Technical Report III

Nemours Children's Hospital as a part of The Nemours Foundation

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Structural Option

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11.16.11

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Executive Summary:

The objective of Technical Report III is to analyze how lateral loads are distributed to each of the shear walls in Nemours Children's Hospital as a part of The Nemours Foundation, NCHTNF. The results of these analyses will be overviewed later in this summary. This report begins with studying the existing conditions and the prevailing codes to understand the design decisions.

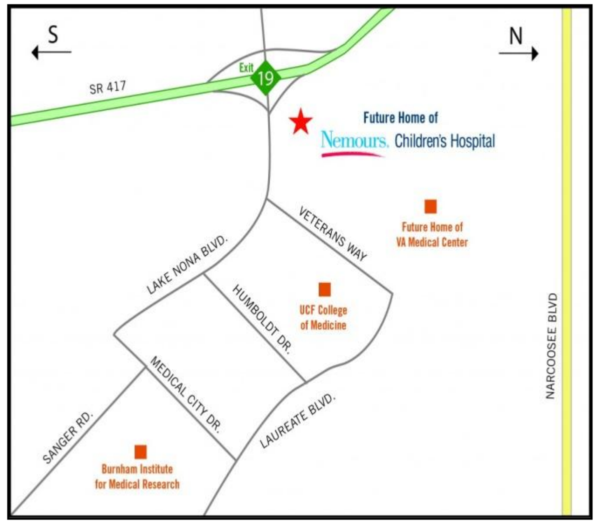
NCHTNF is a 7-story building located in Orlando, Florida. The entire complex consists of a hospital, clinic, loading dock data center, central energy plant (CEP), and parking facility. The 600,000 square foot hospital consists of two components: a bed tower and outpatient center. The combined components will provide 85 beds, emergency department, diagnostics and ambulatory programs, educational and research centers, and an outpatient clinic. Stanly Beaman & Sears and Perkins + Will are the architects of the project. Harris Civil Engineers, Simpson Gumpertz & Heger, AECOM, and TLC Engineering for Architecture are responsible for the engineering design of NCHTNF. Skanska USA Building is acting as the construction manager and general contractor of the design-bid-build project, which is scheduled to be completed July 2012 after ground was broken July 2009.

Gravity loads from ASCE 7-05 are used to determine the wind and seismic loads for NCHTNF. The building's geometry is regularized, so proper analysis of these loads can be completed as outlined in ASCE 7-05. NCHTNF is analyzed and modeled as two separate structures because of an expansion joint running through the building. The two structures will be called hospital and clinic. After analyzing the data, the conclusion is wind controls the design of NCHTNF.

NCHTNF is constructed with 39 shear walls to resist the lateral loading on the building. The lateral loads are transferred from the floor slabs into the shear walls. These loads are then transferred from the shear walls to the foundation. The relative stiffness of each lateral force resisting member is calculated by determining the fraction of the load the member takes from the total applied force.

Different load cases are tested to determine the largest load the NCHTNF could experience. Multiple load cases govern separate situations, which is explained later in the report. ETABS is used to analyze these various cases and determine the displacements, story drifts, center of mass, center of rigidity, and story shears. Further analysis of this data is able to provide torsion, overturning moment, and displacement checks on the building. A typical shear wall is checked to verify the ETABS model, and is found to support the ETABS outputs. After reviewing ETABS' results and this reports' analysis, it is determined that the NCHTNF meets the ASCE 7-05 code requirements.

Building Introduction:



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The design of this $400 million building uses 2007 Florida Building Code with 2009 updates. The Florida Building Code is based on the International Building Code and subsidiary related codes. NCHTNF pays close attention to the standards concerning the high-velocity hurricane zones due to Orlando's location. The building is classified as I-2 because the clinic can be considered business class, but the hospital is industrial because of overnight patients, thus making the entire project industrial. The site is an undeveloped parcel of land that underwent clearing and mass grading to reach its current topography. The site location does not have any restrictions presiding over the NCHTNF's design. The primary structure is concrete with curtain walls dominating the majority of the façade. The glass curtain walls vary between metal sunscreen systems, fritt patterns, and insulated spandrels. Other building materials include ribbed metal panel system, terracotta tile wall system, terrazzo wall panels, and composite metal panels to complement the glass systems in the curtain walls. A curved curtain wall, deep canopies, and two green roof gardens provide additional architectural features to the building design.

NCHTNF is designed to withstand the effects of a category 3 hurricane. The National Oceanic and Atmospheric Administration, NOAA, describes a category 3 hurricane as an event where devastating damage will occur, resulting in injury and death. The Nemours Foundation wants NCHTNF to be listed as a place of refuge, more technically known as an Enhanced Hurricane Protection Area, during a category 3 hurricane. This requires the building's design to at least meet NOAA's classification of a category 3 hurricane, having sustained winds of 111-130 mph. To qualify as an Enhanced Hurricane Protection Area, the hospital is designed to these standards with a factor of safety.

This results in a very extensive design for the building envelope. The modular curtain wall, constructed by Trainor, is designed with 30,000 feet of dual sealant joints to allow weeping between the two joints. A probe test is specified to be conducted after the sealant has cured to ensure the sealant joint is working properly. The north side of the building features a curved curtain wall supported by slanted structural columns. The deep canopies and fritt pattern glass, acting as sunshading devices, are prevalent throughout the building, and provide adequate shading from the Florida sun. NCHTNF incorporates several different roofing systems to accommodate different functions of the roof. A fluid-applied membrane acts as the roofing system for the roof gardens that are accessible to patients. Thermoplastic membrane roofing and SBS-modified bituminous membrane roofing comprise the other roofs on the building. A mock-up of the NCHTNF has been tested in a hurricane testing lab in Florida. A 2-story 10-bay mock-up was required to pass various tests to ensure the building envelope will be able to sustain the effects of a category 3 hurricane. Laminated glass and extensive use of roof fasteners are only a few of the reasons why the building envelope meets the standards of the hurricane test.

The design of NCHTNF follows the USGBC's LEED prerequisites and credits needed for certification based on LEED for New Construction 2.2. The building has two green roof gardens on the second and fourth floor roofs as mentioned in the paragraph above. The green roofs double as outdoor gardens for patients as well as sustainability features for the building. NCHTNF has numerous sunshades to block the sun from the vast glass façades. Deep canopies provide shade for large spaces on the south façade of the building. Fritt pattern and insulated spandrel glass systems are also implemented in the building's design. These devices block some of the intense Florida sun to lessen the load on the HVAC system of the building.

Structural Overview:

NCHTNF bears on spread footings on either improved or natural soils. The hospital and clinic portion of the building are predominately concrete structures with the exception of steel framed mechanical penthouses. The loading dock data center and central energy plant are primarily steel framed structures. The lateral system is comprised of shear walls, which most continue through the entire building height. NCHTNF utilizes unique framing techniques for the wave and sloped curtain wall backup.

Foundation:

PSI, the geotechnical firm, performed nineteen borings across the site in January 2009. The soils generally consist of varying types of fine sands graded relatively clean to slightly silty in composition. The boring blow counts record the upper layers of sand to be of medium dense condition, while the lower layers of sand are generally loose to medium dense condition.

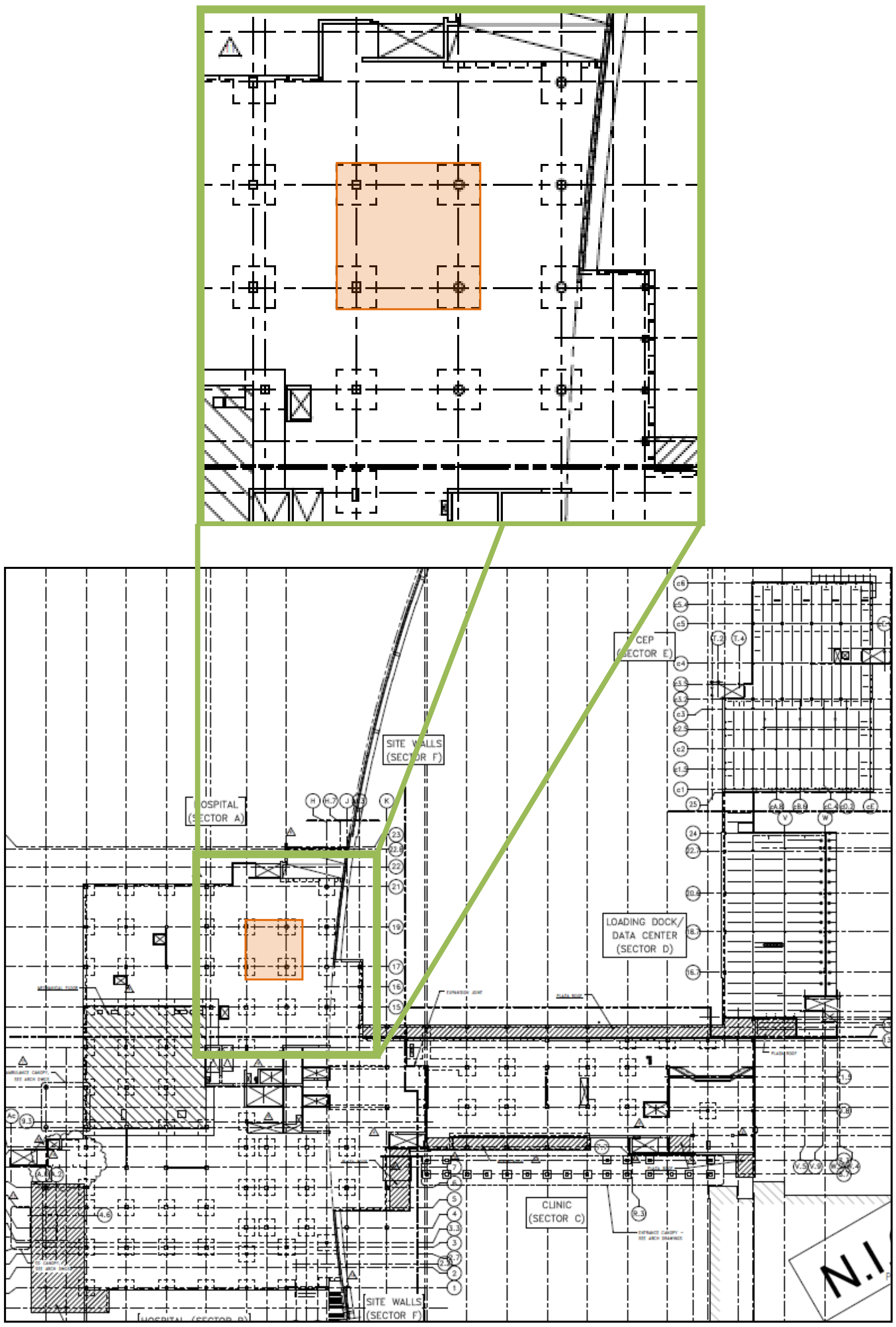
PSI recommends utilizing shallow foundations only if the foundation design implements soil improvement to increase the allowable bearing capacity of the design. PSI proposes another foundation solution, if soil improvement is not desirable implement a pile foundation system. These reinforced augercast piles will withstand a considerably higher foundation loads than the shallow foundation system. The downside of augercast piles are they can bulge or neck where very loose soils are encountered, requiring stringent monitoring and quality control. Due to the specialized nature of the augercast piles for this project, spread footings with soil improvement is chosen as the foundation system for the NCHTNF.

The fact that the water table is measured only 4 feet below the surface raises concerns about excavations. The sump system dewaters shallow excavations while deeper excavations require well-pointing or horizontal sock drains for proper dewatering.

Floor System:

NCHTNF has numerous types of floor construction due to different design requirements in different sections of the building. The building contains 5"-6" normal weight concrete as the slab on grade. A few sections of the foundation system utilize mat foundations, varying from 2' to 4'-3" normal weight concrete. The hospital and clinic are built on normal weight elevated two-way flat slabs, with and without drop panels, varying in depth from 9"-14". A typical structural floor plan detailing a typical 30'x30' bay is shown in Figures 1 and 2. The loading dock data center and central energy plant are constructed with a 4-1/2" 1-way slab on 3"-20 GA. composite metal deck, which is supported by a steel frame system. Some specialty areas, such as the green roof and the slab over the lecture hall, vary slightly from the typical slab in the remainder of the building.

There are 29 different superstructure concrete beams in the NCHTNF. The beams range from 16" x20" to 89" x 48". The hospital and clinic predominately consist of 15' x 30' bays with a few 15' x 15' and 30' x 30' bays to accommodate for the elevator and stair core. The bays in the loading dock data center are far irregular. They vary from the smallest being 21' x 30'-3" to the largest being 30' x 45' – 2". The central energy plant also has a variety of bay sizes, ranging from 22' x 11'-2" to 22' x 26'-7".



Figures 1 & 2 – Level 1 Typical Structural Bay (30'x30') with Key Plan. Courtesy SGH.

Framing System:

The columns supporting the NCHTNF are mostly concrete columns, with steel columns supporting the mechanical penthouses on the 7th floor. The concrete columns supporting the hospital and clinic typically start at a dimension of 30" x 30" and taper to 22" x 22" at Level 6. The mechanical penthouse is constructed with W12x53 columns on both the hospital and clinic. W14x109, W10x49, W10x60, and W14x68 mainly support the loading dock data center. HSS8x8x and HSS12x8 dominate the central energy plant's supporting structure along with a few W12x65 and W12x79 columns.

