Technical Report I

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

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Structural Option

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Executive Summary:

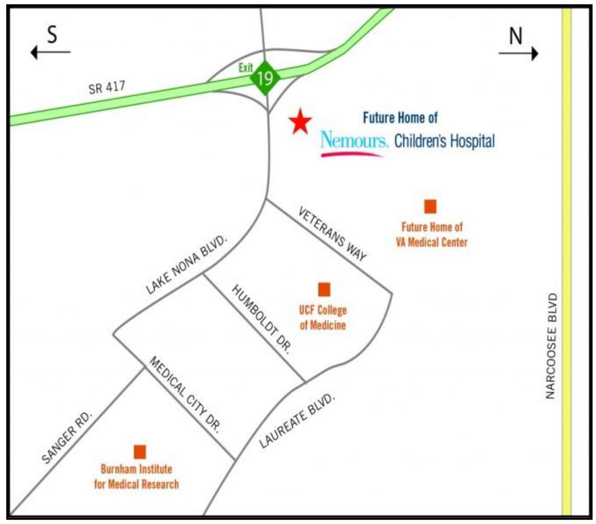
The following senior thesis technical report gives an overview of the structural systems and design procedures for NCHTNF. The structural engineer, Simpson, Gumpertz & Heger, provided all drawings and photographs with the permission of The Nemours Foundation. The analysis begins with studying the existing 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. The wind analysis is performed in both directions to determine a base shear of 2030 k in the North-South direction and 1100 k in the East-West direction for the hospital. The clinic has a base shear of 1740 k in the North-South direction and 657 k in the East-West direction. The overturning moments are found to be 274,000 k-ft and 149,000 k-ft in the North-South and East-West direction respectively for the hospital. The clinic has 235,000 k-ft and 88,700 k-ft in the North-South and East-West direction respectively. The seismic forces are calculated to produce a base shear of 1,510 k and an overturning moment of 111,000 k-ft for the hospital. The clinic seismic forces are calculated to produce a base shear of 497 k and an overturning moment of 39,100 k-ft. After analyzing the data, the conclusion is wind controls the design of NCHTNF.

Spot checks confirm the structural adequacy of the various structural systems. Analysis of the two-way flat slab with drop panels, a typical column, and an end slab condition lead to the conclusion of structural adequacy of the building.

Building Introduction:



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.

The design of this $400 million building uses 2007 Florida Building Code with 2009 updates. The Florida Building Code is based off of 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 sits on top of 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 entirety of the 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.

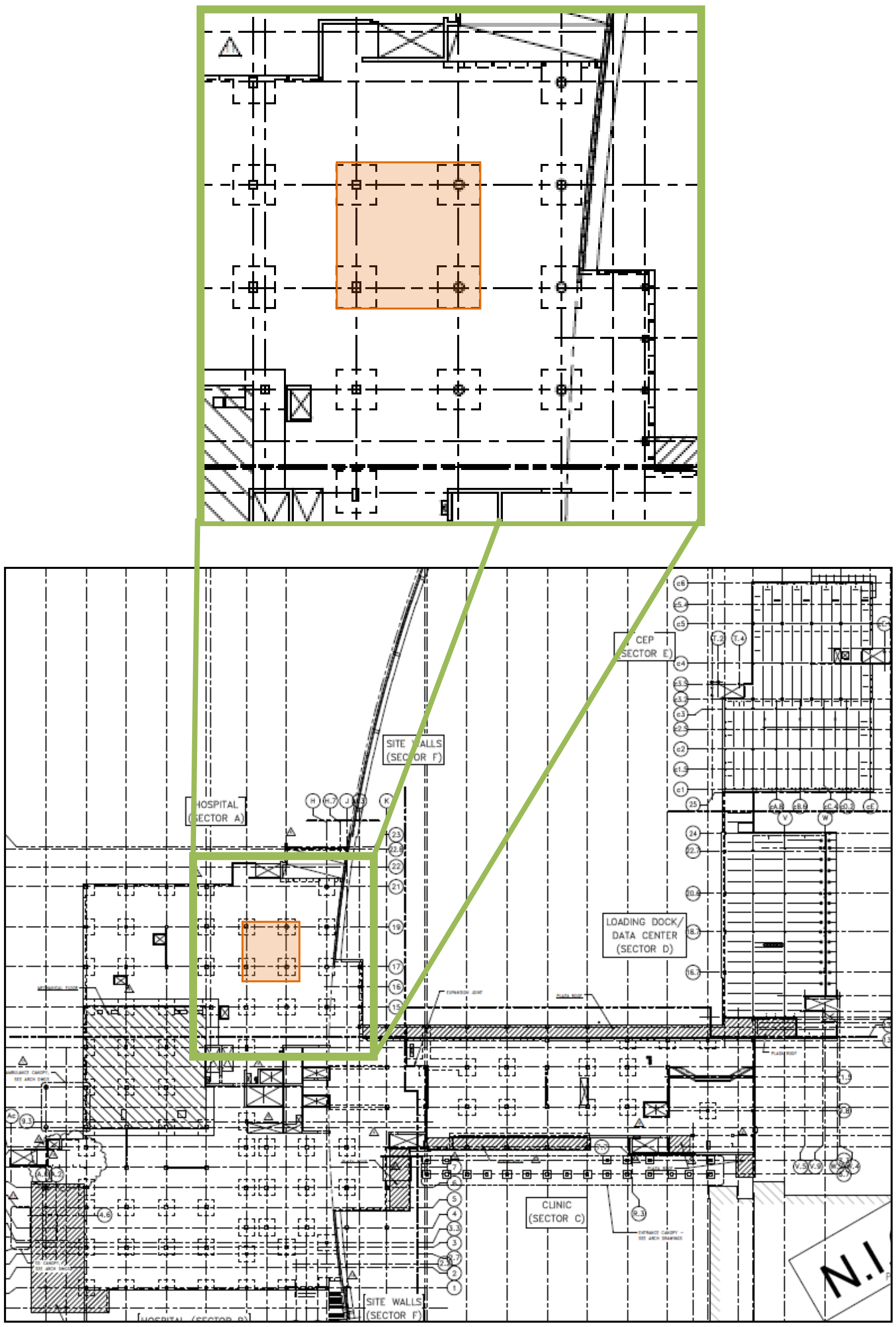
PSI recommends utilizing shallow foundations only if the foundation design implements soil improvement to increase the allowable bearing capacity of the design. PSI proposes another foundation solution, if soil improvement is not desirable, implement a pile foundation system. These reinforced augercast 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 NCHTNF.

