TECHNICAL REPORT 3

Lateral System Analysis and Confirmation Design

Penn State Hershey Medical Center Children's Hospital

Hershey, Pennsylvania

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Figures and Tables

Executive Summary

This technical report focuses on the analysis and confirmation of the existing lateral system of the Children's Hospital. The existing structure consists of a composite steel deck floor system utilizing steel moment frames and concentric braced frames. Pile caps comprised of several micropiles provide foundation support for the superstructure. The overall building dimensions are 359.1 feet by 124.25 feet with a total height of 85.5 feet above grade.

The wind and seismic loads determined from Technical Report 1 were updated to include more precise accounts for building weight and dimensions. These loads were then analyzed using ASCE 7-10 load combinations for strength design. The controlling load combination was determined to be:

$$\boxed{1.2\ (\text{Dead}) + 1.0\ (\text{Earthquake}) + 1.0\ (\text{Live}) + 0.2\ (\text{Snow})}$$

An ETABS model was constructed to examine adequate member strength, story drift, and impact on the foundation. The computer model was simplified to include only the main lateral resisting systems: moment frames and concentric braced frames. Rigid diaphragms were modeled at each story level above grade and assigned a mass based on the weight of elements which include: framing members, columns, composite slabs, facades, and superimposed dead loads.

Serviceability checks were performed to prevent excessive sway as well as to prevent damage to nonstructural components. The drift limit due to seismic loading for an occupancy category IV building is $\Delta_{seismic} = 0.01h_{sx}$. Similarly, the maximum allowable drift under wind loading was taken to be H/400. Story drifts of the structure determined from the model were found to fall within the allowable drift limits, see Tables 16 and 17.

Base reactions due to excessive loads have the potential to cause failure in the foundation. It is necessary to make sure the capacity of the foundation provides enough force to counter the uplift forces in the frames. It was determined that there was no impact on the foundation due to overturning caused by the controlling load case. The allowable tensile capacity of the micropiles is shown to provide sufficient resistance to uplift forces in the columns, see Table 18.

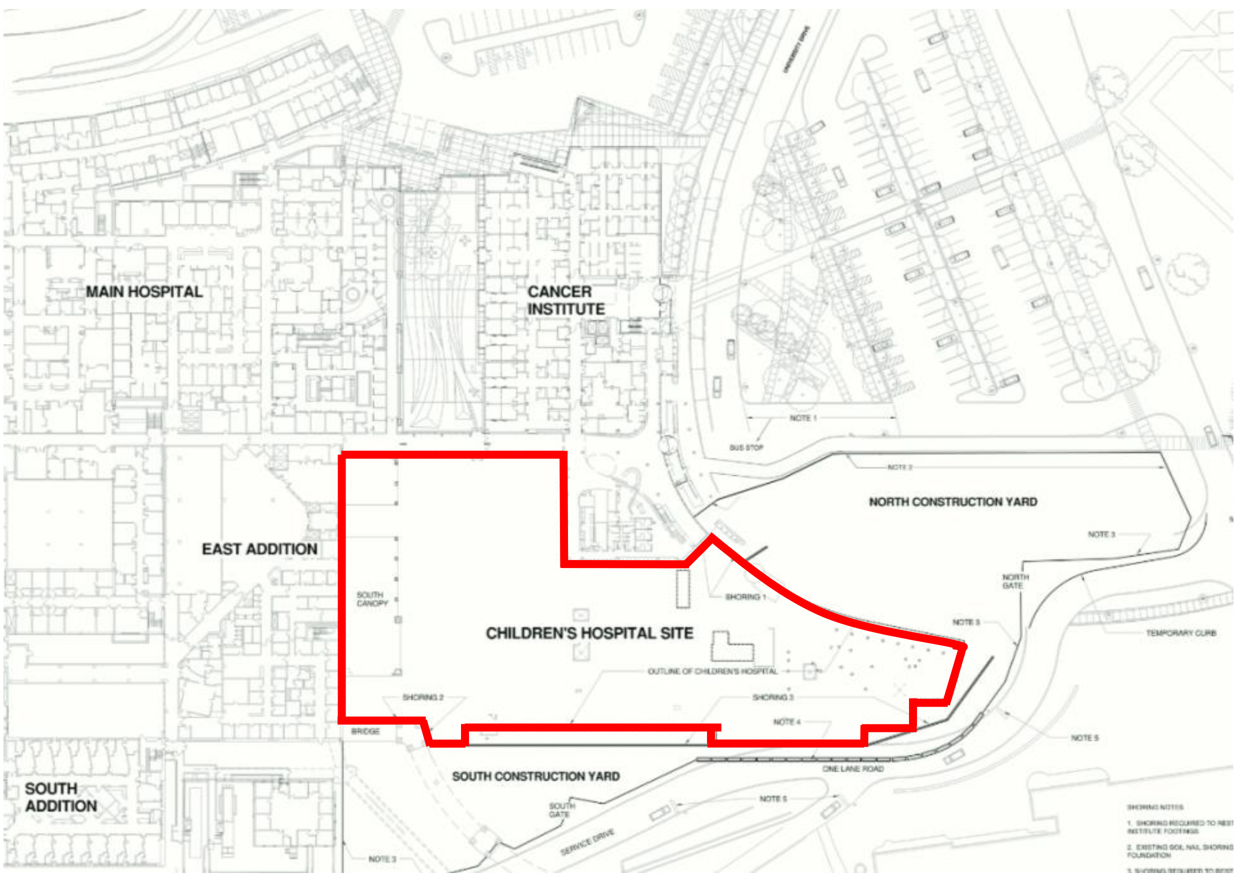
Spot checks were performed where member forces were determined to be greatest. All critical members were determined to be on the compression side of each frame. The selected members are located in Frame D in the East/West direction and Frame 7 in the North/South direction and can be seen in Figure 15. All members were determined to provide sufficient strength under the applied load cases. It would appear that members were designed more for serviceability than strength to prevent unnecessary damage to nonstructural elements.

It is the conclusion of this report that the existing lateral system provides adequate resistance to the applied load cases.

Building Overview

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

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



(Courtesy of: Payette Architects)

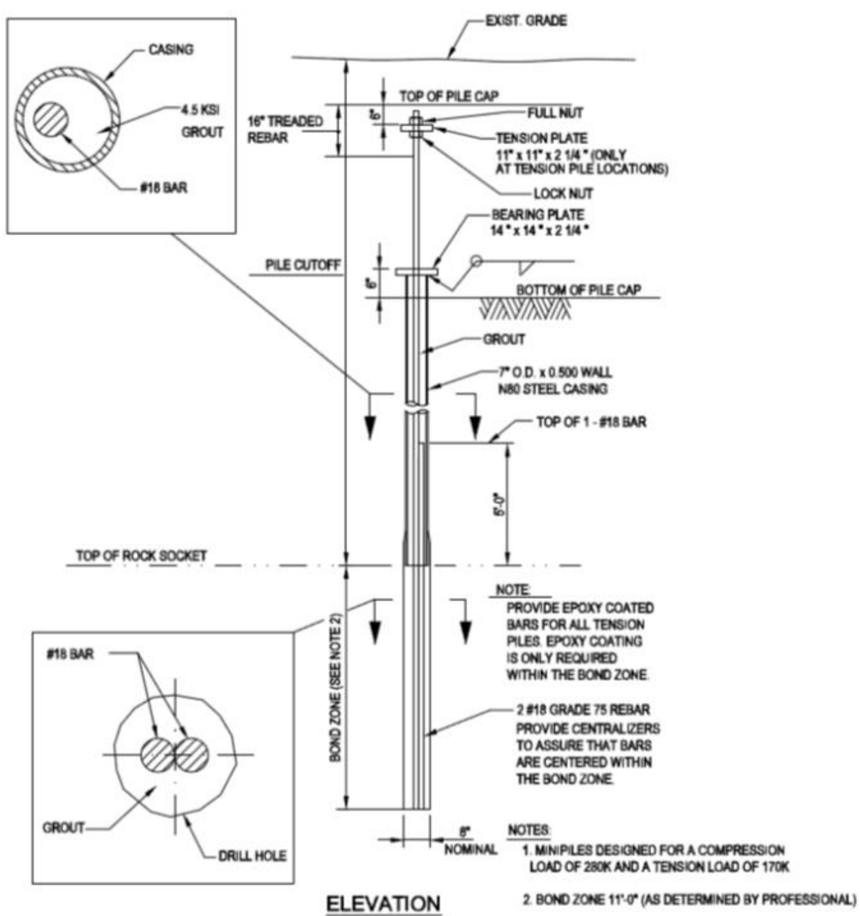
Figure 1 – Site Plan

Introduction to Structural System

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

Foundation

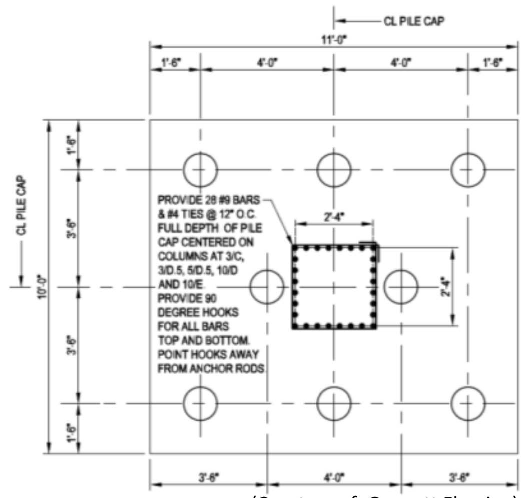
Due to the potential for excessive settlement, micropiles were utilized as recommended in the Geotechnical Report provided by CMT Laboratories. Micropiles consist of a casing that is injected with grout to create a friction bond within the bond zone. The piles that are used in the design are specified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



(Courtesy of: Gannett Fleming)

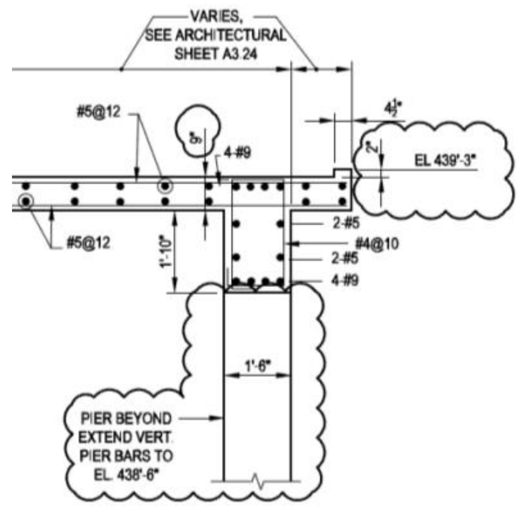
Figure 2 - Micropile Detail

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



(Courtesy of: Gannett Fleming)

Figure 4 - P8 Pile Cap Plan



(Courtesy of: Gannett Fleming)

Figure 3 - Typ. Strut Beam

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

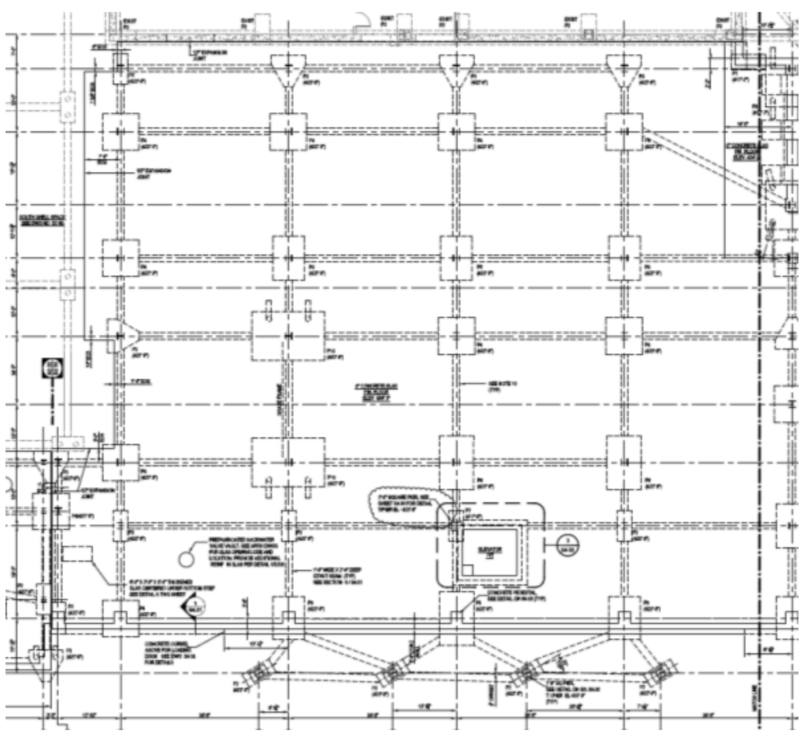


Figure 5 - West End Foundation Plan (Courtesy of: Gannett Fleming)