Lateral System:

Shear walls resist lateral loads in the hospital and clinic of the NCHTNF. These walls are 12-14" thick and tie into mat foundations with dowels matching the typical wall reinforcement, mostly #8 bars. The shear walls are located in the elevator/stair core in the hospital and in the elevator bays and lecture hall in the clinic, which are highlighted below in green in Figure 3. Also, the central energy plant has one shear wall, the rest of the lateral system of the CEP being braced framing which is discussed in the next paragraph. A few shear walls include knockout panels to plan for future openings.

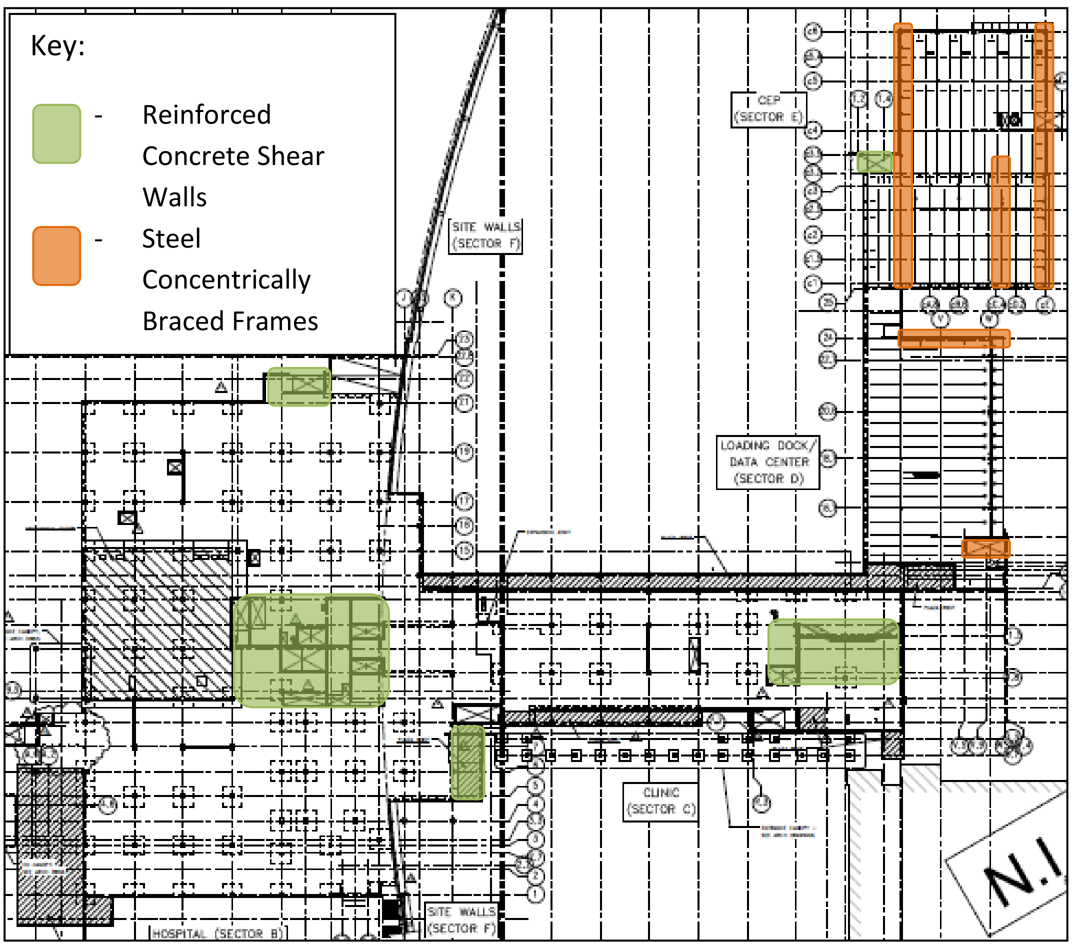


Figure 3 – Level 1 Structural Floor Plan Highlighting the Lateral System. Courtesy SGH.

Steel concentrically braced frames resist lateral loads in the loading dock data center and central energy plant, highlighted above in orange in Figure 3. Diagonal members, HSS6x6 and HSS5x5, brace into W14, W16, and W21 beams in the loading dock data center. Diagonal members, HSS8x8 and HSS8x8, brace into W18 and W21 beams respectively in the central energy plant. As mentioned above, the central energy plant has one shear wall along with the steel concentrically braced frame system.

The load path in NCHTNF starts with the wind load against the façade of the building. Once the load is applied to the façade it is transferred to the diaphragms on each floor. The diaphragms then transfer the load to the lateral elements, being reinforced concrete shear walls in the hospital and clinic and steel concentrically braced frames in the loading dock data center and CEP. These lateral elements transfer the load to the foundation system, the final step of the load path of NCHTNF.

Roof System:

NCHTNF has several different roofing systems to accommodate different functions of the roof. A fluid-applied membrane acts as the roofing system for the roof garden that is accessible to patients and also doubles as a green roof. The fluid-applied membrane utilizes type IV extruded polystyrene board insulation. The other roofs on the building are constructed with thermoplastic membrane roofing and SBS-modified bituminous membrane roofing. Each of these roofs use polyisocyanurate board insulation, which is type II glass fiber mat facer. The other roofing system is 1-1/2" – 18 GA. metal roof deck, located on the loading deck data center, central energy plant, and mechanical penthouses on the 7th floor.

Design Codes:

NCHTNF is designed in compliance with:

Design Codes	
Code	*Description*
Florida Building Code 2007*	With 2009 Updates
Florida Statutes 471 & 553	Main Hospital/Clinic, CEP, & Loading Dock Data Center are all considered "Threshold Buildings"**
ASCE/SEI 7-05	Minimum Design Loads for Buildings and Other Structures
DOE-STD-1020-2002	Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities***
AISC 360-05	Specifications for Structural Steel Buildings
AISC	Code of Standard Practice
AWS D1.1	Structural Welding Code – Steel
ACI	301 – Specification for Structural Concrete 302 – Concrete Floor and Slab Construction 318 – General Design of Reinforced Concrete Not Otherwise Specified

Table 1 – Design Codes

**Note: The 2007 Florida Building Code is based on the International Building Code and subsidiary related codes.*
***Note: "Threshold Buildings" is defined as any building which is greater than 3 stories or 50 feet in height or which has an assembly classification that exceeds 5,000 square feet in area and an occupant content of 500 people or greater.*
****Note: This code is only applicable for the CEP.*

Materials Used:

Table 2 lists the structural materials of NCHTNF as specified in the General Notes (0S1):

Material Properties		
Material		*Strength*
Steel	Grade	fy = ksi
Wide Flange Shapes	A992	50
Hollow Structural Shapes	A500, GR. B	45
Plates	A36	36
Angles	A36	36
Reinforcing Steel	A615	60
Welded Wire Reinforcement	A497	N/A
Welding Electrodes	E70XX	70
Concrete	Weight (pcf)	f'c = psi
Footings/Mat Foundation	145	4,000
Foundation Piers	145	4,000
Foundation Walls ≤ 5' Tall	145	4,000
Foundation Walls > 5' Tall	145	5,000
Slab-On-Grade	145	4,000
Elevated Slabs	145	5,000
Columns	145	6,000
Shear Walls	145	5,000
Beams	145	5,000
Concrete On Metal Deck	145	4,000
Masonry	Grade	Strength = ksi
Concrete Masonry Units	C90	f_y = 2.8
Mortar	C270, Type S	f'_m = 1.8

Table 2 – Material Properties

Building Loads:

Dead Loads:

The general notes in the front end of the structural list the superimposed dead loads. The dead loads are determined using the weights of the components or systems, which the IBC 2009 section 1606.2 states as the proper way to determine dead loads.

Superimposed Dead Loads		
Plan Areas		*Loads (psf)*
	Typical Floors	12
	Mechanical Floors	62
	Light Green Roofs	54
	Medium Green Roofs	209
	Heavy Green Roofs	389
	Typical Roof	24
Special Roofs	Plaza Roof (at grade)	50
Special Roofs	Café Portal Roof	45
Special Roofs	Entry Portal	45
Special Roofs	Ed Low Roof	45
Special Roofs	Clinic Roof Wing	189
Special Roofs	Stitch Roof	20

Table 3 – Superimposed Dead Loads

Live Loads:

The live loads are determined closely following the standard live loads in the IBC 2009 Table 1607.1. The values are listed next to the design values listed below. The mechanical floor allowance is a little high, but the mechanical system for NCHTNF is quite extensive. Also, the design of the building incorporates areas for future expansion for which additional mechanical equipment will be necessary for to control the additional space. These two factors may explain why the live load is above average. The drawings also states live load reduction is taken when code permits.

Live Loads			
	Plan Areas	*Loads (psf) - Design*	*Loads (psf) - IBC*
Hospital/Clinic	Patient Rooms	40	40
	Operating Rooms	60	60
	Corridors, at or below ground floor	100	100
	Corridors, above ground floor	80	80
	Mechanical Floor	150	N/A
	Stairs and Exits	100	100
	Storage – Light	125	125
	Partition Allowance	15	N/A
	Roof Load	20	20
	Light Green Roof	100*	100
	Medium Green Roof	100*	100
	Heavy Green Roof	100*	100
Special Roofs	Plaza Roof	100	100
	Café Portal Roof	20	20
	Entry Portal	20	20
	Ed Low Roof	20	20
	Clinic Roof Wing	20	20
	Stitch Roof	20	20

Table 4 – Live Loads

**Note: These loads are accounting for accessibility to the public.*

Snow Load:

ASCE 7-05 states a snow load is not required for Orlando, Florida.

Rain Load:

ASCE7-05 states "roofs with a slope less than 1/4 in./ft. shall be investigated…" The roof slope on NCHTNF is greater than 1/4 in. so no analysis is required.

Wind Load:

The wind analysis follows chapter 6 in ASCE 7-05 to determine the wind load on NCHTNF. All hand calculations and expanded excel spreadsheets are found in Appendix A. The Design Criteria, as stated in Appendix A, match the criteria on the general notes of the structural drawings. An explanation of design assumptions are as follows:

The building is assumed flexible because the fundamental frequency is below the 1 Hz requirement. Thus, the gust factor is not 0.85, but instead calculated using the equation for the gust factor of a flexible building, outlined in Appendix A. When calculating the gust factor, the damping ratio of the building is assumed to be 1.0. Also, the basic wind speed is not 110 mph as stated in ASCE 7-05, instead V=157mph. The owner wants the building to withstand a category three hurricane, so it is classified as a center of refuge in the event that a category 3 hurricane approaches Orlando, Florida. The building is assumed enclosed because NCHTNF has non-operable windows.

The building geometry is simplified so the height of the building is assumed at 135 ft, the height of the mechanical penthouse. The mechanical penthouse encompasses most of the surface area of the building, confirming my assumption that the building height can be averaged to 135 ft. The building is modeled as two separate structures, the hospital and clinic, divided along the expansion joint shown in Figure 4 below. Two separate wind analyses are calculated for each structure in Appendix A. The calculated values differ from Simpson, Gumpertz & Heger's calculations because their calculations are based on method 3, wind tunnel analysis.

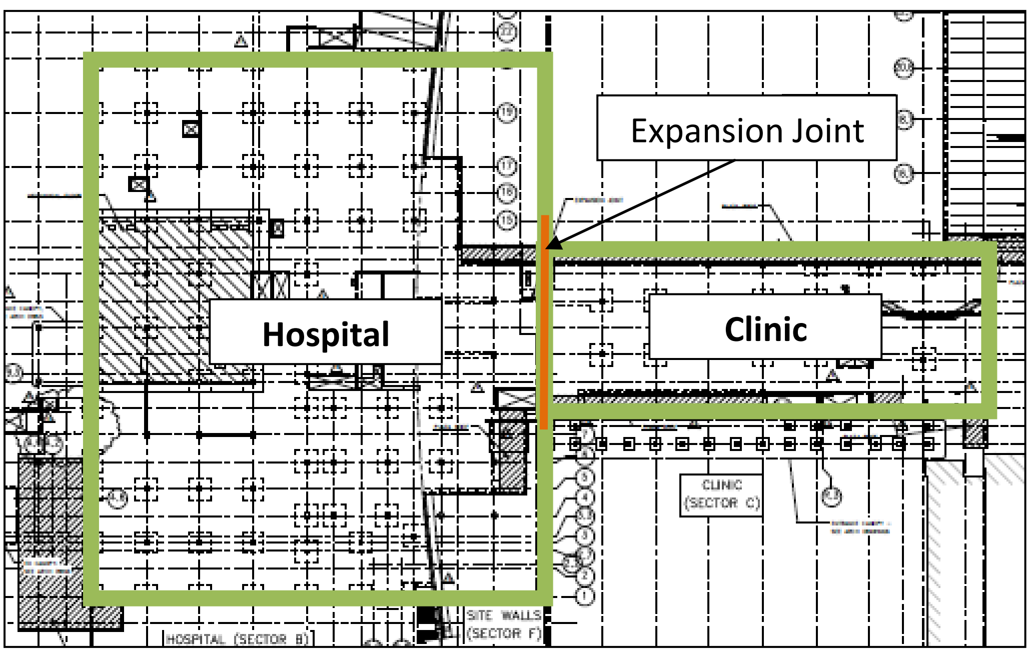


Figure 4 – Generalized Geometry for Wind Analysis. Courtesy SGH.