Due to 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 2-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 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.

Framing System:

The columns supporting 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. HSS8x8 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 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.

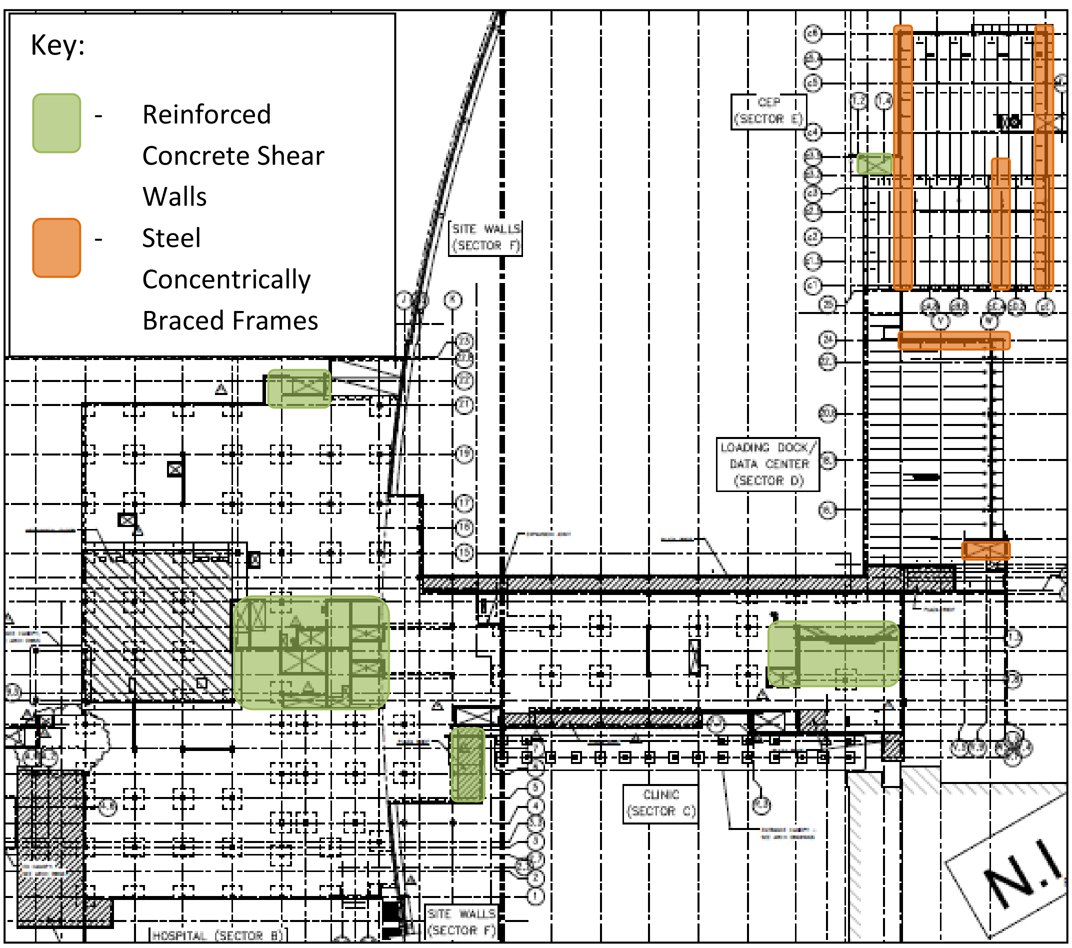


Figure 3 – Level 1 Structural Floor Plan Highlighting the Lateral System.

