Technical Report II

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

Nemours Children's Hospital
As a part of The Nemours Foundation | Orlando, Florida

Caitlin Behm

Structural Option

Advisor: Dr. Boothby

10.19.11

Table of Contents

Executive Summary:

The objective of Technical Report II is to analyze alternate floor systems and compare them to the existing floor system of 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.
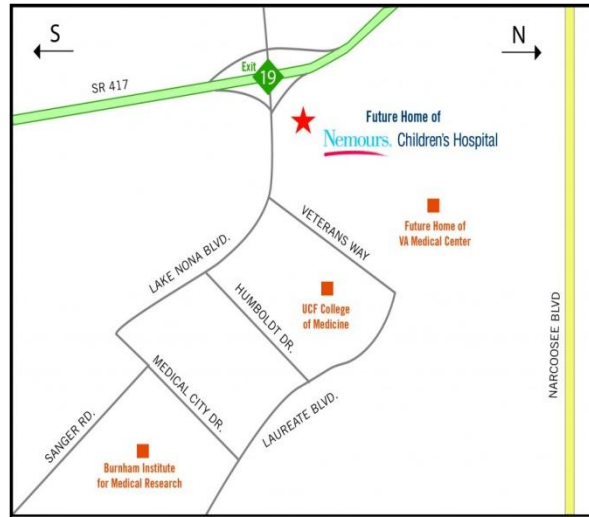
Gravity loads from ASCE 7-05 are used to determine the wind and seismic loads for NCHTNF. The building's geometry is regularized, so proper analysis of these loads can be completed as outlined in ASCE 7-05. NCHTNF is analyzed and modeled as two separate structures because of an expansion joint running through the building. The two structures will be called hospital and clinic. 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 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.

NCHTNF, is constructed with a two-way flat slab with drop panels. The three alternate systems are as follows: pre-cast hollow core planks on steel beams, steel deck on steel beams and girders, and one-way slab with continuous T-beams. Detailed calculations for each system can be found in the Appendix section and individual synopses of each system can be found starting on pg. 17.

A comparison of the systems can be found on pg. 22 of the report. The systems are compared based on categories concerning the feasibility of the construction. In conclusion, it is determined that the two-way and one-way slabs are the most feasible floor systems. These systems are only analyzed using gravity loads, so lateral analysis will need to be performed to analyze which of the two final floor systems is the most beneficial to the design.