The resulting building shear and overturning moment are calculated in the excel spreadsheet, as listed in Appendix A. The applied wind pressures are shown in the North-South and East-West directions in Figures 5 & 6 below.

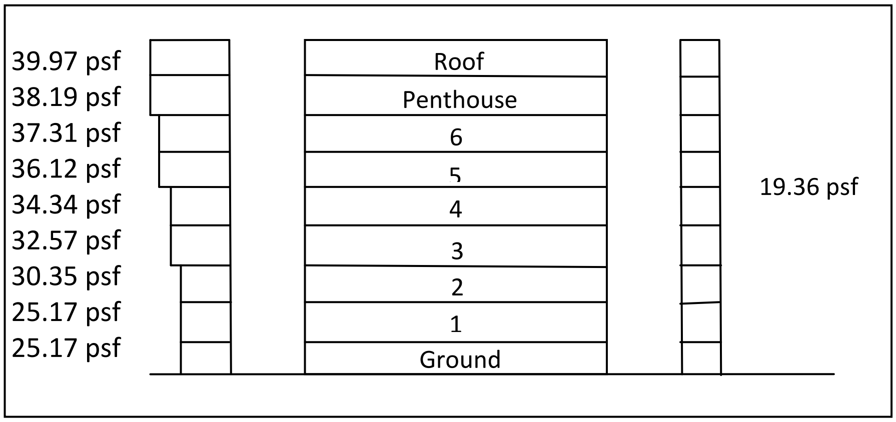


Figure 5 – Wind Pressures Vertical Distribution, North-South Direction

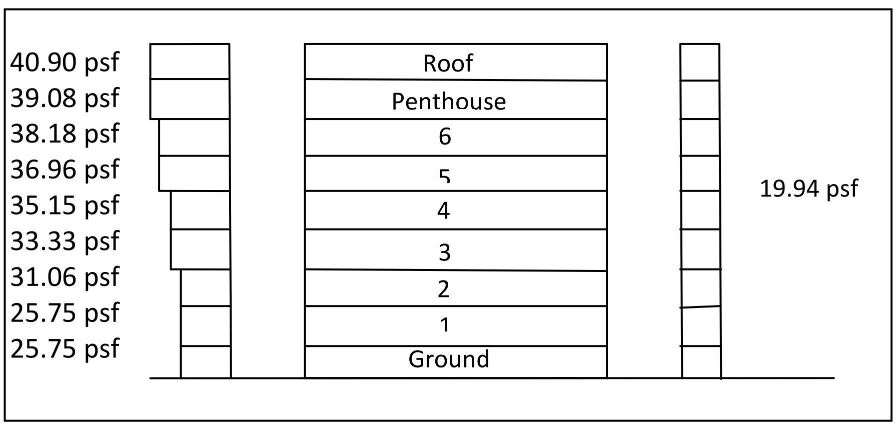


Figure 6 – Wind Pressures Vertical Distribution, East-West Direction

Seismic Load:

The seismic analysis follows chapters 11 and 12 in ASCE 7-05 to determine the seismic load on Nemours Children's Hospital as a part of The Nemours Foundation. The geotechnical report determines the site as site class D, firm soil. Seeing as the building is mostly concrete, the weight of the building is calculated with 145pcf normal weight concrete at 12". Also, typical and specialty roof systems are calculated using the same method, by determining their area and given loading. Of course some errors arise due to this estimate of building weight, but the approximation is within reason.

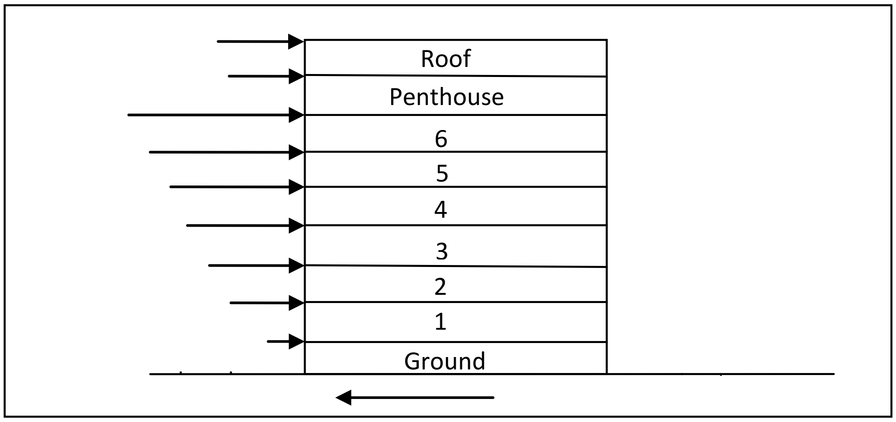
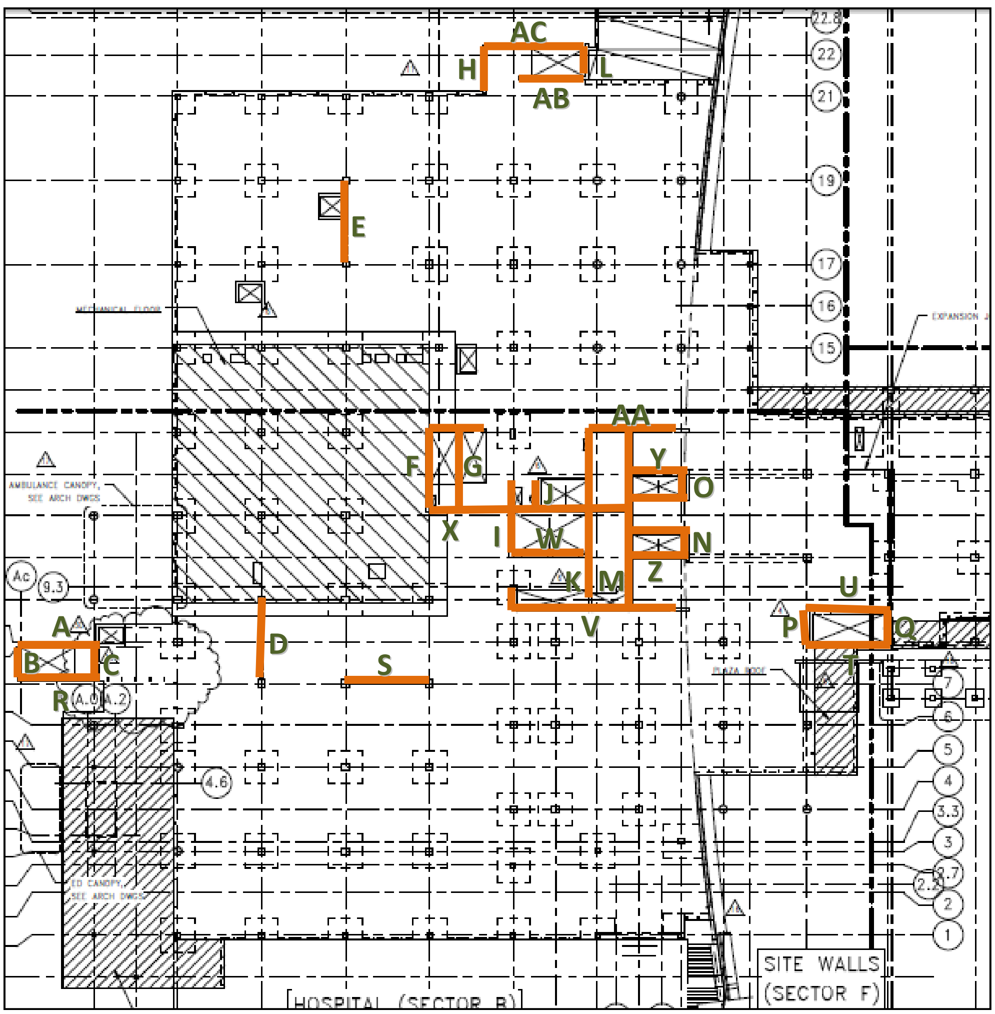


Figure 7 – Seismic Story Forces

The seismic calculations are found in Appendix B. The excel table calculating the resulting base shear is shown above in Figure 7 with the diagram showing the seismic forces acting on the building.

Lateral Load Distribution:

Lateral loads are resisted by 39 shear walls in NCHTNF. The shear walls are shown in Figures 8 & 9, highlighted in orange. The floor plan below also provides a key for the ETABS calculations that label each shear wall numerically. The ETABS model for this report analyzes the building using a rigid diaphragm. The diaphragm transfers the lateral loads to the shear walls, where these walls transfer the lateral load to the foundations. The relative stiffness of each shear wall is subsequently calculated, relating the amount of force seen at that member compared to the total force applied to the floor level, as shown in Appendix C. Due to shear wall height irregularity, the relative stiffness is calculated at each level, so the controlling relative stiffness can be more accurately determined.



KEY:

SW1 = A
SW2 = B
SW3 = C
SW4 = D
SW 5 = E
SW 6 = F
SW 7 = G
SW 8 = H
SW 9 = I
SW 10 = J
SW 11 = K
SW 12 = L
SW 13 = M
SW 14 = N
SW 15 = O
SW 16 = P
SW 17 = Q
SW 18 = R
SW 19 = S
SW 20 = T
SW 21 = U
SW 22 = V
SW 23 = W
SW 24 = X
SW 25 = Y
SW 26 = Z
SW 27 = AA
SW 28 = AB
SW 29 = AC

Figure 8 – Hospital Shear Wall Plan

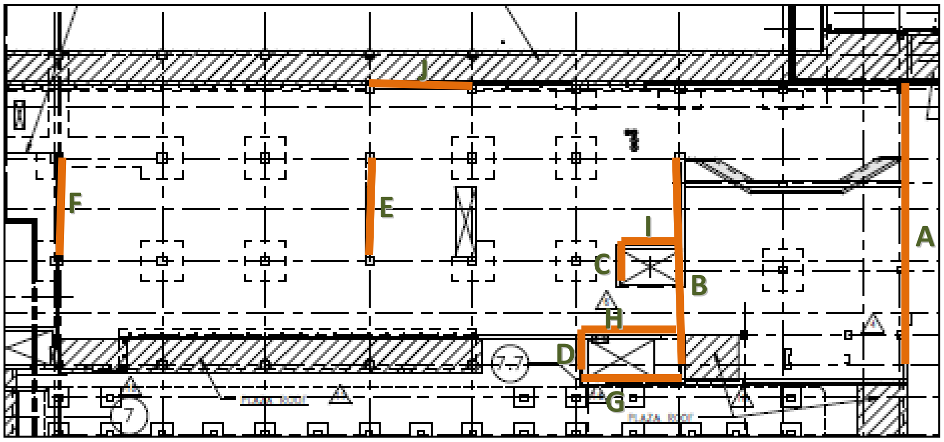


Figure 9 – Clinic Shear Wall Plan

KEY:
SW1 = A
SW2 = B
SW3 = C
SW4 = D
SW5 = E
SW6 = F
SW7 = G
SW8 = H
SW9 = I
SW10 = J

An Excel spreadsheet is used to calculate the distribution of the lateral loads to the 29 shear walls in the hospital and 10 shear walls in the clinic. The calculations considered both the controlling seismic and wind cases for the hospital and clinic. Stiffness is calculated at each floor because some shear walls differ from the typical shear wall height and some shear walls change width in their elevation. See Appendix C to view all of the stiffness tables.

ETABS Model:

As mentioned before, NCHTNF is analyzed with two different models to represent a building expansion joint. The hospital is represented in Figures 10 & 11, while the clinic is shown in Figures 12 & 13. Shear walls are the only elements modeled in ETABS because these walls are the only members in the building to resist lateral loads. The shear walls are meshed to a maximum of 24" instead of 24"x24" to simplify the calculations. The irregularity of the elevation of some of the shear walls cause errors in the ETABS when 24"x24" is specified. The moment of inertia is decreased 50% to account for the cracked section property. Each floor is modeled as a rigid diaphragm with an additional self weight added to represent the weight of the floor system.