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

**Note: The 2007 Florida Building Code is based off of 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:

The chart below 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	$f'_m = 1.8$

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
	Café Portal Roof	45
	Entry Portal	45
	Ed Low Roof	45
	Clinic Roof Wing	189
	Stitch Roof	20

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

**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.

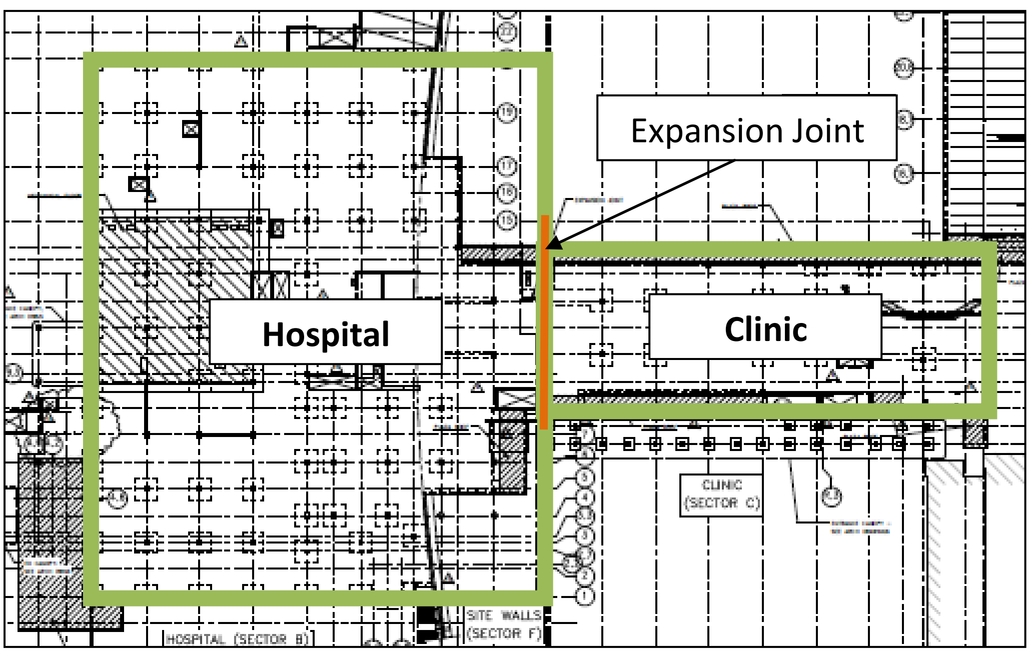


Figure 4 – Generalized Geometry for Wind Analysis.

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.

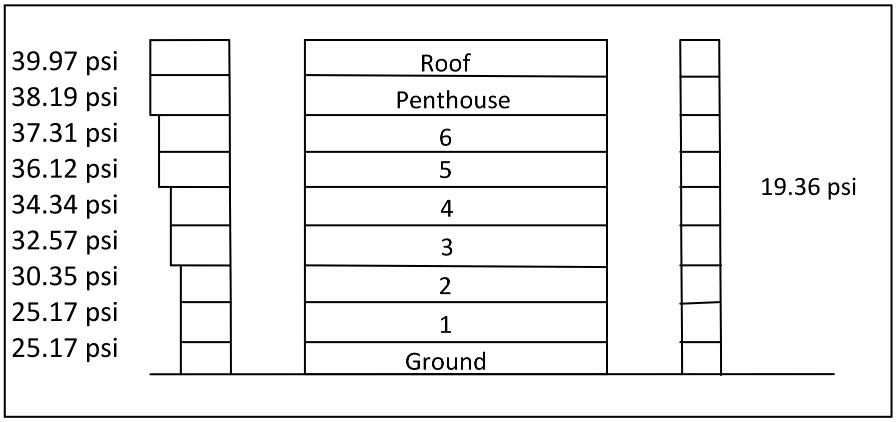


Figure 5 – Wind Pressures Vertical Distribution, North-South Direction

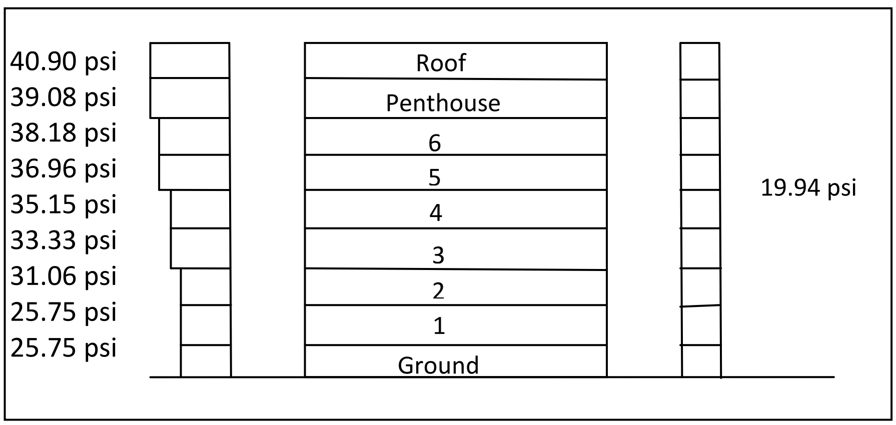


Figure 6 – Wind Pressures Vertical Distribution, East-West Direction

Seismic Load:

The seismic analysis follows chapters 11 and 12 in ASCE 7-05 to determine the seismic load on NCHTNF. 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.

