ECMC Skilled Nursing Facility

Architectural Engineering Senior Thesis 2011

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Executive Summary

The purpose of Technical Report 3 is to evaluate and determine the adequacy of the lateral system in the ECMC Skilled Nursing Facility. This is a new 296,000 square foot skilled nursing facility located on the ECMC campus in Buffalo, NY. The building has unique design features, such as a radial plan geometry and sloped roof layout, and the project cost roughly $95 million to construct. The main framing system consists of composite steel framing with a large mechanical penthouse located on the top floor. The building's main lateral system consists of 16 concentrically braced frames, where 8 frames can be found at the end of each wing while another 8 frames are located surrounding the building core.

The analysis of this technical report begins with a verification of dead, live, and snow loads found within the structural drawings. Afterwards, lateral loads such as wind and seismic were calculated using ASCE 7-10, following both the Main Wind Force Resisting System procedure for wind and the Equivalent Lateral Force procedure for seismic. Once these loads were found, specific load combinations were chosen to determine which load case or combination of load cases controlled the design of the lateral system. It was found that the wind produced a base shear of 1052 kips and seismic produced a base shear of 455 kips in both the N-S and E-W directions. Overturning moments of 54,432 ft-k and 25,063 ft-k were found for both wind and seismic respectively.

With the help of ETABS, a finite element model of the ECMC Skilled Nursing Facility were generated, consisting of 16 brace frames located throughout the building and each floor modeled as a singular rigid diaphragm. The braced frames were oriented in a radial pattern with 8 surrounding the outer edge of the building. The other 8 braced frames are located in a radial pattern surrounding the inner building core. The sloped roof from the original model was simplified in order for wind and seismic loads to stay consistent from both directions. Lateral loads were applied to the model to find the center of rigidity, torsion, story drifts, and overturning. Results were then taken from the ETABS output and compared to hand calculations and allowable limits set forth by code and industry standards.

The displacement and story drifts were found to be within the allowable limits of the code. Overturning considerations discussed that dead load of the building would prevent any uplift from occurring due to lateral loads. Spot checks were performed on two critical members of the braced frame system, a diagonal bracing member in frame C8 and a column in frame A8. Specific load combinations and force directions were considered for the ETABS model until the greatest load case governed. Upon review, it was found that these members in both braced frames were adequately designed and could successfully support the load cases applied to them.

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 Systems 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

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.

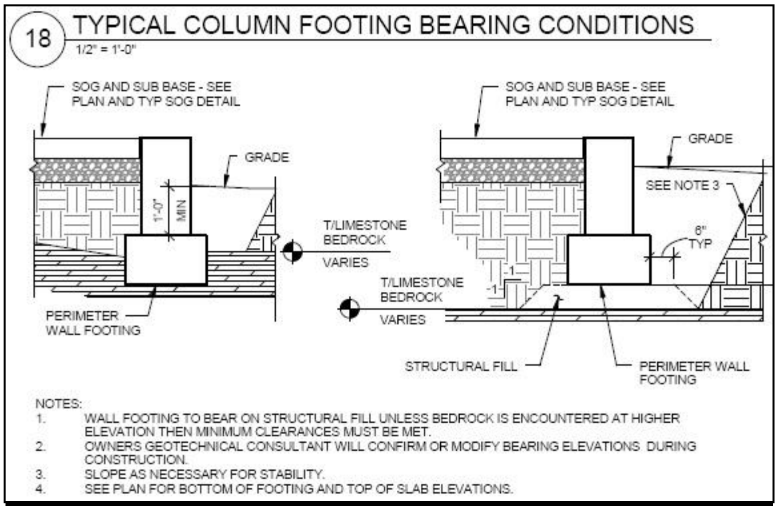


Figure 3: Footing bearing conditions. On bedrock (left detail), and on Structural Fill (right detail). Detail courtesy of Cannon Design.

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.

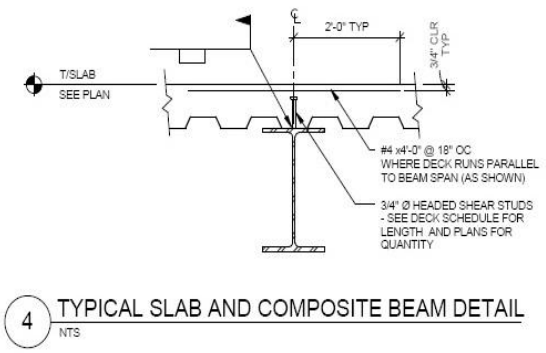


Figure 4: Composite deck system (parallel edge condition). Detail courtesy of Cannon Design.

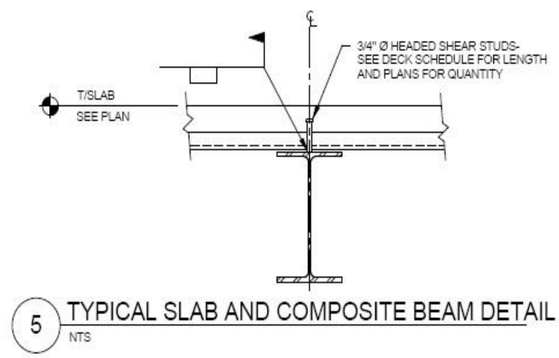


Figure 5: Composite deck system (perpendicular edge condition). Detail courtesy of Cannon Design.

Framing System

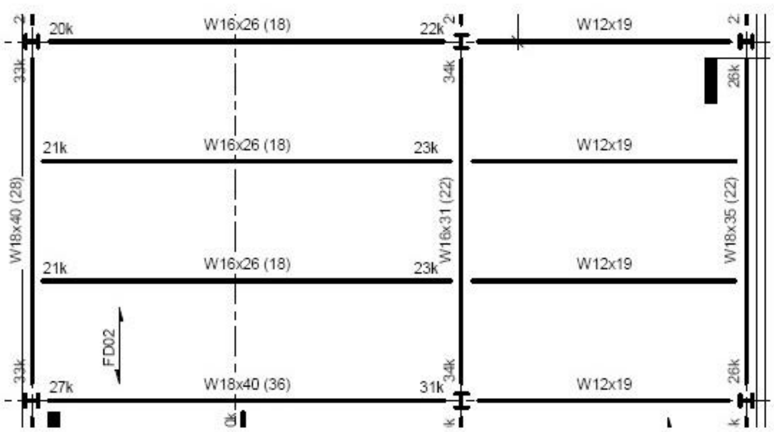


Figure 6: Typical bay layout for building wing. Detail courtesy of Cannon Design.

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.

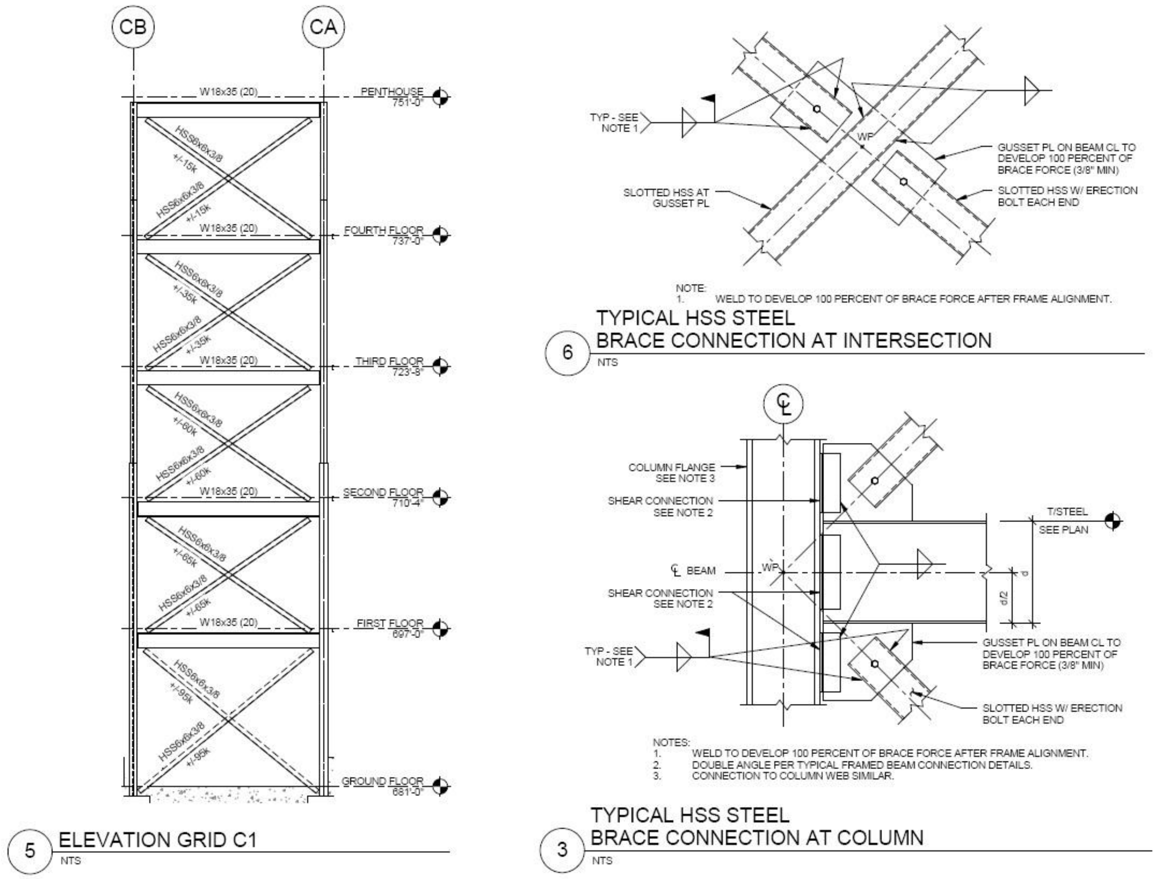


Figure 7: Typical lateral HSS brace frame (left). Typical HSS steel brace connection at intersection (upper right). Typical HSS steel brace connection at column (lower right). Details courtesy of Cannon Design.