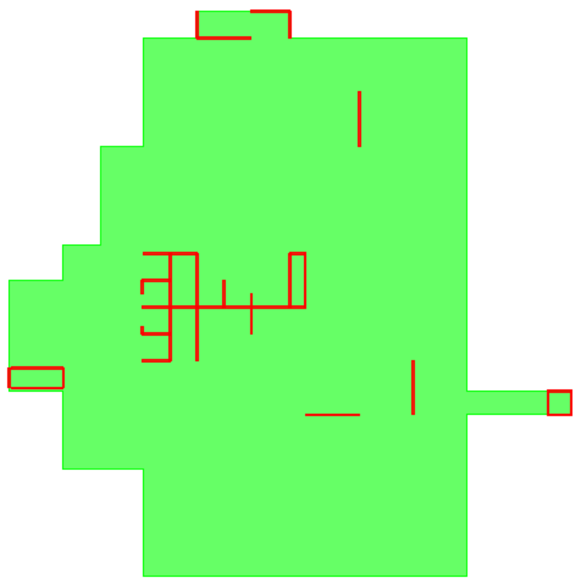


Figure 10 – First Floor Hospital ETABS Model

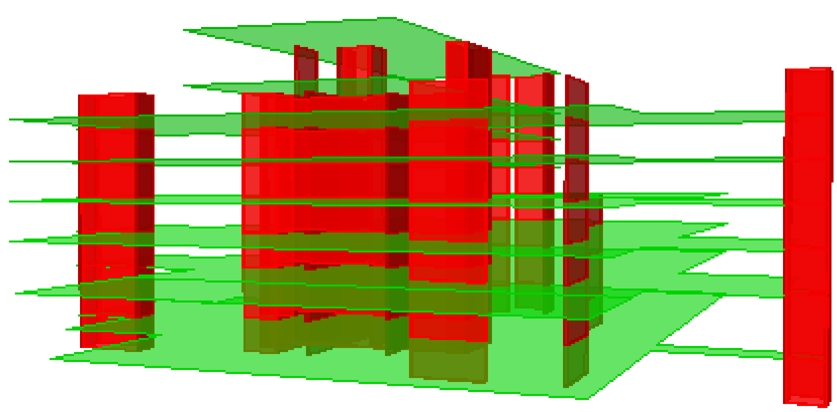


Figure 11 – 3D Hospital ETABS Model

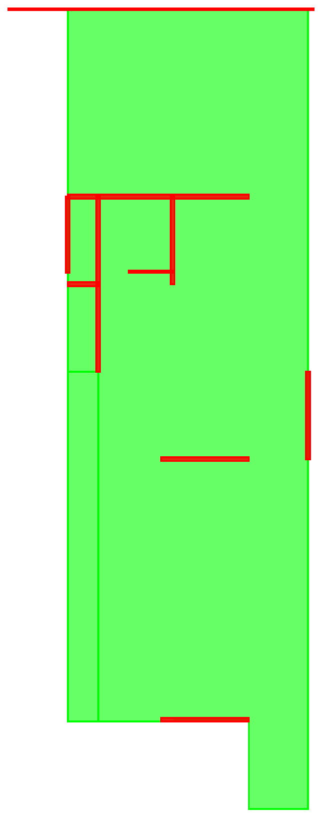


Figure 12 – First Floor Clinic ETABS Model

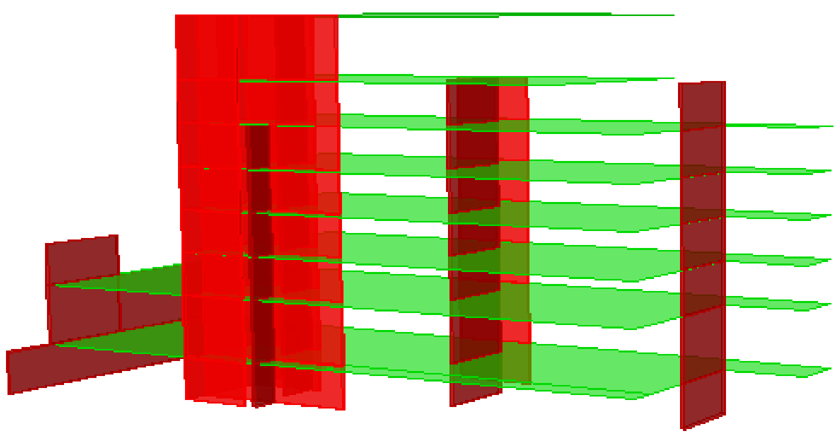


Figure 13 – 3D Clinic ETABS Model

Load Cases:

ASCE 7-05 strength design load combinations are used for the building assessment. The load cases consider both gravity and lateral loads within the NCHTNF. The load combinations utilized in this technical report are listed below.

1.4D
1.2D+1.6L+0.5Lr
1.2D+1.6Lr+0.5W
1.2D+1.0W+1.0L+0.5Lr
1.2D+1.0E+1.0L
0.9D+1.0W
0.9D+1.0E

The load cases are input into the ETABS model where the displacements and drifts determine which case, or cases, govern. NCHTNF has different load cases governing depending on the various shear walls, so all load cases are considered.

Drift & Displacement:

Story drift and lateral displacement are checked in the ETABS model. Referencing ASCE 7-10, the allowable seismic story drift is $0.010h_x$ for category IV. The allowable displacement for wind is L/400. Unfactored loads are used to determine the displacements and story drifts, as shown below in Figures 14 – 17. The actual drifts and displacements are within the limits after comparing the ETABS results with the code.

Hospital Wind Drift and Displacement					
Story	*X Displacement (in)*	*Y Displacement (in)*	*X Story Drift (in)*	*Y Story Drift (in)*	*Allowable Drift (in)*
STORY8	0.1476	0.0354	0.22194	0.1053	4.05
STORY7	0.1145	0.0264	0.243	0.1053	4.05
STORY6	0.1017	0.0027	0.21528	0.09243	4.05
STORY5	0.082	0.0019	0.18018	0.07623	4.05
STORY4	0.0651	0.0051	0.13851	0.05751	4.05
STORY3	0.0461	0.0032	0.09576	0.03843	4.05
STORY2	0.0287	0.0005	0.0513	0.01845	4.05
STORY1	0.0079	0.0002	0.01044	0.00288	4.05

Figure 14 – Hospital Wind Drift & Displacement

Hospital Seismic Drift and Displacement					
Story	*X Displacement (in)*	*Y Displacement (in)*	*X Story Drift (in)*	*Y Story Drift (in)*	*Allowable Drift (in)*
STORY8	0.1022	0.0373	0.15228	0.1539	2.7
STORY7	0.0837	0.0296	0.2376	0.13095	1.8
STORY6	0.0801	-0.0011	0.2106	0.11583	1.8
STORY5	0.0655	-0.0011	0.1782	0.09504	1.8
STORY4	0.0542	0.0043	0.14013	0.0729	1.8
STORY3	0.0388	0.0027	0.09891	0.04914	1.8
STORY2	0.0241	-0.0003	0.05265	0.02475	2.7
STORY1	0.0062	0.0001	0.01008	0.00396	1.8

Figure 15 – Hospital Seismic Drift & Displacement

Clinic Wind Drift and Displacement					
Story	*X Displacement (in)*	*Y Displacement (in)*	*X Story Drift (in)*	*Y Story Drift (in)*	*Allowable Drift (in)*
STORY8	0.9121	0.144	3.18006	1.14048	4.05
STORY7	0.6269	0.0856	1.512	0.5508	4.05
STORY6	0.5859	0.0714	1.58769	0.37206	4.05
STORY5	0.4602	0.0548	1.28997	0.30195	4.05
STORY4	0.3396	0.039	0.97119	0.22599	4.05
STORY3	0.2285	0.0247	1.148805	0.15183	4.05
STORY2	0.1032	0.0124	0.35685	0.06615	4.05
STORY1	0.0246	0.0032	0.04968	0.01134	4.05

Figure 16 – Clinic Wind Drift & Displacement

Clinic Seismic Drift and Displacement					
Story	*X Displacement (in)*	*Y Displacement (in)*	*X Story Drift (in)*	*Y Story Drift (in)*	*Allowable Drift (in)*
STORY8	0.2665	0.0398	0.73872	0.25596	2.7
STORY7	0.1995	0.0274	0.49275	0.01336635	1.8
STORY6	0.1913	0.0234	0.51246	0.12168	1.8
STORY5	0.1509	0.018	0.42174	0.099	1.8
STORY4	0.1116	0.0128	0.31995	0.07452	1.8
STORY3	0.075	0.0081	0.21735	0.04977	1.8
STORY2	0.0336	0.004	0.11655	0.0216	2.7
STORY1	0.0078	0.001	0.01566	0.0036	1.8

Figure 17 – Clinic Seismic Drift & Displacement

Building Torsion:

NCHTNF will experience torsion from the applied lateral loads due to the difference in location of the center of mass and center of rigidity. Due to building geometry irregularities, ETABS calculates the center of rigidity and center of mass of each floor more accurately than a hand calculation. The moment due to torsion is a result of the eccentricity multiplied by the story force. Additionally, the seismic loads are applied at an eccentricity of 5% of the building length, so accidental torsion accounts for this. In Figures 18-21, the accidental torsion and torsion due to eccentricity are added to find a total moment. At this time, the moments at each story are added to determine the total torsion on the hospital and clinic. ETABS accounts for inherent torsion in the building, so a separate calculation to determine this is unnecessary.

Hospital Building Torsion N-S Direction - Seismic Loading							
Floor	*Story Force*	*Location of CR*	*Location of CM*	*e_y (ft)*	*M_t (ft-k)*	*M_a (ft-k)*	*M_{total} (ft-k)*
Story 8	38.10	188.88	194.61	5.73	218.5	285.8	504.2
Story 7	80.30	193.30	194.61	1.31	105.4	602.3	707.7
Story 6	347.00	191.83	178.64	13.19	4576.6	2602.5	7179.1
Story 5	288.00	190.25	178.64	11.61	3344.0	2160.0	5504.0
Story 4	289.00	187.90	165.82	22.08	6381.1	2167.5	8548.6
Story 3	232.00	184.54	162.86	21.69	5031.4	1740.0	6771.4
Story 2	175.00	179.56	158.77	20.80	3639.5	1312.5	4952.0
Story 1	62.60	171.81	165.15	6.66	416.8	469.5	886.3
						Total	35053.2

Figure 18 – Hospital Building Torsion N-S Direction

Hospital Building Torsion E-W Direction - Seismic Loading							
Floor	*Story Force*	*Location of CR*	*Location of CM*	*e_x (ft)*	*M_t (ft-k)*	*M_a (ft-k)*	*M_{total} (ft-k)*
Story 8	38.10	185.79	206.90	21.10	804.1	542.9	1347.0
Story 7	80.30	167.76	206.90	39.14	3142.5	1144.3	4286.8
Story 6	347.00	168.15	154.13	14.02	4865.3	4944.8	9810.0
Story 5	288.00	168.74	154.13	14.61	4208.5	4104.0	8312.5
Story 4	289.00	169.81	172.99	3.17	917.6	4118.3	5035.8
Story 3	232.00	171.24	172.53	1.29	298.6	3306.0	3604.6
Story 2	175.00	172.87	156.19	16.68	2919.4	2493.8	5413.1
Story 1	62.60	174.46	164.70	9.76	610.7	892.1	1502.8
						Total	39312.7

Figure 19 – Hospital Building Torsion E-W Direction

Clinic Building Torsion N-S Direction - Seismic Loading							
Floor	*Story Force*	*Location of CR*	*Location of CM*	*e_y (ft)*	*M_t (ft-k)*	*M_a (ft-k)*	*M_{total} (ft-k)*
Story 8	23.30	492.79	419.50	73.29	1707.6	104.9	1812.5
Story 7	49.00	490.56	419.50	71.06	3482.1	220.5	3702.6
Story 6	121.00	489.63	400.40	89.23	10797.3	544.5	11341.8
Story 5	99.80	489.61	400.40	89.21	8903.5	449.1	9352.6
Story 4	79.70	489.57	400.40	89.17	7106.8	358.7	7465.5
Story 3	60.10	489.95	400.40	89.55	5381.8	270.5	5652.3
Story 2	40.90	501.85	432.51	69.34	2836.0	184.1	3020.1
Story 1	22.90	489.41	436.37	53.04	1214.6	103.1	1317.6
						Total	43664.9

Figure 20 – Clinic Building Torsion N-S Direction

Clinic Building Torsion E-W Direction - Seismic Loading							
Floor	*Story Force*	*Location of CR*	*Location of CM*	*e_x (ft)*	*M_t (ft-k)*	*M_a (ft-k)*	*M_{total} (ft-k)*
Story 8	23.30	136.95	159.00	22.05	513.7	284.3	797.9
Story 7	49.00	136.59	159.00	22.41	1098.0	597.8	1695.8
Story 6	121.00	136.59	160.21	23.62	2857.7	1476.2	4333.9
Story 5	99.80	137.19	160.21	23.02	2297.8	1217.6	3515.4
Story 4	79.70	138.11	160.21	22.10	1761.5	972.3	2733.9
Story 3	60.10	139.63	160.21	20.58	1237.0	733.2	1970.2
Story 2	40.90	142.46	159.90	17.45	713.5	499.0	1212.5
Story 1	22.90	145.11	162.29	17.18	393.3	279.4	672.7
						Total	16932.3

Figure 21 – Clinic Building Torsion E-W Direction

Overturning Moment & Foundation:

The foundation design of a building can be affected by the overturning moments due to the lateral forces. The overturning moments are calculated for the seismic and wind loads in both directions. As seen in Figure 22, the overturning moment due to N-S wind loads generates the greatest for the hospital. Figure 23 shows N-S wind load causes the greatest overturning moment in the clinic. According to the geotechnical report, the maximum allowable soil bearing capacity is 8,000 psf. The foundations cannot exceed this soil capacity so soil failure does not occur.

In the ETABS model, the connection between the shear wall and the foundation is modeled as pinned. The calculations in Appendix D show the axial load on an individual footing is extremely close to the maximum allowable soil bearing capacity. This would leave only marginal capacity for an applied moment; therefore a pinned base more accurately defines the connection instead of a fixed base.

