	145
Slabs-on-Grade	f'c = 3000psi	145
Slabs-on-Steel Deck (Floor Deck 1)	f'c = 3000psi	145
Slabs-on-Steel Deck (Floor Deck 2)	f'c = 3000psi	115
All Other Concrete	f'c = 4000psi	145
Reinforcement		
Typical Bars	ASTM A-615 Grade 60	
Welded Bars	ASTM A-706 Grade 60	
Welded Wire Fabric	ASTM A-185	
Steel Fibers	ASTM A-820 Type 1	
Decking		
Floor Deck (both types)	2" Composite Metal Deck, 20 Ga.	
Roof Deck Type 1	1 1/2" Type B Metal Roof Deck, 20 Ga.	
Roof Deck Type 2	1 1/2" Type B Metal Roof Deck, 18 Ga.	
3/4" Shear Studs	ASTM A-108	

Table 1: This table describes material properties found throughout the building.

Design Loads

Dead and Live Loads

The original structure of the ECMC Skilled Nursing Facility was designed using ASCE 7-02 and the 2007 NYC Building Code. These load cases are compared to the newer ASCE 7-10 standard. Their differences can be seen in Table 2 below. Loads used for thesis analysis are from the ASCE 7-10 standards unless unspecified in the code. Refer to Appendix B for Dead Load Calculations/Assumptions.

Superimposed Dead Loads			
Description	**Location**	**NYC-BC 2007**	**ASCE 7-10**
Roof Deck 1	Roof	2psf	2psf
Roof Deck 2	Penthouse Roof	3psf	2psf
Floor Deck 1	Penthouse Floor	2psf	2psf
Floor Deck 2	Floors 1-4	2psf	2psf
Floor Finishings	Floors 1-4	2psf	2psf
Roofing & Insulation	Roof + Penthouse Roof	8psf	8psf
Leveling Concrete	Floors 1-4	5psf	5psf
Ceilings	Floors 1-4 + Penthouse	5psf	5psf
Typical Suspended MEP	Floors G-4	5psf	5psf
Penthouse Suspended MEP	Penthouse	8psf	8psf
Partitions	Floors 1-4	18psf	18psf
Pavers, Potted Plants	Floors 1-4	80psf	- -
Green Wall (4"thick)	Floors 1-4	20psf	- -
Live Loads			
Description		**NYC-BC 2007**	**ASCE 7-10**
Resident Rooms	Floors G-4	40psf	40psf
Ground Floor Corridors	Floor G	80psf	100psf
Balconies	Floors 1-4	Not Specified	100psf
Resident Corridors	Floors 1-4	80psf	80psf
Penthouse Floor	Penthouse	150psf	150psf
Public Spaces/Exit Corridors/ Stairs/Lobbies	Floors G-Penthouse	100psf	100psf
*Live load reductions used where applicable **Snow drift included where applicable			

Table 2: The table above shows a list of dead and live loads used in the various calculations found in this report, along with a comparison of loads between the NYC BC-2007 versus ASCE 7-10

Wind Load Analysis

Using the Wind Load Directional Procedure in ASCE 7-10, an assumption was made that the ECMC Skilled Nursing Facility façade and geometry were entirely regular with no wings protruding out. The building has a symmetric, radial footprint, so giving the building a square box-like shape seemed to fit. Table 3 shows sample variables used in the wind load calculation, and Table 4 illustrates the summary of wind pressures and story forces. Figure 8 shows the pressure distribution on the building and Figure 9 illustrates story shear forces.

Building Category	III	Damping Ratio(β)	0.02
Basic Wind Speed (V)	120mph	Natural Frequency (n_a)	0.833
Wind Directionality Factor (K_d)	0.85	L/B	1
Exposure Category	B	I_z	0.2764
Topographic Factor (K_{zt})	1	L_z	377.09
α	7	Q	0.7614
Z_{min}	30	V_z	120.7
G_f	0.821	N_1	2.602
K_z	0.96	R_n	0.0762
GC_{pi}	(+/- 18 psf)	R_h	0.3195
Cp(windward walls)	0.8	R_b	0.0895
Cp(leeward walls)	-0.5	R_L	0.0272
Cp(side walls)	-0.7	g_R	4.15
Cp(0-h/2)	-0.9	R	0.2432
Cp(h/2-h)	-0.9	η_h	2.856
Cp(h-2h)	-0.5	η_B	10.92
Cp(>2h)	-0.3	η_L	36.55

Table 3: The table above shows variables and classifications necessary to calculate wind pressures using ASCE 7-10.

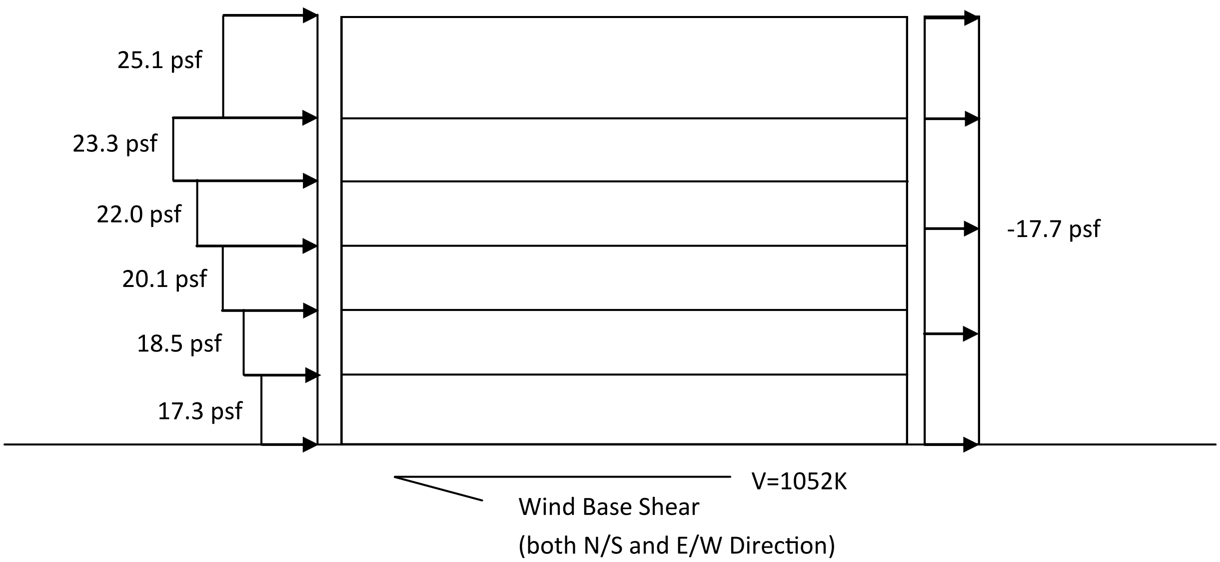


Figure 8: The table above shows the floor wind pressures and forces along with shear/moment forces ion the building.

Wind Loads								
Floor	Story Height (ft)	Height Above Ground (ft)	Controlling Wind Pressure (PSF)		Total Con-trolling Pressure (psf)	Force of Windward Pressure (K)	Story Shear Windward (K)	Moment Windward (ft-k)
			Windward	Leeward				
Penthouse Roof	20	90	25.1	-17.7	42.8	147.2	0	13248
Penthouse Floor	20	70	23.3	-17.7	41	238.9	147.2	16723
4th Floor	13	57	22	-17.7	39.7	177.3	386.1	10106.1
3rd Floor	15	42	20.1	-17.7	37.8	170.2	563.4	7148.4
2nd Floor	13	29	18.5	-17.7	36.2	162.3	733.6	4706.7
1st Floor	13	16	17.3	-17.7	35	156.1	895.9	2497.6
Ground Floor	16	0	0	0	0	0	1052	0
						Σ	**1052**	**54429.8**

Table 4: The table above shows the floor wind pressures and forces along with shear/moment forces ion the building.