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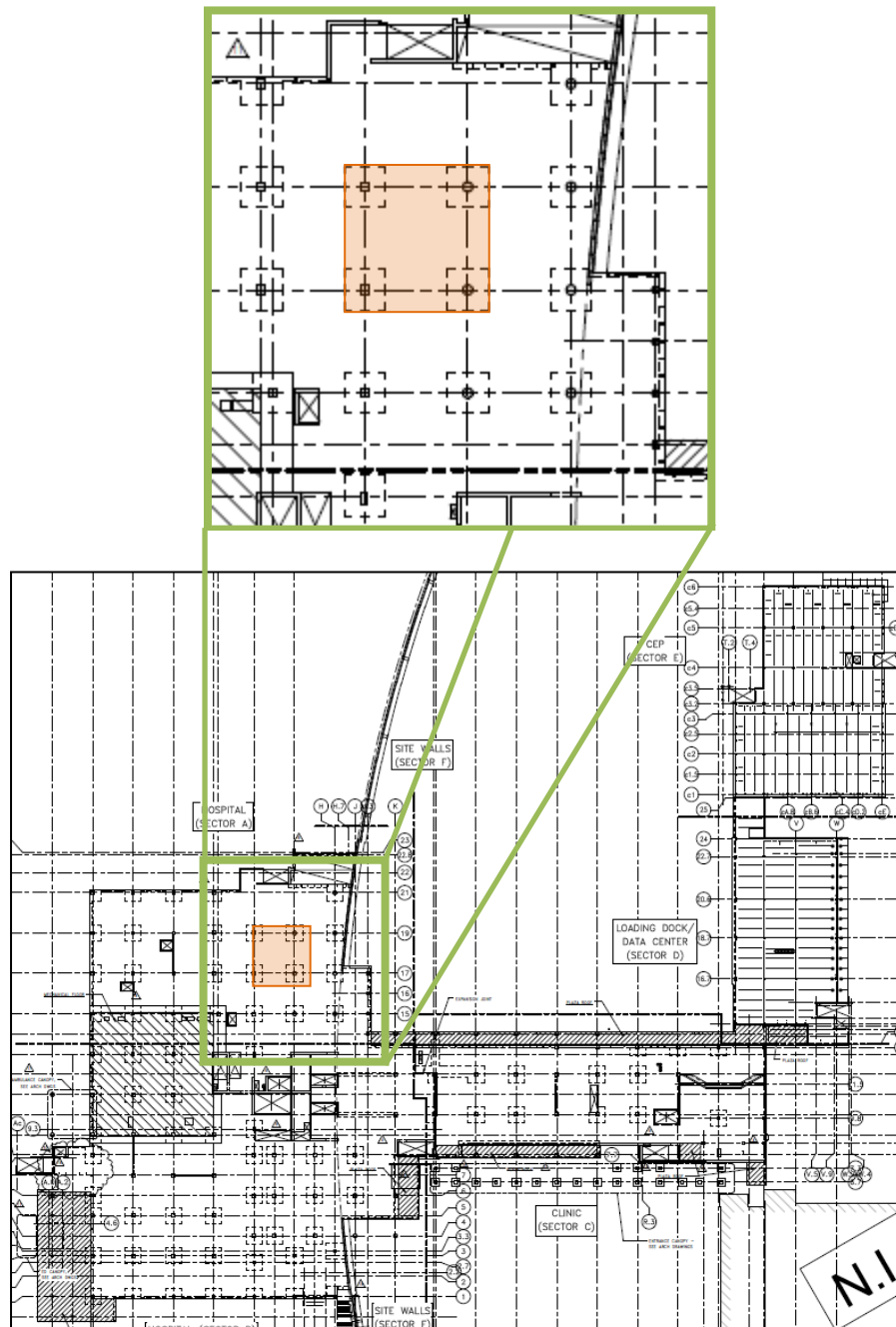
Figures 1 & 2 – Level 1 Typical Structural Bay (30'x30') with Key Plan. Courtesy SGH.

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.

Key:

- Reinforced Concrete Shear Walls
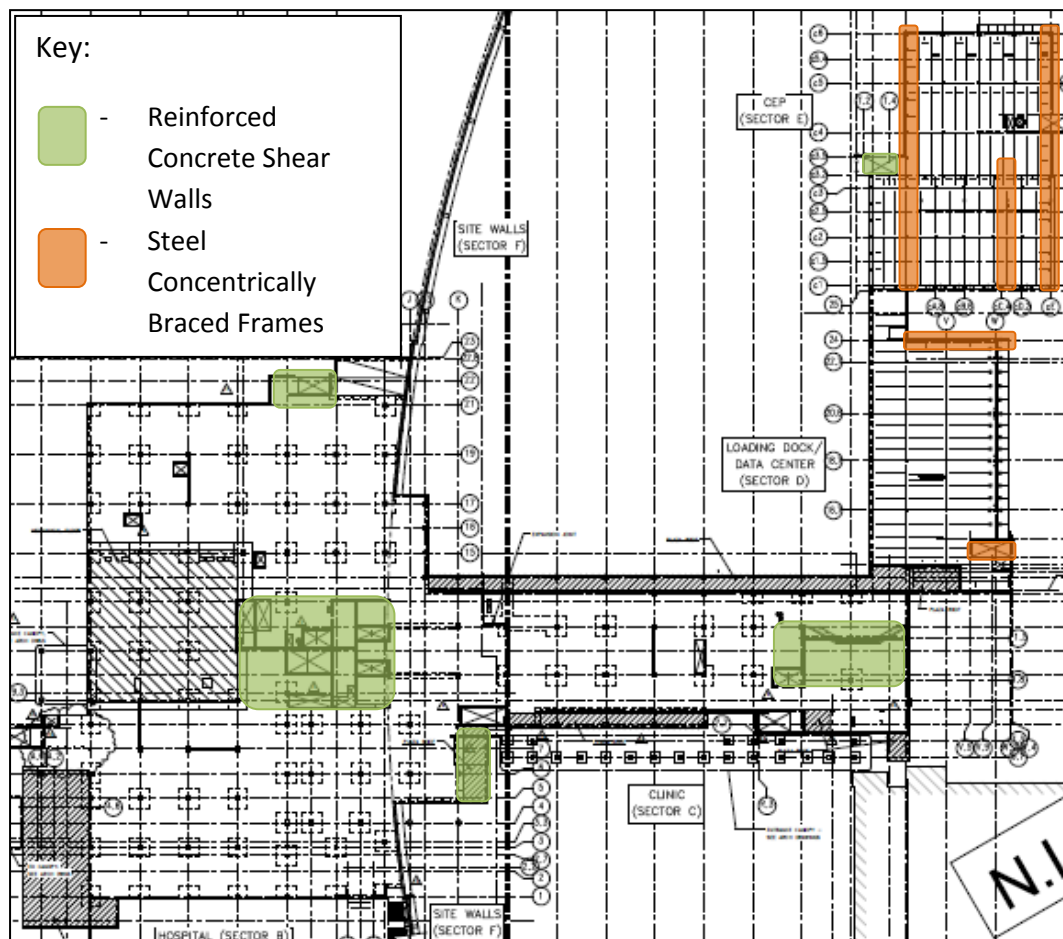- Steel Concentrically Braced Frames