Hospital Overturning Moments							
		Seismic		N-S Wind		E-W Wind	
Floor	Height (ft)	Lateral Force (k)	Moment (ft-k)	Lateral Force (k)	Moment (ft-k)	Lateral Force	Moment (ft-k)
Story 8	135	38.1	5143.5	190	25650	103	13905
Story 7	112.5	80.3	9033.75	308	34650	166	18675
Story 6	97.5	347	33832.5	242	23595	131	12772.5
Story 5	82.5	288	23760	237	19552.5	128	10560
Story 4	67.5	289	19507.5	230	15525	124	8370
Story 3	52.5	232	12180	222	11655	120	6300
Story 2	37.5	175	6562.5	266	9975	143	5362.5
Story 1	15	62.6	939	238	3570	129	1935
	Total Overturning Moment		110958.75		144172.5		77880

Figure 22 – Hospital Overturning Moments

Clinic Overturning Moments							
		Seismic		N-S Wind		E-W Wind	
Floor	Height (ft)	Lateral Force (k)	Moment (ft-k)	Lateral Force (k)	Moment (ft-k)	Lateral Force	Moment (ft-k)
Story 8	135	23.3	3145.5	163	22005	61.6	8316
Story 7	112.5	49	5512.5	263	29587.5	99.6	11205
Story 6	97.5	121	11797.5	207	20182.5	78.5	7653.75
Story 5	82.5	99.8	8233.5	203	16747.5	76.8	6336
Story 4	67.5	79.7	5379.75	197	13297.5	74.4	5022
Story 3	52.5	60.1	3155.25	190	9975	71.9	3774.75
Story 2	37.5	40.9	1533.75	227	8512.5	86.1	3228.75
Story 1	15	22.9	343.5	204	3060	77.1	1156.5
	Total Overturning Moment		39101.25		123367.5		46692.75

Figure 23 – Clinic Overturning Moments

Member Check:

A spot check is carried out on one of the shear walls in the hospital. The spot check is to ensure the wall can withstand the applied gravity and lateral loads that are tested in ETABS. The loads used to verify the member's adequacy are obtained from the ETABS output. The shear wall is found to be adequate for the applied loads in NCHTNF seeing as its displacement passed ASCE 7-05 code standards. A detailed calculation can be found in Appendix E.

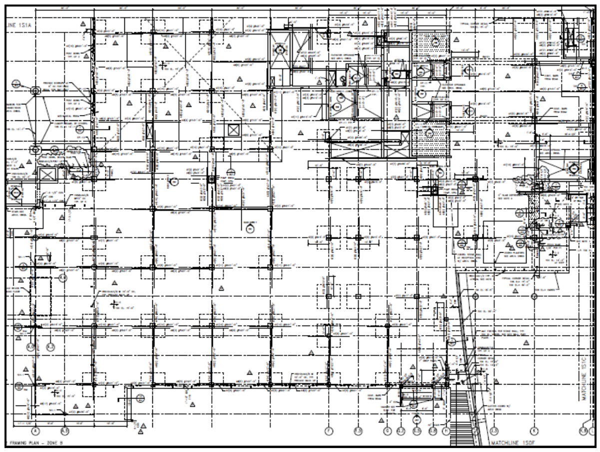


Figure 24 – NCHTNF Hospital Partial Plan

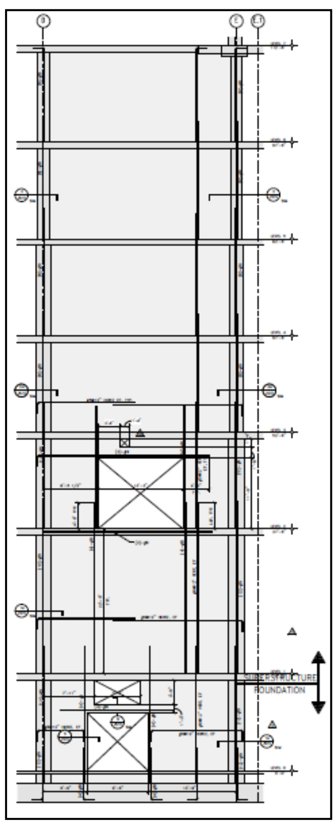


Figure 25 – Hospital Shear Wall #19

Conclusion:

This report analyzes the lateral system in NCHTNF. The use of the ETABS model allows thorough studies of the 39 shears walls in the building because ETABS provides a more detailed and accurate analysis than hand calculations because of irregular building geometry. ETABS' detailed data on each of the shear walls is how the program provides its accurate analysis. Building torsion and overturning moments are also calculated, using ETABS outputs, to study the effect on the foundation from the lateral system.

The ETABS model is also used to calculate story drifts, story displacements, stiffness, and spot checks. After studying the ETABS analysis, NCHTNF meets the ASCE 7-05 standards for drift and displacement. The spot check of the shear wall #19 proves the typical shear wall meet code standards for strength and deflection. In conclusion, NCHTNF's lateral system is adequate for resisting the lateral loads the building will experience.

Appendix A: Wind Load Calculations

A.1 Wind Pressures

Table A.1-1 Hospital North-South Wind Calculations

North - South Hospital (MWFRS)									
Floor	*Elevation*	*z*	*k_z*	*q_z*	*q_h*	*Windward(psf)*	*Leeward (psf)*	*Trib. Area (ft^2)*	*Force (k)*
Ground	89.1	0	0.85	52.43	83.27	25.17	-19.36	2137.5	95
1	104.1	15	0.85	52.43	83.27	25.17	-19.36	5343.75	238
2	126.6	37.5	1.025	63.22	83.27	30.35	-19.36	5343.75	266
3	141.6	52.5	1.1	67.85	83.27	32.57	-19.36	4275	222
4	156.6	67.5	1.16	71.55	83.27	34.34	-19.36	4275	230
5	171.6	82.5	1.22	75.25	83.27	36.12	-19.36	4275	237
6	186.6	97.5	1.26	77.72	83.27	37.31	-19.36	4275	242
Penthouse	201.6	112.5	1.29	79.57	83.27	38.19	-19.36	5343.75	308
Roof	224.1	135	1.35	83.27	83.27	39.97	-19.36	3206.25	190
								?F	2030
								*Overturning Moment (k*ft)*	274000

Table A.1-2 Hospital East-West Wind Calculations

East - West Hospital (MWFRS)									
Floor	*Elevation*	*z*	*k_z*	*q_z*	*q_h*	*Windward(psf)*	*Leeward (psf)*	*Trib. Area (ft^2)*	*Force (k)*
Ground	89.1	0	0.85	52.43	83.27	25.75	-19.94	1125	51
1	104.1	15	0.85	52.43	83.27	25.75	-19.94	2812.5	129
2	126.6	37.5	1.025	63.22	83.27	31.06	-19.94	2812.5	143
3	141.6	52.5	1.1	67.85	83.27	33.33	-19.94	2250	120
4	156.6	67.5	1.16	71.55	83.27	35.15	-19.94	2250	124
5	171.6	82.5	1.22	75.25	83.27	36.96	-19.94	2250	128
6	186.6	97.5	1.26	77.72	83.27	38.18	-19.94	2250	131
Penthouse	201.6	112.5	1.29	79.57	83.27	39.08	-19.94	2812.5	166
Roof	224.1	135	1.35	83.27	83.27	40.90	-19.94	1687.5	103
								?F	1100
								*Overturning Moment (k*ft)*	149000

Table A.1-3 Clinic North-South Wind Calculations

North - South Clinic (MWFRS)									
Floor	Elevation	z	k_z	q_z	q_h	Windward(psf)	Leeward (psf)	Trib. Area (ft^2)	Force (k)
Ground	89.1	0	0.85	52.43	83.27	25.17	-19.36	1830	82
1	104.1	15	0.85	52.43	83.27	25.17	-19.36	4575	204
2	126.6	37.5	1.025	63.22	83.27	30.35	-19.36	4575	227
3	141.6	52.5	1.1	67.85	83.27	32.57	-19.36	3660	190
4	156.6	67.5	1.16	71.55	83.27	34.34	-19.36	3660	197
5	171.6	82.5	1.22	75.25	83.27	36.12	-19.36	3660	203
6	186.6	97.5	1.26	77.72	83.27	37.31	-19.36	3660	207
Penthouse	201.6	112.5	1.29	79.57	83.27	38.19	-19.36	4575	263
Roof	224.1	135	1.35	83.27	83.27	39.97	-19.36	2745	163
								?F	1740
								Overturning Moment (k*ft)	235000

Table A.1-4 Clinic East-West Wind Calculations

East - West Clinic (MWFRS)									
Floor	Elevation	z	k_z	q_z	q_h	Windward(psf)	Leeward (psf)	Trib. Area (ft^2)	Force (k)
Ground	89.1	0	0.85	52.43	83.27	25.75	-19.94	675	31
1	104.1	15	0.85	52.43	83.27	25.75	-19.94	1687.5	77
2	126.6	37.5	1.025	63.22	83.27	31.06	-19.94	1687.5	86
3	141.6	52.5	1.1	67.85	83.27	33.33	-19.94	1350	72
4	156.6	67.5	1.16	71.55	83.27	35.15	-19.94	1350	74
5	171.6	82.5	1.22	75.25	83.27	36.96	-19.94	1350	77
6	186.6	97.5	1.26	77.72	83.27	38.18	-19.94	1350	79
Penthouse	201.6	112.5	1.29	79.57	83.27	39.08	-19.94	1687.5	100
Roof	224.1	135	1.35	83.27	83.27	40.90	-19.94	1012.5	62
								?F	657
								Overturning Moment (k*ft)	88700

A.2 Hand Calculations

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ASCE7-05

Design Criteria

Basic wind speed = 157 mph *** Explanation in wind discussion
occupancy category = IV (Table 1-1)
wind importance factor = 1.15 (Table 6-1)
wind exposure = C (specified in plans)
wind directionality factor = 0.85 (Table 6-4)
topographic factor = 1.0 (section 6.5.7.1)

Building Rigid if $f > 1\text{hz}$

From 12.8.2.1

$T_a = C_t h_n^x$ $\quad C_t = 0.016$ (Table 12.8-2)

$T_a = (0.016)(135')^{0.9} = 1.32$ $\quad x = 0.9$ (Table 12.8-2)

$1/T_a = f = 0.756 < 1.0$ $\quad h_n = 135'$

∴ Building is not rigid, calculate G_f

Gust Effect Factor

*** Note: NS subscript denotes North-South direction
EW subscript denotes East-West direction

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

$I_{\bar{z}} = c(33/\bar{z})^{1/6}$ $\quad c = 0.2$ (Table 6-2)

$I_{\bar{z}} = 0.2(33/81)^{1/6}$ $\quad \bar{z} = 0.6h$

$I_{\bar{z}} = 0.1722$ $\quad \bar{z} = 0.6(135') = 81$

$g_Q = g_v = 3.4$

$$Q = \sqrt{1/[1 + 0.63(B+h/L_{\bar{z}})^{0.63}}$$

$B_{NS} = 465'$

$B_{EW} = 300'$

$L_{\bar{z}} = \ell(\bar{z}/33)^{\bar{\epsilon}}$ $\quad \ell = 500'$ (Table 6-2)

$L_{\bar{z}} = 500'(81/33)^{1/5} = 598.36$ $\quad \bar{\epsilon} = 1/5$ (Table 6-2)

$Q_{NS} = \sqrt{1/[1 + 0.63(465+135/598.36)^{0.63}} = 0.783$

$Q_{EW} = \sqrt{1/[1 + 0.63(300+135/598.36)^{0.63}} = 0.812$

$g_R = \sqrt{2\ln(3{,}600 n_1)} + 0.577/\sqrt{2\ln(3{,}600 n_1)}$ $\quad n_1 = f = 0.756$

$g_R = \sqrt{2\ln(3{,}600(0.756))} + 0.577/\sqrt{2\ln(3600(0.756))}$

$g_R = 4.122$

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

*** Assume Damping Ratio $(\beta) = 1.0$

$R_n = R_l$ when $\eta = 4.6 n_1 h / V_{\bar{z}}$

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

$N_1 = n_1 L_{\bar{z}} / V_{\bar{z}}$

$V_{\bar{z}} = \bar{b} (\bar{z}/33)^{\bar{\alpha}} V (88/60)$ — $\bar{b} = 0.65$ (Table 6-2), $\bar{\alpha} = 1/9$ (Table 6-2)

$V_{\bar{z}} = 0.65 (81/33)^{(1/9)} 157 (88/60)$

$V_{\bar{z}} = 165.376$

$N_1 = 0.756 (598.36) / 165.376$

$N_1 = 2.74$

$R_n = 7.47 (2.74) / (1 + 10.3 (2.74))^{5/3}$

$R_n = 0.074$

$R_h = R_l$ when $\eta = 4.6 n_1 h / V_{\bar{z}}$

$R_h = 1/\eta - 1/2\eta^2 (1 - e^{-2\eta})$

$\eta = 4.6 n_1 h / V_{\bar{z}}$

$\eta = 4.6 (0.756)(135) / 165.374$

$\eta = 2.84$

$R_h = 1/2.84 - 1/2(2.84^2) \cdot (1 - e^{-2(2.84)})$

$R_h = 0.290$

$R_B = R_l$ when $\eta = 4.6 n_1 EB / V_{\bar{z}}$

$R_{B_{NS}} = 1/\eta - 1/2\eta^2 (1 - e^{-2\eta})$

$\eta = 4.6 n_1 EB / V_{\bar{z}}$

$\eta = 4.6 (0.756)(465) / 165.376$

$\eta = 9.781$

$R_{B\,NS} = 1/9.781 - 1/2(9.781^2) \cdot (1 - e^{-2(9.781)})$

$R_{B\,NS} = 0.097$

$R_{B\,EW} = 1/\eta - 1/2\eta^2 (1 - e^{-2\eta})$

$\eta = 4.6 n_1 EB / V_{\bar{z}}$

$\eta = 4.6 (0.756)(300) / 165.376$

$\eta = 6.309$

$R_{B\,EW} = 1/6.309 - 1/2(6.309^2) \cdot (1 - e^{-2(6.309)})$

$R_{B\,EW} = 0.146$

$R_L = R_l$ when $\eta = 15.4 n_1 L / V_{\bar{z}}$

$R_{L\,NS} = 1/\eta - 1/2\eta^2 (1 - e^{-2\eta})$

$\eta = 15.4 n_1 L / V_{\bar{z}}$

$\eta = 15.4 (0.756)(300) / 165.376$

$\eta = 21.122$

$R_{L\,NS} = 1/21.122 - 1/2(21.122^2) \cdot (1 - e^{-2(21.122)})$

$R_{L\,NS} = 0.044$

$R_{L\,EW} = 1/\eta - 1/2\eta^2 (1 - e^{-2\eta})$

$\eta = 15.4 n_1 L / V_{\bar{z}}$

$\eta = 15.4 (0.756)(465) / 165.376$

$\eta = 32.736$

$R_{L\,EW} = 1/32.736 - 1/2(32.736^2) \cdot (1 - e^{-2(32.736)})$

$R_{L\,EW} = 0.030$

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$R_{NS} = \sqrt{(1/\beta) R_n R_h R_B (0.53 + 0.47 R_L)}$