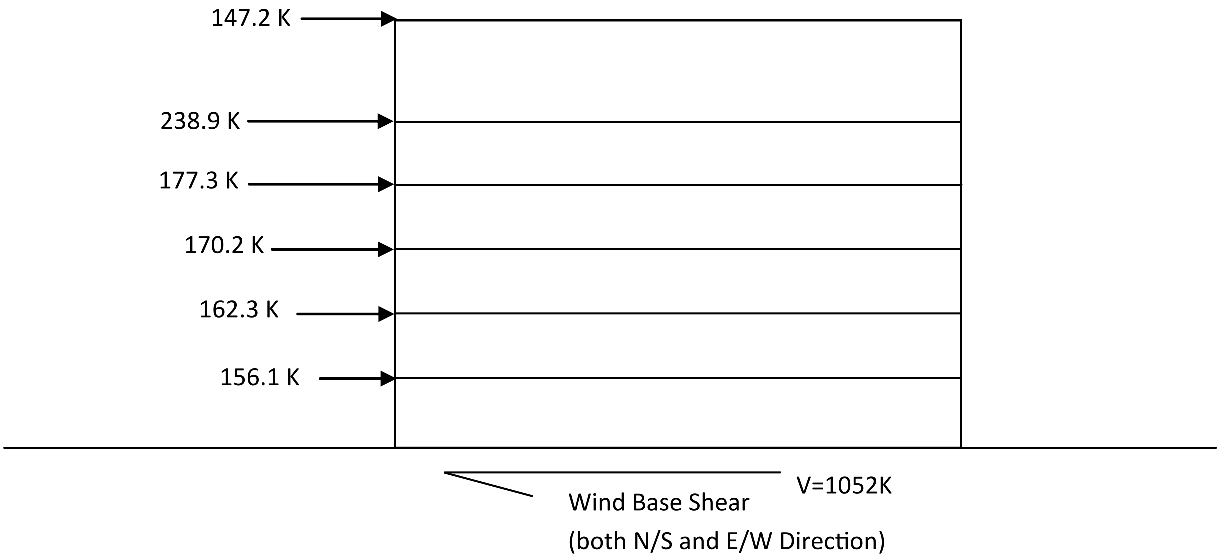


Figure 9: This figure shows the wind shear force at each story in the building.

Wind Load Analysis Conclusion

The wind loads calculated by the structural engineers at Cannon Design were computed using ASCE 7-02. One major difference between the older code and the new ASCE 7-10 code are the increase in basic wind speeds. This change is possibly due to the fact that the Importance factor is no longer included in the computation of velocity pressure q_z. The change also offers a more conservative approach to wind calculation. The base shear found in the thesis study of 1052K is slightly larger than the total building shear caused by wind in the original construction drawings, which is 980K. The larger value found confirms that the new ASCE 7-10 wind load method is a more conservative approach. The original plans do show a slight difference in building shear when the N/S direction is compared against the E/W direction. This difference is likely caused by subtle differences in the buildings radial shape such as overhangs, or the connected canopy over the entrance may create higher building shear due to aerodynamic effects. A step by step calculation of wind loads can be found in Appendix C.

Seismic Load Analysis

The thesis study of the ECMC Skilled Nursing Facility was designed for seismic using ASCE 7-10 Equivalent Lateral Force Procedure found in Section 12.8. Loads used in the analysis consisted of dead loads from floor slabs, roof deck, MEP, and framing. Seismic calculations were performed by hand, and approximate square footages were taken from construction documents. Table 7 shows variables and classifications used in the seismic analysis, and because of the buildings radial geometry, shear forces found in the analysis are assumed to be the same in the N/S direction versus the E/W direction. Table 8 displays results for the seismic analysis from the hand calculations.

Seismic Variable		ASCE 7-10 Reference
S_s	0.211g	USGS WEBSITE
S_1	0.060g	USGS WEBSITE
Site Classification	B	Table 20.3-1
F_A	1.0	Table 11.4-1
F_V	1.0	Table 11.4-2
S_{MS}	0.211	USGS WEBSITE
S_{M1}	0.060	USGS WEBSITE
S_{DS}	0.140	USGS WEBSITE
S_{D1}	0.040	USGS WEBSITE
Occupancy Category	III	Table 1-1
Importance Factor	1.25	Table 1.5-2
Seismic Design Category	A	Table 11.6-1

Table 5: This table shows variables and references to compute a seismic analysis using the Equivalent Lateral Force Procedure in ASCE 7-10.

Equivalent Lateral Force Procedure		
T_L	6 s	Figure 22-12
C_t	0.030	Table 12.8-2
x	0.75	Table 12.8-2
T_a	0.88 s	Section 12.8.2.1
C_u	1.4	Table 12.8-1
R	6	Table 12.2-1
Cs	0.0095	Equation 12.8-5
W	26,045 K	Refer to Appendix C
V	247.4 K	Refer to Appendix C
k	1.19	Section 12.8.3

Table 6: This table shows a summary of variable results for calculations for seismic analysis using the Equivalent Lateral Force Procedure in ASCE 7-10.

Equivalent Lateral Force Procedure following Table 12.6-1							
Floor	Weight w_x (K)	Height h_x (ft)	$w_k h_x^k$ (K)	C_{vx}	Lateral Force F_x (K)	Story Shear V_x (K)	Moment M_x (K)
Penthouse Roof	1,017	90	215,214	0.090	22.3	22.3	2007
Penthouse Floor	4,142	70	649,945	0.271	67.1	89.4	4697
4th Floor	5,221	57	641,571	0.268	66.3	155.7	3779.1
3rd Floor	5,221	43	458,755	0.192	47.5	203.2	2042.5
2nd Floor	5,221	29	287,083	0.120	29.7	232.9	861.3
1st Floor	5,221	16	141,467	0.060	14.8	247.7	236.8
Ground	0	0	0	0	0	0	0
TOTAL	26,043		2,394,036	1	247.7		13623.7

Table 7: This table shows the calculations and processes needed in order to calculate seismic base shear using Equivalent Lateral Force Procedure in ASCE 7-10.

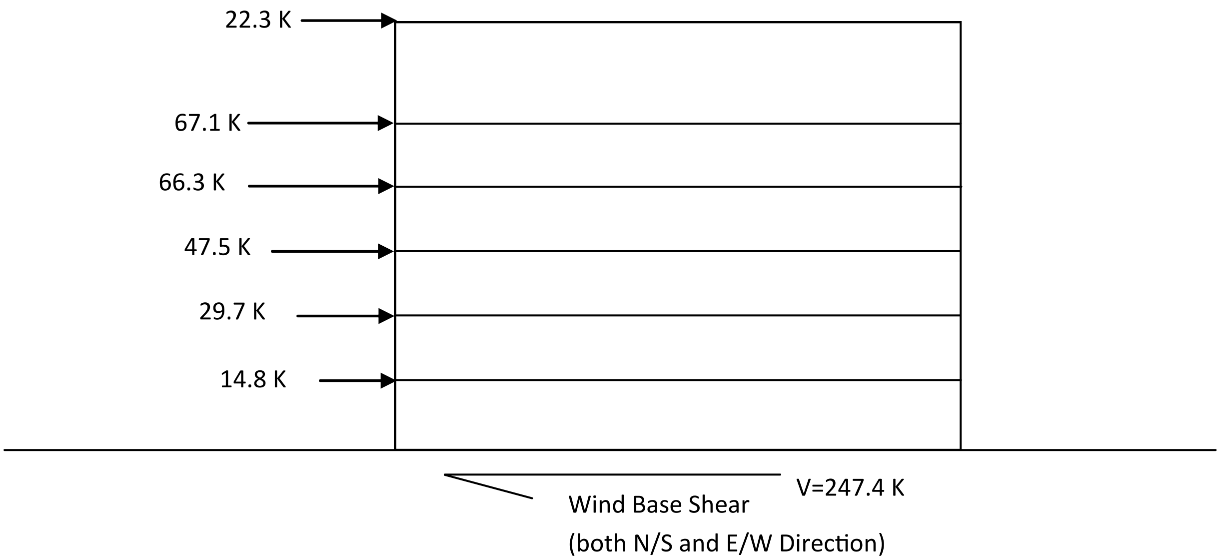


Figure 10: This table shows calculated seismic shear at each story level throughout the building.

Seismic Load Analysis Conclusion

The seismic loads base shear, V=247.4 K, calculated above came out to be a little over half the amount of the base shear caused by seismic forces found within the original drawings, V=430 K. One reason for this large difference in base shear is a possible miscalculation of C_s. The C_s in the calculations was recorded at 0.0095, yet the original specifications state that their C_s is equal to 0.030, a much larger value producing a base shear of 430K. This seems to be the main cause for the large difference in base shear. For seismic hand calculations, please refer to Appendix D.

Snow Load Analysis & Discussion

The snow loads were calculated using various charts and tables found in ASCE 7-10. Table 8 shows the difference in variables and ground snow loads between the original drawings and thesis analysis loads. For more in depth calculations on snow loads, refer to Appendix D.

Snow Loads		
Description	Original Loads	Calculated Loads
P_g	50	50
I_s	1	1.1
C_e	1	0.9
C_t	1	1
P_f	38.5	34.7
P_{drift}	98	95.2

Table 8: This table compares values for snow load between the original construction documents and thesis hand calculated values.

Original loads and calculated loads are closely similar, yet slightly vary. This slight variation is probably due to the slight inconsistencies between the I_s and C_e values. These values differ because of changes in separate versions of ASCE 7. For snow load calculations, please refer to Appendix E.

Gravity System Spot Checks

Typical Slab on Metal Deck

The system described in the construction documents utilized a 5.25" thick lightweight concrete slab on 2" - 20 gage metal decking. Typical dead and live loads were applied to this system and calculations found that this slab is slightly overdesigned, finding that a 2"- 22 gage metal deck at this concrete thickness should be sufficient to carry the loads over the required spans. Figures 11 and 12 illustrate a typical section of a composite concrete slab on deck.