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
ACI	302 – Concrete Floor and Slab Construction
ACI	318 – General Design of Reinforced Concrete Not Otherwise Specified

Table 1 – Design Codes

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:

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
	Café Portal Roof	45
	Entry Portal	45
	Ed Low Roof	45
	Clinic Roof Wing	189
	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.
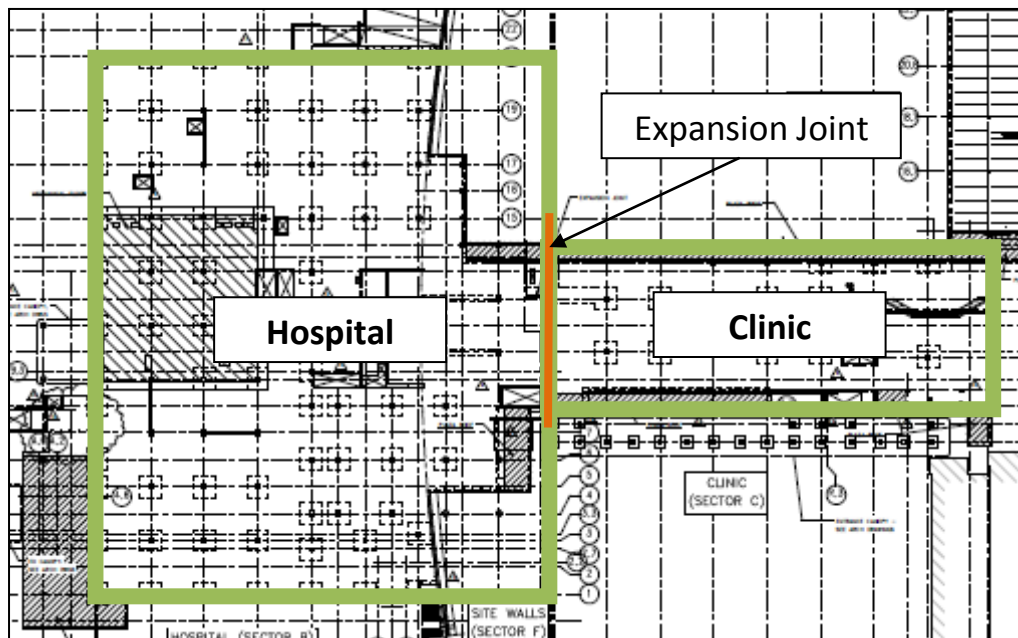

Figure 4 – Generalized Geometry for Wind Analysis. Courtesy SGH.