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

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.

Gravity 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.

Snow Loads

The snow loads were calculated using various charts and tables found in ASCE 7-10. Table 3 shows the difference in variables and ground snow loads between the original drawings and thesis analysis loads.

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

Lateral Loads

Wind Loads

Wind loads were determined using ASCE 7-10. The Main Wind Force Resisting System procedure was used to calculate wind pressures and loads. Due to the radial footprint and complex geometry that each wing created, along with the slanted and staggered roof design, the building was assumed to have a 350' x 350' square plan with a flat roof for simplification. Since the footprint is symmetric and square, wind pressures were only applied from one direction, in this case the East-West direction, to find the equivalent story forces produced by wind. The total base shear calculated was 1052 kips. Detailed calculations of the wind loads can be found in Appendix B.

Building Category	III	Damping Ratio(β)	0.02
Basic Wind Speed (V)	120mph	Natural Frequency (na)	0.833
Wind Directionality Factor (Kd)	0.85	L/B	1
Exposure Category	B	Iz	0.2764
Topographic Factor (Kzt)	1	Lz	377.09
α	7	Q	0.7614
Zmin	30	Vz	120.7
Gf	0.821	N1	2.602
Kz	0.96	Rn	0.0762
GCpi	(+/- 18 psf)	Rh	0.3195
Cp(windward walls)	0.8	Rb	0.0895
Cp(leeward walls)	-0.5	RL	0.0272
Cp(side walls)	-0.7	gR	4.15
Cp(0-h/2)	-0.9	R	0.2432
Cp(h/2-h)	-0.9	ηh	2.856
Cp(h-2h)	-0.5	ηB	10.92
Cp(>2h)	-0.3	ηL	36.55

Table 4: The table above shows variables and classifications necessary to calculate wind pressures using the MWFRS procedure in ASCE 7-10.

Wind Loads								
Floor	Story Height (ft)	Height Above Ground (ft)	Controlling Wind Pressure (PSF)		Total Controlling 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 5: The table above shows the floor wind pressures and forces along with shear/moment forces on the building.

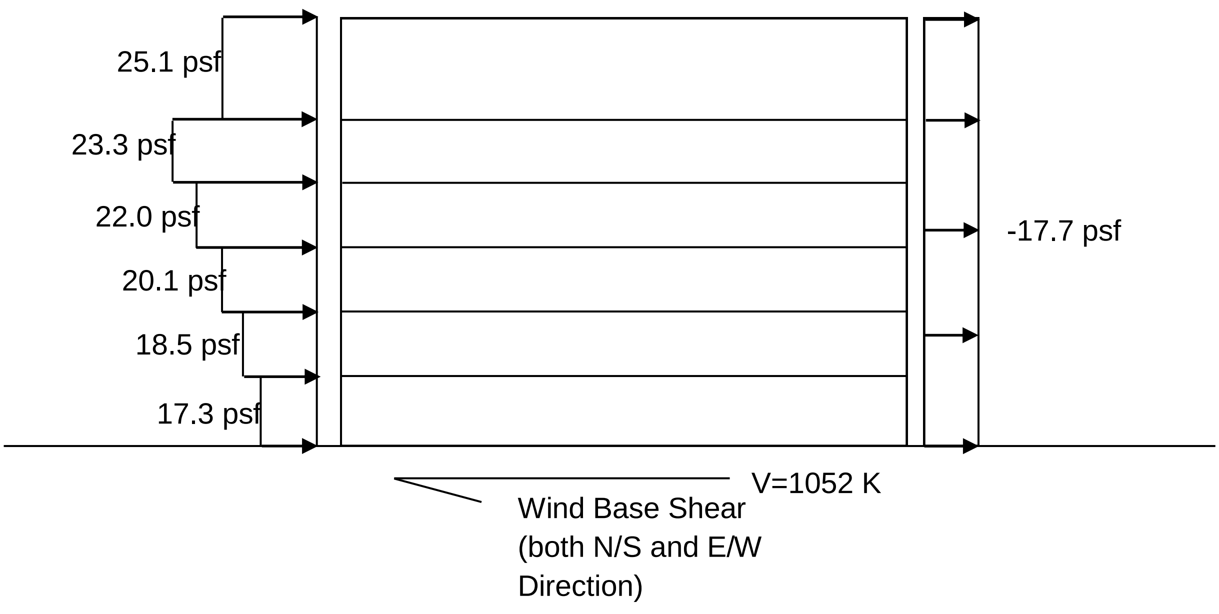


Figure 8: The figure above shows floor wind pressures applied to the windward & leeward side of the building, along with the total base shear.

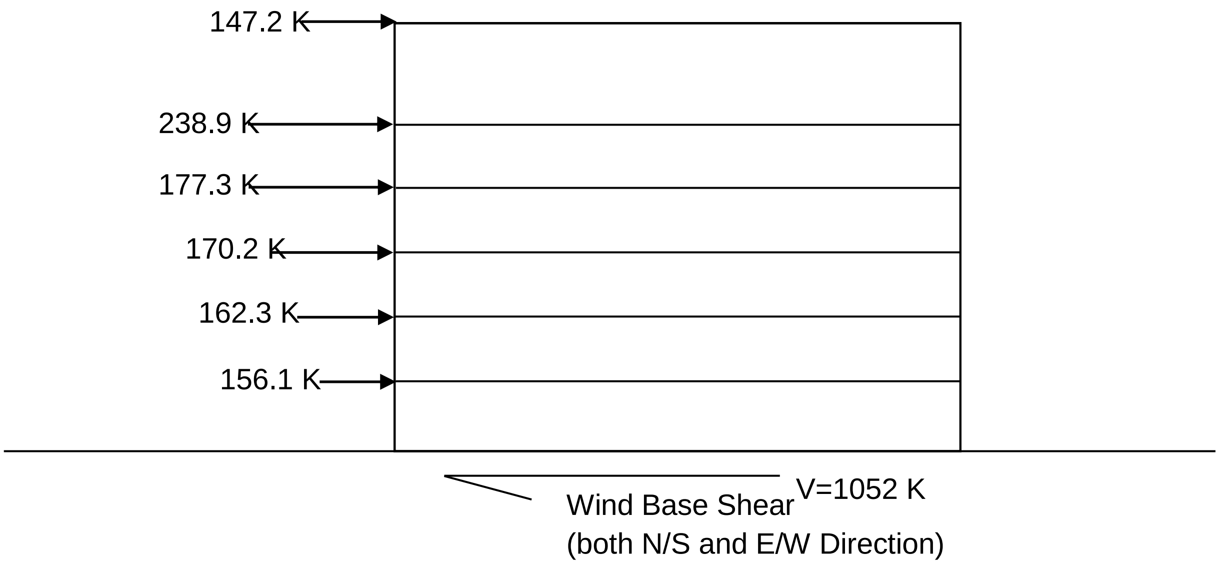


Figure 9: This figure shows the wind story shear force applied to the building.

Seismic Loads

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. The total base shear found from seismic loads was 455.3 kips. A detailed calculation of the seismic forces present can be found in Appendix B.

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	Table 11.4-1
F_V	1	Table 11.4-2
S_{MS}	0.211	USGS WEBSITE
S_{M1}	0.06	USGS WEBSITE
S_{DS}	0.14	USGS WEBSITE
S_{D1}	0.04	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 6: 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	0.03	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	3.25	Table 12.2-1
Cs	0.0175	Equation 12.8-5
W	26,045 K	Refer to Appendix C
V	455.3 K	Refer to Appendix C
k	1.19	Section 12.8.3

Table 7: This table shows a summary of variable results for calculations for seismic analysis using the Equivalent Lateral Force Procedure as 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 (ftK)
Penthouse Roof	1,017	90	215,214	0.09	40.9	40.9	3681
Penthouse Floor	4,142	70	649,945	0.271	123.4	164.3	8638
4th Floor	5,221	57	641,571	0.268	122	286.3	6954
3rd Floor	5,221	43	458,755	0.192	87.4	373.7	3758
2nd Floor	5,221	29	287,083	0.12	54.6	428.3	1583
1st Floor	5,221	16	141,467	0.06	27.3	455.3	437
Ground	0	0	0	0	0	0	0
TOTAL	26,043		2,394,036	1	247.7		25062

Table 8: This table shows the calculations and processes needed in order to calculate seismic base shear using the Equivalent Lateral Force Procedure as in ASCE 7-10.

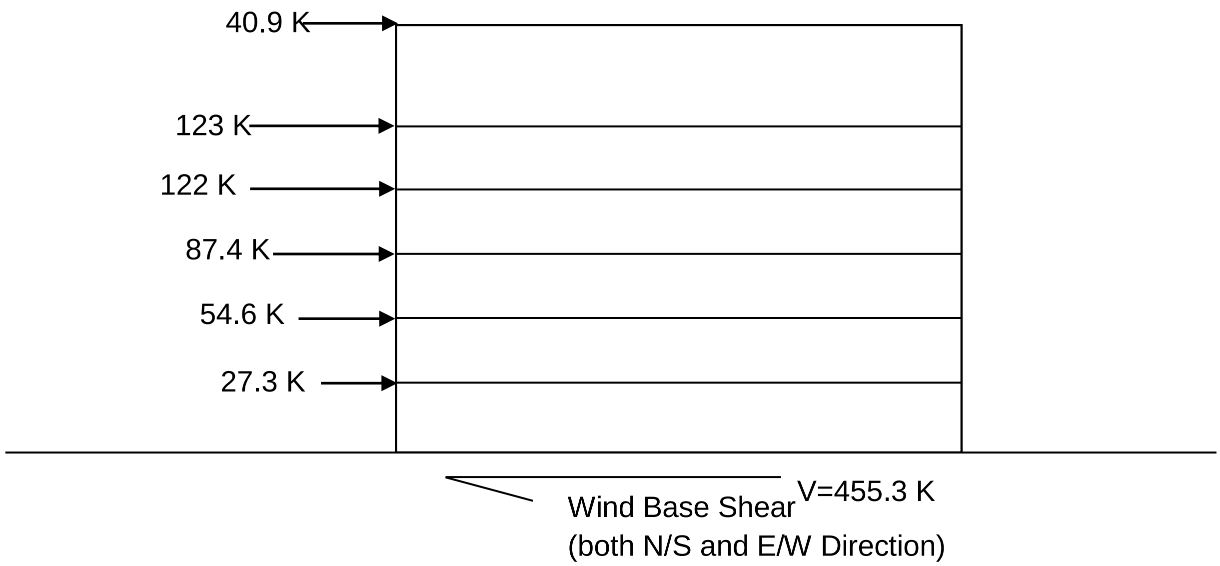


Figure 10: This figure shows calculated seismic story shear at each level throughout the building.