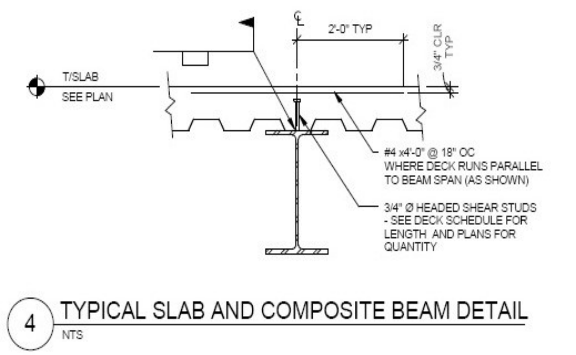


Figure 11: Composite deck system (parallel edge condition). Detail courtesy of Cannon Design.

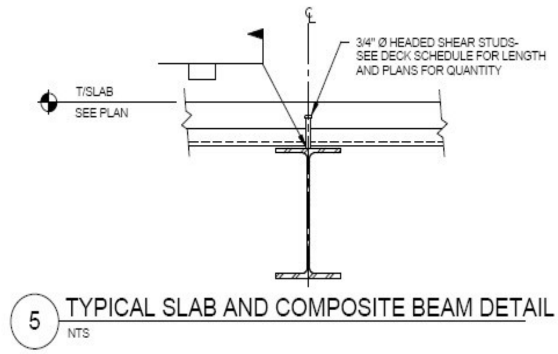


Figure 12: Composite deck system (perpendicular edge condition). Detail courtesy of Cannon Design.

Gravity System Spot Checks

Typical Composite Beam and Girder

According to composite beam and girder spot checks on a typical bay, the designer took a conservative design approach by using a larger live load than required. The calculations uses a live load of 40psf which is specific to ASCE 7-10 code, however the designer used a live load of 80psf. Other reasons for possible discrepancies in beam and girder size are possibly due to selection of beam/girder depth or the number of shear studs selected. The deeper the beam/girder, the more strength capability it has versus flexural strength. Figures 11 and 12 help illustrate how typical framing members interact with typical floor systems.

Gravity System Spot Checks

Typical Column

The column analyzed extended from the ground floor to the roof, with splicing above the 2nd and 4th floors. The column analyzed was on the ground floor, since it would be carrying the largest amount of axial load. The design called for a W10x60 interior column located at gridlines C12-CF. This column supported private residential rooms and a central corridor, using a 40psf live load for both spaces. The column assumed a pin-pin situation, giving it a K=1.0. Also, the unbraced length was assumed to be the floor to floor height of 16ft. Live load reduction was used in computing the maximum axial load P_u. With the use of Table 4-1 in the AISC Steel Construction Manual, 13th edition, calculations showed that a W10x49 column would be sufficient in supporting the loads given. The slight change in size may have to do with the reduction in live load, along with the fact that the designer live load was assumed to be 80psf in the corridor.

For all spot check calculations, please refer to Appendix F.

Final Summary & Conclusion

Although there are differences between ASCE 7-02 and ASCE 7-10, the designer values and the calculated values found within this report were relatively similar. Subtle differences in size or value varied mainly because of load approximation. These discrepancies were usually due to the difference in values found within the IBC and NYC Building Code, as opposed to ASCE 7-10 codes.

Designer wind values were slightly smaller than the calculated thesis values for wind, which was expected. This is largely due to the change between ASCE 7-10 and 7-02, where basic wind speeds were increased drastically throughout the nation. This increase in wind speed creates a larger, more conservative value for wind pressure.

Seismic loads varied greatly, possibly due to a very low calculated C_s. Designer seismic loads were about twice the amount of the hand calculated value. Because of the radial layout of the concentrically brace frames, it is difficult to calculate the building's actual period. This layout could have also caused some type of torsional effect on the building. An approximation for building period was used, which could have poorly described the actual building period. It is assumed that either a miscalculation of Cs or poor approximation of building period (T) could have caused the building shear values to be half of the expected values.

When performing spot checks, it was found that the slab on deck system was very similar between designer and calculated values. The thesis slab on deck only differed by one gage. Typical framing members such as beams and girders were smaller than designer beams and girders. This was mainly due to the differences in live load. Another possible reason is how designers may use computer analysis programs, which factor in the entire systems structural properties instead of evaluating the individual member as in the hand calculations.