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.
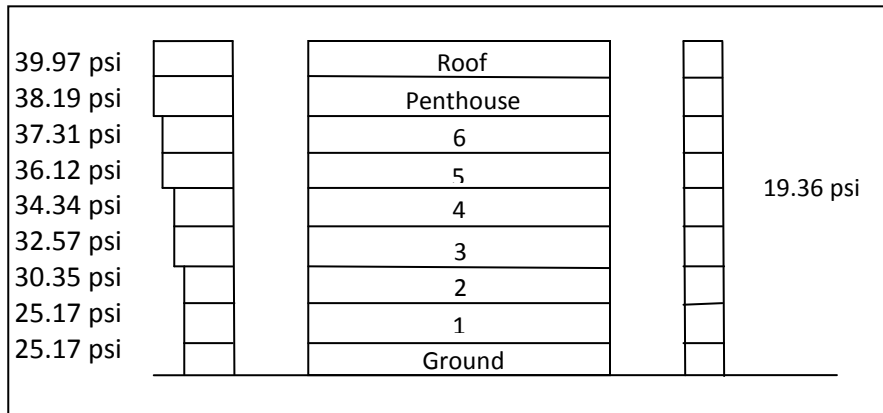
39.97 psi	Roof	
38.19 psi	Penthouse	
37.31 psi	6	
36.12 psi	5	19.36 psi
34.34 psi	4	
32.57 psi	3	
30.35 psi	2	
25.17 psi	1	
25.17 psi	Ground	

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

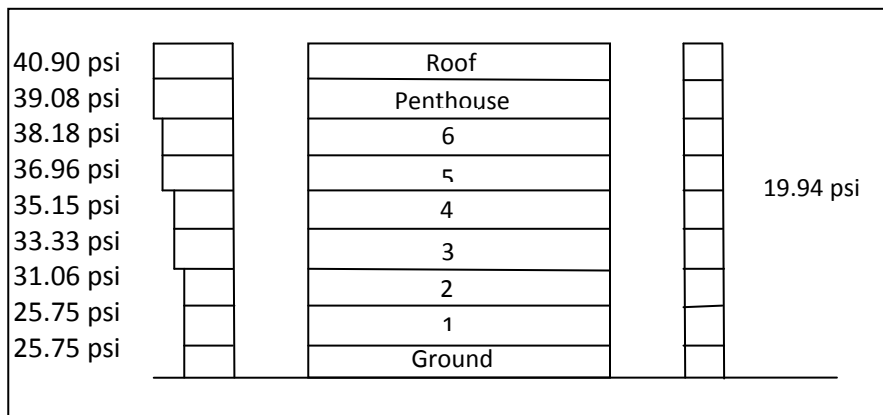|		
40.90 psi	Roof	
39.08 psi	Penthouse	
38.18 psi	6	
36.96 psi	5	19.94 psi
35.15 psi	4	
33.33 psi	3	
31.06 psi	2	
25.75 psi	1	
25.75 psi	Ground	

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.
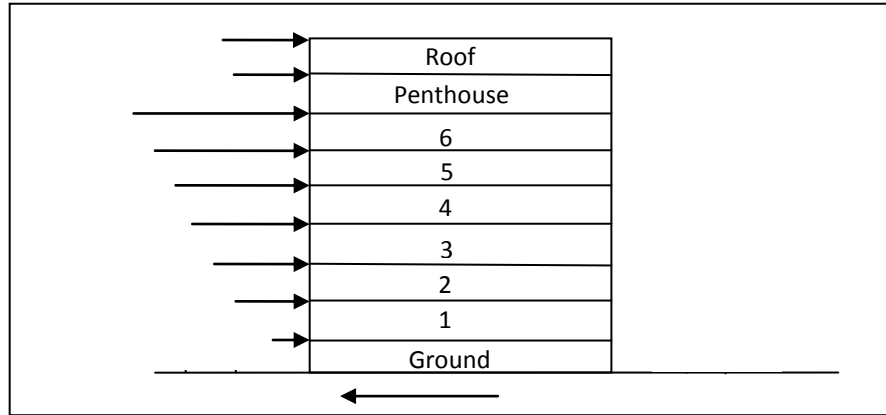
Figure 7 – Seismic Story Forces

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.