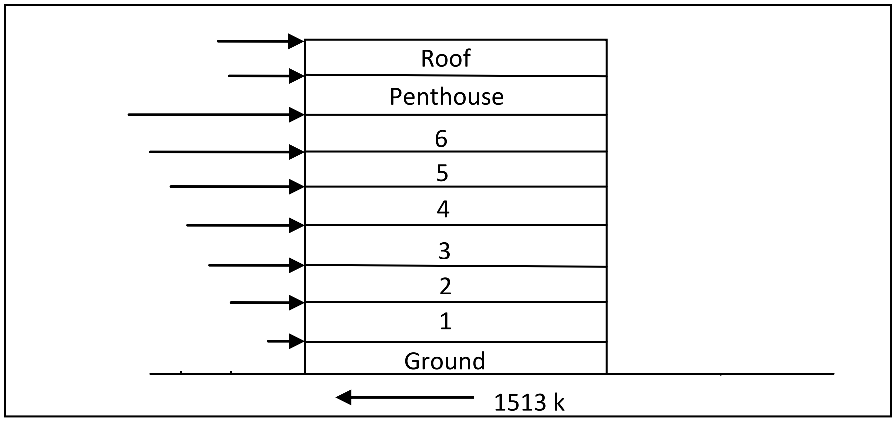


Figure 7 – Seismic Story Forces

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.

Gravity Load Spot Checks:

Spot checks are performed on a typical bay of a typical floor to better understand the design of the structural system. Columns E19, E17, F17, and F19 are the corners of the bay analyzed on level one of NCHTNF. Column E17 on level one is analyzed as one of the typical columns in the building. The torsional stiffness is studied at the end of slab condition on level one as well. Hand calculations are found in Appendix C.

Slab:

NCHTNF utilizes two-way elevated flat slab with drop panels. The analyzed bay on level one has 12" concrete with 12'x12'x6-1/4" drop panels. #6 bars spaced 12" run across the top of the slab while #5 bars spaced 12"run across the bottom of the slab, see Appendix C for a drawing of the rebar layout. The slab does not fail in wide-beam action or in punching shear around the drop panel, but it does fail in punching shear around the column. This is explained by an assumption for the calculation of b_o, which elevates the value of V_c.

Column:

Column E17 is analyzed because it represents one of the typical columns in the building, for size and loading patterns. After a column interaction diagram is created, the loading on the column is summed for floors above the column. These detailed calculations can be found in Appendix C. The column's axial and flexural loadings fit inside the created interaction diagram, proving the column's adequate design.

End of Slab Condition:
The torsional stiffness is calculated at the end of slab condition in NCHTNF. The slab does not have edge beams, so the analysis ends with the determination of the equivalent stiffness of the slab due to the combined effects of the end slab and surrounding columns.

Conclusion:

A more detailed understanding of the structural system of NCHTNF is gained after performing the analyses outlined in this technical report. The various calculations, wind, seismic, and gravity spot checks, prove the existing structural system adequately supports the loads the building requires.

The wind analysis values are much higher due to an over estimate in building geometry and the utilization of Method 2 from ASCE 7-05, instead of Method 3. Wind does control the design, but just not at that high of a value calculated for this report. Changing the basic wind speed from 157mph to the code required 110mph could be interesting to see cost savings.

The slab appears to be adequately designed to carry the dead and live loads of the building. Analysis of alternative slab designs will be explored in Tech II. The columns are extremely over designed seeing as there is great difference between the axial and flexural loads and full capacity. These over designs might be explained by the future plan of expansion or change in occupancy space.

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

Caitlin Behm	AE Senior Thesis	Wind Calcs	1/3

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 $f > 1\,hz$

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)

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

$\therefore$ 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)

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

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

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

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

$g_R = 4.122$

Caitlin Behm | AE Senior Thesis | Wind Calcs | 2/3

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

*** Assume Damping Ratio (β) = 1.0

$R_n = R_\ell$ 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)

$V_{\bar{z}} = 0.65 (81/33)^{(1/9)}\, 157 (88/60)$ — $\bar{\alpha} = 1/9$ (Table 6-2)

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

$N_1 = 0.756 (598.36) / 165.376$

$N_1 = 2.74$

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

$R_n = 0.074$

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

$\eta = 4.6 n_1$

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

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

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

$\eta = 2.84$

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

$R_h = 0.290$

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

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

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

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

$\eta = 9.781$

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

$R_{B\,NS} = 0.097$

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

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

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

$\eta = 6.309$

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

$R_{B\,EW} = 0.146$

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

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

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

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

$\eta = 21.122$

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

$R_{L\,NS} = 0.044$

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

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

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

$\eta = 32.736$

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

$R_{L\,EW} = 0.030$

Caitlin Behm	AE Senior Thesis	Wind Calcs	3/3

$$R_{NS} = \sqrt{(1/\beta) R_n R_h R_B (0.53 + 0.47 R_L)}$$
$$R_{NS} = \sqrt{(0.074)(0.290)(0.097)(0.53 + 0.47(0.040))}$$
$$R_{NS} = 0.034$$
$$R_{EW} = \sqrt{(1/\beta) R_n R_h R_B (0.53 + 0.47 R_L)}$$
$$R_{EW} = \sqrt{(0.074)(0.290)(0.149)(0.53 + 0.47(0.030))}$$
$$R_{EW} = 0.041$$