Floor System

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

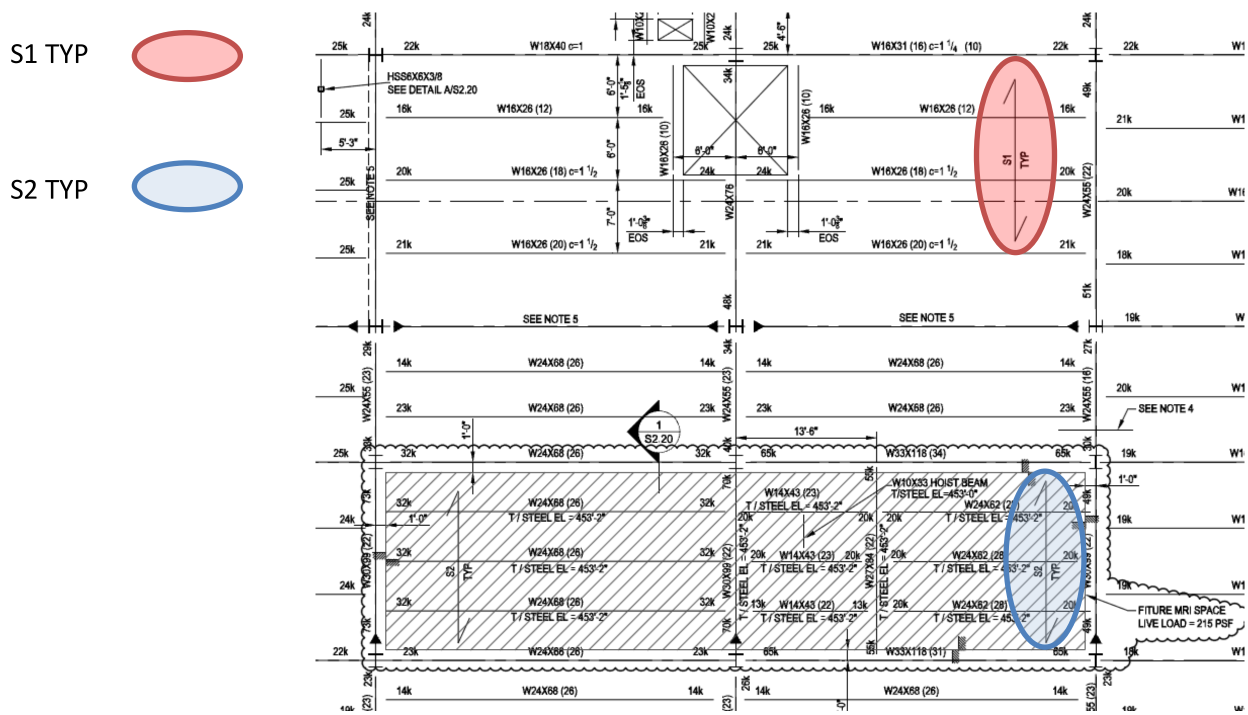
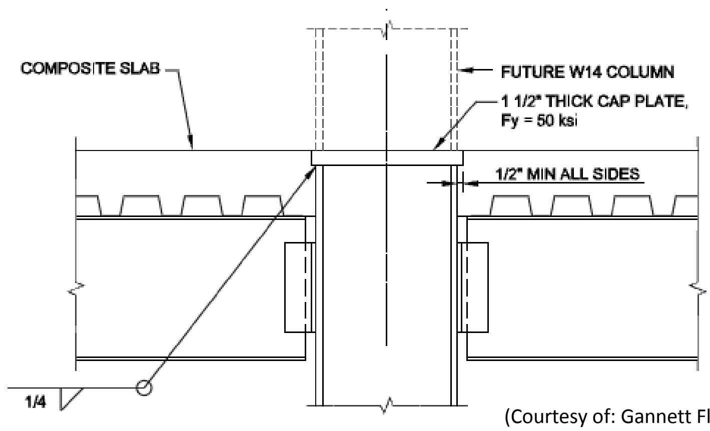


Figure 6 - Level 2 Framing Plan (Courtesy of: Gannett Fleming)

Roof System

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



(Courtesy of: Gannett Fleming)

Figure 7 - Top of Column at Future Sixth Floor

Lateral System

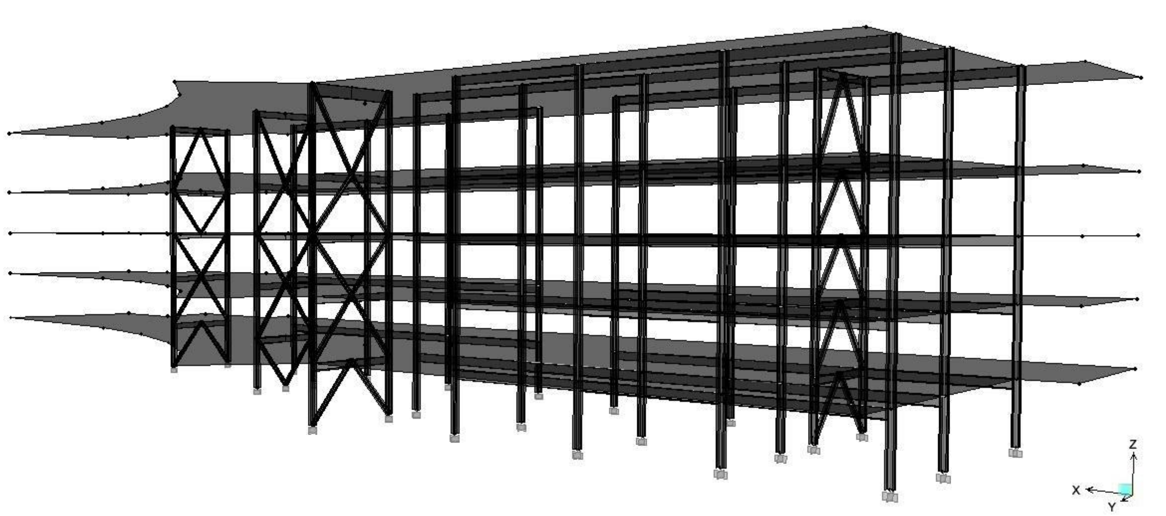
The main lateral force resisting system is composed of several moment frames located at the interior of the floor plan. These moment frames run in the East-West direction along the floor plan and are represented in Figure 8 with red. The purpose in placing the moment frames in these locations is to allow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

The main lateral members used in the moment frame system are wide flange sections, primarily W24x229 and W24x176 while the columns are W14x342 and W14x283. The braced frames used in the structure are comprised of W10x112 and W10x88 bracing members.

Conclusions on Structural System

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

Figure 8 – Framing Plan (Courtesy of: Gannett Fleming)



(Courtesy of: Author)

Figure 9 – ETABS model of Lateral Force Resisting System

Building Codes

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

"International Building Code, 2006 Edition"

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

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

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

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

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

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

"International Building Code, 2009 Edition"

AISC – Steel Construction Manual, 13th Edition

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

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

Allowable Building Drift: $\Delta_{wind} = H/400$

Allowable Story Drift: $\Delta_{seismic} = 0.010h_{sx}$

Materials

Structural Steel	
Wide Flanges	ASTM A992 Grade 50
Plates, Bars, and Angles	ASTM A36
HSS Rectangular Members	ASTM A500 Grade B
HSS Round Members	ASTM A500 Grade B
Anchor Rods	ASTM F1554 Grade 36
¾" High-Strength Bolts	ASTM A325-X
Welding Electrode	E70XX

Concrete	
Pile Caps	f'c = 4000 psi
Slab on Grade	f'c = 4000 psi
Foundation Walls	f'c = 4000 psi
Column Pedestals	f'c = 4000 psi
Strut Beams	f'c = 4000 psi
Note: all concrete is normal weight concrete (145 pcf)	

Reinforcement	
Reinforcing Bars	ASTM A615 Grade 60
Welded Wire Fabric	ASTM A185

Decking	
Floor Deck	2" Composite Metal Deck, 20 Ga.
Roof Deck	1 ½" Metal Roof Deck, 20 Ga.
¾" Shear Studs	ASTM A108
Masonry	
Grout (micropiles)	f'c = 4500 psi

Table 1 - Material Specification

Building Load Summary

Dead and Live Loads

The following live loads were determined using ASCE 7-10 while most of the dead loads are assumed based on the industry standard. The design loads sited in the drawing specifications are also listed to provide comparison between those that the design team used and what the code provides. Where specific gravity loads could not be determined, estimation was made with basic research.

Dead Loads	
Normal Weight Concrete	145 pcf
Structural Steel	490 pcf
2" Deep Metal Deck	69 psf
Superimposed Dead Load	30 psf
Aluminum Cladding	0.75 psf
Note: Superimposed Dead Load includes MEP systems, ceiling weights, and finishes	

Table 2 - Dead Live Loads

Live Loads		
Occupancy or Use	**Original Design**	**ASCE 7-10**
Lobbies/Moveable Seat Areas	100 psf	100 psf
Corridors (First Floor)	100 psf	100 psf
Corridors (Above First Floor)	80 psf	80 psf
Classrooms, Scientific Labs, Offices, Etc.	80 psf	60 psf
Electrical and Mechanical Rooms	250 psf	N/A
Stairs and Landings	100 psf	100 psf
Storage Areas: Light Storage	125 psf	125 psf
Storage Areas: Heavy Storage	250 psf	250 psf
Computer Rooms	100 psf	100 psf
Courtyards	100 psf	100 psf
Future MRI Space	215 psf	N/A

Table 3 - Live Loads

The total building weight was determined to be approximately 26,089 kips. The weight for the beams was tabulated based on the Revit model that was provided by Gannett Fleming. Since the floor to floor heights vary between levels, an average weight per length of columns was calculated. This involved counting and summing all the column weights throughout the structure. This value was then subsequently divided by the number of floors to determine an average weight of 15.484 k/ft for each floor. Slab weight and MEP weights were taken as an area load over each floor. The area for all floors was determined to be 37,297 ft^2. Façade weights were calculated by multiplying the perimeter of the floor plan by the tributary height of the floors for each level. Table 4 shows a summary of the building self-weight.

Level	Beams (k)	Columns (k)	Slab Weight (k)	MEP (k)	Façade (k)	Total (k)
Roof	386.4	170.3	2573.5	1118.9	7.7	4256.8
Penthouse	437.1	286.5	2573.5	1118.9	13	4429.0
4	342.5	232.3	2573.5	1118.9	10.6	4277.8
3	422.5	243.9	2573.5	1118.9	11.1	4369.9
2	468.8	243.9	2573.5	1118.9	11.1	4416.2
Ground	525.05	116.13	2573.5	1118.9	5.3	4338.9
					Total Building Weight=	26088.58

Table 4 - Building Self-Weight

Wind Loads

Wind load analysis is a critical factor in the structural design of the Children's Hospital. The wind forces were determined using ASCE 7-10 for Main Wind Force Resisting Systems (MWFRS). The structure was analyzed as a 359.1ft by 124.25 ft rectangle with a building height of 85.5 ft to the top of the parapet. The wind pressures were calculated for each face and then distributed to each story level. The total base shear and overturning moment were subsequently calculated for the building.

From Figure 10, the total base shear was calculated to be 1549.21 kips for the North-South wind loading. The total base shear for the East-West wind loading was determined to be 492.58 kips in Figure 11. The large difference in base shear is attributed to the face of the building normal to each wind direction. The North and South facades have about three times larger surface area than the East and West facades. This would explain why the base shear is about three times greater in the North and South direction compared to the East and West direction.

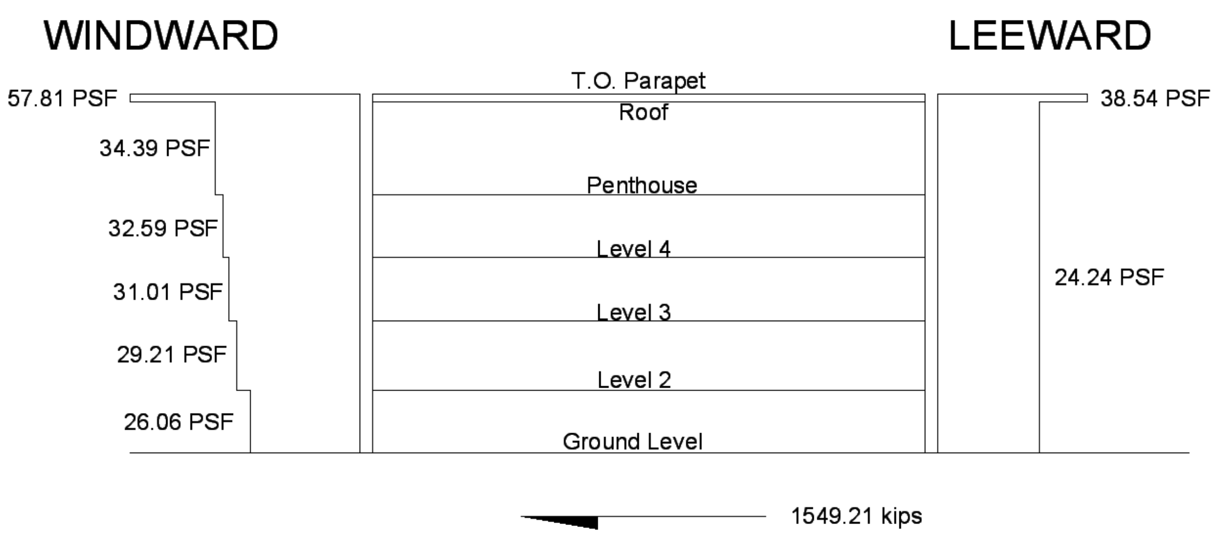


Figure 10 - North/South Wind Loading (Courtesy of: Author)

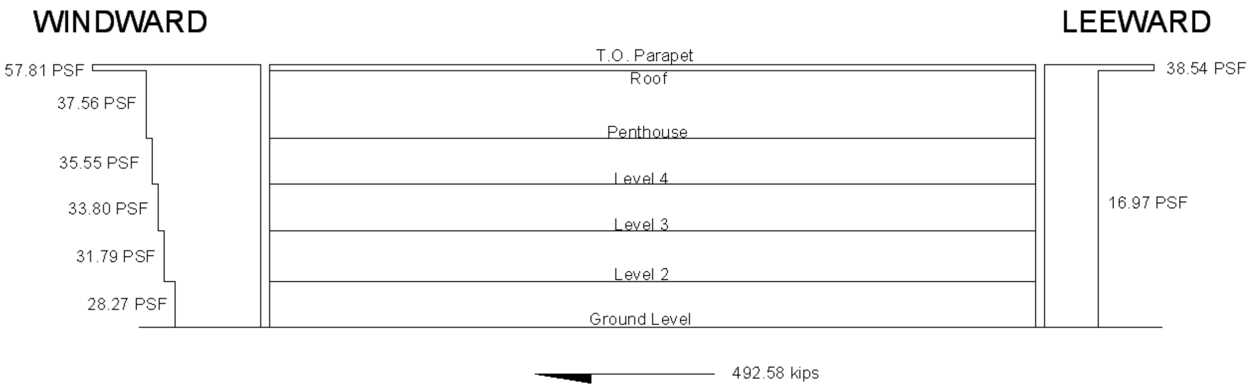


Figure 11 - East/West Wind Loading (Courtesy of: Author)

Refer to Tables 5 and 6 for the wind pressures and design forces, shears, and moments due to wind loading. Further factors and hand calculations for the wind analysis can be found in Appendix A of this report.