Lateral Load Distribution

The ECMC Skilled Nursing Facility is broken up into four large reference areas. Figure 11 shows the location of these areas. Figure 12 shows the locations of four concentrically braced frames, highlighted in red, which are used to resist any lateral loads. Each area has a similar layout of braced frames, and when viewed in a full radial building plan, they form an exterior ring and interior ring of braced frames.

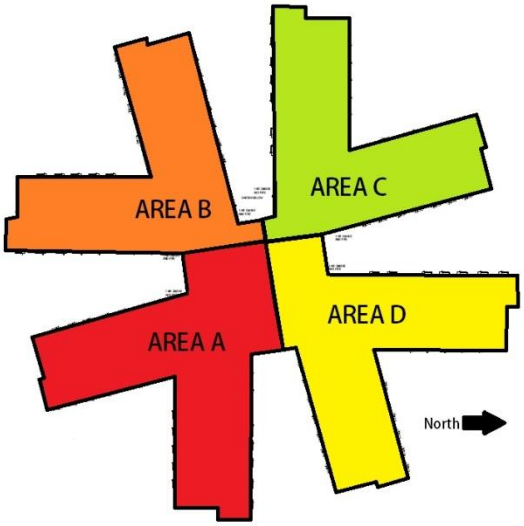


Figure 11: Areas A, B, C, and D of the ECMC Skilled Nursing Facility with North arrow.

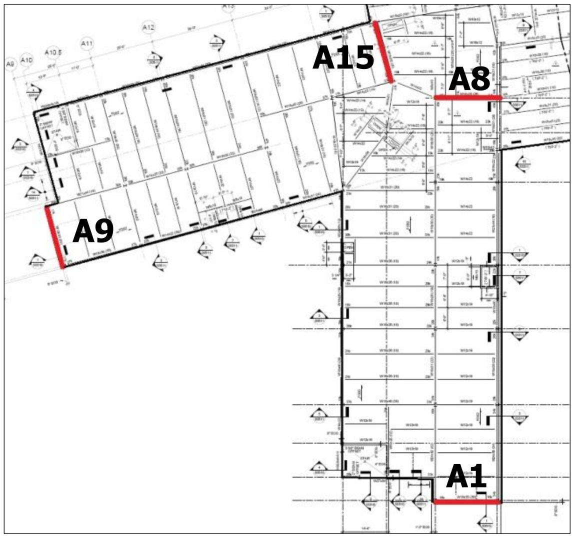


Figure 12: Area A shown with typical braced frame locations highlighted in red. Similar areas will follow the same numbering pattern.

In this report, each floor system was modeled in ETABS as a rigid diaphragm. This allows story shears produced by wind or seismic to transfer through the floor slab directly into the concentrically braced frames. The loads transfer from the braced frames downward into the buildings foundation system. In order to calculate the relative stiffness for each braced frame, a 1 kip horizontal load was applied to the top of the frame, and then finding the displacement associated with that force. Using the relative stiffness, further calculations determined the total load capacity for each braced frame.

ETABS Model

In order to find an accurate center of mass and center of rigidity for the ECMC Skilled Nursing Facility, a finite elements computer model was generated using ETABS. Only the concentrically braced frames were modeled, since these are the main elements in the building that resist lateral loads. Each floor system was created as a rigid diaphragm, with an added area mass to account for the floor dead loads. Line elements were used to model the columns, beams, and cross bracing. The beams and columns consist of W-Flange steel shapes and the cross bracing is comprised of square steel HSS tubing. The model was created using 8 local grids, where 4 of those grids are rotated 15 degrees to match the angles of each wing. Figures 13 and 14 both show a three-dimensional view of the ETABS model that was created for this technical report. Figure 15 and 16 show the locations of the braced frames as seen on a typical floor plan from the ETABS model.

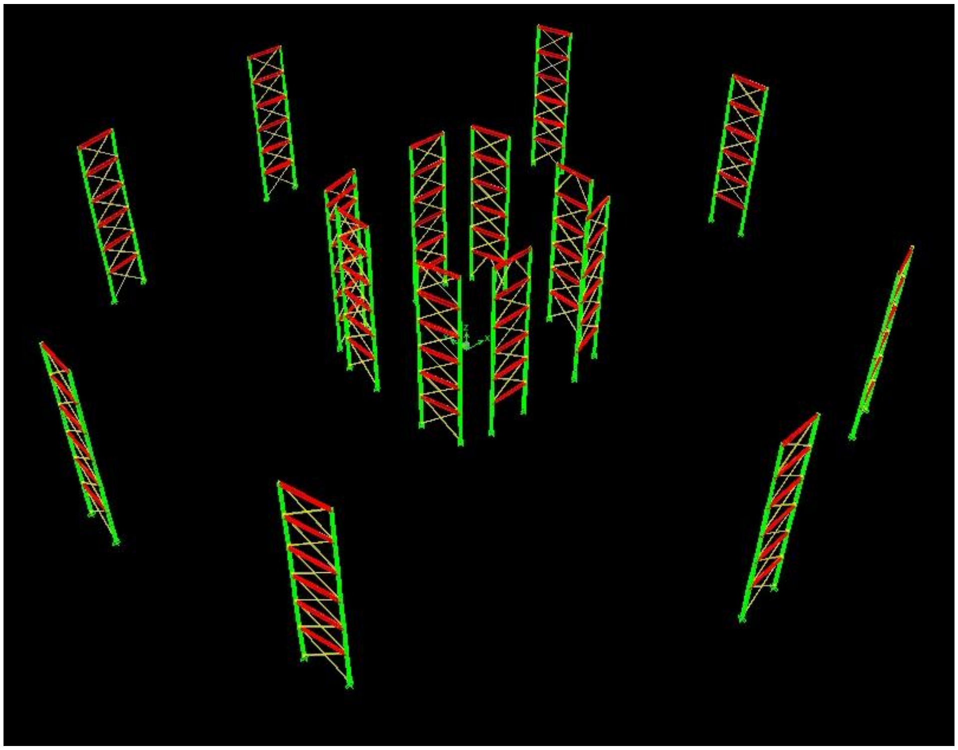


Figure 13: ETABS 3D Model of Concentrically Braced Frames (Diaphragms not shown)

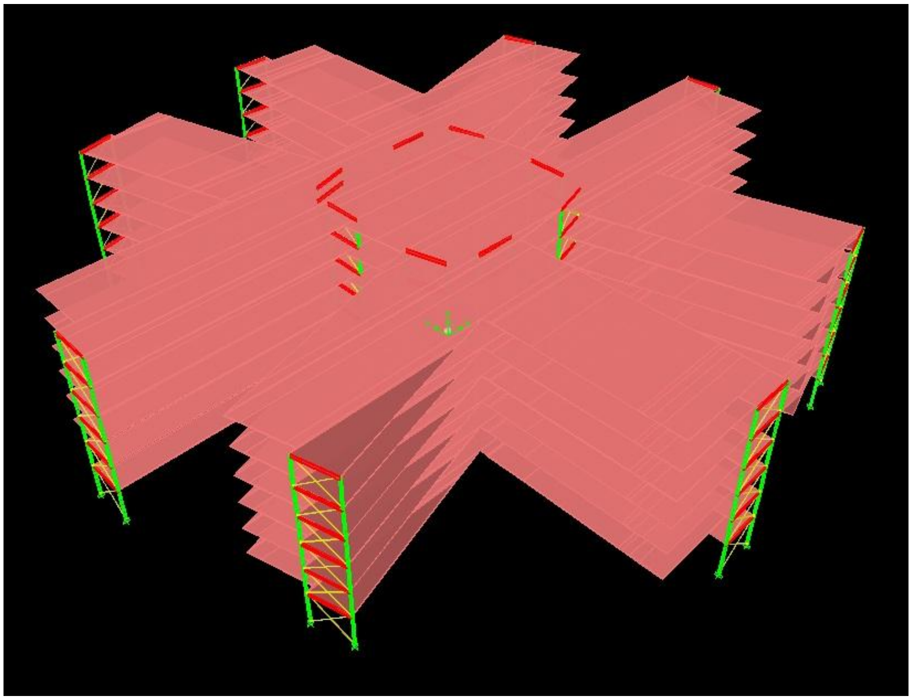
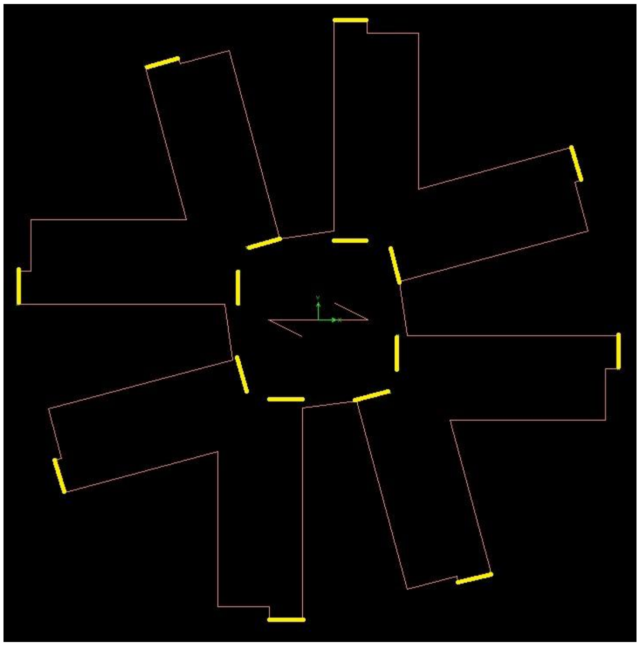