$$G_{f_{NS}} = 0.925([1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}] / [1 + 1.7 g_v I_{\bar{z}}])$$
$$G_{f_{NS}} = 0.925([1 + 1.7(0.1722)\sqrt{3.4^2 (0.783)^2 + 4.122^2 (0.034)^2}] / [1 + 1.7(3.4)(0.1722)])$$
$$\boxed{G_{f_{NS}} = 0.825}$$
$$G_{f_{EW}} = 0.925([1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}] / [1 + 1.7 g_v I_{\bar{z}}])$$
$$G_{f_{EW}} = 0.925([1 + 1.7(0.1722)\sqrt{3.4^2 (0.812)^2 + 4.122^2 (0.041)^2}] / [1 + 1.7(3.4)(0.1722)])$$
$$\boxed{G_{f_{EW}} = 0.839}$$

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

Caitlin Behm | AE Senior Thesis | Seismic Calcs | 1/1

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

$S_{ms} = F_a S_s$

$S_{ms} = (1.6)(0.096) = 0.15$

$S_{m1} = F_v S_1$

$S_{m1} = (2.4)(0.038) = 0.09$

$F_a = 1.6$ (Table 11.4-1)

$F_v = 2.4$ (Table 11.4-2)

$S_{DS} = 2/3\, S_{ms}$

$S_{DS} = 2/3(0.15) = 0.10$

$S_{D1} = 2/3\, S_{m1}$

$S_{D1} = 2/3(0.09) = 0.06$

$T_0 = 0.2(S_{D1}/S_{DS})$

$T_0 = 0.2(0.06/0.10) = 0.12$

$T_s = S_{D1}/S_{DS}$

$T_s = 0.06/0.10 = 0.6$

$T_L = 8s$ (figure 22-15)

$T_s > T_0$

hospitals & other healthcare facilities

Occupancy category = IV (Table 1-1)

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: Gravity Spot Checks

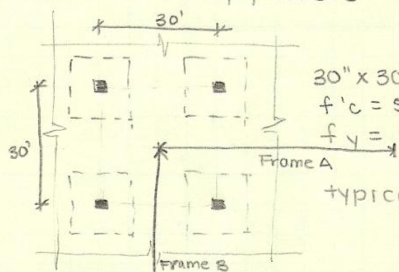
C.1 Hand Calculations

Caitlin Behm | AE Senior Thesis | Gravity Spot Checks | 1/8

ACI 318.08

$\boxed{\text{Slab check}}$

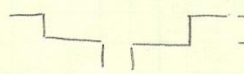
12" NW concrete elevated two-way flat slab w/ 6 1/4" drop panels



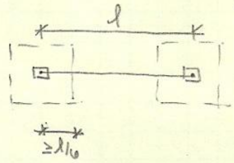
30" x 30" columns
$f'_c = 5{,}000$ psi
$f_y = 60{,}000$ psi

typical interior bay

ACI 13.2.5 when reducing the amount of negative moment reinforcement over a column or minimum required slab thickness, a drop panel shall

a) 

$a \geq 1/4$ adjacent slab thickness
$a \geq 1/4\,(12") = 3"$
$a = 6\,1/4"$ ✓

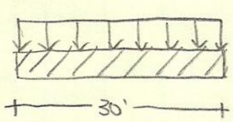
b) 

$l = 30'$
$l/6 = 5' \rightarrow$ panels project out 6' ✓

∴ Drop panel → thickness of drop panel below slab shall not be assumed to be greater than 1/4 the distance from edge of drop panel to face of column.

For a generalized gravity spot check Direct Design Method will be used even though not all the categories requirements were met.

SDL = 12 psf (typical floor) → see building load section
DL = self weight of concrete
LL = 125 psf (service → storage) → see building load section



$W_u = 1.2\,W_D + 1.6\,W_L$
$W_u = 1.2\,[(12"/12")(145\text{ pcf}) + 12\text{ psf}] + 1.6\,[125\text{ psf}]$
$W_u = 395$ psf

Slab thickness

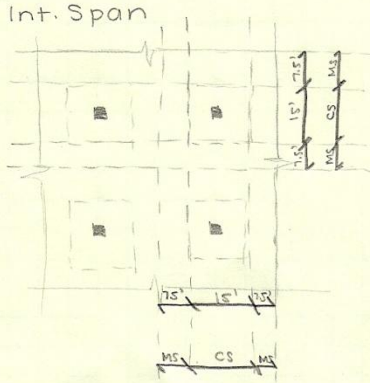
$l_n/36$ (Table 9.5c w/ drop panels Int. panels)
$(30' - 30"/12") \times 12"/36 = 9.17" \Rightarrow 10"$ slab (12" slab used in current design) ✓

Caitlin Behm | AE Senior Thesis | Gravity Spot Check | 2/8

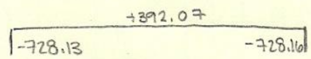
Frame A = Frame B b/c of symmetry

$$M_o = \tfrac{1}{8} w_u \ell_2 \ell_n^2 = \tfrac{1}{8}(0.395\text{ ksf})(30')(30' - 30''/12'')^2 = 1120.2 \text{ ft}\cdot\text{K}$$