	Level	Height (ft)	K_z	q_z	Wind Pressure	
					N-S (psf)	E-W (psf)
Windward	T.O. Parapet	85.5	1.23	38.54	57.81	57.81
	Roof	83.5	1.22	38.23	34.39	37.56
	Penthouse	61.5	1.14	35.72	32.59	35.55
	4	46.5	1.07	33.53	31.01	33.80
	3	31.5	0.99	31.02	29.21	31.79
	2	15	0.85	26.63	26.06	28.27
	Ground	0	0.85	26.63	26.06	28.27
Leeward	T.O. Parapet	85.8	1.23	38.54	38.54	38.54
	Ground to Roof	83.5	1.23	38.54	24.24	16.97

Table 5 - Wind Pressures

Level	Height Above Ground (ft)	Floor Height (ft)	Total Pressure (psf)		Wind Forces					
					Load (kips)		Shear (kips)		Moment (ft-kips)	
			N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W
T.O. Parapet	85.5	0	96.35	96.35	34.60	11.97	34.60	11.97	0	0
Roof	83.5	2	58.63	54.53	252.64	81.31	287.24	93.28	69.20	23.94
Penthouse	61.5	22	56.83	52.52	377.53	120.73	664.76	214.01	6388.43	2076.03
4	46.5	15	55.25	50.77	297.62	94.61	962.38	308.62	16359.90	5286.13
3	31.5	15	53.45	48.76	302.32	95.42	1264.71	404.04	30795.68	9915.45
2	15	16.5	50.30	45.24	284.51	88.54	1549.21	492.58	51663.33	16582.06
Ground	0	15	50.30	45.24	0.00	0.00	1549.21	492.58	74901.54	23970.72

Table 6 - Wind Design Forces

Seismic Loads

Despite the site location being in an area of the country where the effects of earthquakes are minimal, it is still necessary to analyze the structure in terms of its seismic response. Seismic analysis was performed using ASCE 7-10 for seismic design criteria. To determine the base shear for the structure, the total weight for all floors above grade was calculated, see Appendix B. The weight was estimated to be around 26,089 kips. The base shear was calculated by finding the seismic response coefficient and multiplying that by the weight of the structure. The seismic response coefficient C_s was determined to be 4.6% which is comparable for a five story building. The calculations for determining the seismic response coefficient can be found in Appendix C.

The base shear for the structure was determined to be 1200.1 kips. Table 7 and Figure 12 show how each level experiences a different percent of the base shear based on the weight of that floor in relation to the overall weight. Comparing the base shear under wind loads to the base shear under seismic loads, the wind loads were determined to have a greater base shear. Since the site is located on the East Coast where predominantly wind controls, it is not surprising that this is the case. Both combined wind and seismic factors will be looked at under the controlling load combinations of this report.

Level	Height h_x (ft)	Story Weight w_x (kips)	$w_x*h_x^k$	C_{vx}	Lateral Force F_i	Story Shear V_x (kips)	Moment M_x (ft-k)
Roof	83.5	4256.6	983415.9	0.379	454.8	454.8	37978.7
Penthouse	61.5	4429	702460.8	0.271	324.9	779.7	47953.2
4	46.5	4277.8	481047.1	0.185	222.5	1002.2	46602.9
3	31.5	4369.9	304364.1	0.117	140.8	1143.0	36004.0
2	15	4416.2	123492.6	0.048	57.1	1200.1	18001.5
Total		**21749.5**	**2594780.5**		**1200.1**		**186540.4**

Table 7 - Seismic Forces

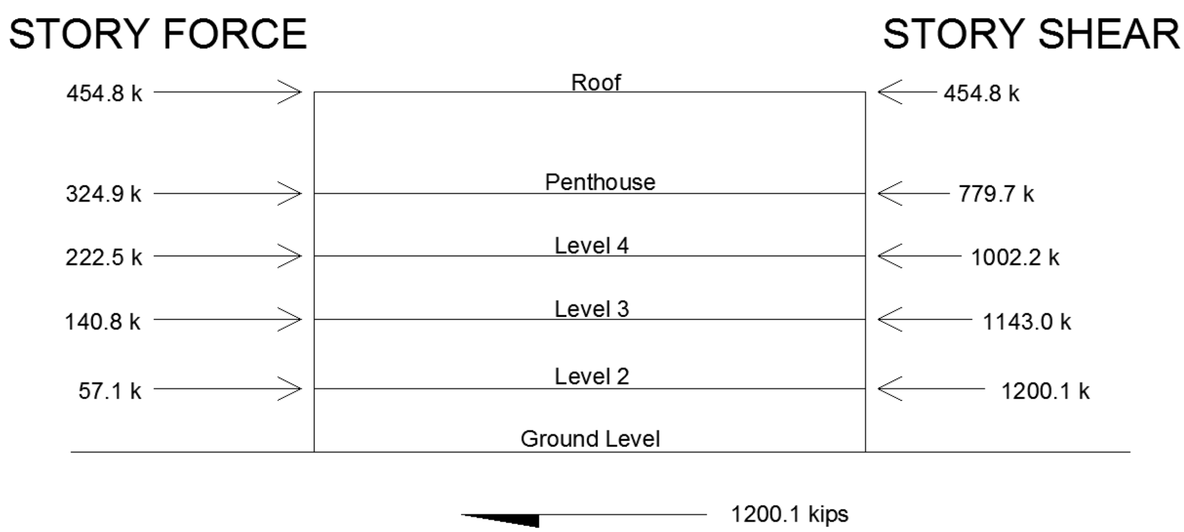


Figure 12 - Seismic Forces (Courtesy of: Author)

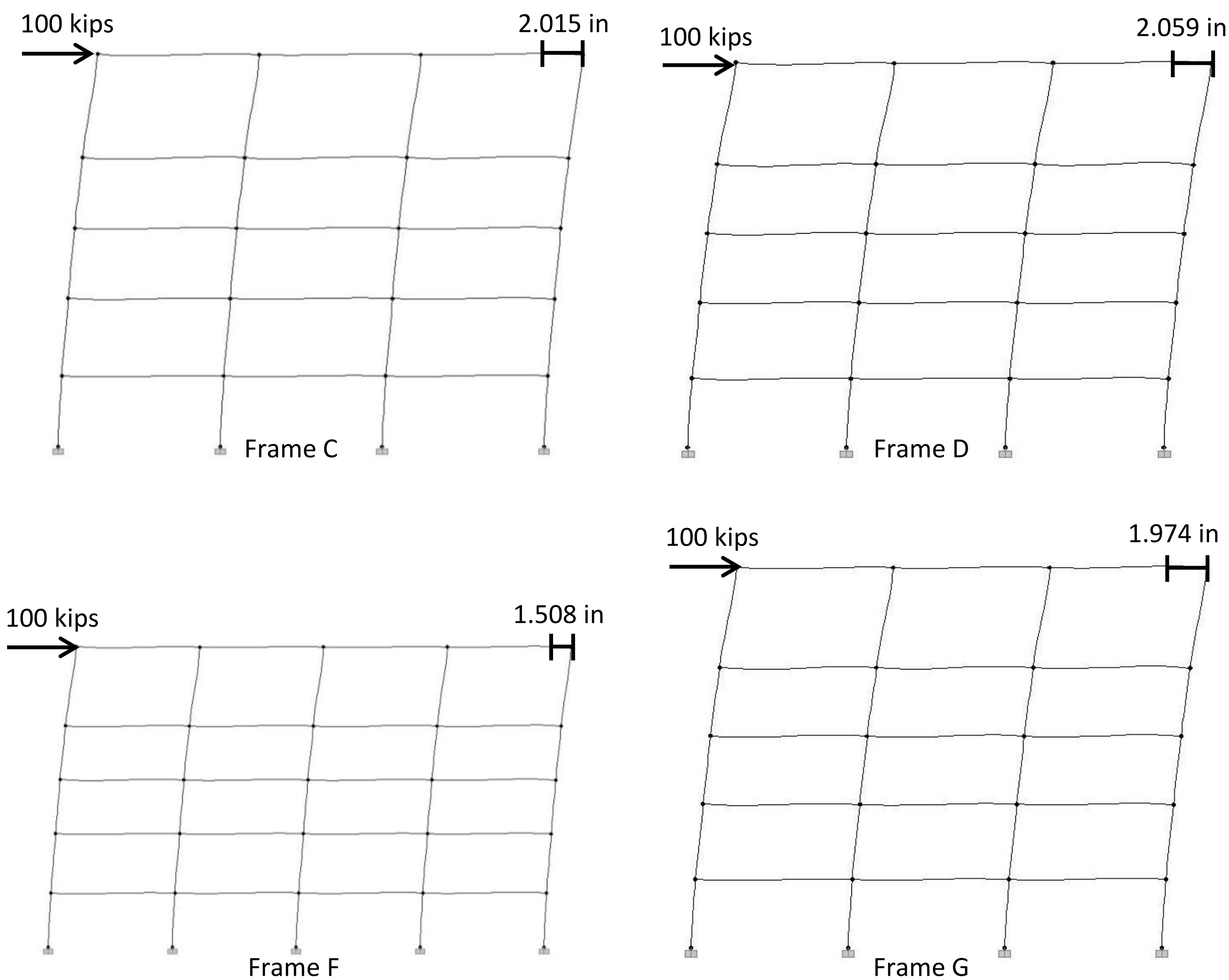
Lateral System In-Depth Analysis

Relative Stiffness of Lateral Elements

The relative stiffness was calculated for each lateral resisting frame in each direction. An ETABS model was constructed to determine the reactions of the system. To determine the stiffness, a 100 kip load was applied at the top story of each frame and the resulting displacement was recorded. The equation for the stiffness of each frame is:

$$K_i = \frac{P}{\Delta}$$

The stiffness of each frame can be used to show the amount of participation each frame contributes to the lateral resisting system. The following are the results for frames running East-West assuming a fixed base condition.

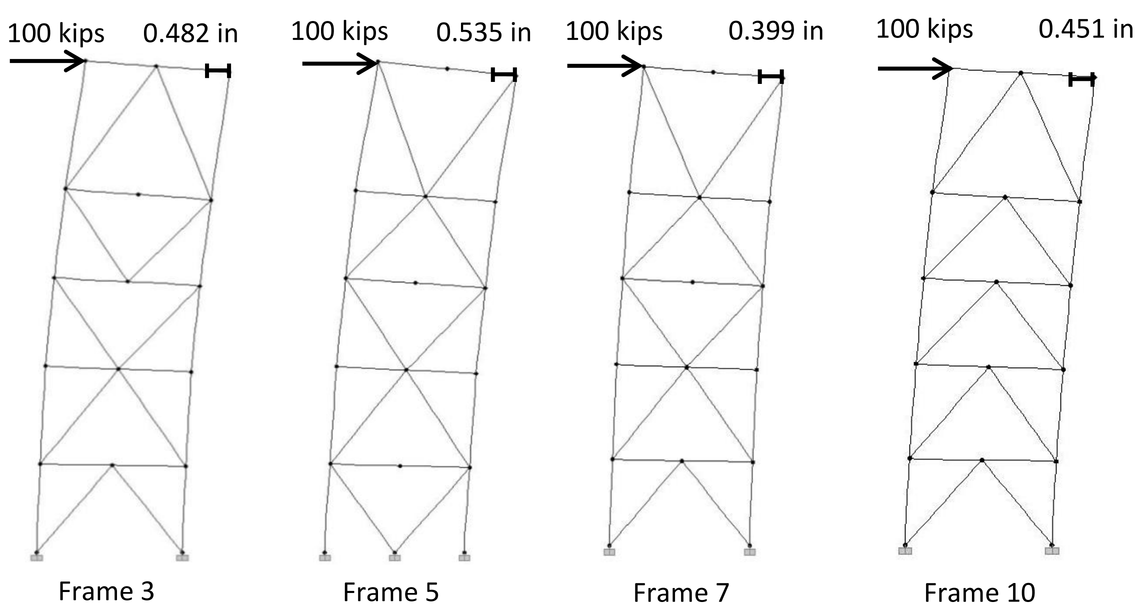


(Courtesy of: Author)

Relative Stiffness				
Frame	**Load (Kips)**	**Δ (in)**	**K (k/in)**	**Relative**
C	100	2.015	49.62779	23.06%
D	100	2.059	48.56727	22.57%
F	100	1.508	66.313	30.82%
G	100	1.974	50.65856	23.54%
		Total =	215.17	100.00%

Table 8 - E-W Frame Stiffness (Fixed)

The following are the results for frames running North-South assuming a fixed base condition.