Figure 14: ETABS 3D Model of Concentrically Braced Frames (Diaphragms shown)



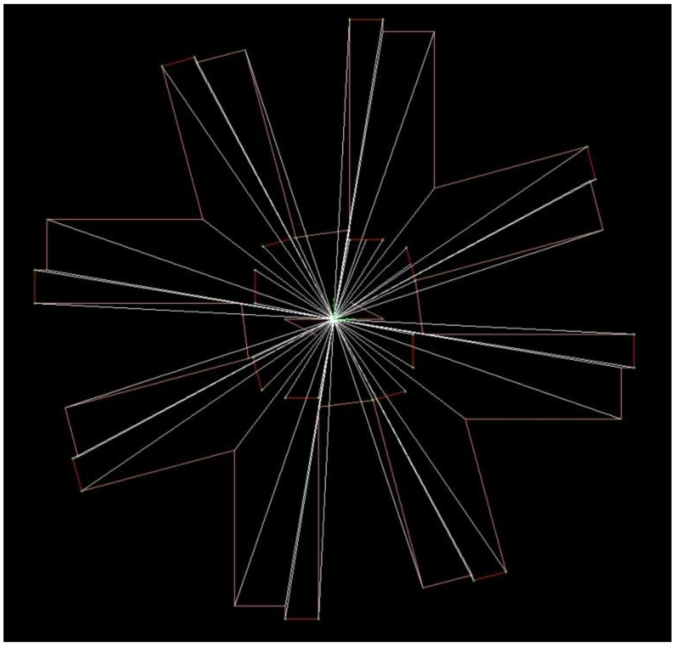


Figure 15: The image on the left shows an ETABS Model of Typical Floor Plan with braced frames highlighted in yellow. The right image shows the center of mass for each diaphragm.

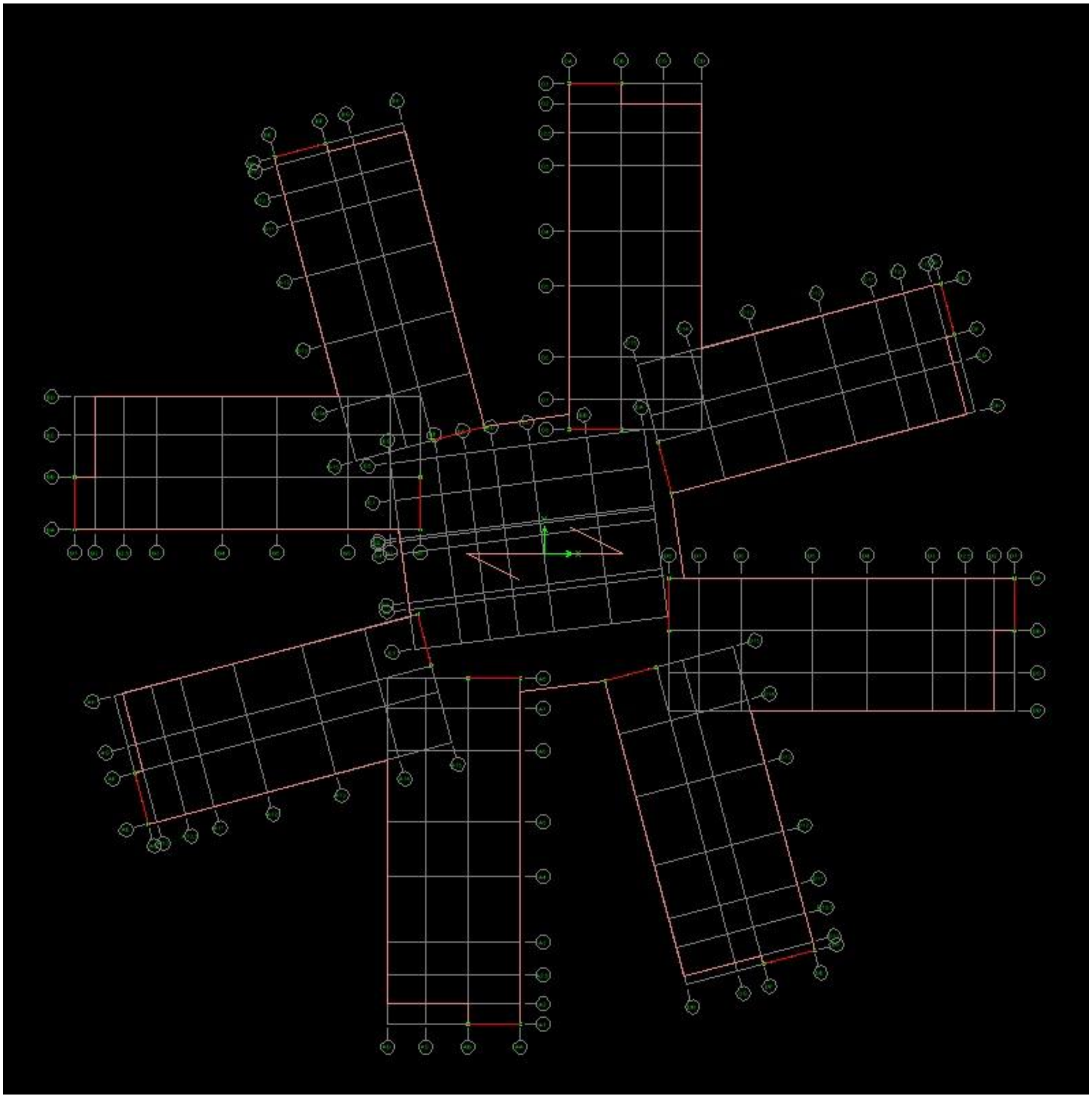


Figure 16: The plan layout above shows the separate local grids used to model each wing at the specific angle and location necessary to replicate the model adequately.

Load Case Combinations

Load combinations from ASCE 7-10 for strength design were considered for this technical report. The load combinations have changed in ASCE 7-10 as compared to ASCE 7-02, where these load cases include both gravity and lateral loads. The load combinations that were considered in this report include the following:

1. 1.4D
2. 1.2D + 1.6L + 0.5Lr
3. 1.2D + 1.6Lr + 0.5W
4. 1.2D + 1.0W + 1.0L + 0.5Lr
5. 1.2D + 1.0E + 1.0L
6. 0.9D + 1.0W
7. 0.9D + 1.0E

It was found in most cases wind controlled the design of the lateral system due to its excessive amount of load on the building, essentially twice the force of seismic. In this case, load cases 4 and 6 governed due to wind and were used in the ETABS model to show the worst case scenarios on the lateral system. Load case 4 was used for strength and deflection checks while case 6 was considered for any uplift effects.

Story Drift and Total Displacement

Story drift and total lateral displacements of the building were checked for this report. From ASCE 7-10, the allowable seismic story drift for a building in Occupancy Category III is $0.015h_{sx}$. The acceptable standard for total building displacement for wind loads is L/400. Using the ETABS finite element building model, it was found that the braced frames in the building met acceptable drift requirements for both wind and seismic load cases. Tables 9 and 10 are outputs of displacement and drift under the calculated seismic loads while Tables 11 and 12 are similar outputs due to wind load.

Seismic Story Drift & Displacement N-S Direction				
Floor	**Displacement (in)**	**Story Drift (in)**	**Allowable Story Drift (in)**	**Is this OK?**
Roof	0.9476	0.001171	1.35	yes
PH Floor	0.7783	0.001256	1.05	yes
4th Floor	0.5959	0.001276	0.855	yes
3rd Floor	0.4183	0.001152	0.63	yes
2nd Floor	0.2564	0.000935	0.435	yes
1st Floor	0.1244	0.000671	0.24	yes

Table 9: The table above shows seismic story drifts and total displacement in the N-S direction.

Seismic Story Drift & Displacement E-W Direction				
Floor	**Displacement (in)**	**Story Drift (in)**	**Allowable Story Drift (in)**	**Is this OK?**
Roof	0.9005	0.001037	1.35	yes
PH Floor	0.7383	0.001124	1.05	yes
4th Floor	0.5627	0.001155	0.855	yes
3rd Floor	0.3912	0.001058	0.63	yes
2nd Floor	0.2343	0.000872	0.435	yes
1st Floor	0.105	0.000675	0.24	yes

Table 10: The table above shows seismic story drifts and total displacement in the E-W direction.

Wind Story Drift & Displacement N-S Direction				
Floor	**Displacement (in)**	**Story Drift (in)**	**Allowable Story Drift (in)**	**Is this OK?**
Roof	2.0525	0.00239	2.7	yes
PH Floor	1.6721	0.00251	2.1	yes
4th Floor	1.2717	0.00244	1.71	yes
3rd Floor	0.8982	0.00217	1.26	yes
2nd Floor	0.5628	0.00179	0.87	yes
1st Floor	0.2841	0.00158	0.48	yes

Table 11: The table above shows wind story drifts and total displacement in the N-S direction.

Wind Story Drift & Displacement E-W Direction				
Floor	**Displacement (in)**	**Story Drift (in)**	**Allowable Story Drift (in)**	**Is this OK?**
Roof	1.9567	0.002266	2.7	yes
PH Floor	1.5885	0.002391	2.1	yes
4th Floor	1.1994	0.002341	1.71	yes
3rd Floor	0.8359	0.002099	1.26	yes
2nd Floor	0.509	0.001758	0.87	yes
1st Floor	0.2346	0.001371	0.48	yes

Table 12: The table above shows wind story drifts and total displacement in the E-W direction.