$R_{NS} = \sqrt{(0.074)(0.290)(0.097)(0.53 + 0.47(0.046))}$

$R_{NS} = 0.034$

$R_{EW} = \sqrt{(1/\beta) R_n R_h R_B (0.53 + 0.47 R_L)}$

$R_{EW} = \sqrt{(0.074)(0.290)(0.149)(0.53 + 0.47(0.030))}$

$R_{EW} = 0.041$

$G_{f_{NS}} = 0.925([1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}] / [1 + 1.7 g_v I_{\bar{z}}])$

$G_{f_{NS}} = 0.925([1 + 1.7(0.1722)\sqrt{3.4^2(0.783)^2 + 4.122^2(0.034)^2}] / [1 + 1.7(3.4)(0.1722)])$

$\boxed{G_{f_{NS}} = 0.825}$

$G_{f_{EW}} = 0.925([1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}] / [1 + 1.7 g_v I_{\bar{z}}])$

$G_{f_{EW}} = 0.925([1 + 1.7(0.1722)\sqrt{3.4^2(0.812)^2 + 4.122^2(0.041)^2}] / [1 + 1.7(3.4)(0.1722)])$

$\boxed{G_{f_{EW}} = 0.839}$

Enclosed flexible building
↳ nonoperable windows ∴ enclosed

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

$q = q_z$ for windward walls
$q = q_h$ for leeward walls
$C_p = 0.8$ (windward walls) (fig. 6-6)
$= -0.5$ (leeward walls) (fig. 6-6)
$q_i = q_z$
$GC_{pi} = \pm 0.18$ (fig. 6-5)
$q_z = 0.00256\, K_z K_{zt} K_d V^2 I$
K_z = Table 6.3 (varies w/ height)
$K_{zt} = 1.0$
$K_d = 0.85$
$V^2 = 110$ mph
$I = 1.15$

see pg. 1 of wind calcs for data location

*** remainder of wind calcs on excel spread sheet stated in wind calc discussion.

Appendix B: Seismic Load Calculations

B.1 Seismic Loads

Table B.1 Hospital Seismic Calculations

Seismic Calculations (Hospital)								
Floor	*Height (ft)*	*System Weight (k)*	*Total Weight (k)*	*$w*h^k$*	*C_{vx}*	*F_x (k)*	*V_i (k)*	*M (ft-k)*
1	15	9527.31	9530	202000	0.04	62.60	62.60	939
2	37.5	9447.04	9450	564000	0.12	175.00	237.60	6560
3	52.5	8579.13	8580	748000	0.15	232.00	469.60	12200
4	67.5	8045.68	8050	932000	0.19	289.00	758.60	19500
5	82.5	6400.50	6400	929000	0.19	288.00	1046.60	23800
6	97.5	6394.50	6390	1120000	0.23	347.00	1393.60	33800
Penthouse	112.5	1255.50	1260	259000	0.05	80.30	1473.90	9030
Roof	135	486.00	486	123000	0.03	38.10	1512.00	5140
? Totals			50100	4880000		1510		111000

Table B.2 Clinic Seismic Calculations

Seismic Calculations (Clinic)								
Floor	*Height (ft)*	*System Weight (k)*	*Total Weight (k)*	*$w*h^k$*	*C_{vx}*	*F_x (k)*	*V_i (k)*	*M (ft-k)*
1	15	3492.70	3490	74000	0.02	22.90	22.90	344
2	37.5	2218.50	2220	132000	0.03	40.90	63.80	1530
3	52.5	2218.50	2220	194000	0.04	60.10	123.90	3160
4	67.5	2218.50	2220	257000	0.05	79.70	203.60	5380
5	82.5	2218.50	2220	322000	0.07	99.80	303.40	8230
6	97.5	2218.50	2220	389000	0.08	121.00	424.40	11800
Penthouse	112.5	767.25	767	158000	0.03	49.00	473.40	5510
Roof	135	297.00	297	75100	0.02	23.30	496.70	3150
? Totals			15700	1600000		497		39100

B.2 Hand Calculations

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11.4 seismic ground motion

site class D (firm soil) according to geotech report

$S_s = 0.096$, $S_1 = 0.038$ — from usgs.gov ground motion calculator based on ASCE7-05

$F_a = 1.6$ (Table 11.4-1)
$F_v = 2.4$ (Table 11.4-2)

$S_{MS} = F_a S_s$
$S_{MS} = (1.6)(0.096) = 0.15$
$S_{M1} = F_v S_1$
$S_{M1} = (2.4)(0.038) = 0.09$

$S_{DS} = 2/3\, S_{MS}$
$S_{DS} = 2/3(0.15) = 0.10$
$S_{D1} = 2/3\, S_{M1}$
$S_{D1} = 2/3(0.09) = 0.06$

$T_0 = 0.2(S_{D1}/S_{DS})$
$T_0 = 0.2(0.06/0.10) = 0.12$
$T_s = S_{D1}/S_{DS}$
$T_s = 0.06/0.10 = 0.6$
$T_L = 8s$ (figure 22-15)

$T_s > T_0$

occupancy category = IV (Table 1-1) → hospitals & other healthcare facilities
Importance factor ⇒ IV = 1.5 (Table 11.5-1)

seismic design category
$S_{DS} < 0.167$ $S_{DS} = 0.10$ ✓ ∴ A (Table 11.6-1)
seismic design category
$S_{D1} < 0.067$ $S_{D1} = 0.06$ ✓ ∴ A (Table 11.6-2)

12.8 Equivalent Lateral Force Procedure

$V = C_s W$

$W = 69{,}485^k$ (calculated using spreadsheet) → see bottom note

$C_s = S_{DS}/(R/I)$ for $T \le T_L$
$T = 0.756s \le T_L = 8s$ ✓
↳ calculated for wind calcs

$R = 5$ (ordinary reinforced concrete shear walls) (Table 12.2-1)

$C_s = 0.10/(5/1.5)$
$C_s = 0.03 > 0.01$ ✓

$F_x = C_{vx} V$

$C_{vx} = w_x h_x^k / \sum_{i=1}^{n} w_i h_i^k$

$k = 1.128$ (interpolation) (sec. 12.8.3)

*** remainder of seismic calcs on excel spreadsheet stated in seismic discussion.

Note: weight calculated using 12" slab across each floor as weight estimate.

Appendix C: Stiffness Tables

C.1 Hospital Wind Stiffness Tables

Story	Pier	Shear	$K_{relative}$
STORY8	SW6	50.63	0.266
STORY8	SW7	32.07	0.169
STORY8	SW10	11.11	0.058
STORY8	SW8	8.83	0.046
STORY8	SW11	25.88	0.136
STORY8	SW12	16.35	0.086
STORY8	SW13	45.12	0.237
STORY8	SW24	5.22	0.027
STORY8	SW26	7.21	0.038
STORY8	SW28	-4.2	0.022
STORY8	SW29	-8.22	0.043
	Total	190	

Story	Pier	Shear	$K_{relative}$
STORY7	SW1	0	0.000
STORY7	SW2	0	0.000
STORY7	SW3	0	0.000
STORY7	SW4	0	0.000
STORY7	SW6	39.67	0.080
STORY7	SW7	8.33	0.017
STORY7	SW8	-5.93	0.012
STORY7	SW9	6.11	0.012
STORY7	SW11	207.12	0.416
STORY7	SW10	-12.9	0.026
STORY7	SW12	-9.5	0.019
STORY7	SW13	265.07	0.532
STORY7	SW14	0	0.000
STORY7	SW15	0	0.000
STORY7	SW16	0	0.000
STORY7	SW17	0	0.000

STORY7	SW18	0	0.000
STORY7	SW19	0.74	0.001
STORY7	SW20	0	0.000
STORY7	SW21	0	0.000
STORY7	SW22	-16.39	0.033
STORY7	SW23	-15.99	0.032
STORY7	SW24	-87.51	0.176
STORY7	SW25	18.37	0.037
STORY7	SW26	0.85	0.002
STORY7	SW27	60.08	0.121
STORY7	SW28	24.61	0.049
STORY7	SW29	15.24	0.031
	Total	497.97	

Story	Pier	Shear	$K_{relative}$
STORY6	SW1	9.15	0.012
STORY6	SW2	17.25	0.023
STORY6	SW3	-0.65	0.001
STORY6	SW4	20	0.027
STORY6	SW6	62.02	0.084
STORY6	SW7	28.91	0.039
STORY6	SW8	7.13	0.010
STORY6	SW9	14.03	0.019
STORY6	SW10	-3.12	0.004
STORY6	SW11	259.54	0.351
STORY6	SW12	6.01	0.008
STORY6	SW13	324.91	0.439
STORY6	SW14	-1.57	0.002
STORY6	SW15	-5.47	0.007
STORY6	SW16	10.73	0.015
STORY6	SW17	0.25	0.000
STORY6	SW18	-8.75	0.012
STORY6	SW19	-1.15	0.002

STORY6	SW20	-13.25	0.018
STORY6	SW21	13.88	0.019
STORY6	SW22	-28.09	0.038
STORY6	SW23	-21.63	0.029
STORY6	SW24	-126.02	0.170
STORY6	SW25	30.58	0.041
STORY6	SW26	4.21	0.006
STORY6	SW27	81.53	0.110
STORY6	SW28	38.9	0.053
STORY6	SW29	20.63	0.028
	Total	739.96	

Story	Pier	Shear	$K_{relative}$
STORY5	SW1	8.66	0.009
STORY5	SW2	26.02	0.027
STORY5	SW3	5.86	0.006
STORY5	SW4	35.6	0.036
STORY5	SW6	91.46	0.094
STORY5	SW7	60.7	0.062
STORY5	SW8	16.42	0.017
STORY5	SW9	19.71	0.020
STORY5	SW10	4.5	0.005
STORY5	SW11	308.8	0.316
STORY5	SW12	16.48	0.017
STORY5	SW13	381.78	0.391
STORY5	SW14	-0.75	0.001
STORY5	SW15	-3.1	0.003
STORY5	SW16	13.26	0.014
STORY5	SW17	0.23	0.000
STORY5	SW18	-11.49	0.012
STORY5	SW19	-4.98	0.005
STORY5	SW20	-21.28	0.022
STORY5	SW21	16.16	0.017

STORY5	SW22	-30.37	0.031
STORY5	SW23	-19.96	0.020
STORY5	SW24	-137.97	0.141
STORY5	SW25	28.71	0.029
STORY5	SW26	5.3	0.005
STORY5	SW27	85.72	0.088
STORY5	SW28	53	0.054
STORY5	SW29	28.47	0.029
	Total	976.94	

Story	Pier	Shear	$K_{relative}$
STORY4	SW1	7.46	0.006
STORY4	SW2	28.28	0.023
STORY4	SW3	9.28	0.008
STORY4	SW4	46.49	0.039
STORY4	SW5	47.52	0.039
STORY4	SW6	115.24	0.095
STORY4	SW7	89.93	0.075
STORY4	SW8	20.14	0.017
STORY4	SW9	22.66	0.019
STORY4	SW10	8.54	0.007
STORY4	SW11	353.2	0.293
STORY4	SW12	20.73	0.017
STORY4	SW13	432.34	0.358
STORY4	SW14	-0.76	0.001
STORY4	SW15	-2.51	0.002
STORY4	SW16	15.59	0.013
STORY4	SW17	0.26	0.000
STORY4	SW18	-11.55	0.010
STORY4	SW19	-7.36	0.006
STORY4	SW20	-26.88	0.022
STORY4	SW21	17.3	0.014
STORY4	SW22	-32.66	0.027

STORY4	SW23	-19.33	0.016
STORY4	SW24	-141.32	0.117
STORY4	SW25	25.69	0.021
STORY4	SW26	5.18	0.004
STORY4	SW27	86.4	0.072
STORY4	SW28	62.74	0.052
STORY4	SW29	34.34	0.028
	Total	1206.94	