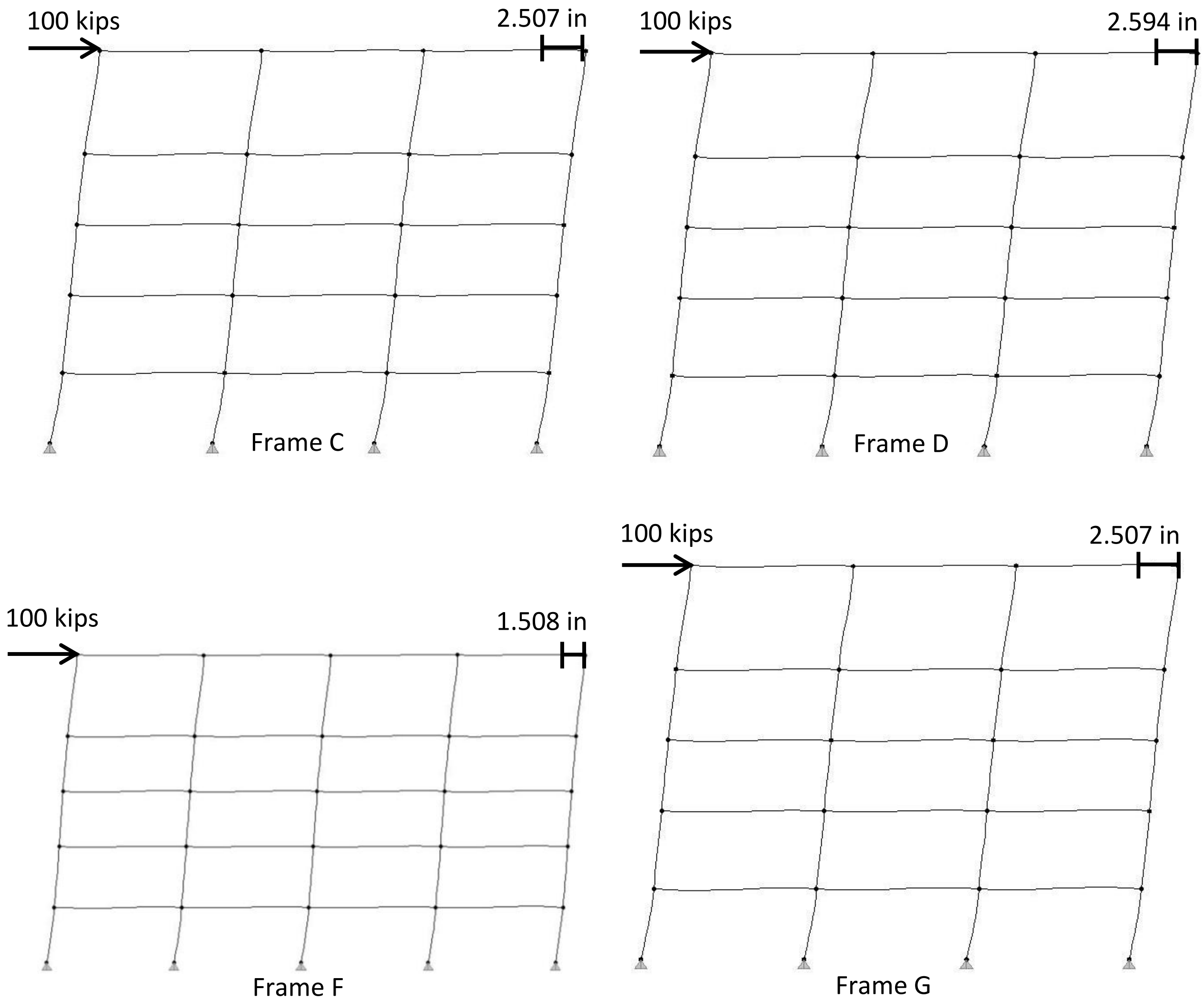


(Courtesy of: Author)

Relative Stiffness				
Frame	**Load (Kips)**	**Δ (in)**	**K (k/in)**	**Relative**
3	100	0.482	207.4689	23.94%
5	100	0.535	186.9159	21.57%
7	100	0.399	250.6266	28.92%
10	100	0.451	221.7295	25.58%
		Total =	866.74	100.00%

Table 9 - N-S Frame Stiffness (Fixed)

The following are the results for frames running East-West assuming a pinned base condition.

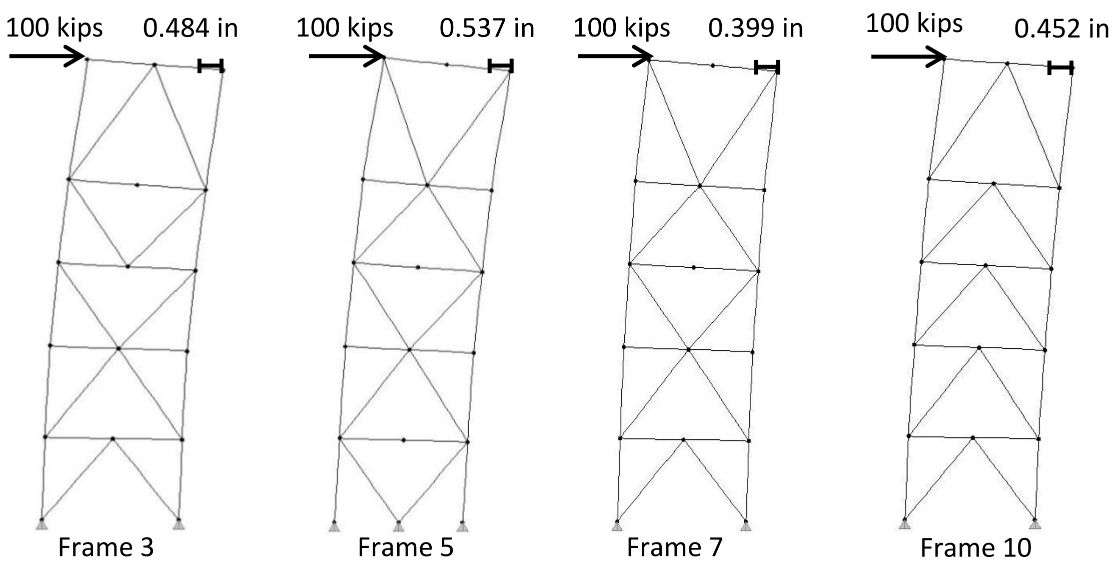


(Courtesy of: Author)

Relative Stiffness				
Frame	**Load (Kips)**	**Δ (in)**	**K (k/in)**	**Relative**
C	100	2.507	39.88831	18.54%
D	100	2.594	38.5505	17.92%
F	100	1.918	52.13764	24.23%
G	100	2.507	39.88831	18.54%
		Total =	170.46	100.00%

Table 10 - E-W Frame Stiffness (Pinned)

The following are the results for frames running North-South assuming a pinned base condition.



(Courtesy of: Author)

Relative Stiffness				
Frame	**Load (Kips)**	**Δ (in)**	**K (k/in)**	**Relative**
3	100	0.484	206.6116	23.8941%
5	100	0.537	186.2197	21.5358%
7	100	0.399	250.6266	28.9843%
10	100	0.452	221.2389	25.5857%
		Total =	864.70	100.00%

Table 11 - N-S Frame Stiffness (Pinned)

In comparison, there was little overall variance in the displacements for the frames oriented in the North-South direction. The overall stiffness of the concentrically braced frames is large enough to overcome the change from a fixed a pinned condition. The use of pinned connections had a greater effect on the displacement of the moment frames running in the East-West direction. The moment frames experienced an average decrease of about 20% in stiffness by changing to a pinned connection. It is also important to note the relative stiffness of any frame has a maximum difference of 10% with respect to other frames in the same direction. This demonstrates how evenly the distribution of forces will be to each frame.

Center of Rigidity

The center of rigidity was determined by taking into account the stiffness of each frame in relation to its distance to an origin point. The X and Y coordinates of the center of rigidity were calculated using the equations:

$$X_R = \frac{\Sigma R_i x_i}{\Sigma R_i} \qquad Y_R = \frac{\Sigma R_i y_i}{\Sigma R_i}$$

The Table 12 shows the coordinates of the center of rigidity and center of mass for both the fixed and pinned conditions calculated by hand.

Center of Rigidity and Eccentricity						
	COR_x (ft)	COR_y (ft)	COM_x (ft)	COM_y (ft)	e_x (ft)	e_y (ft)
Fixed Base	172.3	69.2	160.56	58.84	11.74	10.36
Pinned Base	172.18	68.97	160.56	58.84	11.62	10.13

Table 12 - Center of Rigidity and Eccentricity

From the ETABS model, the center of rigidity was tabulated for each floor. The average center of rigidity from the ETABS model was determined to be (174.5 ft, 69.2 ft) for the fixed base condition and (175.6 ft, 69.1 ft) for the pinned condition. This is close to the hand calculated values for the center of rigidity in Table 12. Looking at the coordinates for both the fixed and pinned cases, the center of rigidities are almost the same. It is apparent by this comparison that although the individual frame stiffness may change by switching base conditions, the overall center of rigidity remains relatively unaffected.

Load Combinations

It was necessary to determine the overall load combination. Wind loads and seismic loads were applied at the center of pressure and center of mass respectively for each level. According to ASCE 7-10 Chapter 12.8.4.2 for seismic design, accidental torsion should be considered caused by a distance equal to 5% of the building width perpendicular to the direction of the applied forces. Live and dead loads were applied as uniform loads to the diaphragms. The following load combinations were considered from ASCE 7-10 Chapter 2.3 – "Combining Factored Loads Using Strength Design":

1.) 1.4D
2.) $1.2D+1.6L+0.5L_r$
3.) $1.2D+1.6L_r+(1.0L \text{ or } 0.5W)$
4.) $1.2D+1.0W+1.0L+0.5L_r$
5.) 1.2D+1.0E+1.0L+0.2S
6.) 0.9D+1.0W
7.) 0.9D+1.0E

Four variations of the wind loading were considered according to Figure 27.4-8 in ASCE 7-10. Figure 13 shows these design wind load cases as part of the Main Wind Force Resisting System. These cases include:

Case 1: 100% of wind pressure acting in the North/South or East/West direction

Case 2: 75% of wind pressure acting in the North/South or East/West direction with torsion

Case 3: 75% of wind pressure acting on all faces concurrently

Case 4: 56.3% of wind pressure acting on all faces concurrently with torsion

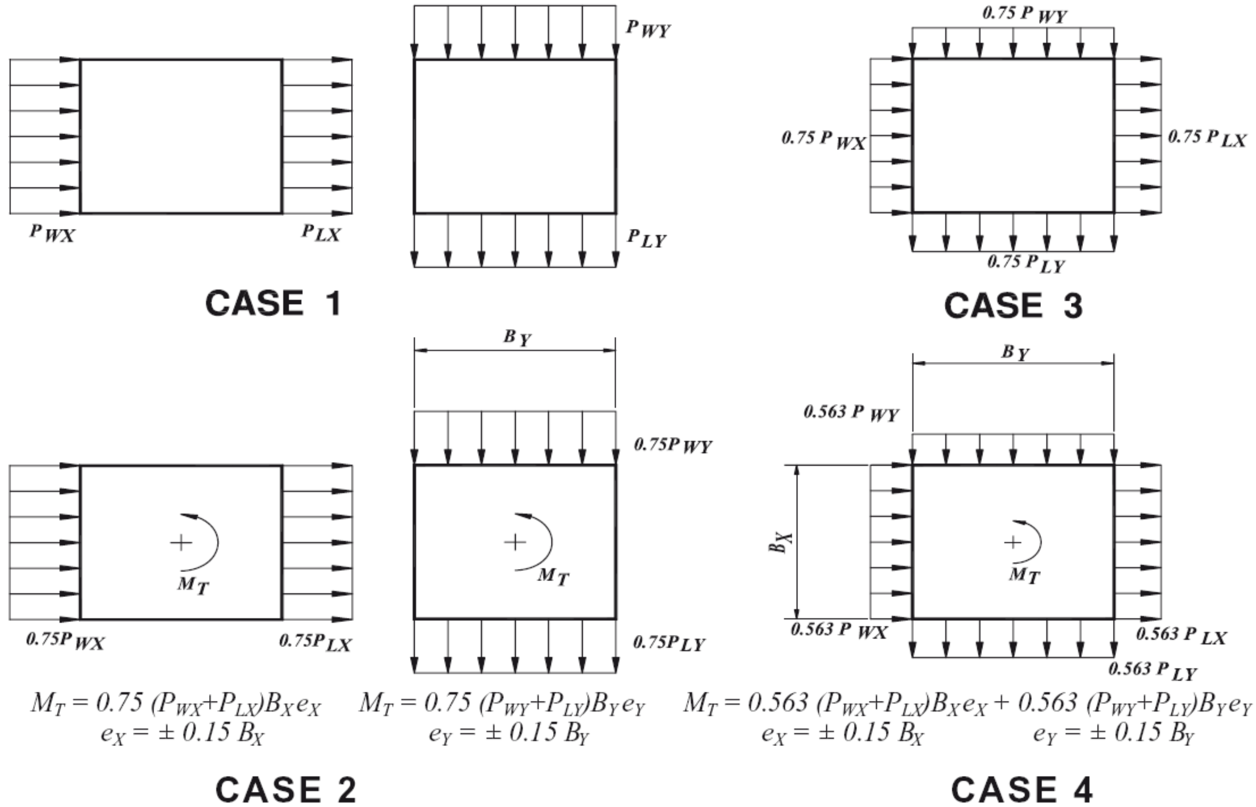


Figure 13 - Design Wind Load Cases (Courtesy of: AISC/SEI 7-10)

To determine which of these wind cases controlled, the story forces in Table 6 were applied to each diaphragm. For cases 1 and 3, the forces were applied at the center of mass. For cases 2 and 4, eccentricity had to be considered. The eccentricity was determined by considering 15% of the overall width, see Table 13. Since rotation occurs around the center of rigidity, both positive and negative eccentricity had to be considered to determine which eccentricity would have a greater impact. Figure 14 shows the positive sign convention that was used for the wind load cases.