Torsional Effects

The ECMC Skilled Nursing Facility will see some slight torsional effects due to torsion, however nothing overly significant. Because of the buildings radial geometry in plan along with the circular layout of each braced frame, the buildings center of mass is relatively in the same location as the buildings center of rigidity. The ETABS model was used to obtain both the center of mass and rigidity for each floor. ETABS applies an eccentricity of 5% of the building length when checking seismic torsional effects, which accounts for accidental torsion that occurs in the building. The tables below show building torsion in the N-S and E-W directions under seismic loading.

Building Torsion N-S Direction -Seismic Loading							
Floor	Story Force (k)	Location of COR	Location of COM	e_x (ft)	M_t (ft-k)	M_a (ft-k)	M_{tot} (ft-k)
Roof	40.9	1.282	0.174	1.108	45.3	122.3	167.6
PH Floor	164.3	1.273	0.174	1.099	180.6	491.3	671.9
4th Floor	286.3	1.282	0.174	1.108	317.2	856.1	1173.3
3rd Floor	373.7	1.279	0.174	1.105	412.9	1117.4	1530.4
2nd Floor	428.3	1.265	0.174	1.091	467.3	1280.6	1747.9
1st Floor	455.3	1.261	0.174	1.087	494.9	1362.2	1857.2
						Total	7148.2

Table 13: This table shows building torsional effects in the N-S Direction due to seismic.

Building Torsion E-W Direction -Seismic Loading							
Floor	Story Force (k)	Location of COR	Location of COM	e_x (ft)	M_t (ft-k)	M_a (ft-k)	M_{tot} (ft-k)
Roof	40.9	0.776	0.095	0.681	27.9	66.8	94.6
PH Floor	164.3	0.821	0.095	0.726	119.3	268.2	387.5
4th Floor	286.3	0.769	0.095	0.674	193.0	467.4	660.4
3rd Floor	373.7	0.787	0.095	0.692	258.6	610.1	868.7
2nd Floor	428.3	0.802	0.095	0.707	302.8	699.2	1002.0
1st Floor	455.3	0.778	0.095	0.683	311.0	743.8	1054.7
						Total	4067.9

Table 14: This table shows building torsional effects in the E-W Direction due to seismic.

Overturning & Foundation Considerations

Often when a building is subject to lateral loads, whether it be from wind or seismic, it becomes essential to check for an overturning moment which could cause multiple issues within a building, including possible foundation uplift. Load cases 6 and 7 from the combination loads section of this report are used to calculate overturning. Table 15 below lists the overturning moments calculated on the building. The overturning moments were calculated by hand for the seismic and wind loads; and since the hand calculations were simplified into a symmetric square plan, the overturning moments in the E-W direction experienced the same load as in the N-S direction. As seen in the table, the wind overturning moment controlled since it produced much larger lateral loads than seismic. Since the building has such a large and wide base as opposed to its height, it is unlikely that the building will overturn. However, moment transferred to the foundations via the lateral system can cause possible soil failures if the bearing capacity is exceeded on the soil, thus it is important to check overturning moments.

Overturning Moments					
		Seismic		Wind	
Floor	Height (ft)	Lateral Force (k)	Moment (ft-k)	Lateral Force (k)	Moment (ft-k)
Roof	90	40.9	3681	147.2	13248
PH Floor	70	123.4	8638	238.9	16723
4th Floor	57	122	6954	177.3	10106
3rd Floor	42	87.4	3758	170.2	7148
2nd Floor	29	54.6	1583	162.3	4707
1st Floor	16	27.3	437	156.1	2498
	Total Overturning Moment		25062		54430

Table 15: This table shows calculated overturning moments due to both seismic and wind.

Critical Member Checks

Spot checks were performed on two members, a brace and a column, that underwent specific loading to produce maximum stress cases. Several load cases were considered and the controlling load case differed depending on which member was being observed. The ETABS model was used to obtain loads on the members. The first check involved a bracing member found on the ground floor at braced frame #C8. This frame experienced the largest diagonal member compressive/tensile loads under the wind loads given. The member was checked for axial tension and compressive strength and it was found that the bracing member could sufficiently support the worst load case. The second check involved a column located on the ground floor at braced frame #A8. Bracing wasn't used on the first floor in this bay, making the column undergo the largest combined axial and bending load case. Upon analysis, it was found that the column was adequate to support the loads. Members checked are highlighted in Figures 16 below. Detailed calculations for the member checks can be found in Appendix B.

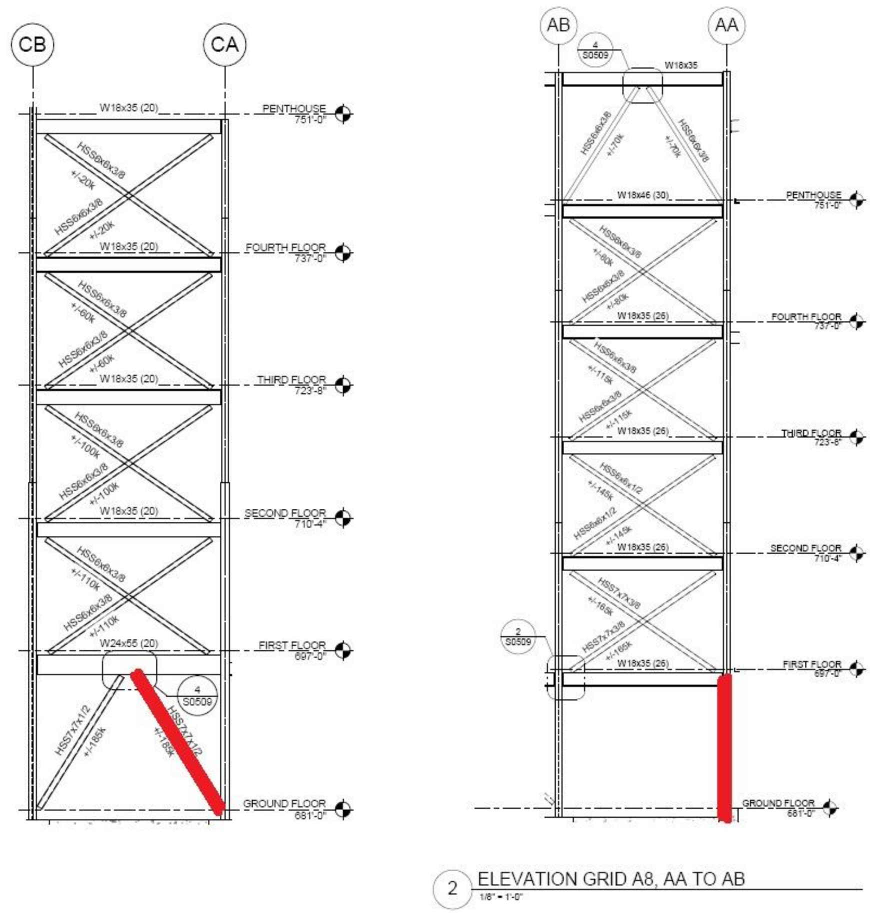


Figure 16: The two figures above show both members checked for strength. (Frame C8 on left, Frame A8 on right)

Conclusion

After a thorough analysis of the ECMC Skilled Nursing Facility, it was found that the building's lateral system was sufficient to carry the combinations of forces it was likely to experience. This conclusion is based upon a finite element computer model analysis, multiple hand calculations, and spot checks that were conducted for this technical report. Wind loads were determined via ASCE 7-10 using the Main Wind Force Resisting System procedure and seismic forces were found using the Equivalent Lateral Force procedure. Wind forces produced a base shear of 1052 kips and tended to be the dominating load case for the lateral system analysis, however seismic was still included in the analysis, producing a base shear of 455 kips. These values were similar to those found in construction documents.

A finite element computer model was created using ETABS software to provide a better understanding of the structural behavior of the building's lateral system. The model was designed as a rigid diaphragm that transferred lateral story shears into 16 concentrically braced frames scattered throughout the structure. These braced frames then transferred the lateral load down through the frame members into the foundations of the building. Eight of the frames were located on the outskirts of the building perimeter at the end of each wing, while the other eight frames surrounded and supported the building's central core.

Using ASCE 7-10, there was a significant increase in wind and seismic loads applied to the structure compared to that from ASCE 7-02. Even with the increase in loads between the different versions of ASCE 7, the lateral system of the building was still adequate in resisting these loads. The center of rigidity and the center of mass of the building were found to be relatively close to one another and located mainly in the center of the building, possibly due to the radial layout of braced frames and the symmetric geometry of the building. Although accidental torsion within the building did cause some significant moments, the building was sufficient in carrying any additional torsional effects.

Overall building drift and displacement were calculated using the ETABS finite element model and were checked against allowable drift limits of 0.015h and L/400 respectively. Upon inspection, it was found that the building's lateral system met the allowable limits of the code. Overturning moments were checked and it was found that the building possessed enough self-weight to resist these moments. In conclusion, it was determined that the concentrically braced frame lateral system found in the ECMC Skilled Nursing Facility was satisfactory to resist the various combinations of loads that it experienced.