Int. Span

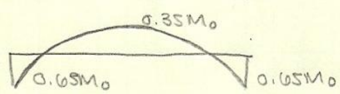


Frame A = frame B

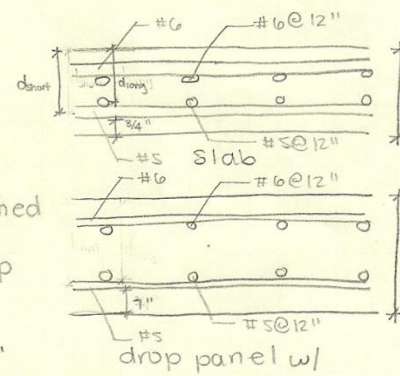


Column strip
$\ell_2/2 = 30'/2 = 15'$

Middle strip
$30' - 15' = 15'/2 = 7.5'$ on each side of CS



Distribution of moments
$\alpha = 0$ b/c no beams
positive moment ⇒ 60%
negative moment ⇒ 75%



$d_{short} = 12 - 3/4 - 0.5(0.625) = \boxed{10.94''}$
$12''\ d_{long} = 12 - 3/4 - 0.625 - 0.5(0.625) = \boxed{10.31''}$

same as above

7" = cover & drop panel depth

*** thickness of drop panel below slab shall not be assumed to be greater than 1/4 the distance from edge of drop panel to face of column.

$\tfrac{1}{4}[(6' \times 12''/1') - 15''] = 14.25''$

(6.25" droppanel used in current design) ✓

Frame A = frame B

Total Moment	-728.13	392.07	-728.13
Moment in CS	-436.88	294.05	-436.88
Moment in MS	-291.25	98.02	-291.25

Total width = 30'
col strip: 15'
mid strip: 7.5'

Caitlin Behm | AE Senior Thesis | Gravity Spot Check | 3/8

Reinforcement Design & Distribution

(frame A Column strip)

Description	Int. Span Neg M.	Int. Span Pos. M
width of strip (b)	180"	180"
effective depth	10.94"	10.94"
$R = M_u/bd^2$	243.35	163.79
ρ (from Table A.5a)	0.0042	0.0028
Ⓐ $A_s = \rho bd$	8.27 in^2	5.51 in^2
Ⓑ $A_{s,min} = 0.0018bt$	5.91 in^2	5.91 in^2
Ⓒ N = larger A or B	$\boxed{8.27}$ in^2	$\boxed{5.91}$ in^2
Ⓓ N_{min} = width strip/2t	4.93 in^2	4.93 in^2
N = (larger C or D)/0.31		

adequate reinforcing exceeding these values. See slab detail

(frame A middle strip)

Description	Int. Span Neg M.	Int. Span Pos. M
width of strip (b)	180"	180"
effective depth	10.94"	10.94"
$R = M_u/bd^2$	162.23	54.60
ρ (from Table A.5a)	0.0027	0.0009
Ⓐ $A_s = \rho bd$	5.32 in^2	1.77 in^2
Ⓑ $A_{s,min} = 0.0018bt$	3.89 in^2	3.89 in^2
Ⓒ N = larger A or B	5.32 in^2	3.89 in^2
Ⓓ N_{min} = width strip/2t	$\boxed{7.5}$ in^2	$\boxed{7.5}$ in^2
N = (larger C or D)/0.31		

adequate reinforcing exceeding these values. see slab detail.

(frame B column strip)

Description	Int. Span Neg M	Int. Span Pos M
width of strip (b)	180"	180"
effective depth	10.31"	10.31"
$R = M_u/bd^2$	274.00	184.42
ρ (from Table A.5a)	0.0047	0.0031
Ⓐ $A_s = \rho bd$	8.72 in^2	5.75 in^2
Ⓑ $A_{s,min} = 0.0018bt$	5.91 in^2	5.91 in^2
Ⓒ N = larger A or B	$\boxed{8.72}$ in^2	$\boxed{5.91}$ in^2
Ⓓ N_{min} = width strip/2t	4.93 in^2	4.93 in^2
N = (larger C or D)/0.31		

adequate reinforcing exceeding these values. see slab detail.

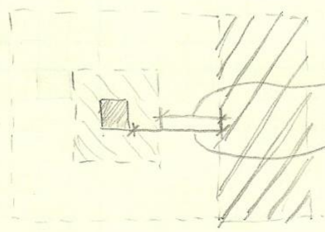
(frame B middle strip)

Description	Int. Span Neg M	Int. Span Pos M
width of strip (b)	180"	180"
effective depth	10.31"	10.31"
$R = M_u/bd^2$	102.67	61.46
ρ (from Table A.5a)	0.0031	0.0010
Ⓐ $A_s = \rho bd$	5.75 in^2	1.86 in^2
Ⓑ $A_{s,min} = 0.0018bt$	3.89 in^2	3.89 in^2
Ⓒ N = larger A or B	5.75 in^2	3.89 in^2
Ⓓ N_{min} = width strip/2t	$\boxed{7.5}$ in^2	$\boxed{7.5}$ in^2
N = (larger C or D)/0.31		

adequate reinforcing exceeding these values. See slab detail.