Appendix A: Framing Plans and Elevations

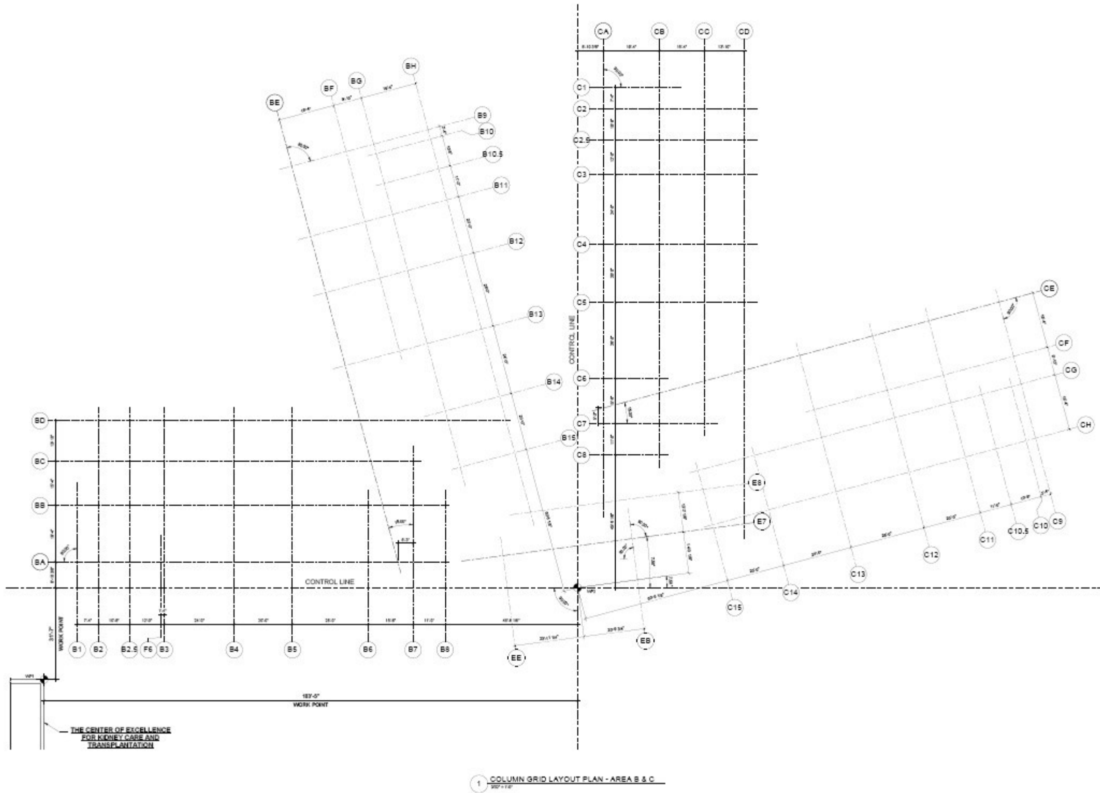
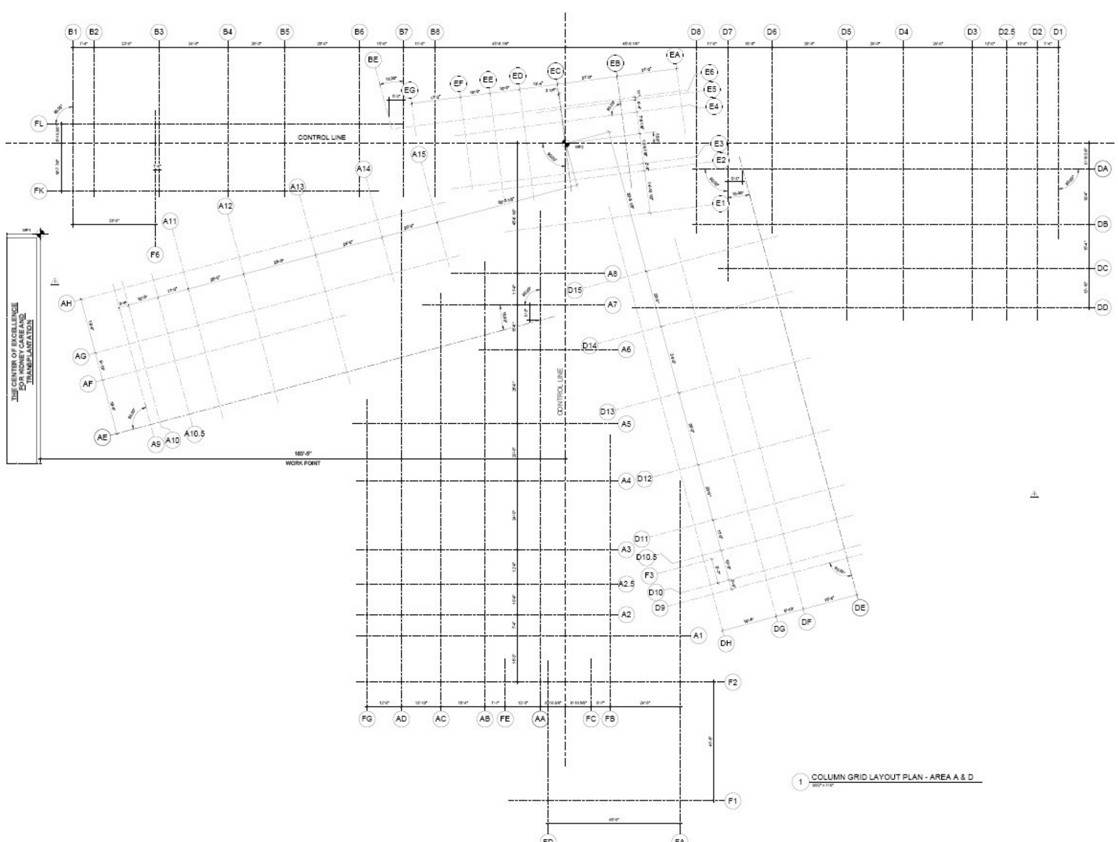
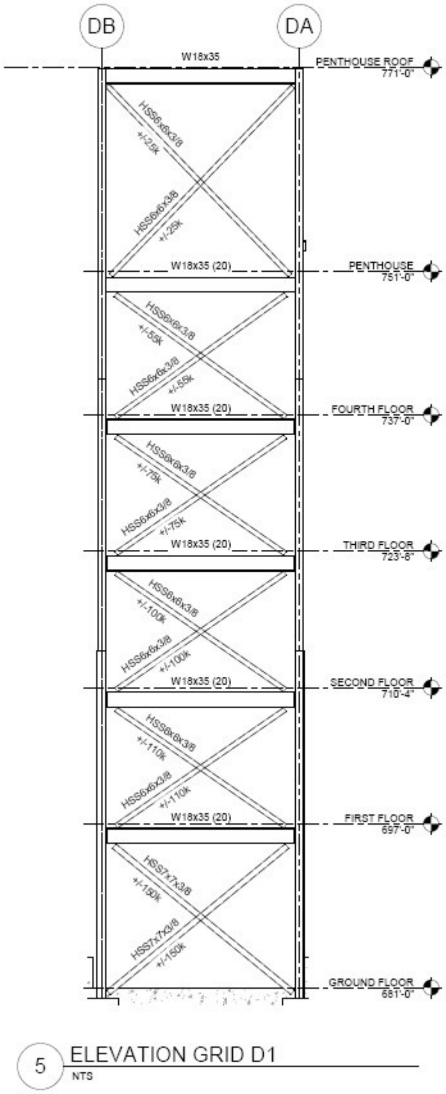
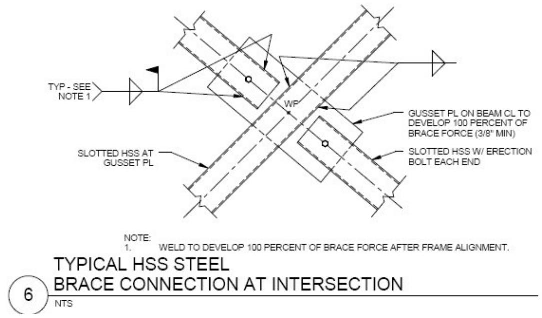
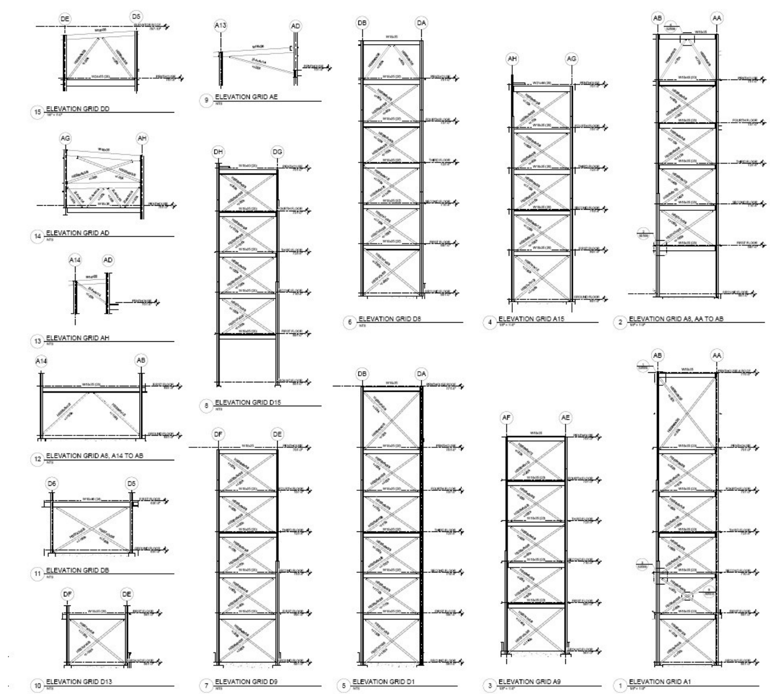


Figure 13: Column Grid Layout Plans (East End on bottom, West End on top) Details courtesy of Cannon Design.









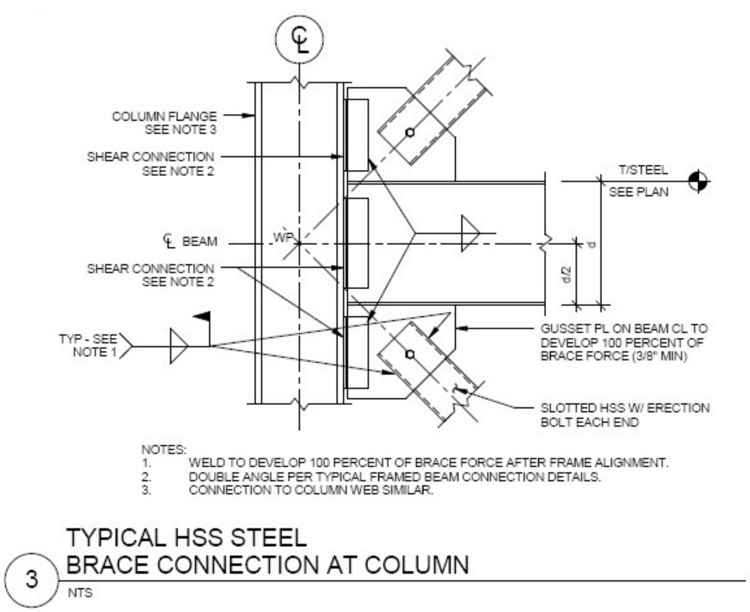


Figure 14: Concentric HSS Brace Frames and Connection Details. Details courtesy of Cannon Design.

Appendix B: Dead & Live Load Calculations

Dead/Live Load	Tech 1 Report	BRIAN BRUNNET	1

* Using ASCE 7-10

Dead Loads:

- Roof DL
  + MTL deck: 2.15 psf
  + Insulation: 2psf
  + MEP: 18 psf
  + Framing: 10psf

  32.2 psf

- Penthouse Floor DL
  + MTL Deck: 2psf
  + NWC topping: 145pcf × $\frac{6.5''}{12''}$ = 78.5psf
  + Blended Fiber Reinf.: 24pcf × $\frac{6.5''}{12''}$ = 13psf
  + MEP: 20 psf
  + Framing: 10psf

  123.5psf

- Floors (1 → 4) DL
  + MTL Deck: 2psf
  + LWC topping: 115pcf × $\frac{5.25''}{12''}$ = 50.3psf
  + Blended Fiber Reinf.: 24pcf × $\frac{5.25}{12}$ = 10.5psf
  + MEP: 18psf
  + Framing: 10psf

  90.8psf

Live Loads: ASCE 7-10

Corridors: 140psf

Lobbies: 100psf

Balconies: 100psf

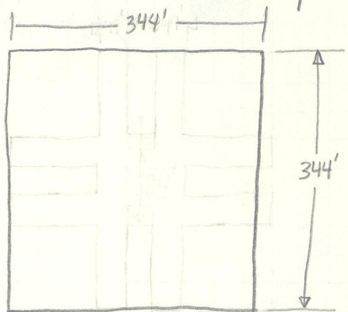
Resident Rooms: 40psf

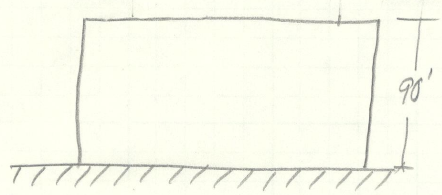
Stairs & Exits: 100psf

1st floor Corridor: 100psf

Appendix C: Wind Load Calculations

<u>Note:</u> Because of symmetric radial pattern for building footprint, I assume that pressures caused by wind in N-S direction will be similar to pressures experienced by wind in E-W direction.