Analysis of Floor Systems:

This analysis compares the existing floor system to three alternative floor systems. NCHTNF is designed using a two-way flat slab with drop panels. The three alternative floor systems include: pre-cast hollow core planks on steel beams, steel deck with steel beams and girders, and one-way slab with continuous T-beams. The typical 30'x30' interior bay that is analyzed for each floor system is shown in Figures 8 and 9.
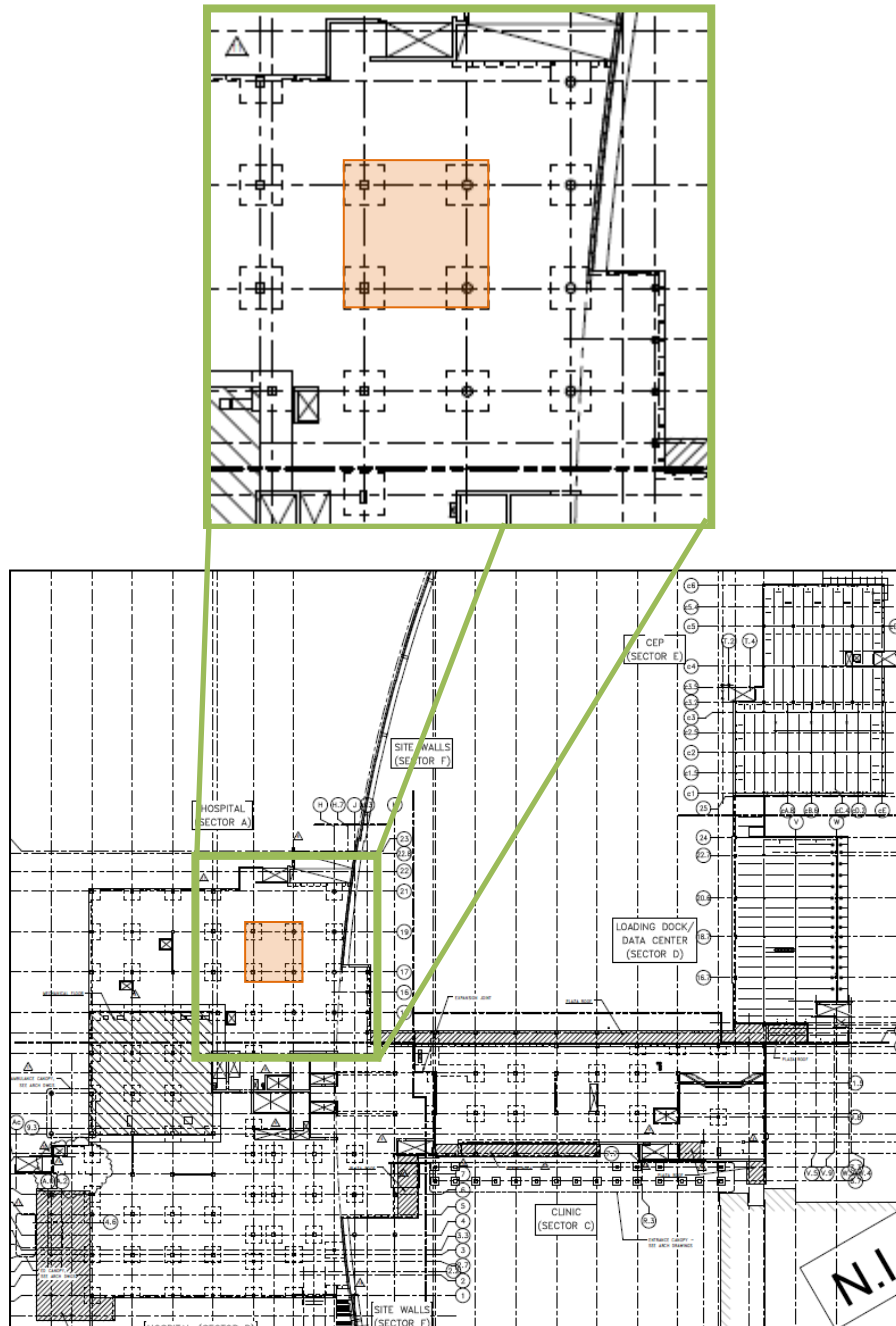
Figures 8 & 9 – Level 1 Typical Structural Bay (30'x30') with Key Plan. Courtesy SGH.

Note: Gravity loads are the only loads used to analyze the floor systems. Additional considerations and calculations will have to be taken into account for lateral forces, which is not part of the scope of this report. All hand calculations for the analyses can be found in the Appendix section of this report.