Appendix A: Building Plans and Schedules

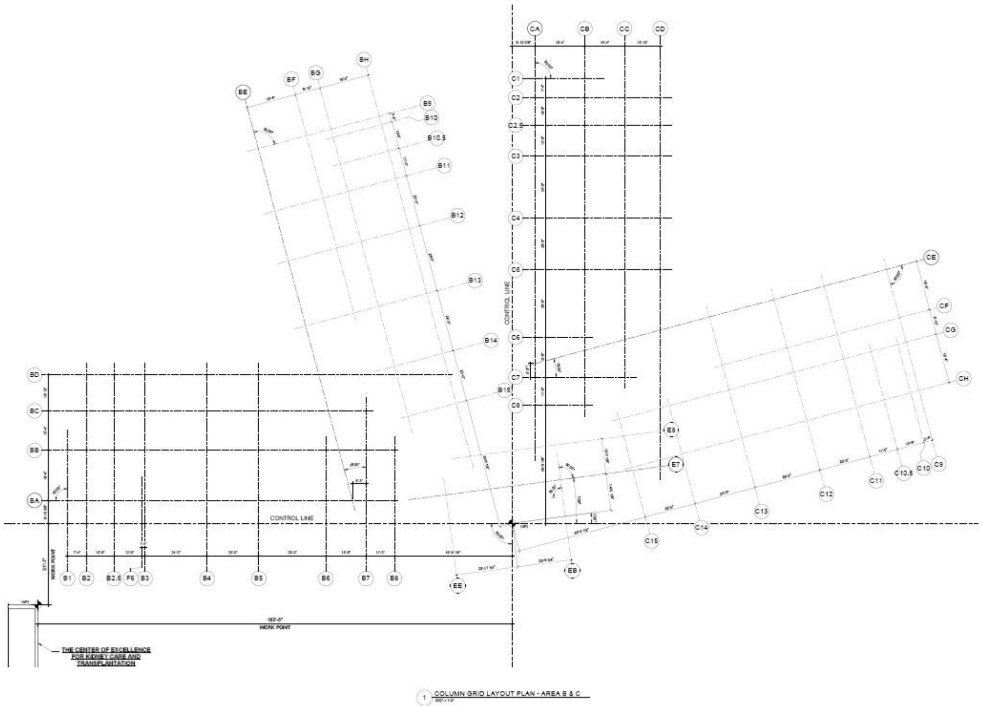
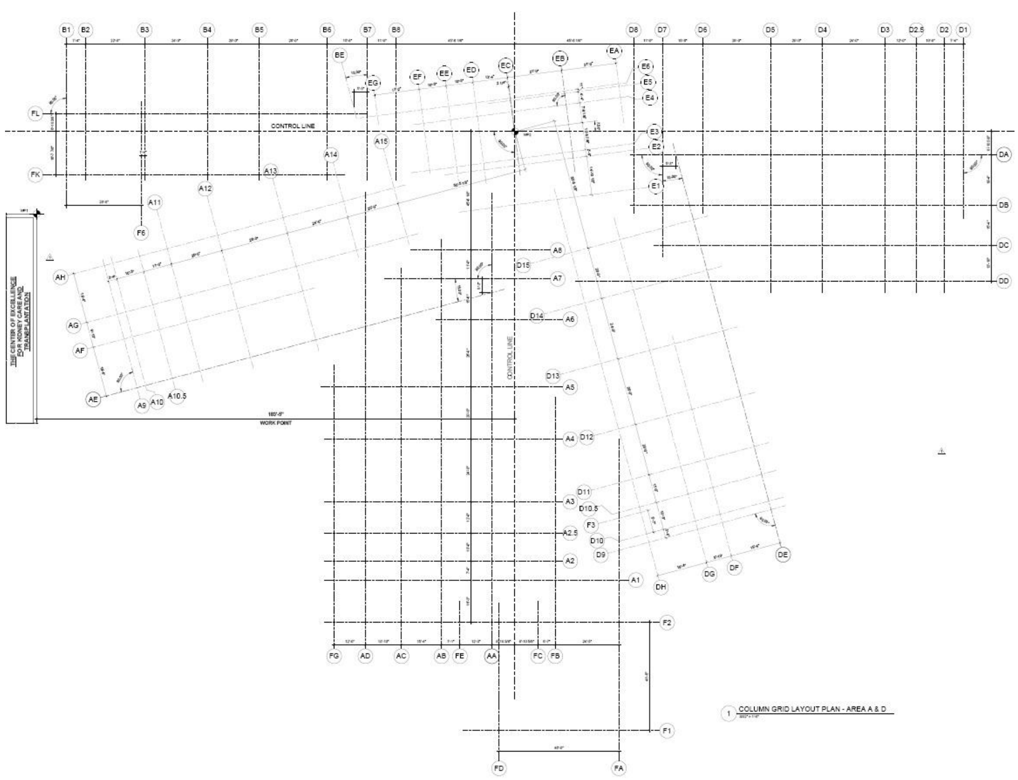
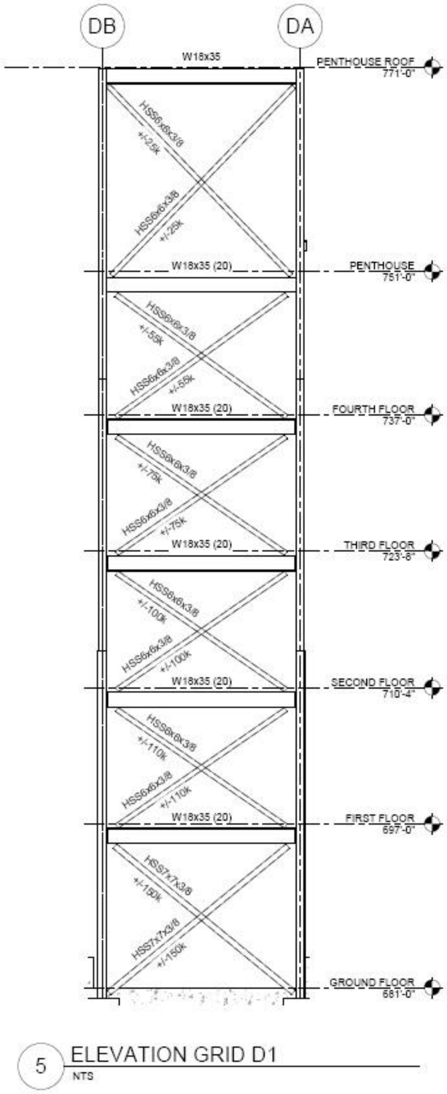
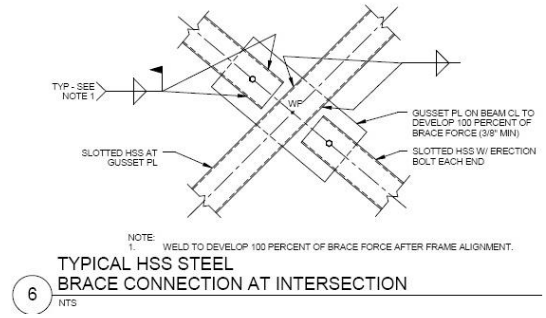
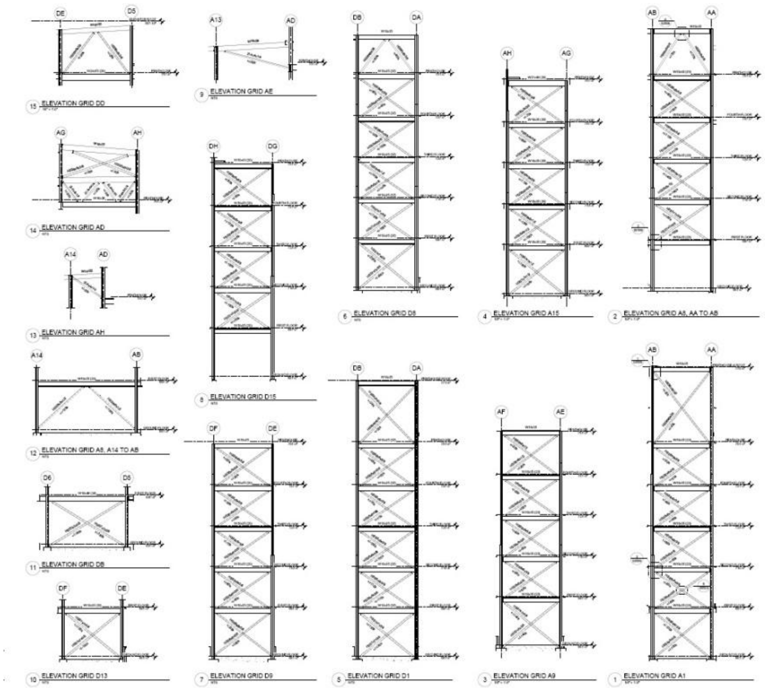


Figure 17: Column Grid Layout Plans (East End on bottom, West End on top) Details courtesy of Cannon Design.









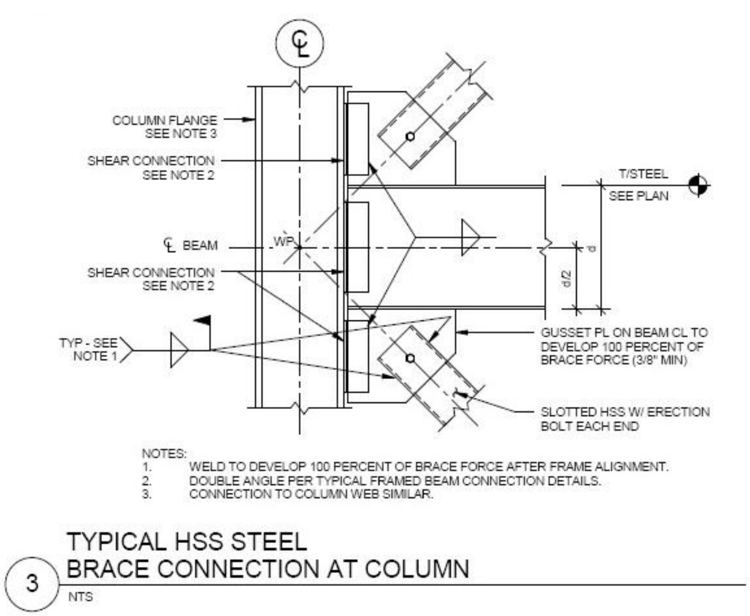


Figure 18: Concentric HSS Brace Frames and Connection Details. Details courtesy of Cannon Design.