<u>Assume:</u> Building Footprint (Symmetric)

max. $h = 90\text{ ft}$

<u>STEPS TO FIND MWFRS WIND LOADS (Table 27.2-1)</u>

1.) Determine Risk Category (Tab 1.4-1)

Category III

2.) Determine Wind Speed

$V = 120\text{ mph}$

3.) Determine Wind Load Parameters:

$K_d = 0.85$

Exposure Category: B

$K_{zt} = 1.00$

<u>For G:</u> Calculate Approx. Natural Frequency (n_1)

$$g_r = \sqrt{2\ln(3600(.83))} + \frac{.577}{\sqrt{2\ln(3600(.83))}}$$

→ For concrete or steel bldgs with other lateral force resisting systems: $g_r = \underline{4.15}$

$$n_1 = \frac{75}{H} = \frac{75}{90'} = 0.833\text{Hz} < 1\text{Hz} \longrightarrow \text{FLEXIBLE}$$

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

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

$$L_{\bar{z}} = l\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$$

$$I_{\bar{z}} = 0.3\left(\frac{33}{0.6(90')}\right)^{1/6} = \underline{0.28}$$

$$L_{\bar{z}} = 320\left(\frac{54}{33}\right)^{\frac{1}{3}}$$

$$L_{\bar{z}} = 377$$

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

$$G_f = 0.925\left(\frac{1 + 1.7(0.28)\sqrt{(3.4)^2(.76)^2 + (4.15)^2(R)^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$$

$$Q = \sqrt{\frac{1}{1 + .63\left(\frac{344' + 90'}{377}\right)}} = \underline{0.76}$$

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

$$R = \sqrt{\frac{1}{.02} (.084)(.289)(.087)(.53 + .47(.027))}$$

$R = 0.239$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{10}\right)^{\bar{a}}\left(\frac{88}{60}\right)V$$

$$\bar{V}_{\bar{z}} = (0.45)\left(\frac{54}{10}\right)^{\frac{1}{4}}\left(\frac{88}{60}\right)(120\text{ mph}) = \underline{120.7 \text{ mph}}$$

for R_h, $\eta = 4.6 n_1 \frac{h}{\bar{V}_{\bar{z}}} = 4.6(.833)\frac{90}{120.7} = 2.86$

for R_B, $\eta = 4.6 n_1 \frac{B}{\bar{V}_{\bar{z}}} = 4.6(.833)\frac{344}{120.7} = 10.92$

for R_L, $\eta = 15.4 n_1 \frac{L}{\bar{V}_{\bar{z}}} = 15.4(.833)\frac{344}{120.7} = 36.6$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{(.833)(320)}{120.7} = 2.21$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(2.21)}{(1 + 10.3(2.21))^{5/3}} = \underline{0.084}$$

$$R_h = \frac{1}{2.86} - \frac{1}{2(2.86)^2}\left(1 - e^{-2(2.86)}\right) = \underline{0.289}$$

$$R_B = \frac{1}{10.92} - \frac{1}{2(10.92)^2}\left(1 - e^{-2(10.92)}\right) = \underline{0.087}$$

$$R_L = \frac{1}{36.6} - \frac{1}{2(36.6)^2}\left(1 - e^{-2(36.6)}\right) = 0.0269$$

From previous:

$$G_f = 0.925 \frac{1 + 1.7(.28)\sqrt{(3.4)^2(.76)^2 + (4.15)^2(.239)^2}}{1 + 1.7(3.4)(0.28)} = \underline{\underline{0.819}}$$

Enclosure Classification: <u>Fully Enclosed</u> $(GC_{pi} = \pm 0.18)$

4.) Determine velocity pressure exposure coefficient:

$K_z = K_h = \underline{0.96}$ at 90', (Exposure B)

5.) Determine velocity pressure:

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

$$q_z = 0.00256(0.96)(1.0)(0.85)(120)^2 = \underline{30.08 \text{ psf}}$$

6.) Determine external pressure coefficient C_p or C_N:

Wall C_p:

Windward walls: $C_p = 0.8$

Leeward walls: $C_p = -0.5$

Side walls: $C_p = -0.7$

$$\frac{L}{B} = \frac{344'}{344'} = 1.0$$ symmetric plan

Roof C_p: Slope $\Rightarrow \frac{3}{4}'' / 12'' = 3.58^\circ < 10^\circ$

$$\frac{h}{L} = \frac{90'}{344'} = 0.262 \leq 0.5$$

Horiz. Dist From Windward edge:

$0 - h/2 \longrightarrow C_p = -0.9$, -0.18

$h/2 - h \longrightarrow C_p = -0.9$, -0.18

$h - 2h \longrightarrow C_p = -0.5$, -0.18

$2h \longrightarrow C_p = -0.3$, -0.18

7.) Calculate wind pressure, p, on each building surface:

MWFRS Pressures: $p = qG_fC_p - q_i(GC_{pi})$ [psf]

Controlling Pressures:

Windward Walls:

$p = (30.08)(0.819)(0.8) - (30.08)(\pm 0.18) = 19.7 \pm 5.4$ psf $\rightarrow$ $+25.1$ psf

Leeward Walls:

$p = (30.08)(.819)(-.5) - (30.08)(\pm 0.18) = -12.3 \pm 5.4$ psf $\rightarrow$ -17.7 psf

Roof:

$p = (30.08)(.819)(-0.9) - (30.08)(\pm 0.18) = -22.2 \pm 5.4$ psf

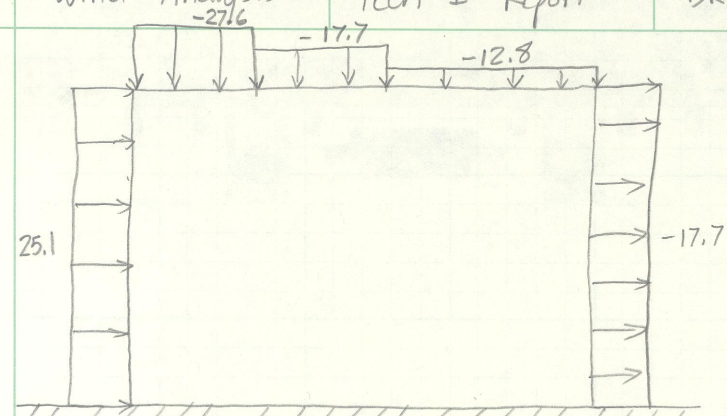
$= (-22.2) \pm 5.4$ psf for 0 to 90' $\longrightarrow$ -27.6 psf

$= (-12.3) \pm 5.4$ psf for 90' to 180' $\longrightarrow$ -17.7 psf

$= (-7.4) \pm 5.4$ psf for $> 180'$ $\longrightarrow$ -12.8 psf

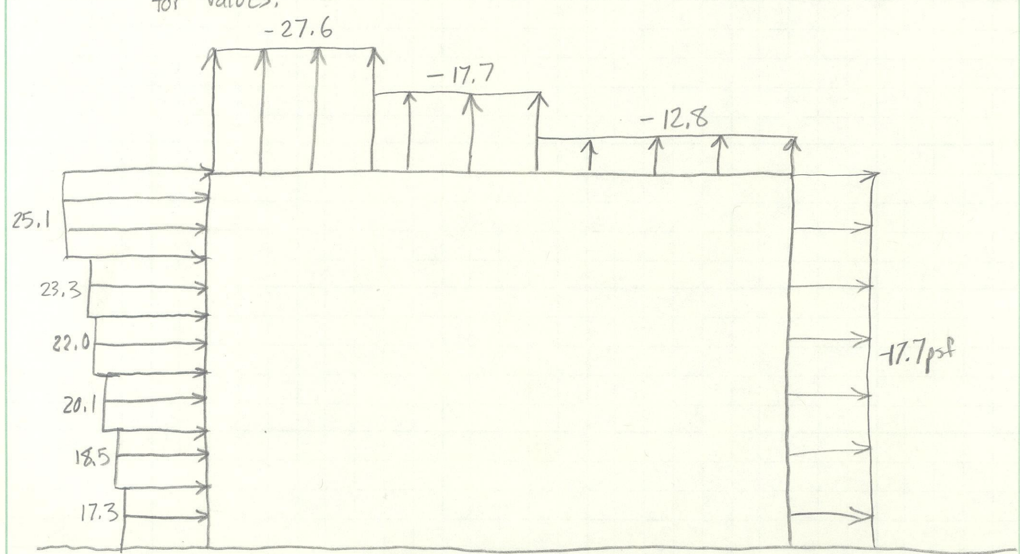
side walls:

$p = (30.08)(.819)(-.7) - (30.08)(\pm .18) = -17.2 \pm 5.4$ psf $\longrightarrow$ -22.6 psf



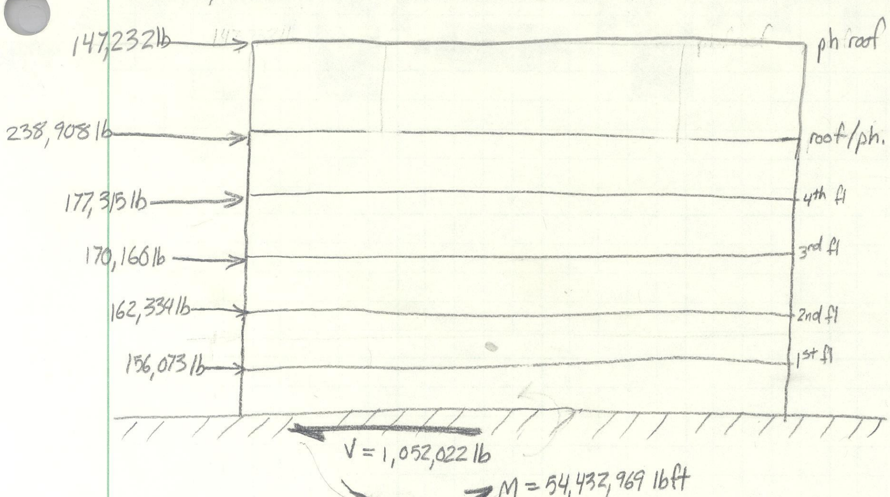
Values in (psf)

→ Note: These values show wind pressures ONLY at the height of 90'. Using Microsoft Excel as a time-saving tool, I repeated the following wind load analysis to attain a more accurate load distribution. Each new repetition is calculated at each story level, providing accurate pressures at 16', 29', 42', 57', 70', and 90' above grade for windward, leeward, + side walls. The sketch below should depict the more accurate wind load diagram. Refer to excel spreadsheet for values.



values in (psf)

Building Shear / Floor Shear:



$$F_{ph} = (25.1\,psf)\left(\tfrac{20'}{2}\right)(344') + (17.7\,psf)\left(\tfrac{20'}{2}\right)(344') = \underline{147{,}232\ lb}$$

$$F_{rf} = \left[(25.1\,psf)\left(\tfrac{20'}{2}\right) + (23.3\,psf)\left(\tfrac{13'}{2}\right)\right](344') + (17.7)\left(\tfrac{20}{2} + \tfrac{13}{2}\right)(344) = \underline{238{,}908\ lb}$$

$$F_4 = \left[(23.3 + 22.0)\left(\tfrac{13'}{2}\right)\right](344') + 17.7(13')(344) = \underline{177{,}315\ lb}$$

$$F_3 = (22 + 20.1)\left(\tfrac{13}{2}\right)(344) + 17.7(13)(344) = \underline{170{,}160\ lb}$$

$$F_2 = (20.1 + 18.5)\left(\tfrac{13}{2}\right)(344) + 17.7(13)(344) = \underline{162{,}334\ lb}$$

$$F_1 = (18.5 + 17.3)\left(\tfrac{13}{2}\right)(344) + 17.7(13)(344) = \underline{156{,}073\ lb}$$

base shear → $V = 1{,}052{,}022\ lb$

Overturning Moment: (M)

$$M = (147{,}232)(90') + (238{,}908)(70') + (177{,}315)(57') + (170{,}160)(42) + (162{,}334)(29) + (156{,}073)(16)$$

$$M = \underline{54{,}432{,}969\ lbft}$$

Appendix D: Seismic Load Calculations

Seismic Analysis	Tech 1 Report	BRIAN BRUNNET	1

Location: Buffalo, NY

Roof DL: 32.2 psf

Floor 4 DL: 123.5 psf

Floors 1-3 DL: 90.8 psf

Snow Load: 50 psf

Exterior Walls DL: 30psf

Using USGS's U.S. Seismic "DesignMaps" Web Application:

$S_s = 0.211g$ $S_{ms} = 0.211g$ $S_{DS} = 0.140g$

$S_1 = 0.060g$ $S_{m1} = 0.060g$ $S_{D1} = 0.040g$

+ Seismic Design Category: A
+ $F_a = 1.0$
+ $F_v = 1.0$
+ $T_L = 6.0$ sec $T_0 = 0.057$sec $T_s = 0.284$sec
+ $PGA = 0.123g$
+ $C_{rs} = 0.876$ $C_{r1} = 0.913$

$V = C_s W$

$$C_s = \frac{S_{DS}}{\left[\frac{R}{I_e}\right]}$$

Concentrically Braced

$R = 6$ (12.2-1)

$I_e = 1.25$ (1.5-2)

$$C_s = \frac{0.140}{\left[\frac{6}{1.25}\right]}$$

$C_s = 0.029$

$T_a = C_T h_w^x$

$= 0.03(90')^{0.75}$

$T_a = 0.88$ sec $< T_L = 6$ sec

C_s should be $< \frac{S_{D1}}{\left(\frac{R}{I}\right)T} = \frac{0.040}{\left(\frac{6}{1.25}\right)(.88)} = 0.0095 < 0.029$

controls

C_s should be > 0.01 ☑

$C_s = 0.0095$

roof DL = 32.2 psf , roof snow load = 50 psf

Penthouse → DL = 123.5 psf , wall DL = 30 psf

Floors 1-4 DL = 90.8 psf , wall DL = 30 psf

Roof / Penthouse:

20% snow load

$W_{rf} = (36{,}003\,sf)(32.2\,psf + 10\,psf) + 2(231.4' + 231.4')\left(\frac{20'}{2} + \frac{13'}{2}\right)(30) + (17{,}527\,sf)(123.5\,psf)$

$W_{rf} = 1{,}519{,}327 + 458{,}172 + 2{,}164{,}585 = \underline{4{,}142{,}084\ lb}$

Floor 4:

$W_4 = (53{,}530\,sf)(90.8\,psf) + 4(231.4')(13')(30\,psf)$

$W_4 = 4{,}860{,}524 + 360{,}984 = \underline{5{,}221{,}508\ lb}$

Penthouse Roof:

$W_{ph} = (17{,}527\,sf)(32.2 + 10\,psf) + 4(231.4)\left(\frac{20'}{2}\right)(30\,psf)$

$W_{ph} = \underline{1{,}017{,}319\ lb}$

Floors 1-3:

$W_{1/3} = (53{,}530\,sf)(90.8\,psf) + 4(231.4')(13')(30\,psf)$

$W_{1/3} = 4{,}860{,}524 + 360{,}984 = \underline{5{,}221{,}508\ lb}$

Total Dead Load: $W = W_{ph} + W_{rf} + W_4 + 3W_{1/3} = 1{,}017{,}319 + 4{,}142{,}084 + 4(5{,}221{,}508)$

$W = 26{,}045{,}435\ lb$

$V = C_s W = (0.0095)(26{,}045{,}435\ lb) = 247{,}432\ lb$

$F_x = C_{vx} V$

where $C_{vx} = \frac{w_x h_x^{\,k}}{\sum w_i h_i^{\,k}}$; $k = 1.0$ for $T \le 0.5\,sec$; $k = 2.0$ for $T > 2.5\,sec$

$T = 0.88\,sec$

Using interpolation:

$k = 1.19$

$$\Sigma w_i h_i^{\,k} = (1{,}017{,}319)(90')^{1.19} + (4{,}142{,}084)(70')^{1.19} + (5{,}221{,}508)(57')^{1.19}$$
$$+ (5{,}221{,}508)(43')^{1.19} + (5{,}221{,}508)(29')^{1.19} + (5{,}221{,}508)(16')^{1.19}$$

$$\Sigma w_i h_i^{\,k} = 215{,}281{,}020 + 649{,}958{,}413 + 641{,}633{,}493$$
$$+ 458{,}800{,}001 + 287{,}110{,}824 + 141{,}481{,}198$$

$$\Sigma w_i h_i^{\,k} = 2{,}394{,}264{,}949$$

$$C_{ph} = \frac{215{,}281{,}020}{\Sigma w_i h_i^{\,k}} = 0.090 \qquad F_{ph} = (.090)(247{,}432\text{ lb}) = 22{,}270\text{ lb}$$

$$C_{rf} = \frac{649{,}958{,}413}{\Sigma w_i h_i^{\,k}} = 0.271 \qquad F_{rf} = (.271)(247{,}432\text{ lb}) = 67{,}054\text{ lb}$$

$$C_4 = \frac{641{,}633{,}493}{\Sigma w_i h_i^{\,k}} = 0.268 \qquad F_4 = (.268)(247{,}432\text{ lb}) = 66{,}312\text{ lb}$$