Two-Way Flat Slab with Drop Panels

Description:
The existing system is a 12" concrete two-way flat slab with drop panels. The drop panels sit 6-1/4" below the 12" slab depth and span 12'x12'. The doubly reinforced slab has #6 bars spaced 12" O.C. on the top and #5 bars spaced 12" O.C. for the bottom reinforcement. The localized slab over the columns requires an additional (7) #8 bars spaced 6" O.C. on the top in the North-South direction and (15) #8 bars spaced 6" O.C. on the top in the East-West direction. Figure 10 shows a typical bay of the system while Figure 11 shows a section cut through the drop panel specific to NCHTNF. Hand calculations can be found in Appendix C.
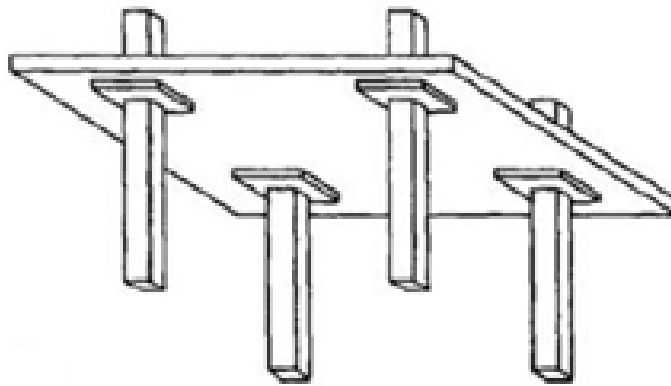

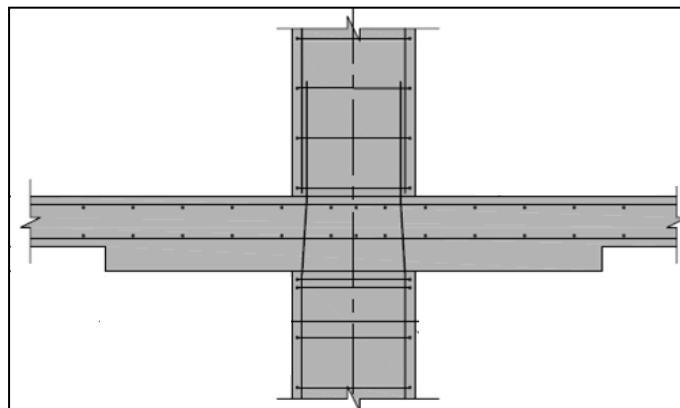Figure 10 - Two-Way Flat Slab with Drop Panels. Courtesy engcastle.com

Figure 11 - Drop Panel Detail (Behm)

Advantages:
The drop panels allow a greater floor-to-floor height because they replace the beams that other systems require as supports, which are usually much deeper than the drop panels. Additionally, the two-way system does not require any fire-proofing because concrete is inherently fire-rated. Also, Orlando is a pro-concrete city, so concrete is readily available with skilled laborers in the surrounding area.

Disadvantages:
The two-way flat slab with drop panels is a heavier system than a steel deck with steel beams and girders system. This will result in larger columns and thicker foundations to support the weight of the floor system, and thus an increase in project cost. Also, the drop panels require formwork and a longer lead time than steel and pre-cast floor systems.