FOOTING SCHEDULE					
ALLOWABLE BEARING PRESSURE : 16000 PSF					
MARK	WIDTH	LENGTH	DEPTH	REINFORCING (EW BOT UON)	REMARKS
F3.5	3'-6"	3'-6"	1'-8"	5-#5	
F4.5	4'-6"	4'-6"	1'-10"	8-#5	
F4.5-6.5	4'-6"	6'-6"	1'-10"	8-#5 LW, 10-#7 SW	
F5.5	5'-6"	5'-6"	2'-2"	6-#7	
F6.5	6'-6"	6'-6"	2'-6"	9-#7	
F6.5-8	6'-6"	8'-0"	2'-6"	9-#7 LW, 12-#7SW	
F7-A3	7'-0"	7'-0"	2'-8"	10-#7 B, 12-#7 T	HOOK TOP BARS, SEE DETAILS 15 & 16
F7-A4	7'-0"	7'-0"	2'-8"	10-#7 B, 12-#7 T	HOOK TOP BARS, SEE DETAILS 15 & 16

FOOTING SCHEDULE NOTES:

1. SEE PLANS AND DETAILS FOR TOP OF FOOTING ELEVATIONS.
2. SEE S0001 FOR FOUNDATION AND CONCRETE NOTES.

4 FOOTING SCHEDULE AND NOTES

1/2" = 1'-0"

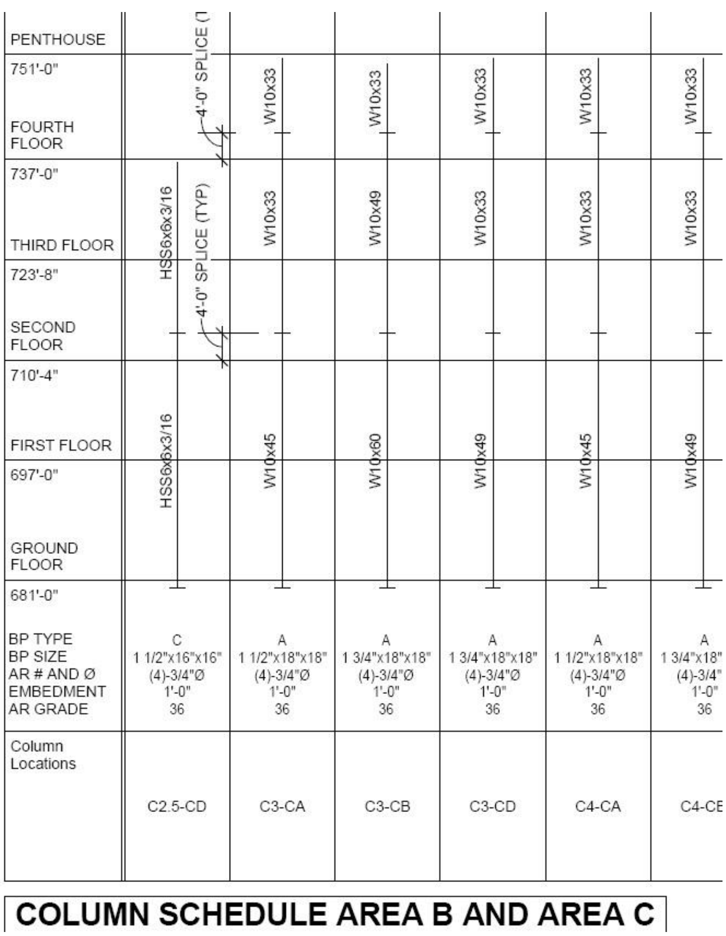
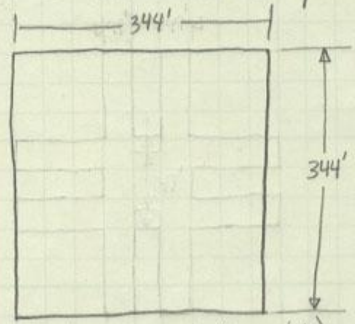


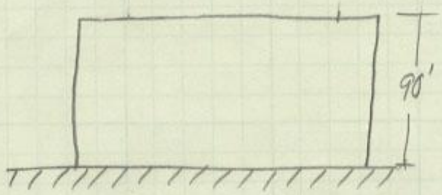
Figure 19: Footing Schedule (above) and Partial Column Schedule (left).

Appendix B: Calculations

Wind Analysis	Tech 3 Report	BRIAN BRUNNET	1

Note: 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.





Assume: Building Footprint (Symmetric)

max. h = 90 ft

STEPS TO FIND MWFRS WIND LOADS (Table 27.2-1)

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

Category III

2.) Determine Wind Speed

$V = 120$ mph

3.) Determine Wind Load Parameters:

$K_d = 0.85$

Exposure Category: B

$K_{zt} = 1.00$

For G: 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: $g_r = \underline{4.15}$

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

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

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

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

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

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

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

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

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

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

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

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

$$R = 0.239$$

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

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

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: Fully Enclosed ($GC_{pi} = \pm 0.18$)

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 $\Rightarrow \frac{3}{4}''/12'' = 3.58° < 10°$

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

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: $p = q G_f C_p - q_i (GC_{pi})$ [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

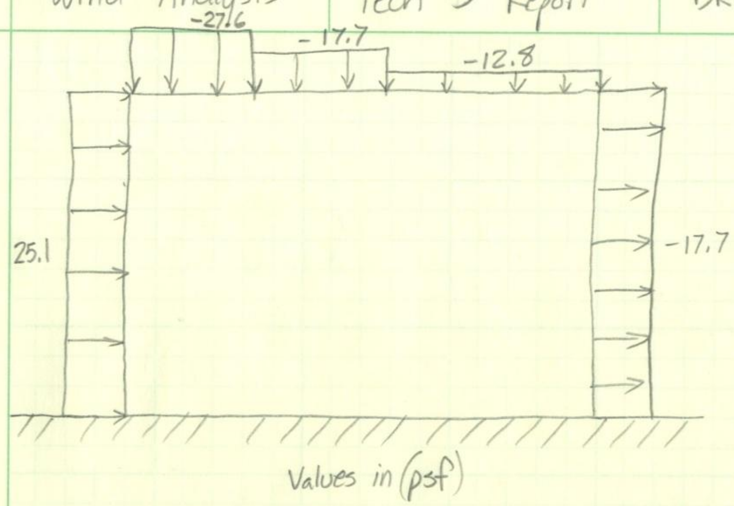
$= (-22.2) \pm 5.4$ psf for 0 to 90' $\longrightarrow$ -27.6 psf

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

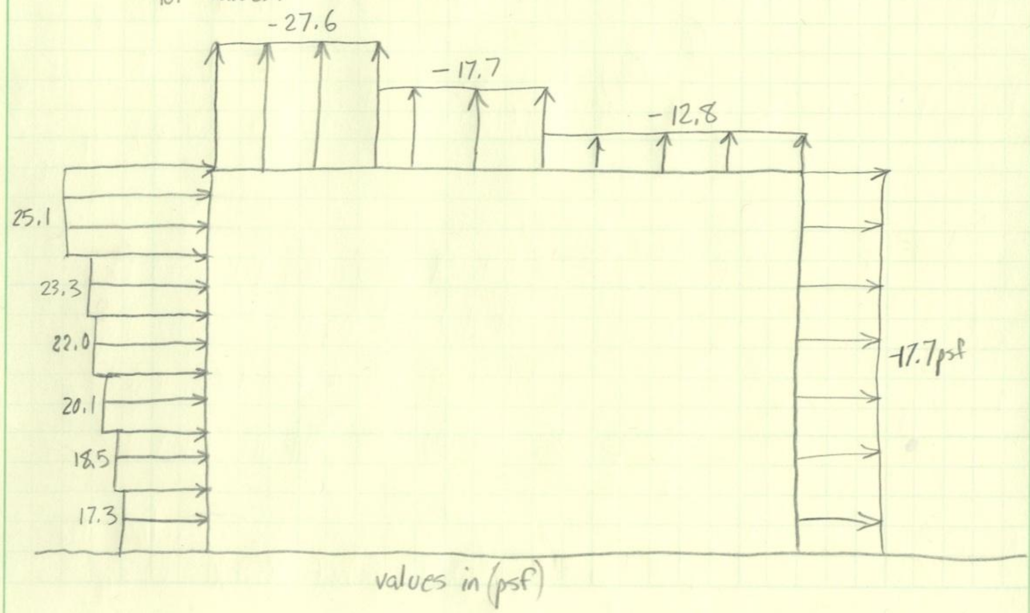
$= (-7.4) \pm 5.4$ psf for $> 180'$ $\longrightarrow$ -12.8 psf

side walls:

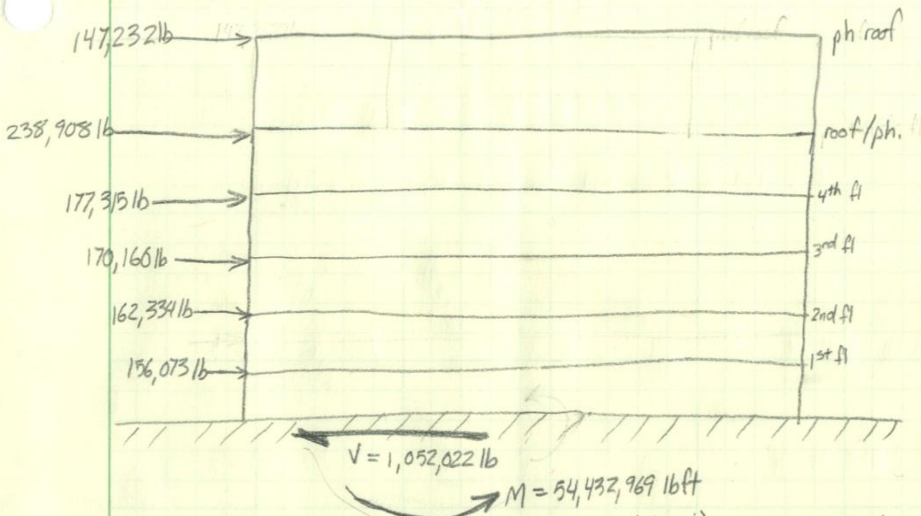
$p = (30.08)(.819)(-.7) - (30.08)(\pm .18) = -17.2 \pm 5.4$ psf $\longrightarrow$ -22.6 psf



* 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.