Story	Pier	Shear	$K_{relative}$
STORY3	SW1	5.46	0.004
STORY3	SW2	31.73	0.022
STORY3	SW3	15.49	0.011
STORY3	SW4	63.59	0.045
STORY3	SW5	80.82	0.057
STORY3	SW6	138.65	0.097
STORY3	SW7	121.07	0.085
STORY3	SW8	24.06	0.017
STORY3	SW9	28.99	0.020
STORY3	SW10	14.31	0.010
STORY3	SW11	390.68	0.273
STORY3	SW12	24.31	0.017
STORY3	SW13	478.32	0.335
STORY3	SW14	-0.54	0.000
STORY3	SW15	-1.59	0.001
STORY3	SW16	18.99	0.013
STORY3	SW17	0.07	0.000
STORY3	SW18	-10.78	0.008
STORY3	SW19	-9.06	0.006
STORY3	SW20	-31.9	0.022
STORY3	SW21	18.95	0.013
STORY3	SW22	-35.55	0.025
STORY3	SW23	-18.1	0.013

STORY3	SW24	-136.31	0.095
STORY3	SW25	20.82	0.015
STORY3	SW26	4.57	0.003
STORY3	SW27	83.44	0.058
STORY3	SW28	68.53	0.048
STORY3	SW29	39.92	0.028
	Total	1428.94	

Story	Pier	Shear	$K_{relative}$
STORY2	SW1	2.85	0.002
STORY2	SW2	42.1	0.025
STORY2	SW3	30.85	0.018
STORY2	SW4	95.29	0.056
STORY2	SW5	101.97	0.060
STORY2	SW6	165.91	0.098
STORY2	SW7	157.57	0.093
STORY2	SW8	34.54	0.020
STORY2	SW9	47.97	0.028
STORY2	SW10	29.28	0.017
STORY2	SW11	414.09	0.244
STORY2	SW12	34.07	0.020
STORY2	SW13	511.75	0.302
STORY2	SW14	-0.42	0.000
STORY2	SW15	0.58	0.000
STORY2	SW16	27.82	0.016
STORY2	SW17	1.59	0.001
STORY2	SW18	-8.4	0.005
STORY2	SW19	-9.12	0.005
STORY2	SW20	-33.98	0.020
STORY2	SW21	20.55	0.012
STORY2	SW22	-36.09	0.021
STORY2	SW23	-12.19	0.007
STORY2	SW24	-116.01	0.068

STORY2	SW25	10.3	0.006
STORY2	SW26	3.38	0.002
STORY2	SW27	69.79	0.041
STORY2	SW28	66.31	0.039
STORY2	SW29	42.61	0.025
	Total	1694.96	

Story	Pier	Shear	$K_{relative}$
STORY1	SW1	0.32	0.000
STORY1	SW2	65.79	0.034
STORY1	SW3	60.35	0.031
STORY1	SW4	124.65	0.064
STORY1	SW5	125.68	0.065
STORY1	SW6	177.73	0.092
STORY1	SW7	175.27	0.091
STORY1	SW8	58.62	0.030
STORY1	SW9	79.67	0.041
STORY1	SW10	56.94	0.029
STORY1	SW11	391.77	0.203
STORY1	SW12	56.79	0.029
STORY1	SW13	479.55	0.248
STORY1	SW14	3.34	0.002
STORY1	SW15	13.57	0.007
STORY1	SW16	44.42	0.023
STORY1	SW17	18.8	0.010
STORY1	SW18	-5.13	0.003
STORY1	SW19	-6.64	0.003
STORY1	SW20	-27.6	0.014
STORY1	SW21	17.99	0.009
STORY1	SW22	-25.63	0.013
STORY1	SW23	-2.07	0.001
STORY1	SW24	-83.08	0.043
STORY1	SW25	-0.34	0.000

STORY1	SW26	2.35	0.001
STORY1	SW27	42.07	0.022
STORY1	SW28	51.46	0.027
STORY1	SW29	36.29	0.019
	Total	1932.93	

C.2 Hospital Quake Stiffness Tables

Story	Pier	Shear	$K_{relative}$
STORY8	SW6	24.84	0.65214
STORY8	SW7	-11.87	0.31163
STORY8	SW8	2.25	0.05907
STORY8	SW10	1.2	0.0315
STORY8	SW11	7.35	0.19296
STORY8	SW12	4.07	0.10685
STORY8	SW13	10.25	0.2691
STORY8	SW24	-6.35	0.16671
STORY8	SW26	5.52	0.14492
STORY8	SW28	2.14	0.05618
STORY8	SW29	-1.31	0.03439
	Total	38.09	

Story	Pier	Shear	$K_{relative}$
STORY7	SW1	0	0
STORY7	SW2	0	0
STORY7	SW3	0	0
STORY7	SW4	0	0
STORY7	SW6	26.38	0.2228
STORY7	SW7	-25.87	0.2185
STORY7	SW8	-7.78	0.06571
STORY7	SW9	-7.3	0.06166
STORY7	SW10	-13.43	0.11343
STORY7	SW11	101.98	0.86132
STORY7	SW12	-6	0.05068
STORY7	SW13	50.41	0.42576

STORY7	SW14	0	0
STORY7	SW15	0	0
STORY7	SW16	0	0
STORY7	SW17	0	0
STORY7	SW18	0	0
STORY7	SW19	0.44	0.00372
STORY7	SW20	0	0
STORY7	SW21	0	0
STORY7	SW22	-5.24	0.04426
STORY7	SW23	-7.7	0.06503
STORY7	SW24	-68.86	0.58159
STORY7	SW25	11.17	0.09434
STORY7	SW26	7.11	0.06005
STORY7	SW27	44.24	0.37365
STORY7	SW28	14.95	0.12627
STORY7	SW29	3.9	0.03294
	Total	118.4	

Story	Pier	Shear	$K_{relative}$
STORY6	SW1	11.01	0.02366
STORY6	SW2	16.66	0.0358
STORY6	SW3	-7.08	0.01521
STORY6	SW4	9.15	0.01966
STORY6	SW6	69.1	0.14849
STORY6	SW7	16.17	0.03475
STORY6	SW8	10.79	0.02319
STORY6	SW9	9.7	0.02084
STORY6	SW10	0.93	0.002
STORY6	SW11	173.83	0.37354
STORY6	SW12	11.61	0.02495
STORY6	SW13	162.08	0.34829
STORY6	SW14	-1.1	0.00236
STORY6	SW15	-4.73	0.01016

STORY6	SW16	-0.32	0.00069
STORY6	SW17	-1.42	0.00305
STORY6	SW18	-12.73	0.02736
STORY6	SW19	-3.56	0.00765
STORY6	SW20	-5.94	0.01276
STORY6	SW21	1.51	0.00324
STORY6	SW22	-14.94	0.0321
STORY6	SW23	-13.32	0.02862
STORY6	SW24	-112.44	0.24162
STORY6	SW25	21.25	0.04566
STORY6	SW26	8.48	0.01822
STORY6	SW27	65.39	0.14051
STORY6	SW28	36.94	0.07938
STORY6	SW29	18.34	0.03941
	Total	465.36	

Story	Pier	Shear	$K_{relative}$
STORY5	SW1	11.6	0.0154
STORY5	SW2	26.17	0.03474
STORY5	SW3	-0.25	0.00033
STORY5	SW4	27.92	0.03706
STORY5	SW6	100.03	0.13278
STORY5	SW7	51.53	0.0684
STORY5	SW8	17.14	0.02275
STORY5	SW9	17.07	0.02266
STORY5	SW10	5.42	0.00719
STORY5	SW11	239.6	0.31805
STORY5	SW12	18.28	0.02426
STORY5	SW13	247.71	0.32881
STORY5	SW14	-0.48	0.00064
STORY5	SW15	-2.25	0.00299
STORY5	SW16	5.24	0.00696
STORY5	SW17	0.23	0.00031

STORY5	SW18	-14.83	0.01969
STORY5	SW19	-6.61	0.00877
STORY5	SW20	-12.5	0.01659
STORY5	SW21	3.16	0.00419
STORY5	SW22	-20.45	0.02715
STORY5	SW23	-13.66	0.01813
STORY5	SW24	-132.17	0.17544
STORY5	SW25	21.11	0.02802
STORY5	SW26	8.43	0.01119
STORY5	SW27	72.54	0.09629
STORY5	SW28	54.81	0.07276
STORY5	SW29	28.56	0.03791
	Total	753.35	

Story	Pier	Shear	$K_{relative}$
STORY4	SW1	9.68	0.00929
STORY4	SW2	33.76	0.03239
STORY4	SW3	8.54	0.00819
STORY4	SW4	48.25	0.04629
STORY4	SW5	41.19	0.03952
STORY4	SW6	125.19	0.1201
STORY4	SW7	83.75	0.08035
STORY4	SW8	22.98	0.02205
STORY4	SW9	22.79	0.02186
STORY4	SW10	9.84	0.00944
STORY4	SW11	295	0.28302
STORY4	SW12	23.63	0.02267
STORY4	SW13	320.18	0.30717
STORY4	SW14	-0.53	0.00051
STORY4	SW15	-1.68	0.00161
STORY4	SW16	9.23	0.00886
STORY4	SW17	0.22	0.00021
STORY4	SW18	-15.54	0.01491

STORY4	SW19	-10.52	0.01009
STORY4	SW20	-20.39	0.01956
STORY4	SW21	4.33	0.00415
STORY4	SW22	-25.48	0.02444
STORY4	SW23	-14.55	0.01396
STORY4	SW24	-142.13	0.13636
STORY4	SW25	19.51	0.01872
STORY4	SW26	7.91	0.00759
STORY4	SW27	76.71	0.07359
STORY4	SW28	71.39	0.06849
STORY4	SW29	39.08	0.03749
	Total	1042.34	

Story	Pier	Shear	$K_{relative}$
STORY3	SW1	6.83	0.00536
STORY3	SW2	39.24	0.03079
STORY3	SW3	17.28	0.01356
STORY3	SW4	68.69	0.0539
STORY3	SW5	76.37	0.05993
STORY3	SW6	144.4	0.11331
STORY3	SW7	112.39	0.08819
STORY3	SW8	28.33	0.02223
STORY3	SW9	29.16	0.02288
STORY3	SW10	15.02	0.01179
STORY3	SW11	332.03	0.26055
STORY3	SW12	28.08	0.02203
STORY3	SW13	371.95	0.29188
STORY3	SW14	-0.42	0.00033
STORY3	SW15	-0.97	0.00076
STORY3	SW16	12.69	0.00996
STORY3	SW17	0.11	8.6E-05
STORY3	SW18	-15.13	0.01187
STORY3	SW19	-14.18	0.01113

STORY3	SW20	-27.49	0.02157
STORY3	SW21	5.19	0.00407
STORY3	SW22	-29.21	0.02292
STORY3	SW23	-14.6	0.01146
STORY3	SW24	-140.35	0.11014
STORY3	SW25	16.28	0.01278
STORY3	SW26	6.94	0.00545
STORY3	SW27	75.79	0.05947
STORY3	SW28	82.32	0.0646
STORY3	SW29	47.6	0.03735
	Total	1274.34	

Story	Pier	Shear	$K_{relative}$
STORY2	SW1	2.77	0.00191
STORY2	SW2	46.99	0.03242
STORY2	SW3	31.25	0.02156
STORY2	SW4	93.07	0.06421
STORY2	SW5	93.5	0.06451
STORY2	SW6	157.14	0.10842
STORY2	SW7	136.97	0.0945
STORY2	SW8	35.84	0.02473
STORY2	SW9	41.56	0.02867
STORY2	SW10	24.83	0.01713
STORY2	SW11	343.41	0.23694
STORY2	SW12	34.01	0.02347
STORY2	SW13	391.56	0.27016
STORY2	SW14	-0.37	0.00026
STORY2	SW15	0.44	0.0003
STORY2	SW16	18.27	0.01261
STORY2	SW17	0.88	0.00061
STORY2	SW18	-12.97	0.00895
STORY2	SW19	-16.34	0.01127
STORY2	SW20	-31.17	0.02151

STORY2	SW21	5.53	0.00382
STORY2	SW22	-29.68	0.02048
STORY2	SW23	-11.05	0.00762
STORY2	SW24	-119.9	0.08273
STORY2	SW25	8.96	0.00618
STORY2	SW26	5.27	0.00364
STORY2	SW27	63.94	0.04412
STORY2	SW28	82.82	0.05714
STORY2	SW29	51.82	0.03575
	Total	1449.35	

Story	Pier	Shear	$K_{relative}$
STORY1	SW1	-1.24	0.00082
STORY1	SW2	60.39	0.03994
STORY1	SW3	52.45	0.03469
STORY1	SW4	106.23	0.07026
STORY1	SW5	103.05	0.06816
STORY1	SW6	150.15	0.09931
STORY1	SW7	139.36	0.09217
STORY1	SW8	48.2	0.03188
STORY1	SW9	59.52	0.03937
STORY1	SW10	41.16	0.02722
STORY1	SW11	312.53	0.2067
STORY1	SW12	44.32	0.02931
STORY1	SW13	345.77	0.22869
STORY1	SW14	2.03	0.00134
STORY1	SW15	8.68	0.00574
STORY1	SW16	27.47	0.01817
STORY1	SW17	10.64	0.00704
STORY1	SW18	-9.18	0.00607
STORY1	SW19	-13.67	0.00904
STORY1	SW20	-25.84	0.01709
STORY1	SW21	4.79	0.00317