wide beam action

drop panel = 12' x 12'



$\pi d^2/4 = a^2$
$\sqrt{\pi d^2/4} = a$
$\sqrt{\pi(12')^2/4} = 10.63'$

$15' - 10.63'/2 - 10.63"/12 =$
$= 8.80'$

$V_u = w_u \times 8.80 \times l_2$
$V_u = (0.395)(8.80)(30)$
$V_u = 104.28K$

$\pi d^2/4 = a^2$
$\sqrt{\pi d^2/4} = a$
$\sqrt{\pi(12')^2/4} = 10.63'$

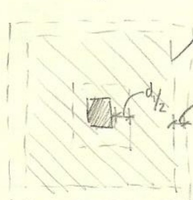
$15' - 10.63'/2 - 67.63"/12 =$
$= 8.80'$

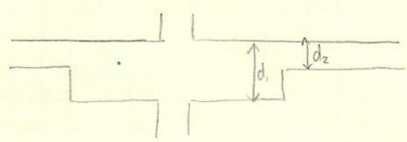
$V_u = w_u \times 8.80 \times l_2$
$V_u = (0.395)(8.80)(30)$
$V_u = 104.28K$

$\phi V_n = 0.6(2\sqrt{5000})(30 \times 12)(10.63)(1/1000)$
$\phi V_n = 324.71K > V_u$ ✓

same in other direction b/c of symmetry.

punching shear





$b_o = 2(d_1/2) + \text{col. width}$
$b_o = 2(18.25/2) + 30 = 48.25$
$4(48.25) = 193"$ (perimeter)

$V_c = 4\sqrt{f'c}\, b_o d$
$V_c = 4\sqrt{5000}(193)(18.25)(1/1000)$
$V_c = 996.24K$

$V_u = W_u \cdot \text{Area}$
$V_u = (0.395)(48.25^2)$
$V_u = 919.58K$

$V_u < V_c$ NG →

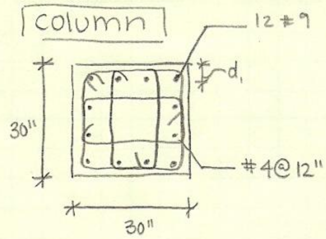
$b_o = 2(d_2/2) + \text{col width}$
$b_o = 2(12/2) + 30 = 42$
$4(42) = 168"$ (perimeter)

$V_c = 4\sqrt{f'c}\, b_o d$
$V_c = 4\sqrt{5000}(193)(12)(1/1000)$
$V_c = 655.06K$

$V_u = W_u \cdot \text{Area}$
$V_u = (0.395)(42^2)$
$V_u = 696.78K$

$V_u > V_c$ ✓

Column



$f'c = 6{,}000$ psi
$f_y = 60{,}000$ psi

$\varepsilon_y = f_y / E_s = 60/29000 = 0.0021$

need column interaction diagram

Axial strength, P_o

$P_o = 0.85 f'c\, A_c + A_s f_y$
$P_o = 0.85(6)(30 \times 30 - 12 \times 1) + (12 \times 1)(60) = \boxed{5248.8\text{K}}$

Balanced - Strain Strength, M_b, P_b

$d_1 = 5"$ (assumption)

$c = \varepsilon_u / \varepsilon_u + \varepsilon_y \cdot (30 - d_1)$
$c = 0.003/(0.003 + 0.0021) \cdot (30 - 5) = 14.71$

$a = \beta_1 \cdot c = (0.75)(14.71) = 11.03$

$\varepsilon_{s1} = 0.003/(14.71) \cdot (14.71 - 5) = 0.0020 < \varepsilon_y$ ✓
$f_{s1} = (0.002)(29000) = 58$ ksi

$\varepsilon_{s2} = 0.003/(14.71) \cdot (14.71 - 15) = -0.00006$
$f_{s2} = (-0.00006)(29000) = -1.72$ ksi

$\varepsilon_{s3} = 0.003/(14.71) \cdot (14.71 - 25) = -0.0021$
$f_{s3} = -60$ ksi

$P_b = (0.85)(6)(30)(0.75)(14.71) + 4(1)(58) + 2(1)(-1.72) + 4(1)(-60)$
$P_b = \boxed{1676.5\text{K}}$
$M_b = (0.85)(6)(30)(0.75)(14.71)\left(15 - \frac{(0.75)(14.71)}{2}\right) + 4(1)(58)(15-5) +$
$2(1)(-1.72)(15-15) + 4(1)(-60)(15-25)$
$M_b = 20728.31$ in·K $= \boxed{1727.36\text{ ft}\cdot\text{K}}$

Pure bending, M_o

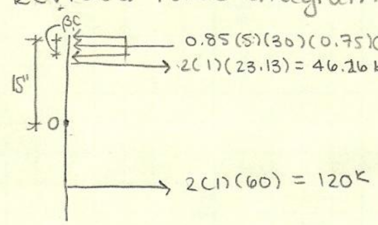
· assume ε_{s_1} does not yield
· assume ε_{s_2} & ε_{s_3} yield