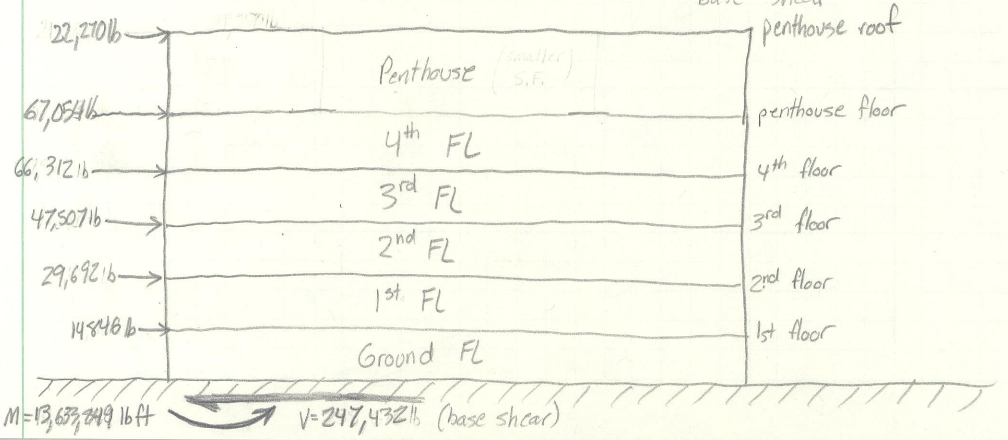
$$C_3 = \frac{458{,}800{,}001}{\Sigma w_i h_i^{\,k}} = 0.192 \qquad F_3 = (.192)(247{,}432\text{ lb}) = 47{,}507\text{ lb}$$

$$C_2 = \frac{287{,}110{,}824}{\Sigma w_i h_i^{\,k}} = 0.120 \qquad F_2 = (.120)(247{,}432\text{ lb}) = 29{,}692\text{ lb}$$

$$C_1 = \frac{141{,}481{,}198}{\Sigma w_i h_i^{\,k}} = 0.060 \qquad F_1 = (.060)(247{,}432\text{ lb}) = 14{,}846\text{ lb}$$

$$\underline{1.000} \qquad V = 247{,}432\text{ lb}$$

Base Shear



Overturning Moment:

$$M = (22432)(90) + (67054)(70) + (66312)(57) + (47507)(43) + (29,692)(29) + (14,846)(16)$$

$$\underline{\underline{M = 13,633,849 \text{ lbft}}}$$

Appendix E: Snow Load Calculations

Snow Load | Tech 1 Report | BRIAN BRUNNET | 1

Flat Roof snow load using ASCE 7-10:

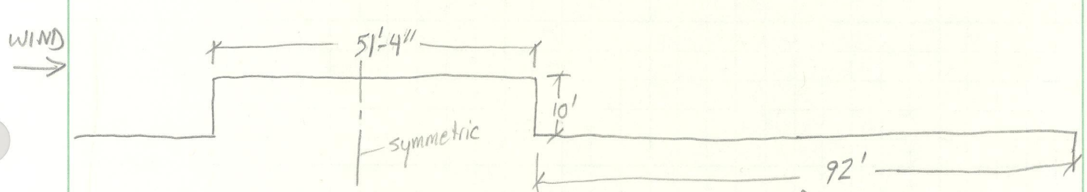
$p_f = 0.7\, C_e C_t I_s p_g$ → Exposure B (26.7.3)

$C_e = 0.9$ (for Fully Exposed)

$C_t = 1.0$ $I_s = 1.10$ (Risk Categ. III)

$p_g = 50\text{psf}$ $p_f = 0.7(0.9)(1.0)(1.10)(50) = \underline{34.7\text{ psf}}$

Drift onto Penthouse: LEEWARD DRIFT



$\gamma = 0.13 p_g + 14 = 0.13(50) + 14 = 20.5\text{ pcf}$ $h_b = \frac{p_s}{\gamma} = \frac{34.7}{20.5} = \underline{1.69'} > 0.2$

$h_d = 0.43 \sqrt[3]{l_u}\, \sqrt[4]{p_g + 10} - 1.5$ $l_u = 51'\text{-}4"$

$p_g = 50$

$h_d = 0.43 \sqrt[3]{(51.33)}\, \sqrt[4]{50+10} - 1.5$

$p_d = h_d \gamma = (2.95')(20.5) = \underline{60.5\text{psf}}$ (leeward)

$h_d = \underline{2.95\text{ ft}}$

Drift onto Penthouse: Windward Drift

$h_d = \left[0.43 \sqrt[3]{(92')}\, \sqrt[4]{50+10} - 1.5\right] \times \frac{3}{4}$

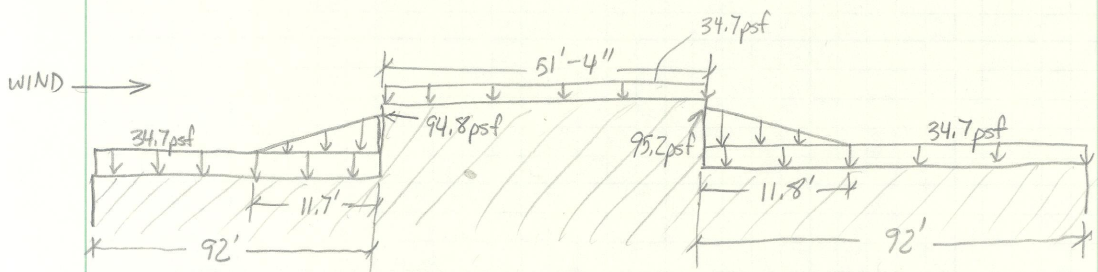
$h_d = \underline{2.93\text{ ft}}$

$p_d = h_d \gamma = (2.93)(20.5) = \underline{60.1\text{ psf}}$ (windward)

- since $h_d < h_c$:

$$w = 4h_d = 4(2.95) = 11.8' \text{ (leeward)}$$

$$w = 4h_d = 4(2.93) = 11.7' \text{ (windward)}$$



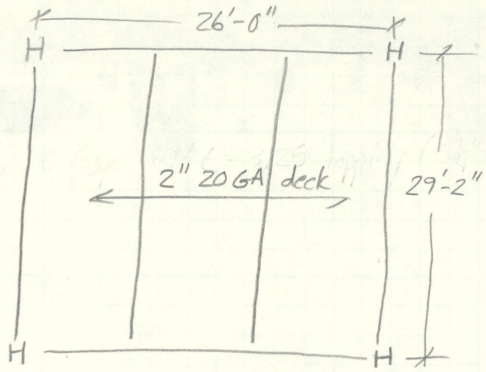
Appendix F: Gravity System Spot Check Calculations

Typical Deck

Loads:

LL = 40 psf
MEP = 18 psf
Superimp. DL = 20 psf
partitions = 18 psf

96 psf



$\frac{26'}{3} = 8.67'$ span

Assume: 2-hr rating, LWC, unprotected deck

Try: 2VLI Deck w/ $3\frac{1}{4}$" LW topping (Total depth: $5\frac{1}{4}$")

	3span Condit:	Load:	
2VLI22:	9'-7"	168 psf	← I chose this
→ 2VLI20:	10'-11"	190 psf	

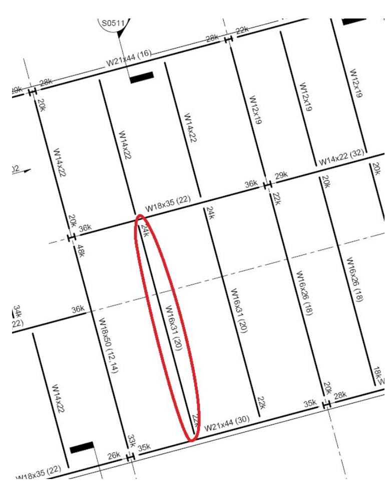
what original design calls for

$\boxed{\text{USE 2VLI22 w/ } 5\frac{1}{4}\text{" LWC}}$

168 psf > 96 psf ☑ ok

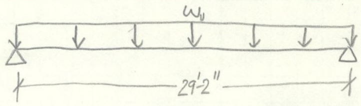
Max unsh. length: 9'-7" > 8'-6" ☑ ok

⇒ Possibly used 2VLI20 for larger 3span length of 10'-11"



Spot Checks	Tech 1 Report	BRIAN BRUNNET	1

TYPICAL BEAM



Given: [Using AISC Steel Construction Manual, 13th edit.]
- Wide Flange (ASTM A992 Steel)
- Use of Shear Studs ($3/4"\phi$)
- Use LRFD to Design

LOADS: From DL + LL calculations:
- Floor DL = 90.8psf (includes bm wt)
- Floor LL = 40psf (residential corridor)

Can LL be reduced?:

$K_{LL} A_T = (2)(8.67' \times 29.167') = 505.6\,ft^2 > 400\,ft^2$ ☑ can be reduced