Eccentricity (e=0.15B)				
Level	**B_x (ft)**	**e_x (ft)**	**B_y (ft)**	**e_y (ft)**
Roof	359.1074	53.86611	149.0582	22.35873
Penthouse	359.1074	53.86611	149.0582	22.35873
4	359.1074	53.86611	149.0582	22.35873
3	359.1074	53.86611	149.0582	22.35873
2	359.1074	53.86611	149.0582	22.35873

Table 13 - Wind Case Eccentricity

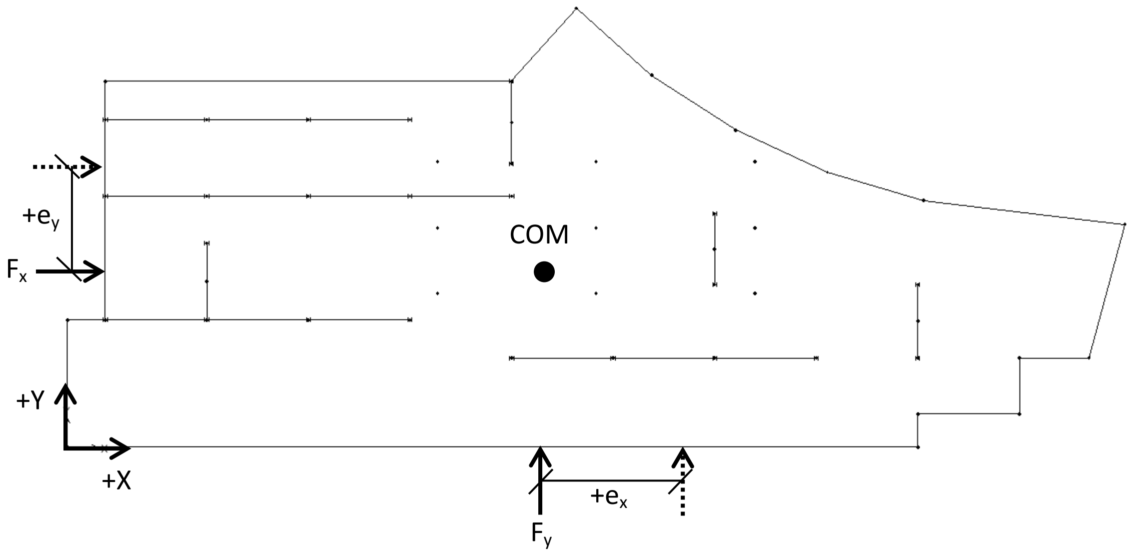


Figure 14 - Wind Case Sign Convention (Courtesy of: Author)

The positive and negative eccentricities were taken from the center of mass for each wind direction. The following tables show the locations where each of the wind forces were applied for case 2.

Case 2 North/South Positive Eccentricity

Level	Fx	Fy	XCCOM	YCCOM
Roof	0	189.48	233.42	74.53
Penthouse	0	283.15	233.42	74.53
4	0	223.22	233.42	74.53
3	0	226.74	233.42	74.53
2	0	213.38	233.42	74.53

Case 2 North/South Negative Eccentricity

Level	Fx	Fy	XCCOM	YCCOM
Roof	0	189.48	125.69	74.53
Penthouse	0	283.15	125.69	74.53
4	0	223.22	125.69	74.53
3	0	226.74	125.69	74.53
2	0	213.38	125.69	74.53

Case 2 East/West Positive Eccentricity

Level	Fx	Fy	XCCOR	YCCOR
Roof	60.98	0	179.55	96.89
Penthouse	90.55	0	179.55	96.89
4	70.96	0	179.55	96.89
3	71.57	0	179.55	96.89
2	66.41	0	179.55	96.89

Case 2 East/West Negative Eccentricity

Level	Fx	Fy	XCCOR	YCCOR
Roof	60.98	0	179.55	52.17
Penthouse	90.55	0	179.55	52.17
4	70.96	0	179.55	52.17
3	71.57	0	179.55	52.17
2	66.41	0	179.55	52.17

For case 4, eccentricities had to be considered in both directions simultaneously. The following tables show the locations where each of the wind forces were applied for case 4.

Case 4 North/South and East/West (e_x pos. e_y pos)

Level	Fx	Fy	XCCOR	YCCOR
Roof	45.78	142.24	233.42	96.89
Penthouse	67.97	212.55	233.42	96.89
4	53.27	167.56	233.42	96.89
3	53.72	170.21	233.42	96.89
2	49.85	160.18	233.42	96.89

Case 4 North/South and East/West (e_x neg. e_y neg)

Level	Fx	Fy	XCCOR	YCCOR
Roof	45.78	142.24	125.69	52.17
Penthouse	67.97	212.55	125.69	52.17
4	53.27	167.56	125.69	52.17
3	53.72	170.21	125.69	52.17
2	49.85	160.18	125.69	52.17

Case 4 North/South and East/West (e_x pos. e_y neg)

Level	Fx	Fy	XCCOR	YCCOR
Roof	45.78	142.24	233.42	52.17
Penthouse	67.97	212.55	233.42	52.17
4	53.27	167.56	233.42	52.17
3	53.72	170.21	233.42	52.17
2	49.85	160.18	233.42	52.17

Case 4 North/South and East/West (e_x neg. e_y pos)

Level	Fx	Fy	XCCOR	YCCOR
Roof	45.78	142.24	125.69	96.89
Penthouse	67.97	212.55	125.69	96.89
4	53.27	167.56	125.69	96.89
3	53.72	170.21	125.69	96.89
2	49.85	160.18	125.69	96.89

After applying the wind load cases to the ETABS model, it was necessary to determine the overall controlling wind case. Table 14 shows the displacements for the roof level based on each of the cases. The wind case that controlled was case 1, where 100% of the wind load is applied in each direction.

Story	Load	X-Disp. (in)	Y-Disp. (in)
PH ROOF	CASE1NS	0.0062	0.7434
PH ROOF	CASE1EW	1.0346	0.0012
PH ROOF	CASE2NSPOS	0.0429	0.5173
PH ROOF	CASE2NSNEG	-0.0336	0.5978
PH ROOF	CASE2EWPOS	0.7709	0.0062
PH ROOF	CASE2EWNEG	0.781	-0.0044
PH ROOF	CASE3	0.7806	0.5584
PH ROOF	CASE4POSPOS	0.6109	0.393
PH ROOF	CASE4NEGNEG	0.5611	0.4454
PH ROOF	CASE4POSNEG	0.6185	0.385
PH ROOF	CASE4NEGPOS	0.5535	0.4534

Table 14 - Controlling Wind Case

After analyzing the ETABS model under the applicable load combinations, it was determined that load combination 5 (1.2D+1.0E+1.0L+0.2S) controlled. Table 15 shows the roof level displacements for all load combinations. Load combination 8 represents 1.2D+1.0E_x+1.0L+0.2S, which considers the earthquake loading in the x-direction. Similarly, load combination 9 represents 1.2D+1.0E_y+1.0L+0.2S, which considers the earthquake loading in the y-direction. It is important to note that COMB12 and COMB13 have the same displacements as the controlling load case. These two combinations represent the 0.9D+1.0E load case. These displacements are the same since only the earthquake load contributes to lateral displacement. Gravity forces such as dead, live, and snow loads, will be important when comparing forces in typical members. It is therefore appropriate to select the 1.2D+1.0E+1.0L+0.2S load combination over 0.9D+1.0E since the former would have the greatest impact on members due to combined loads.

Story	Load	X-Disp. (in)	Y-Disp. (in)
PH ROOF	COMB1	0	0
PH ROOF	COMB2	0	0
PH ROOF	COMB3	0	0
PH ROOF	COMB4	0.5173	0.0006
PH ROOF	COMB5	0.0031	0.3717
PH ROOF	COMB6	1.0346	0.0012
PH ROOF	COMB7	0.0062	0.7434
PH ROOF	COMB8	3.6067	-0.005
PH ROOF	COMB9	0.0065	0.843
PH ROOF	COMB10	1.0346	0.0012
PH ROOF	COMB11	0.0062	0.7434
PH ROOF	COMB12	3.6067	-0.005
PH ROOF	COMB13	0.0065	0.843

Table 15 - Controlling Load Combination

Story Shears

Story shears were considered for the 1.2D+1.0E+1.0L+0.2S load combination. Since the base shear due to earthquake can act in both the x and y directions, it was necessary to check shear in the frames for both loading directions. The results for the shear forces in each frame can be found on page 27. The tables are divided to show the differentiation between the fixed and pinned base condition. For most cases, the pinned base condition experienced slightly greater shear forces in the frames. The summation of the shear forces at the ground level in each table are approximately 1200.1 kips, which is the seismic base shear determined in Figure 12. Looking at the relative stiffness of each frame determined in Tables 8 to 11, Frame F has the greatest stiffness in the x-direction. By inspection, it is appropriate that frame F would take the greatest force due to earthquake loading in the x-direction. Similarly, Frame 7 has the greatest relative stiffness in the y-direction. Therefore, it would take the greatest amount of force due to earthquake loading in the y-direction.

Load Combination 1.2D+1.0E$_x$+1.0L+0.2S with Fixed Base

Story	East/West Frames: Shear (kips)				North/South Frames: Shear (kips)			
	Frame C	Frame D	Frame F	Frame G	Frame 3	Frame 5	Frame 7	Frame 10
Roof	0	0	0	0	0	0	0	0
Penthouse	110.5	102.62	140.82	107.2	-1.9	-1.9	-1.88	-0.65
4	183.49	171.49	236.49	178.87	2.59	-3.49	2.53	1.68
3	238.54	235.29	307.75	232.52	-3.63	-3.49	-3.56	-1.22
2	49.66	252.59	333.76	255.27	12.38	11.61	11.61	6.24
Ground	237.68	238.2	299.92	232.29	64.57	51.17	50.29	25.99

Load Combination 1.2D+1.0E$_x$+1.0L+0.2S with Pinned Base

Story	East/West Frames: Shear (kips)				North/South Frames: Shear (kips)			
	Frame C	Frame D	Frame F	Frame G	Frame 3	Frame 5	Frame 7	Frame 10
Roof	0	0	0	0	0	0	0	0
Penthouse	110.47	102.6	140.84	107.17	-1.87	-1.89	-1.87	-0.65
4	184.6	172.51	238.08	179.99	1.07	1.2	1.21	1.03
3	237.3	234.48	306.8	231.27	-1.54	-2.54	-2.73	-0.85
2	295.46	287.05	384.9	291.02	-36.09	-32.3	-31.75	-15.31
Ground	247.46	248.41	316.63	246.45	42.92	39.63	39.19	19.43

Load Combination 1.2D+1.0E$_y$+1.0L+0.2S with Fixed Base

Story	East/West Frames: Shear (kips)				North/South Frames: Shear (kips)			
	Frame C	Frame D	Frame F	Frame G	Frame 3	Frame 5	Frame 7	Frame 10
Roof	0	0	0	0	0	0	0	0
Penthouse	0.16	1.4	0.24	0.2	104.02	80.47	121.41	146.91
4	-0.71	1.16	-0.88	-0.7	147.38	170.45	227.83	235.17
3	2.89	4.24	2.07	1.48	194.07	209.49	289.36	298.61
2	-3.47	1.62	-1.99	-1.33	220.37	233.57	360.22	334.03
Ground	12.15	15.48	16.22	13.09	274.93	161.25	378.64	328.34

Load Combination 1.2D+1.0E$_y$+1.0L+0.2S with Pinned Base

Story	East/West Frames: Shear (kips)				North/South Frames: Shear (kips)			
	Frame C	Frame D	Frame F	Frame G	Frame 3	Frame 5	Frame 7	Frame 10
Roof	0	0	0	0	0	0	0	0
Penthouse	0.14	1.39	0.23	0.19	104.18	80.22	121.49	146.97
4	-0.57	1.23	-0.75	-0.6	147.65	169.85	227.86	235.03
3	2.22	3.79	1.36	0.93	194.46	209.23	290.43	299.78
2	-0.89	3.92	1.11	1.14	221.25	230.84	356.77	328.86
Ground	-0.88	-0.77	-2.16	-1.85	270.76	168.59	402.69	363.72

Serviceability Check

Wind Drift

Serviceability checks are implemented to prevent excessive sway as well as to prevent damage to nonstructural components. Story drifts were determined for the applicable case 1 wind loading in both the East/West and North South directions. The story drift values were computed from the ETABS model and checked against the allowable drift of H/400. Since the structure has a relatively low story height, the story drifts were expected to be fairly low. All story drift values were determined to be acceptable under the drift limitations.