$f_{s_1} = 0.003/c \cdot (c-5)(29000)$
$f_{s_2} = -60$
$f_{s_3} = -60$

$\Sigma F = 0 = 0.85(6)(30)(0.75)c + 4f_{s_1} + 2f_{s_2} + 4f_{s_3}$
$0 = 0.85(6)(30)(0.75)c + 4(1)(0.003/c \cdot (c-5)(29000)) + 2(1)(-60) + 4(1)(-60)$
$0 = 114.75c + 348 - 1740/c - 120 - 240$
$0 = 114.75c - 12 - 1740/c$
$-114.75c^2 + 12c + 1740 = 0$
$c = 3.95$

verify assumptions:
$f_{s_1} = (0.003/3.95)(3.95-5)(29000) = -23.13$ yields NG

Revised force diagram b/c assumption wrong

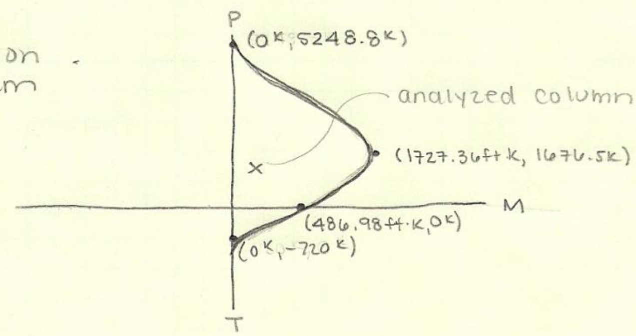


$M_o = 377.72\left(15 - \frac{(0.75)(3.95)}{2}\right) - 46.26(15-5) - 120(15-25)$
$M_o = 5843.70 \text{ in}\cdot k = \boxed{486.98 \text{ ft}\cdot k}$

Pure tension

$T_o = \Sigma A_{si} \cdot f_{si}$
$T_o = 12(1)(-60) = \boxed{-720^k}$

Interaction diagram



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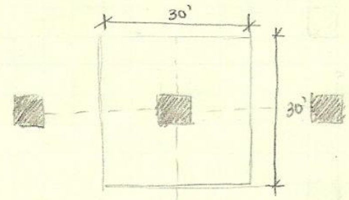
Loading on Column



Live Load Reduction:

$[0.25 + 15/\sqrt{A_i}\,]$ $\quad A_T = 30 \cdot 30 = 900\,ft^2$

$[0.25 + 15/\sqrt{(900)(4)(6)}\,] = 0.35 \quad \therefore$ use 0.4

SDL = 12 psf (typical floor)
DL = self weight of concrete

$P_L = 0.4(125)(900)(6) = 270\,K$

$P_{D_{FL}} = 150pcf \times (12''/12) \times (900) = 135K \times (6) = 810^K$

$P_{D_{FL}} = 12\,(900) = 10.8K \times (6) = 64.8^K$

$P_{D_R} = 20\,(900) = 18\,K$

$P_u = 1.2(810 + 64.8 + 18) + 1.6(270) = \boxed{1503.36K}$

Live Load Reduction

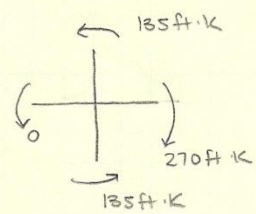
$[0.25 + 15/\sqrt{A_i}\,]$

$[0.25 + 15/\sqrt{2(30)(30)}\,] = 0.60$

$w_{LL} = 0.60(125)(30) = 2.25\,klf$

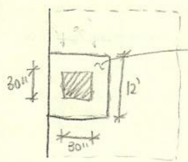
$FEM = \frac{2.25(30)^2}{12}(1.6) = 270\,ft \cdot K$

$\boxed{M_u = 135\,ft \cdot K}$



Column P_u, M_u is within column interaction diagram $\therefore$ it is adequate.

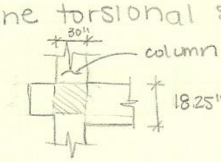
Edge Slab Condition



drop panel

12" concrete slab
6 1/4" drop panels

Determine torsional stiffness of slab



Torsional coefficient

$C = (1 - 0.63(18.25/30))((18.25)^3(30)/3) = 37488.47$

Stiffness of slab

$K_t = 2[9E_{cs}C/\ell_2(1 - c_2/\ell_2)^3$

$K_t = 2[9E_{cs}(37488.47)/[(30 \times 12)[1 - 30/(30 \times 12)]^3]$

$K_t = 2433.51 E_c$

Stiffness of exterior column

$I_c = (30)(30)^3/12 = 67500 \text{ in}^4$

$K_c = 4E_cI_c/L - 2(\text{slab thickness})$

$K_c = 4E_c(67500)/(264 - (12 + 18.25))$

$K_c = 1155.08 E_c$

columns flanking slab

$1/K_{ec} = 1/\Sigma K_c + 1/K_t$

$1/K_{ec} = 1/2(1155.08E_c) + 1/2433.51E_c$

$1/K_{ec} = 0.000844/E_c$

$E_c = 0.000844 K_{ec}$

$\boxed{K_{ec} = 1185.12 E_c}$ ← torsional stiffness of slab

*** this was calculated because the flat slab system does not have beams or girders