Building Shear / Floor Shear:



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

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

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

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

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

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

base shear → $\underline{V = 1,052,022\ lb}$

Overturning Moment: (M)

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

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

Seismic Analysis	Tech 3 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: 30 psf

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

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

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

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

$V = C_s W$

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

Concentrically Braced

$R = 3.25\,(12.2-1)$

$I_e = 1.25\,(1.5-2)$

$$C_s = \frac{0.140}{\left[\frac{3.25}{1.25}\right]}$$

$C_s = 0.054$

$T_a = C_T h_w^x$

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

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

C_s should be $< \frac{S_{D1}}{\left(\frac{R}{I}\right)T} = \frac{0.040}{\left(\frac{3.25}{1.25}\right)(.88)} = 0.0175 < 0.054$

controls

C_s should be > 0.01 ☑

$C_s = 0.0175$

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

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

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

Roof/Penthouse:

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

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

Floor 4:

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

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

Penthouse Roof:

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

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

Floors 1-3:

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

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

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

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

$V = C_s W = (0.0175)(26{,}045{,}435 lb) = \boxed{455{,}340 lb}$

$F_x = C_{vx} V$

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

$T = 0.88\,sec$

Using interpolation:

$K = 1.19$

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

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

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

$C_{ph} = \dfrac{215{,}281{,}020}{\Sigma w_i h_i^{\,k}} = 0.090 \qquad F_{ph} = (.090)(455{,}340 \text{ lb}) = 40{,}981 \text{ lb}$

$C_{rf} = \dfrac{649{,}958{,}413}{\Sigma w_i h_i^{\,k}} = 0.271 \qquad F_{rf} = (.271)(455{,}340 \text{ lb}) = 123{,}397 \text{ lb}$

$C_4 = \dfrac{641{,}633{,}493}{\Sigma w_i h_i^{\,k}} = 0.268 \qquad F_4 = (.268)(455{,}340 \text{ lb}) = 122{,}031 \text{ lb}$

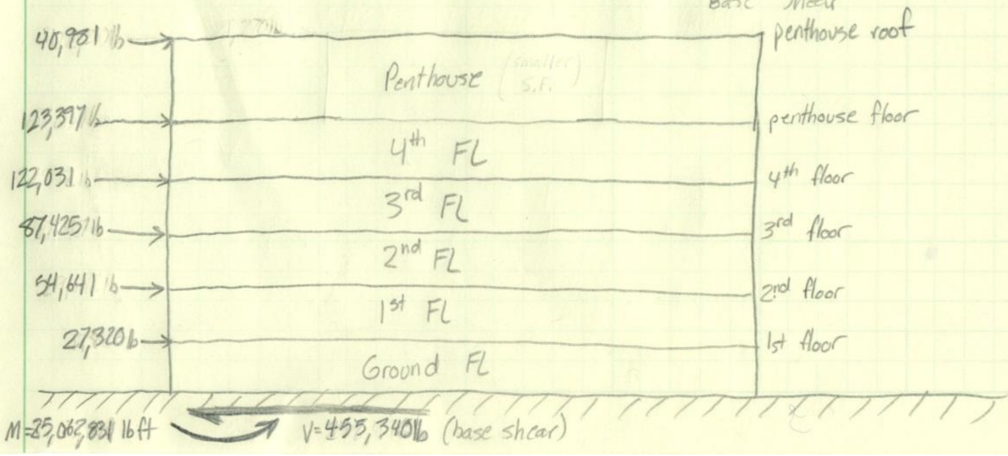
$C_3 = \dfrac{458{,}800{,}001}{\Sigma w_i h_i^{\,k}} = 0.192 \qquad F_3 = (.192)(455{,}340 \text{ lb}) = 87{,}425 \text{ lb}$

$C_2 = \dfrac{287{,}110{,}824}{\Sigma w_i h_i^{\,k}} = 0.120 \qquad F_2 = (.120)(455{,}340 \text{ lb}) = 54{,}641 \text{ lb}$

$C_1 = \dfrac{141{,}481{,}198}{\Sigma w_i h_i^{\,k}} = 0.060 \qquad F_1 = (.060)(455{,}340 \text{ lb}) = 27{,}320 \text{ lb}$

$\qquad\qquad 1.000 \ \boxed{\checkmark} \qquad\qquad V = 455{,}340 \text{ lb} \ \boxed{\checkmark}$

Base Shear



Overturning Moment:

$$M = (40{,}981)(90) + (123{,}357)(70) + (122{,}031)(57) + (157{,}425)(43) + (54{,}641)(29) + (27{,}320)(16)$$

$$M = \boxed{25{,}062{,}831 \text{ lbft}}$$

Flat Roof snow load using ASCE 7-10:

→ Exposure B (26.7.3)

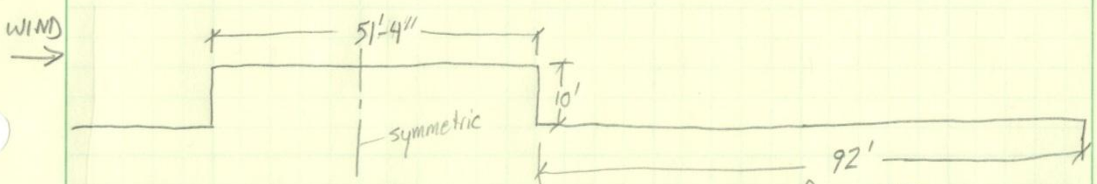
$p_f = 0.7\, C_e C_t I_s p_g$

$C_e = 0.9$ (for Fully Exposed)

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

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

Drift onto Penthouse: LEEWARD DRIFT



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

$h_d = 0.43 \sqrt[3]{l_u}\, \sqrt[4]{p_g + 10} - 1.5$ $\quad l_u = 51'\text{-}4"$, $p_g = 50$

$h_d = 0.43 \sqrt[3]{(61.33)}\, \sqrt[4]{50+10} - 1.5$

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

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

Drift onto Penthouse: Windward Drift

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

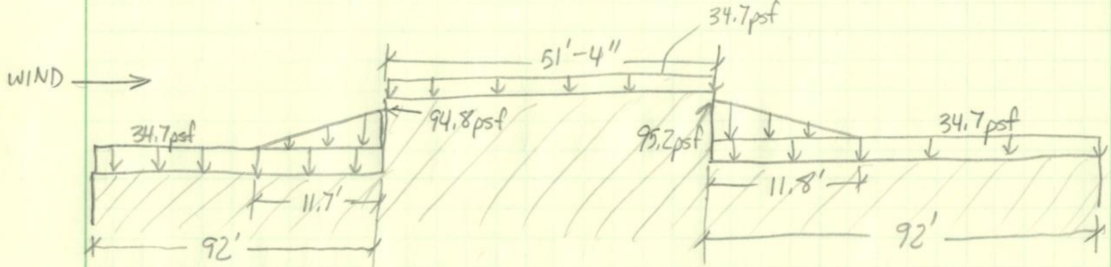
$h_d = \underline{2.93\text{ ft}}$

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

- since $h_d < h_c$:

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

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



* Using ASCE 7-10

Dead Loads:

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

32.2 psf

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

123.5 psf

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

90.8 psf

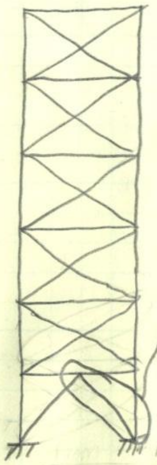
Live Loads: ASCE 7-10

Corridors: 40 psf
Lobbies: 100 psf
Balconies: 100 psf
Resident Rooms: 40 psf
Stairs & Exits: 100 psf
1st floor Corridor: 100 psf

Member checks	Tech 3 Report	BRIAN BRUNNET	1

Check of HSS Cross bracing member in brace frame #C8

worst case brace frame brace for Wind (N-S Direct.)

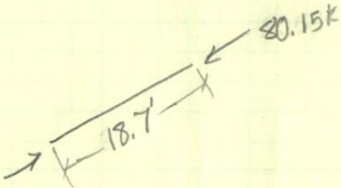


HSS 7×7×0.5" ASTM A500 GRADE B

$F_y = 46$ ksi

Max Axial Load = 178.1^k

Load Case 4: 1.2D + 1.0W + 1.0L Governs



Tension:

$\phi P_n = \phi F_y A_g = 0.9(46)(11.6\,in^2)$

$\phi P_n = 480.2^k > 178.1^k$ ☑ ok

Compression:

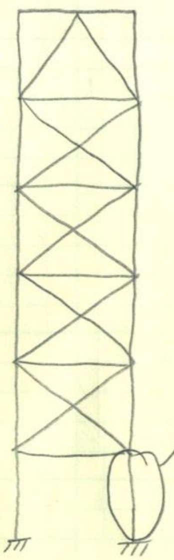
$I = 80.5\,in^4$

$P_{cr} = \frac{\pi^2 EI}{L^2} = \frac{\pi^2(29000)(80.5)}{(224.4)^2} = 457.6^k$

$\phi P_n = 0.9(457.6) = 411.8^k > P_u = 178.1^k$ ☑ ok

Check of W-flange column in braced frame #A8

worst case brace frame column for wind (N-S direct.)



W10×100 $F_y = 50$ ksi

From Wind

Load Case 4 controls

$1.2D + 1.0W + 1.0L$

$1.2(D) = 1.2(120^K) = 144.5^K$

$1.0(L) = 23.2^K$

Table 6-1 AISC Manual

$p = 1.11 \times 10^{-3}$

$b = 1.93 \times 10^{-3}$

$p(P_r) = (1.11 \times 10^{-3})(483.45) = 0.537 > 0.2$ ✓

$p(P_r) + b_x M_{rx} + b_y M_{ry} \leq 1.0$

$0.537 + (1.93 \times 10^{-3})(18.55) = 0.572 \leq 1.0$ ✓ ok