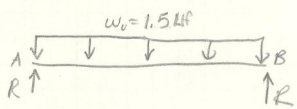
$L = L_0\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) = 40\left(.25 + \frac{15}{\sqrt{505.6}}\right) = \underline{36.7\text{ psf}} > 0.5(L_0) = 20\text{psf}$ ☑ ok

Total Load:

$W_u = 1.2D + 1.6L = 1.2(90.8) + 1.6(36.7\text{psf}) = 167.7\text{ psf}$

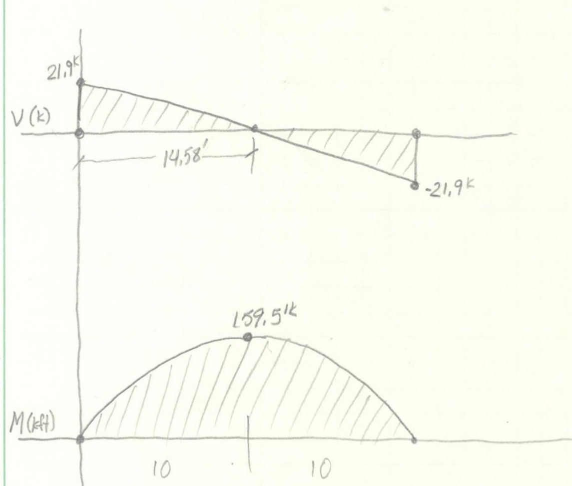
$w_u = (167.7\text{psf})(8.67') = 1453.6\text{ plf} = \underline{1.5\text{ klf}}$

trib width



$\Sigma F_y : (1.5)(29.167') = 2R$

$\boxed{R = 21.9^k}$



$M_u = \frac{1}{2}(21.9^k)(14.58')$

$\boxed{M_u = 159.5^{k\text{ft}}}$

Typical Beam

Given: Metal Deck = 2VLI20 w/ $5\frac{1}{4}''$ total thickness, $\perp$, $\underbrace{\text{weak position, 1 stud/rib}}_{\text{assumption}}$ $\frac{3}{4}''\phi$ shear studs

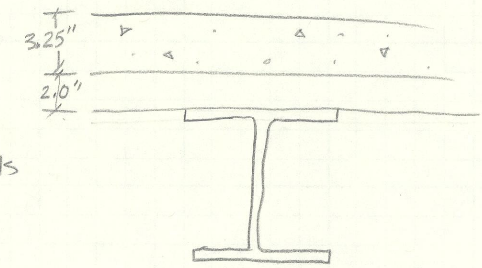
$f'_c = 3^{ksi}$

$Q_n = 17.2^k/\text{stud}$

$M_u = 159.5^{'k}$ $\quad \frac{159.5^{'k}}{17.2^k} = 10$ studs / side $\longrightarrow$ 20 total studs

$\Sigma Q_n = (17.2^k)(10 \text{ studs})$

$= 172^k$



$b_{eff} = \min \begin{cases} \frac{(29.167')(12)}{8} = 43.8'' \text{ controls} \\ \frac{8.67(12)}{2} = 52.0'' \end{cases}$

$a = \frac{(172^k)}{0.85(3)(43.8)} = 1.54''$

$Y2 = 5.25'' - \frac{a}{2} = 5.25 - \frac{1.54}{2} = 4.48'' \sim 4.5''$

$\Sigma Q_n = 172^k$ Table 3-19 $\Rightarrow$ Try W12×19

$b_{eff} = 43.8''$ $\quad \phi M_n = 172^k > 159.5^k$

$a = 1.54''$ $\quad Y2 = 4.5''$

$Y2 = 4.5''$ $\quad Y1 = 0.350''$

$\Delta_{LL} = \frac{L}{360} = \frac{(29.167)(12)}{360} = 0.97''$

$\Delta_{LL} = \frac{5wL^4}{384EI_{LB}} = \frac{5(0.6\text{ klf})(29.167)^4}{384(29000)(338)} \times (1728) = 1.00$ ☒ use W12×22

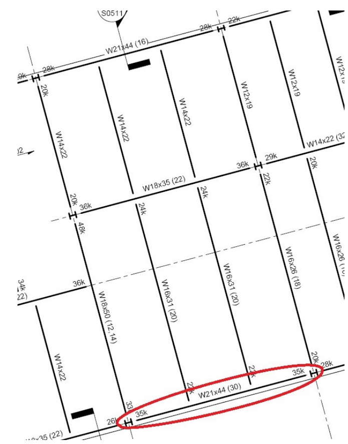
$\phi M_n = 198 > 159.5^k$

$Y2 = 4.5''$

$Y1 = .425''$

$\Delta_{LL} = \frac{5(.6)(29.167)^4(1728)}{384(29000)(392)} = 0.86'' < 0.97''$ ☑ ok

$\boxed{\text{USE W12×22 (20)}}$



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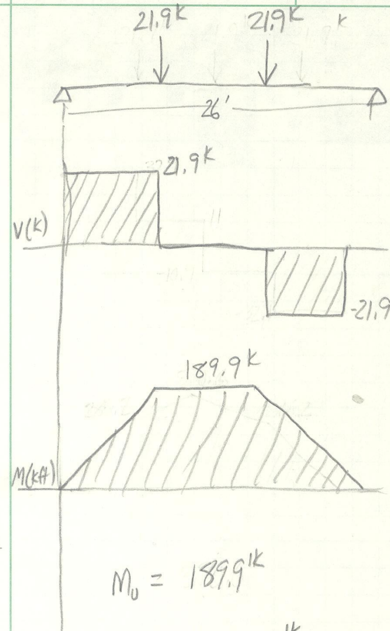
Typical Girder



$\Sigma F_y: 21.9(2) = 2(R)$

$R = 21.9^k$

$Q_n = 17.1$ (table 3-21)

$\Sigma Q_n = 17.1(12) = 205.2^k$

b_{eff} $\frac{(26)(12)}{8} = 39$ controls

min $\frac{26(12)}{2} = 156$

$a = \frac{(205.2^k)}{0.85(3)(39)} = 2.06''$

$M_u = 189.9^{1k}$

$\frac{189.9^{1k}}{Q_n} = \frac{189.9^{1k}}{17.1} = 12$ shear studs/side

(24) total studs

$Y2 = 5.25 - \frac{2.06}{2} = 4.22'' \sim 4.25''$

$\Sigma Q_n = 205.2^k$

$b_{eff} = 39''$

$a = 2.06''$

$Y2 = 4.25''$

Table 3-19 → Try W14x34

$\phi M_n = 345^{1k} > 189.9^{1k}$

$Y2 = 4.25''$

$Y1 = 0.455''$

$\Delta_{LL} = \frac{L}{360} = \frac{(26)(12)}{360} = 0.867''$

$\Delta_{LL} = \frac{PL^3}{28EI_{LB}} = \frac{(21.9)(26)^3(1728)}{28(29000)(708)} = 1.17''$ ☒

Solve for I_{LB} @ $Y2 = 4.25''$, $Y1 = BFL$

$\Delta_{LL} = \frac{PL^3}{28EI_{LB}} = \frac{(21.9)(26)^3(1728)}{28(29000)\,I_B} = 0.867''$

$I_B = 945 in^4$

Try W16x40: $I_{LB} = 1020 in^4$

$\phi M_n = 422 > 189.9$ ✓

$Y2 = 4.25''$

$Y1 = 0.505''$

$\Delta_{LL} = \frac{(21.9)(26)^3(1728)}{28(29000)(1020)} = 0.803'' < .867''$ ☑ ok

$\boxed{\text{USE W16x40(24)}}$

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Typical Column

$$\text{Trib. area} = \left(\frac{19'4''}{2} + \frac{29'2''}{2}\right)\left(\frac{26'}{2} + \frac{20'}{2}\right) = (24.25)(23) = 557.75\ ft^2$$

$$LL_{red} = 0.25 + \frac{15}{\sqrt{4(5)557.75}} = 0.39 \longrightarrow \underline{\underline{0.4}}$$

$W_{rf} = 32.2psf$

$W_{fl} = 90.8psf$

$W_{sn} = 39\ psf$

$W_L = 40psf$

$$P_L = 0.4\,(40psf)(5)(557.75ft^2) = 44.6^k$$

$$P_s = (39psf)(557.75ft^2) = 21.8^k$$

$$P_D = (32.2psf)(557.8) + (90.8)(4)(557.8) = 220.6^k$$

$$P_u = 1.2(220.6) + 1.6(44.6) + 0.5(21.8) = 347^k$$

$K_x = 1.0 \quad L_x = 16' \quad KL_x = 16' \quad KL_y = \frac{16}{r_x/r_y}$

→ USING Table 4-1:

Try: W10 × 49 ($\phi P_n = 428^k$

$\phi P_n = 428^k > P_u = 347^k$ ☑ ok

$KL_y = \frac{16}{1.71} = 9.4' < 16'$ ☑ ok

$$\boxed{\text{USE W10×49 for column @ C12-CF}}$$