Wind Drift: East/West Direction

Story	Story Height (ft)	Story Drift (in)		Allowable Story Drift Δ_{wind} (in) = H/400		Total Drift (in)		Allowable Total Drift Δ_{wind} (in) = H/400	
Roof	83.5	0.000701	<	0.66	Acceptable	0.005349	<	2.51	Acceptable
Penthouse	61.5	0.001017	<	0.45	Acceptable	0.004648	<	1.85	Acceptable
4	46.5	0.001232	<	0.45	Acceptable	0.003631	<	1.40	Acceptable
3	31.5	0.001449	<	0.495	Acceptable	0.002399	<	0.95	Acceptable
2	15	0.00095	<	0.45	Acceptable	0.00095	<	0.45	Acceptable

Wind Drift: North/South Direction

Story	Story Height (ft)	Story Drift (in)		Allowable Story Drift Δ_{wind} (in) = H/400		Total Drift (in)		Allowable Total Drift Δ_{wind} (in) = H/400	
Roof	83.5	0.000897	<	0.66	Acceptable	0.004424	<	2.51	Acceptable
Penthouse	61.5	0.001011	<	0.45	Acceptable	0.003527	<	1.85	Acceptable
4	46.5	0.00104	<	0.45	Acceptable	0.002516	<	1.40	Acceptable
3	31.5	0.000772	<	0.495	Acceptable	0.001476	<	0.95	Acceptable
2	15	0.000704	<	0.45	Acceptable	0.000704	<	0.45	Acceptable

Table 16 - Allowable Wind Drifts

Earthquake Drift

Story drifts were determined for the controlling earthquake loading in both the East/West and North/South directions. The drift limitation according to ASCE 7-10 for an occupancy category IV building is $0.01h_{sx}$. All story drift values were determined to be acceptable under the drift limitations.

Controlling Earthquake Drift: East/West Direction									
Story	**Story Height (ft)**	**Story Drift (in)**	**Allowable Story Drift $\Delta_{seismic}$ (in) = $0.01h_{sx}$**			**Total Drift (in)**	**Allowable Total Drift $\Delta_{seismic}$ (in) = $0.01h_{sx}$**		
Roof	83.5	0.003476	<	0.22	Acceptable	0.018362	<	0.835	Acceptable
Penthouse	61.5	0.003959	<	0.15	Acceptable	0.014886	<	0.615	Acceptable
4	46.5	0.004141	<	0.15	Acceptable	0.010927	<	0.465	Acceptable
3	31.5	0.004276	<	0.165	Acceptable	0.006786	<	0.315	Acceptable
2	15	0.00251	<	0.15	Acceptable	0.00251	<	0.15	Acceptable

Controlling Earthquake Drift: North/South Direction									
Story	**Story Height (ft)**	**Story Drift (in)**	**Allowable Story Drift $\Delta_{seismic}$ (in) = $0.01h_{sx}$**			**Total Drift (in)**	**Allowable Total Drift $\Delta_{seismic}$ (in) = $0.01h_{sx}$**		
Roof	83.5	0.001125	<	0.22	Acceptable	0.005089	<	0.835	Acceptable
Penthouse	61.5	0.001087	<	0.15	Acceptable	0.003964	<	0.615	Acceptable
4	46.5	0.001111	<	0.15	Acceptable	0.002877	<	0.465	Acceptable
3	31.5	0.001072	<	0.165	Acceptable	0.001766	<	0.315	Acceptable
2	15	0.000694	<	0.15	Acceptable	0.000694	<	0.15	Acceptable

Table 17 - Allowable Earthquake Drifts

Overturning and Foundation Impact

Overturning moment is caused by how the reactions at the base of the supports act in response to the loading condition. Proper design of the foundation is important to prevent overturning of the structure. In order to determine whether overturning moment will be a concern, it was necessary to determine the reactions at the supports. The uplift forces (FZ) were taken from ETABS for earthquake loading in the x and y directions. The columns rest on pile caps which consist of several micropiles. Each micropile is specified to have an allowable tension load of 170 kips. The capacity was determined based on the number of micropiles in the pile cap. A layout of the critical points considered for uplift can be found in Appendix E.

X-Direction Loading (E_x)				Y-Direction Loading (E_y)			
Point	**FZ (k)**	**Pile Cap**	**Capacity (k)**	**Point**	**FZ (k)**	**Pile Cap**	**Capacity (k)**
1	**-143.81**	**P5**	**850**	1	1.73		
2	19.74			2	-0.25		
3	-19.74			3	0.25		
4	143.81			4	-1.73		
5	**-135.8**	**P4**	**680**	5	-8.03		
6	125.9			**6**	**-819.39**	**P10**	**1700**
7	-25.11			7	-10.49		
8	143.58			8	0.08		
10	19.28			10	0.24		
12	**-140.46**	**P4**	**680**	12	-1.72		
15	-19.28			15	-0.24		
16	140.46			16	1.72		
25	**-141.46**	**P4**	**680**	25	-0.62		
26	18.1			26	0.08		
27	0			27	0		
28	-18.1			28	-0.08		
29	141.46			29	0.62		
30	-108.57			30	837.83		
32	105.8			32	602.8		
33	-105.8			**33**	**-602.8**	**P10**	**1700**
36	22.23			**36**	**-783.85**	**P8**	**1360**
37	-22.23			37	783.85		
39	-35.28			**39**	**-568.68**	**P8**	**1360**
40	35.28			40	568.68		
41	0			41	0		

Table 18 - Foundation Impact

Since the capacities of the micropiles at the critical points is greater than the reactions at those points, impact due to overturning forces can be neglected. It can be assumed that the foundation provided adequate resistance to the uplift forces of the structure.

Lateral Member Spot Checks

Member spot checks were performed for moment frame D due to ASCE load case 5 acting in the x-direction. Concentric braced frame 7 was checked due to ASCE load case 5 acting in the y-direction. Figure 15 shows which members were checked for strength requirements based on loading from the ETABS model. Combined axial and bending moments in the columns were checked using Table 6-1 in the AISC Steel Manual. The bending moment capacity was checked for the beam using Table 3-2. Axial compression was checked in bracing members using Table 4-1.

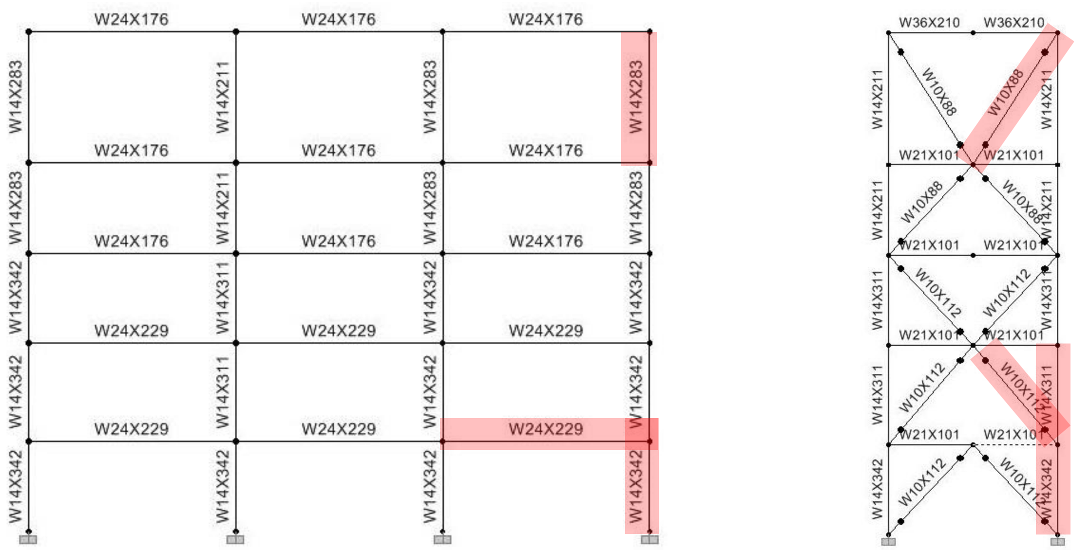


Figure 15 – Member Spot Checks (Courtesy of: Author)

It was determined that these members provide more than sufficient strength under the controlling load case. It is more probable that lateral member were designed to control under serviceability requirements. See Appendix F for all hand calculations pertaining to member spot checks.

Evaluations and Summary

The lateral forces determined in the first technical report were updated to include more accurate building dimensions and weights. An ETABS model was constructed to analyze the lateral force resisting system. Applicable loads were factored using the ASCE 7-10 load combinations for strength design. It was determined from the ETABS output that the load combination 1.2 (Dead) + 1.0 (Earthquake) + 1.0 (Live) + 0.2 (Snow) controlled in both the North/South and East/West directions. Since it the building location in a non-seismic region, it is more probable that the building was designed around the wind forces.

Story drifts were determined using ETABS and compared with the limits taken from ASCE 7-10. The drift limit due to seismic loading for an occupancy category IV building is $\Delta_{seismic} = 0.01h_{sx}$. Both story drift and total drift, as seen in Table 17, were determined to be well within the limitations. Similarly, the maximum allowable drift under wind loading was taken to be H/400. Table 16 shows the results under wind loading and were determined to meet the requirements for serviceability.

The impact on the foundation was checked to determine if overturning moment was a factor under the controlling load case. The reactions at the supports in the z-direction were determined and checked against the capacity of the foundation. Since each column is supported by a pile cap consisting of several micropiles. The allowable tensile capacity of each micropile is 170 kips as shown in the drawing specifications. It was determined that at all critical locations where tension forces were greatest that the foundation provided adequate strength. Therefore it can be assumed that no necessary changes would need to be made to the foundation.

Spot checks were performed for selected members of Frame D in the East/West direction and Frame 7 in the North/South Direction. Column members were checked for combined axial compression and bending. Beams in the moment frames were checked based on allowable bending moments and shear. Bracing members in the concentrically braced frames were checked for pure axial compression. Hand calculations can be found in Appendix F of this report. All members which were checked were determined to provide enough strength under the applicable load combinations. This report concludes that the lateral force resisting system was adequately designed according to ASCE 7-10.

APPENDIX

Appendix A: Wind Calculations

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

Gust Effect Factor		
	N-S	E-W
B (ft)	359.1	124.25
L (ft)	124.25	359.1
h (ft)	85.5	85.5
n_1	0.632	0.632
β (assumed 1%)	0.01	0.01
Structure (η_1 < 1 Hz)	Flexible	Flexible
g_Q	3.4	3.4
g_v	3.4	3.4
g_R	4.08	4.08
z	51.3	51.3
L_z	546.12	546.12
I_z	0.186	0.186
Q	0.802	0.862
V_z	122.43	122.43
N_1	2.82	2.82
R_n	0.073	0.073
η for R_h	2.03	2.03
R_h	0.373	0.373
η for R_B	8.53	2.95
R_B	0.11	0.28
η for R_L	9.88	28.55
R_L	0.096	0.034
R	0.415	0.646
G_f	0.898	1.001

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GENERAL REQUIREMENTS:

OCCUPANCY CATEGORY : IV
BASIC WIND SPEED (V) : 120 MPH
WIND DIRECTIONALITY FACTOR (K_d) : 0.85
EXPOSURE CATEGORY : C
TOPOGRAPHIC FACTOR (K_{zt}) : 1.0

GUST EFFECT FACTOR :

FOR N/S WIND DIRECTION : B = 359.1 FT, L = 124.25 FT, H = 85.5 FT
FOR E/W WIND DIRECTION : B = 124.25 FT, L = 359.1 FT, H = 85.5 FT

26.9.2.1 – LIMITATIONS FOR APPROXIMATE NATURAL FREQUENCY

① BUILDING HEIGHT = 85.5 FT < 300 FT ∴ OK
② BUILDING HEIGHT = 85.5 FT < $4L_{eff}$

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

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

(N/S) $L_{eff} = \dfrac{124.25\text{ FT}(15 + 31.5 + 46.5 + 61.5 + 85.5\text{ FT})}{(15 + 31.5 + 46.5 + 61.5 + 85.5\text{ FT})} = 124.25\text{ FT}$

85.5 FT < 4(124.25 FT) ∴ OK

(E/W) L_{eff} = 359.1 FT

85.5 FT < 4(359.1 FT) ∴ OK

n_a SHALL BE CALCULATED BY 26.9.3

26.9.3 — APPROXIMATE NATURAL FREQUENCY

FOR STRUCTURAL STEEL MOMENT-RESISTING FRAME BUILDINGS:

$n_a = 22.2/h^{0.8}$

$= 22.2/(85.5)^{0.8} = 0.632$ Hz < 1.0 Hz ∴ FLEXIBLE

FOR STRUCTURAL STEEL WITH OTHER LATERAL RESISTING SYSTEMS:

$n_a = 75/h$

$= 75/85.5 = 0.877$ Hz < 1.0 Hz ∴ FLEXIBLE

26.9.5 — FLEXIBLE BUILDINGS

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

(N/S) $g_Q = g_v = 3.4$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}}$$ WHERE $n_1 = 0.632$ Hz

$= 4.08$

MATT VANDERSALL | TECH REPORT #3 | WIND LOADING | 2/3

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

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)V$$

WHERE $\bar{z} = \max \begin{vmatrix} 0.6h = 0.6(85.5) = 51.3 \text{ FT} \\ z_{min} = 15 \text{ FT} \end{vmatrix}$

$\bar{b} = 0.65$, $\bar{\alpha} = 1/6.5$ — FROM TABLE 26.9-1

$$= 0.65\left(\frac{51.3}{33}\right)^{1/6.5}\left(\frac{88}{60}\right)120$$

$$= 122.4$$

$R_\ell = R_h$: $\eta = \dfrac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \dfrac{4.6(0.632)(85.5 \text{ FT})}{122.4} = 2.03 > 0$

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

$$\underline{= 0.373}$$

$R_\ell = R_B$: $\eta = \dfrac{4.6 n_1 B}{\bar{V}_{\bar{z}}} = \dfrac{4.6(0.632)(359.1)}{122.4} = 8.53 > 0$

$$R_B = \frac{1}{8.53} - \frac{1}{2(8.53)^2}\left(1 - e^{-2(8.53)}\right)$$

$$\underline{= 0.11}$$

$R_\ell = R_L$: $\eta = \dfrac{15.4 n_1 L}{\bar{V}_{\bar{z}}} = \dfrac{15.4(0.632)(124.25)}{122.4} = 9.88 > 0$

$$R_L = \frac{1}{9.88} - \frac{1}{2(9.88)^2}\left(1 - e^{-2(9.88)}\right)$$

$$\underline{= 0.096}$$

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

WHERE $L_{\bar{z}} = \ell\left(\dfrac{\bar{z}}{33}\right)^{\bar{\epsilon}}$ — FROM TABLE 26.9-1: $\ell = 500$ FT, $\bar{\epsilon} = 1/5$

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

$$L_{\bar{z}} = 500\left(\frac{51.3}{33}\right)^{1/5}$$

$$L_{\bar{z}} = 546.1$$

$$\underline{R_n = 0.073}$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{(0.632)(546.1)}{122.4} = 2.82$$

ASSUME $\beta = 0.01$ FOR STEEL BUILDINGS

$$R = \sqrt{\frac{1}{0.01}(0.073)(0.373)(0.11)(0.53 + 0.47(0.096))} = 0.415$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = 0.802$$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.2\left(\frac{33}{51.3}\right)^{1/6} = 0.186$$

$$G_f = 0.925\left(\frac{1 + 1.7(0.186)\sqrt{(3.4)^2(0.802)^2 + (4.08)^2(0.415)^2}}{1 + 1.7(3.4)(0.186)}\right)$$

$$G_f = 0.898$$

FOR E/W GUST FACTOR SEE SPREADSHEET

MATT VANDERSALL | TECH REPORT #3 | WIND LOADING | 3/3

ENCLOSURE CLASSIFICATION : ENCLOSED

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

VELOCITY PRESSURE EXPOSURE COEFFICIENT (K_z OR K_h) : SEE SPREADSHEET

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

EXTERNAL PRESSURE COEFFICIENT (C_p OR C_N)

(N/S)

SURFACE	L/B	C_P	USE WITH
WINDWARD	ALL	0.8	q_z
LEEWARD	0.35	−0.5	q_h
SIDE	ALL	−0.7	q_h

(E/W)

SURFACE	L/B	C_P	USE WITH
WINDWARD	ALL	0.8	q_z
LEEWARD	2.89	−0.26	q_h
SIDE	ALL	−0.7	q_h

WIND PRESSURE FOR ENCLOSED FLEXIBLE BUILDINGS:

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

FOR PARAPET : $P_p = q_p (GC_{pn})$ WHERE $GC_{pn} = \begin{matrix} +1.5 \\ -1.0 \end{matrix}$

$$q_p = 0.00256(1.23)(1.0)(0.85)(120)^2 = 38.54$$

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

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

DESIGN WIND PRESSURES:

(N/S) WINDWARD : $P = q_z(0.898)(0.8) - 38.54(\pm 0.18)$

$P = 0.718\, q_z + 6.94$

LEEWARD : $p = (38.54)(0.898)(-0.5) - (38.54)(\pm 0.18)$

$p = -24.24$ PSF

(E/W) WINDWARD : $p = q_z(1.001)(0.8) - 38.54(\pm 0.18)$

$p = 0.801\, q_z + 6.94$

LEEWARD : $p = (38.54)(1.001)(-0.26) - (38.54)(\pm 0.18)$

$p = -16.97$ PSF

Appendix B: Floor Weights

MATT VANDERSALL | TECH REPORT #3 | FLOOR WEIGHTS | 1/2

ASSUMPTIONS: FRAMING MEMBERS TAKEN FROM REVIT MODEL
TOTAL COLUMN WEIGHT FROM TECH 1 = 15.484 K/FT PER FLOOR
SLAB WEIGHT FROM VULCRAFT CATALOG = 69 PSF
MEP LOAD = 30 PSF
FACADE = 0.75 PSF
FLOOR AREA = 37296.6 FT^2 / PERIMETER = 937.9 FT

LEVEL 1 — AT GROUND LEVEL

FRAMING MEMBERS = 525.05 K

COLUMNS = (15.484 K/FT)(7.5 FT) = 116.13 K

SLAB WEIGHT = (69 PSF)(37296.6 FT^2)/1000 = 2573.5 K

MEP = (30 PSF)(37296.6 FT^2)/1000 = 1118.9 K

FACADE = (0.75 PSF)(937.9 FT)(7.5 FT)/1000 = 5.3 K

$$\text{WEIGHT PER FLOOR AREA} = \frac{4338.9\ K}{37296.6\ FT^2} = 0.1163\ KSF$$

$$\text{DIAPHRAGM MASS ASSIGNMENT} = \frac{0.1163\ KSF}{32.2\ FT/SEC^2}\ \frac{1}{(12\ IN)^3} = \underline{2.09E-6\ K \cdot IN}$$

LEVEL 2

FRAMING MEMBERS = 468.8 K

COLUMNS = (15.484 K/FT)(7.5 FT + 8.25 FT) = 243.9 K

SLAB WEIGHT = 2573.5 K

MEP = 1118.9 K

FACADE = (0.75 PSF)(937.9 FT)(7.5 FT + 8.25 FT)/1000 = 11.1 K

$$\text{WEIGHT PER FLOOR AREA} = \frac{4416.2\ K}{37296.6\ FT^2} = 0.1184\ KSF$$

$$\text{DIAPHRAGM MASS ASSIGNMENT} = \frac{0.1184\ KSF}{32.2\ FT/SEC^2}\ \frac{1}{(12\ IN)^3} = \underline{2.13E-6\ K \cdot IN}$$

LEVEL 3

FRAMING MEMBERS = 422.5 K

COLUMNS = 243.9 K

SLAB WEIGHT = 2573.5 K

MEP = 1118.9 K

FACADE = 11.1 K

$$\text{WEIGHT PER FLOOR AREA} = \frac{4369.9\ K}{37296.6\ FT^2} = 0.1172\ KSF$$

$$\text{DIAPHRAGM MASS ASSIGNMENT} = \frac{0.1172\ KSF}{32.2\ FT/SEC^2}\ \frac{1}{(12\ IN)^3} = \underline{2.11E-6\ K \cdot IN}$$

MATT VANDERSALL | TECH REPORT #3 | FLOOR WEIGHTS | 2/2

LEVEL 4

FRAMING MEMBERS = 342.5 K

COLUMNS = (15.484 K/FT)(15 FT) = 232.3 K

SLAB WEIGHT = 2573.5 K

MEP = 1118.9 K

FACADE = (0.75 PSF)(937.9 FT)(15 FT)/1000 = 10.6 K

WEIGHT PER FLOOR AREA = $\frac{4277.8\ k}{37296.6\ ft^2}$ = 0.1147 KSF

DIAPHRAGM MASS ASSIGNMENT: $\frac{0.1147\ ksf}{32.2\ ft/sec^2}\frac{1}{(12 in)^3}$ = 2.06E-6 k·in

PENTHOUSE LEVEL

FRAMING MEMBERS = 437.1 K

COLUMNS = (15.484 K/FT)(7.5 FT + 11 FT) = 286.5 K

SLAB WEIGHT = 2573.5 K

MEP = 1118.9 K

FACADE = (0.75 PSF)(937.9 FT)(7.5 FT + 11 FT)/1000 = 13 K

WEIGHT PER FLOOR AREA = $\frac{4429\ k}{37296.6\ ft^2}$ = 0.1188 KSF

DIAPHRAGM MASS ASSIGNMENT = $\frac{0.1188\ ksf}{32.2\ ft/sec^2}\frac{1}{(12 in)^3}$ = 2.14 E-6 k·in

PENTHOUSE ROOF

FRAMING MEMBERS = 386.4 K

COLUMNS = (15.484 K/FT)(11 FT) = 170.3 K

SLAB WEIGHT = 2573.3 K

MEP = 1118.9 K

FACADE = (0.75 PSF)(937.9 FT)(11 FT)/1000 = 7.7 K

WEIGHT PER FLOOR AREA = $\frac{4256.6\ k}{37296.6\ ft^2}$ = 0.1141 KSF

DIAPHRAGM MASS ASSIGNMENT = $\frac{0.1141\ ksf}{32.2\ ft/sec^2}\frac{1}{(12 in)^3}$ = 2.05 E-6 k·in

Appendix C: Seismic Calculations

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

SEISMIC USE GROUP : IV

SITE CLASS : D

SPECTRAL RESPONSE ACCEL. SHORT $(S_s) = 0.209g$ (USGS)

SPECTRAL RESPONSE ACCEL. LONG $(S_1) = 0.055g$ (USGS)

SITE COEFFICIENT $(F_a) = 1.6$

$(F_v) = 2.4$

SOIL MODIFIED ACCEL ; $S_{MS} = F_a S_s$

$= (1.6)(0.209) = 0.3344\,g$

SOIL MODIFIED ACCEL : $S_{M1} = F_v S_1$

$= (2.4)(0.055) = 0.132g$

DESIGN SPECTRAL RESPONSE, SHORT: $S_{DS} = 2/3\, S_{MS}$

$= 2/3(0.3344) = 0.223g$

DESIGN SPECTRAL RESPONSE, LONG : $S_{D1} = 2/3\, S_{M1}$

$= 2/3(0.132) = 0.088g$

RESPONSE MODIFICATION FACTOR (R) : 3

IMPORTANCE FACTOR (I_e) : 1.50

SEISMIC DESIGN CATEGORY (SDC) : C

[TABLES 11.6-1 + 2 : RISK CATEGORY IV

$0.167 \le S_{DS} \le 0.33 \longrightarrow C$

$0.067 \le S_{DS} \le 0.133 \rightarrow C$]

APPROX. FUNDAMENTAL PERIOD :

$T_a = C_T h_n^x$ WHERE $C_T = 0.02$, $x = 0.75$ FROM TABLE 12.8-2

$= 0.02(85.5\text{ FT})^{0.75}$

$= 0.562$

$C_u = 1.7$ [UPPER LIMIT ON CALCULATED PERIOD – 12.8-1]

$T = C_u T_a = (1.7)(0.562) = 0.955$

$T_b = 1.9135$ FROM FUNDAMENTAL MODE OF VIBRATION AFTER MODAL ANALYSIS

$$T_{min} = \begin{vmatrix} 0.955 \leftarrow \text{CONTROLS} \\ 1.9135 \end{vmatrix}$$

CHECK LONG-PERIOD TRANSITION PERIOD : $T_L = 6$ sec

SEISMIC RESPONSE COEFFICIENT :

$$C_s \begin{vmatrix} S_{DS}/\left(\frac{R}{I}\right) = 0.223/\left(\frac{3}{1.5}\right) = 0.112 \\ \text{FOR } T \le T_L : S_{D1}/\left[T \cdot \frac{R}{I}\right] = 0.088/\left[0.955 \cdot \frac{3}{1.5}\right] = 0.046 \leftarrow \text{CONTROLS} \end{vmatrix}$$