STORY1	SW22	-21.1	0.01396
STORY1	SW23	-3.76	0.00249
STORY1	SW24	-84.32	0.05577
STORY1	SW25	1.43	0.00095
STORY1	SW26	3.54	0.00234
STORY1	SW27	39.05	0.02583
STORY1	SW28	65.5	0.04332
STORY1	SW29	44.83	0.02965
	Total	1511.98	

C.3 Clinic Wind Stiffness Tables

Story	Pier	Shear	$K_{relative}$
STORY8	SW2	14.53	0.08916
STORY8	SW3	148.44	0.91084
STORY8	SW7	-103.87	0.63736
STORY8	SW8	-181.35	1.11278
STORY8	SW9	285.22	1.75014
	Total	162.97	

Story	Pier	Shear	$K_{relative}$
STORY7	SW2	218.22	0.51219
STORY7	SW3	-36.51	0.08569
STORY7	SW5	244.31	0.57343
STORY7	SW6	0	0
STORY7	SW7	31.64	0.07426
STORY7	SW8	-214.84	0.50426
STORY7	SW9	40.35	0.09471
STORY7	SW10	142.88	0.33536
	Total	426.05	

Story	Pier	Shear	$K_{relative}$
STORY6	SW2	333.53	0.52691
STORY6	SW3	-49.41	0.07806
STORY6	SW4	11.01	0.01739

STORY6	SW5	65.71	0.10381
STORY6	SW6	272.15	0.42994
STORY6	SW7	114.69	0.18119
STORY6	SW8	-107.24	0.16942
STORY6	SW9	-35.04	0.05536
STORY6	SW10	27.59	0.04359
	Total	632.99	

Story	Pier	Shear	$K_{relative}$
STORY5	SW2	378.09	0.45227
STORY5	SW3	-3.28	0.00392
STORY5	SW4	10.7	0.0128
STORY5	SW5	97.61	0.11676
STORY5	SW6	352.87	0.4221
STORY5	SW7	80.69	0.09652
STORY5	SW8	-123.54	0.14778
STORY5	SW9	-9.18	0.01098
STORY5	SW10	52.03	0.06224
	Total	835.99	

Story	Pier	Shear	$K_{relative}$
STORY4	SW2	428.3	0.28932
STORY4	SW3	19.09	0.0129
STORY4	SW2	428.3	0.28932
STORY4	SW3	19.09	0.0129
STORY4	SW4	12.36	0.00835
STORY4	SW5	167.05	0.11284
STORY4	SW6	406.19	0.27438
STORY4	SW7	46.44	0.03137
STORY4	SW8	-164.96	0.11143
STORY4	SW9	24.22	0.01636
STORY4	SW10	94.3	0.0637
	Total	1480.38	

Story	Pier	Shear	$K_{relative}$
STORY3	SW1	0	0
STORY3	SW2	495.76	0.40537
STORY3	SW3	19.29	0.01577
STORY3	SW4	4.05	0.00331
STORY3	SW5	234.57	0.1918
STORY3	SW6	469.32	0.38375
STORY3	SW7	27.09	0.02215
STORY3	SW8	-196.77	0.16089
STORY3	SW9	47.6	0.03892
STORY3	SW10	122.08	0.09982
	Total	1222.99	

Story	Pier	Shear	$K_{relative}$
STORY2	SW1	-307.8	0.21228
STORY2	SW2	903.21	0.62292
STORY2	SW3	85.02	0.05864
STORY2	SW4	39.17	0.02701
STORY2	SW5	302.79	0.20883
STORY2	SW6	427.56	0.29488
STORY2	SW7	51.38	0.03544
STORY2	SW8	-158.53	0.10933
STORY2	SW9	43.46	0.02997
STORY2	SW10	63.69	0.04393
	Total	1449.95	

Story	Pier	Shear	$K_{relative}$
STORY1	SW1	-188.06	0.1137
STORY1	SW2	793.27	0.47961
STORY1	SW3	117.49	0.07103
STORY1	SW4	68.16	0.04121
STORY1	SW5	355.94	0.2152
STORY1	SW6	507.18	0.30664

STORY1	SW7	13.99	0.00846
STORY1	SW8	-140.02	0.08466
STORY1	SW9	42.96	0.02597
STORY1	SW10	83.08	0.05023
	Total	1653.99	

C.4 Clinic Quake Stiffness Tables

Story	Pier	Shear	$K_{relative}$
STORY8	SW2	2.68	0.11507
STORY8	SW3	20.61	0.88493
STORY8	SW7	-11.66	0.50064
STORY8	SW8	-31.97	1.37269
STORY8	SW9	43.63	1.87334
	Total	23.29	

Story	Pier	Shear	$K_{relative}$
STORY7	SW2	38.97	0.52956
STORY7	SW3	-4.28	0.05816
STORY7	SW5	38.9	0.5286
STORY7	SW6	0	0
STORY7	SW7	6.07	0.08248
STORY7	SW8	-47.47	0.64506
STORY7	SW9	15.04	0.20438
STORY7	SW10	26.36	0.3582
	Total	73.59	

Story	Pier	Shear	$K_{relative}$
STORY6	SW2	93.14	0.47138
STORY6	SW3	-1.9	0.00962
STORY6	SW4	1.94	0.00982
STORY6	SW5	27.07	0.137
STORY6	SW6	77.34	0.39142
STORY6	SW7	20.33	0.10289
STORY6	SW8	-42.94	0.21732

STORY6	SW9	7.42	0.03755
STORY6	SW10	15.19	0.07688
	Total	197.59	

Story	Pier	Shear	$K_{relative}$
STORY5	SW2	127.51	0.42418
STORY5	SW3	7.06	0.02349
STORY5	SW4	3.87	0.01287
STORY5	SW5	46.86	0.15589
STORY5	SW6	115.3	0.38357
STORY5	SW7	15.75	0.0524
STORY5	SW8	-54.57	0.18154
STORY5	SW9	13.83	0.04601
STORY5	SW10	24.99	0.08313
	Total	300.6	

Story	Pier	Shear	$K_{relative}$
STORY4	SW2	156.38	0.40885
STORY4	SW3	11.28	0.02949
STORY4	SW4	4.93	0.01289
STORY4	SW5	68.38	0.17878
STORY4	SW6	141.52	0.37
STORY4	SW7	10.82	0.02829
STORY4	SW8	-66.94	0.17501
STORY4	SW9	20.02	0.05234
STORY4	SW10	36.1	0.09438
	Total	382.49	

Story	Pier	Shear	$K_{relative}$
STORY3	SW1	0	0
STORY3	SW2	182.51	0.4108
STORY3	SW3	10.18	0.02291
STORY3	SW4	2.54	0.00572

STORY3	SW5	86.36	0.19438
STORY3	SW6	162.7	0.36621
STORY3	SW7	7.98	0.01796
STORY3	SW8	-73.55	0.16555
STORY3	SW9	23.21	0.05224
STORY3	SW10	42.35	0.09532
	Total	444.28	

Story	Pier	Shear	$K_{relative}$
STORY2	SW1	-99.71	0.20505
STORY2	SW2	303.55	0.62423
STORY2	SW3	28.78	0.05918
STORY2	SW4	12.96	0.02665
STORY2	SW5	100.31	0.20628
STORY2	SW6	140.39	0.2887
STORY2	SW7	17.45	0.03588
STORY2	SW8	-56.01	0.11518
STORY2	SW9	17.79	0.03658
STORY2	SW10	20.77	0.04271
	Total	486.28	

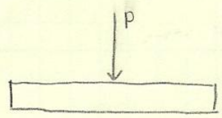
Story	Pier	Shear	$K_{relative}$
STORY1	SW1	-61.81	0.12122
STORY1	SW2	253.19	0.49656
STORY1	SW3	35.91	0.07043
STORY1	SW4	20.47	0.04015
STORY1	SW5	108.66	0.2131
STORY1	SW6	153.47	0.30099
STORY1	SW7	6.17	0.0121
STORY1	SW8	-45.67	0.08957
STORY1	SW9	15.05	0.02952
STORY1	SW10	24.45	0.04795
	Total	509.89	

Appendix D: Foundation Model Check

D.1 Hand Calculations

Caitlin Behm | AE Senior Thesis | Foundation Model | 1/1

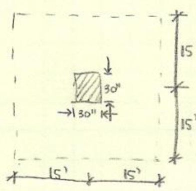
Fixed / Pinned Foundation Determination



P/A ≈ soil bearing pressure , pinned base
P/A << soil bearing pressure , fixed base

• also, consider anchor bolts sizes & rebar details

Typical Interior Column (Both Hospital & Clinic):



Trib Area = $900 ft^2$
Floors 1-6
SDL = 12 psf (typical floor)
DL = self weight of concrete = 145 pcf (12"/1') = 145 psf
LL = 80 psf (servery + storage)

Floor 7 (PH)
SDL = 12 psf (typical floor)
DL = 62 psf (mechanical)
LL = 150 psf (mechanical)

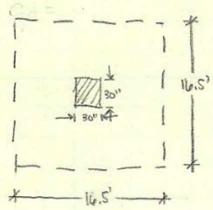
Floor 8 (roof)
SDL = 24 psf (typical roof)
DL = 75 psf (deck weight)
LL = 20 psf (typical roof)

$W_u = 1.2 W_D + 1.6 W_L$
$W_u = 1.2 [145 psf + 12 psf] + 1.6 [80 psf]$
$W_u = 316.4 psf$ ← typical for floors 1-6

$W_u = 1.2 W_D + 1.6 W_L$
$W_u = 1.2 [12 psf + 62 psf] + 1.6 [150 psf]$
$W_u = 328.8 psf$ ← typical for floor 7

$W_u = 1.2 W_D + 1.6 W_L$
$W_u = 1.2 [24 psf + 75 psf] + 1.6 [20 psf]$
$W_u = 150.8 psf$ ← typical for floor 8

$P_{u@\,foundation} = 6 [(900 ft^2)(316.4 psf)] + [(900 ft^2)(328.8 psf)] + [(900 ft^2)(150.8 psf)]$
$= 2140 K$



Footing Area = $272.25 ft^2$

$P/A = 2{,}140{,}000 lbs / 272.25 ft^2 = 7860.4 psf$
$q_a = 8000 psf$

$P/A \approx q_a$, $\boxed{\text{pinned base}}$

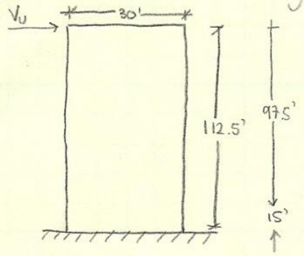
anchor bolts : #5 @ 12"
reinforcement : 12 #9

Appendix E: Member Check

E.1 Hand Calculations

Caitlin Behm | AE Senior Thesis | Spot check | 1/1

Shear Wall Strength Check (Typical)



Given: $h = 14"$ thick
$f'c = 5{,}000$ psi
$f_y = 60{,}000$ psi

Hospital SW19

Check maximum permitted strength:

$V_u < \phi V_n = \phi 10 \sqrt{f'c}\, hd$

$d = 0.8(30\times12) = 288"$

$\phi V_n = 0.75(10)\sqrt{5{,}000}\,(14")(288")/1000 = 2138.3K$

$V_u < \phi V_c$ ✓

$V_u = 50^K$ ← From ETABS

critical section
$a \le l_w/2 = 30/2 = 15'$ ← governs
$\le h_w/2 = 112.5/2 = 56.25'$

Shear strength provided by V_c:

$V_c = 2\sqrt{f'c}\, hd = 2\sqrt{5000}(14")(288")/1000 = 570.2K$

$V_c \le 3.3\sqrt{f'c}\, hd + N_u d / l_w$ $\quad N_u = 0$ b/c no axial

$3.3\sqrt{5000}\,(14")(288") + 0 = 940.8K$ ✓

$V_c \le \left[0.6\sqrt{f'c} + \left[l_w(1.25\sqrt{f'c} + 0.2\, N_u/l_w h)\right]/(M_u/V_u - l_w/2)\right] hd$ $\quad M_u = 97.5' \times V_u \times 12 = 58500$ in·k

$0.6\sqrt{5000} + [30\times12(1.25\sqrt{5000})/(58500/50 - 30\times12/2)]](14")(288")$

$V_c = 42.4^K$

Deflection Check

$\Delta_{cantilever} = \Delta_c = \Delta_{shear}$

$\Delta_{fixed} = \Delta_f = \Delta_{flexure}$

$\Delta_c = Ph^3/3E_m I + 1.2Ph/E_v A$

$\Delta_c = (50k)(112.5\times12)^3/3(57000\sqrt{5000})(2.7E7) +$
$(1.2)(50k)(112.5\times12)/(1.61E6)(4032)$

$\Delta_c = 0.39$ in

$A = t\cdot d = (14")(288") = 4032 in^2$
$I = t\cdot d^3/12 = (14")(288")^3/12 = 2.7E7$
$E_v = 0.4(57000\sqrt{5000}) = 1.61E6$

$\Delta_f = Ph^3/12E_m I + 1.2Ph/E_v A$

$(50k)(112.5\times12)^3/12(57000\sqrt{5000})(2.7E7) +$
$(1.2)(50k)(112.5\times12)/(1.61E6)(4032)$

$\Delta_f = 0.11$ in

$\Delta_c + \Delta_f = \Delta$

$0.39 + 0.11 = 0.5"$ ✓

$l/400 = 112.5'\times12/400 = 3.375"$