BASE SHEAR : $V_b = C_s W$

$= (0.046)(26088.4\text{ K})$

$\boxed{= 1200.1\text{ K}}$

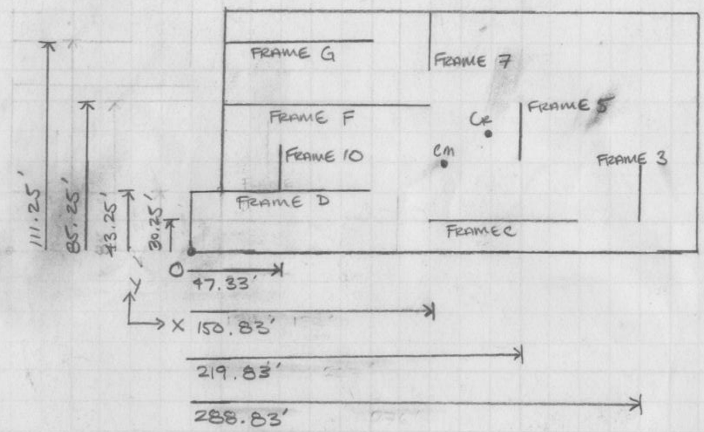
STRUCTURAL PERIOD EXPONENT : $0.5 \le T = 0.955 \le 2.5$

THROUGH INTERPOLATION – $k = 1.23$

Appendix D: Center of Rigidity

MATT VANDERSALL   TECH REPORT #3   CENTER OF RIGIDITY   1

FRAME LAYOUT:



CENTER OF RIGIDITY FOR FIXED BASE:

STIFFNESS OF FRAMES

FRAME C = 49.63 k/in
FRAME D = 48.57 k/in
FRAME F = 66.31 k/in
FRAME G = 50.66 k/in

FRAME 3 = 207.5 k/in
FRAME 5 = 186.9 k/in
FRAME 7 = 250.6 k/in
FRAME 10 = 221.7 k/in

$$X_R = \frac{\Sigma R_i x_i}{\Sigma R_i} = \frac{(221.7)(47.33)+(250.6)(150.83)+(186.9)(219.83)+(207.5)(288.83)}{221.7+250.6+186.9+207.5}$$

$$= 172.3 \text{ FT}$$

$$Y_R = \frac{\Sigma R_i y_i}{\Sigma R_i} = \frac{(49.63)(30.25)+(48.57)(43.25)+(66.31)(85.25)+(50.66)(111.25)}{49.63+48.57+66.31+50.66}$$

$$= 69.2 \text{ FT}$$

AVERAGE COR FROM ETABS: $X_R = 174.5$ FT ∴ OK
$Y_R = 69.2$ FT ∴ OK

CENTER OF RIGIDITY FOR PINNED BASE:

STIFFNESS OF FRAMES

FRAME C = 39.89 k/in
FRAME D = 38.55 k/in
FRAME F = 52.14 k/in
FRAME G = 39.89 k/in

FRAME 3 = 206.61 k/in
FRAME 5 = 186.22 k/in
FRAME 7 = 250.63 k/in
FRAME 10 = 221.24 k/in

$$X_R = \frac{\Sigma R_i x_i}{\Sigma R_i} = \frac{(221.24)(47.33)+(250.63)(150.83)+(186.22)(219.83)+(206.61)(288.83)}{221.24+250.63+186.22+206.61}$$

$$= 172.18 \text{ FT}$$

$$Y_R = \frac{\Sigma R_i y_i}{\Sigma R_i} = \frac{(39.89)(30.25)+(38.55)(43.25)+(52.14)(85.25)+(39.89)(111.25)}{39.89+38.55+52.14+39.89}$$

$$= 68.97 \text{ FT}$$

AVERAGE COR FROM ETABS: $X_R = 175.6$ FT ∴ OK
$Y_R = 69.1$ FT ∴ OK

MATT VANDERSALL	TECH REPORT # 3	CENTER OF RIGIDITY	2

CENTER OF MASS TAKEN FROM ETABS: $X_m = 160.56$ FT
$Y_m = 58.84$ FT

ECCENTRICITY FOR FIXED BASE:

$e_x = X_R - X_m = 172.3 - 160.56 = 11.74$ FT

$e_y = Y_R - Y_m = 69.2 - 58.84 = 10.36$ FT

ECCENTRICITY FOR PINNED BASE:

$e_x = 172.18 - 160.56 = 11.62$ FT

$e_y = 68.97 - 58.84 = 10.13$ FT

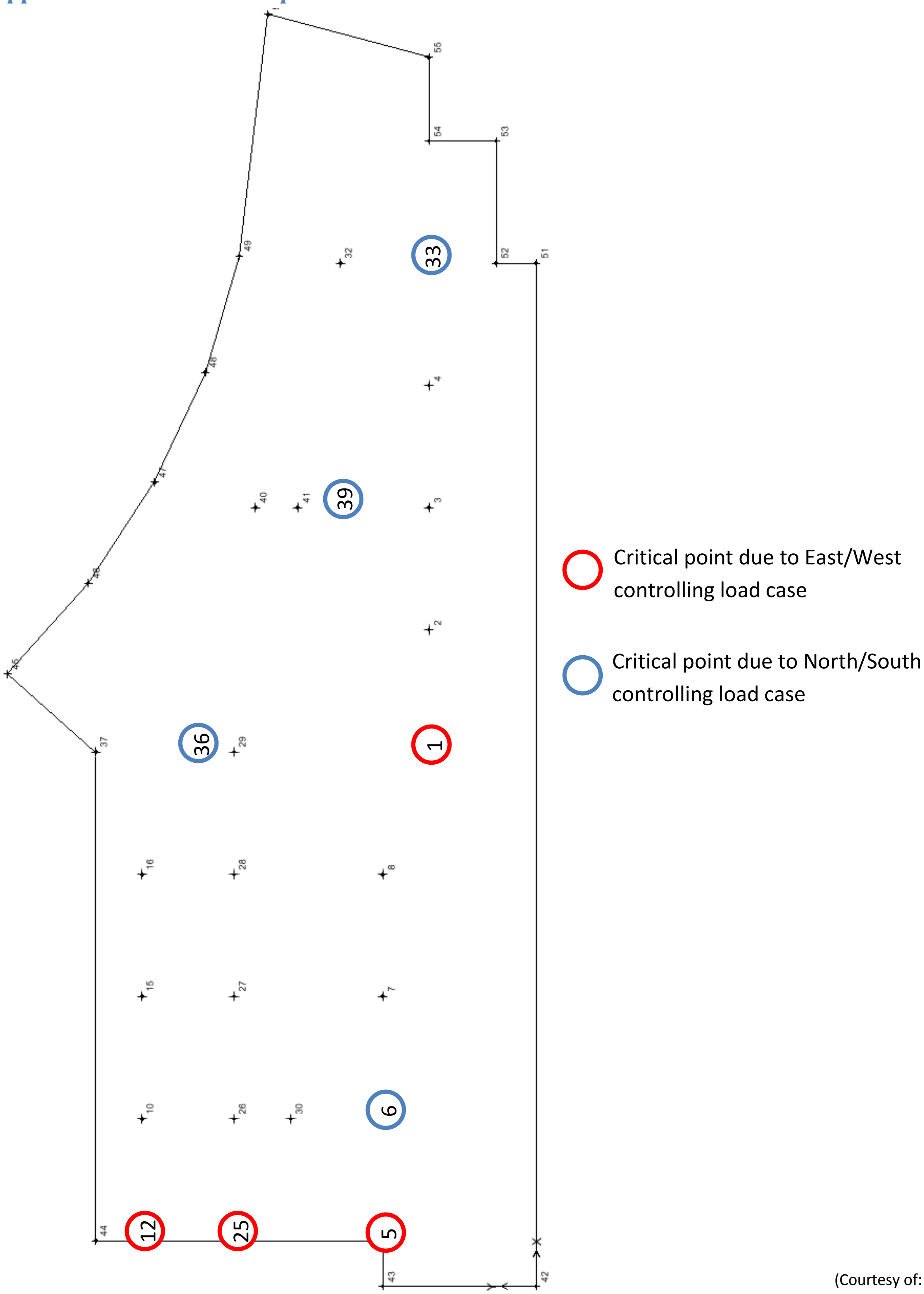
TORSIONAL MOMENT OF INERTIA:

$J = \Sigma R_i d_i^2$ FOR CALCULATIONS SEE TABLE

FOR FIXED BASE: $J = 7032494.91 \frac{k \cdot FT^2}{IN}$

FOR PINNED BASE: $J = 6967485.14 \frac{k \cdot FT^2}{IN}$

Appendix E: Foundation Impact



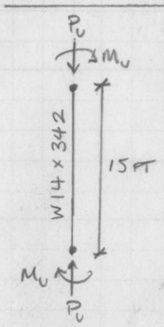
(Courtesy of: Author)

Appendix F: Member Spot Checks

MATT VANDERSALL	TECH REPORT #3	MEMBER CHECKS	1/2

LOADING IN X-DIRECTION

COLUMN MEMBER AT FRAME D LEVEL 1



$P_u = 143.6$ KIPS
$M_{u_x} = 559.3$ K·FT

FROM TABLE 6-1: COMBINED AXIAL AND BENDING

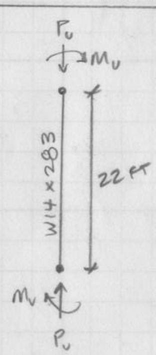
$KL = (15\text{ FT})\frac{r_x}{r_y} = (15\text{ FT})(1.65) = 24.75\text{ FT}$

$p \times 10^{-3} = 0.315$

$b_x \times 10^{-3} = 0.365$

$\frac{143.6(0.315)}{1000} + \frac{559.3(0.365)}{1000} = 0.25 < 1.0 \therefore$ OK

COLUMN MEMBER AT FRAME D PENTHOUSE LEVEL



$P_u = 14.21$ KIPS
$M_{ux} = 223.88$ K·FT

FROM TABLE 6-1: COMBINED AXIAL AND BENDING

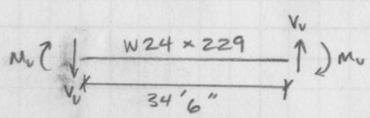
$KL = (22\text{ FT})\frac{r_x}{r_y} = (22\text{ FT})(1.63) = 35.86\text{ FT}$

$p \times 10^{-3} = 0.582$

$b_x \times 10^{-3} = 0.479$

$\frac{14.21(0.582)}{1000} + \frac{223.88(0.479)}{1000} = 0.12 < 1.0 \therefore$ OK

BEAM MEMBER AT FRAME D AT LEVEL 3



$V_u = 39.12$ KIPS
$M_u = 710.88$ K·FT

$M_n = M_p - BF(L_b - L_p)$

FROM TABLE 3-2: $M_p = 2530$ K·FT
$BF = 28.9$ K
$L_b = 34'6"$
$L_p = 11'$

$M_n = 2530 - 28.9(34.5' - 11')$

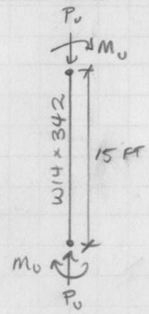
$M_n = 1850.85$ K·FT > 710.88 K·FT $\therefore$ OK

$V_n = 749$ K > 39.12 K $\therefore$ OK

MATT VANDERSALL | TECH REPORT #3 | MEMBER CHECKS | 2/2

LOADING IN Y-DIRECTION

COLUMN MEMBER AT FRAME 7 AT LEVEL 1



$P_u = 586.32$ k
$M_u = 52.08$ k·FT

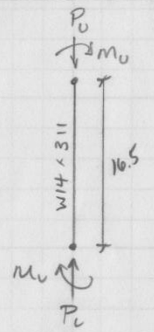
FROM TABLE 6-1: COMBINED AXIAL AND BENDING

$KL = (15\text{ FT})\frac{r_x}{r_y} = (15\text{ FT})(1.65) = 24.75$ FT

$p \times 10^{-3} = 0.315$
$b_x \times 10^{-3} = 0.365$

$\frac{586.32(0.315)}{1000} + \frac{52.08(0.365)}{1000} = 0.20 < 1.0$ ∴ OK

COLUMN MEMBER AT FRAME 7 AT LEVEL 2



$P_u = 373.28$ k
$M_u = 16.74$ k·FT

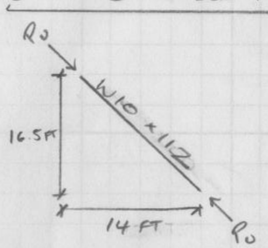
FROM TABLE 6-1: COMBINED AXIAL AND BENDING

$KL = (16.5\text{ FT})\frac{r_x}{r_y} = (16.5)(1.64) = 27.06$ FT

$p \times 10^{-3} = 0.377$
$b_x \times 10^{-3} = 0.412$

$\frac{373.28(0.377)}{1000} + \frac{16.74(0.412)}{1000} = 0.15 < 1.0$ ∴ OK

BRACING MEMBER AT FRAME 7 AT LEVEL 2

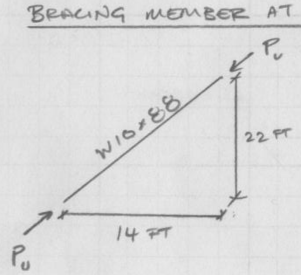


$P_u = 275.73$ k
$L_b = 21.64$ FT

FROM TABLE 4-1: $\phi P_n = 746.28$ k > 275.73 k ∴ OK

$\frac{P_u}{\phi P_n} = \frac{275.73}{746.28} = 0.37 < 1.0$ ∴ OK

BRACING MEMBER AT FRAME 7 AT PENTHOUSE LEVEL



$P_u = 110.84$ k
$L_b = 26.08$ FT

FROM TABLE 4-1: $\phi P_n = 413.68$ k > 110.84 k ∴ OK

$\frac{P_u}{\phi P_n} = \frac{110.84}{413.68} = 0.27 < 1.0$ ∴ OK