Pre-cast Hollow Core Planks on Steel Beams

Description:
The pre-cast hollow core planks are pre-stressed concrete members that allow longer spans and support higher loads. From the *Nitterhouse Pre-Stressed Catalog*, a 10"x4'-0" module with 7-1/2" diameter strands are used to support the loads across the 30' span. This pre-cast system has an additional 2" topping to provide a 2-hour fire rating, which is required by code. The pre-cast hollow core planks are supported by W24x84 steel beams. These beams carry the load of the planks to the columns without exceeding maximum deflection. Figure 12 shows the section of the pre-cast hollow core plank used in this floor system design. Hand calculations can be found in Appendix D.
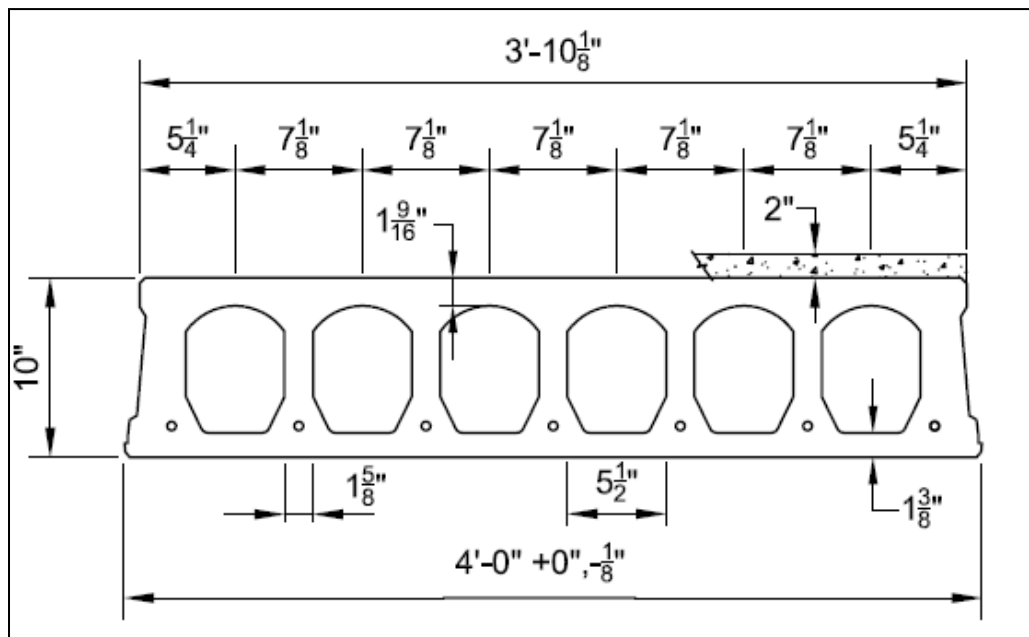

Figure 12 - Pre-cast Hollow Core Plank Section. Courtesy Nitterhouse.

Advantages:
The pre-cast hollow core planks on steel beams are able to span lengths ranging between 16' to 40', which encompasses the typical bay length of 30'. The voids in the pre-cast planks reduce the weight of the system as compared to solid concrete slab systems. The voids also reduce sound and heat transmissions throughout NCHTNF. Additionally, the pre-cast panels will allow the construction process to be accelerated because the planks arrive on site at full strength.

Disadvantages:
A typical fault of the pre-cast hollow core planks is differential cambering. This causes the joints to displace, which leads to long term maintenance issues for the floor system. The column spacing will need to change from 30' to 32' because the pre-cast planks are constructed in 4' modules. Also, NCHTNF has irregular façades that dictate the floor plan layout, the issue being the pre-cast hollow core planks are a regularized size. The planks will require sawcutting to construct the unique shapes of the floor system.

Steel Deck with Steel Beams and Girders

Descriptions:
This floor system is constructed using a 1.5" deep, 18-gage composite metal deck with 2" topping. W21x55 support the deck and topping, while W30x90 support the beams. A detailed drawing of the section of the deck and beam is shown in Figure 13. Hand calculations can be found in Appendix E.
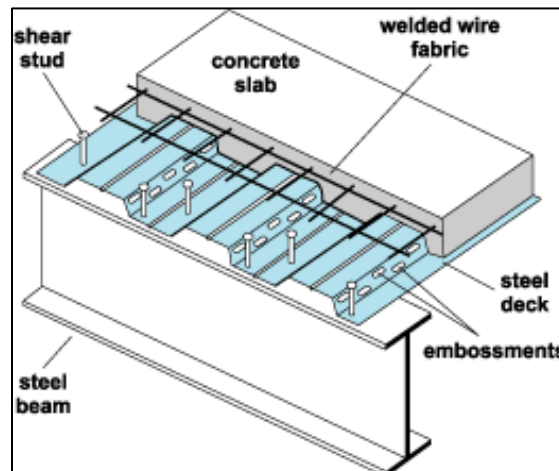

Figure 13 – Steel Deck with Steel Beam Section Cut. Courtesy answers.com.

Advantages:
The steel deck with steel beams and girders is a lightweight system in comparison to concrete floor systems. There is no required formwork for the concrete because the metal deck acts as the formwork for the 2" topping. Also, the composite action between the metal deck and the concrete allows for a shallower deck and topping depth as compared to a concrete slab. The shallower slab, and therefore lightweight deck system, requires smaller steel members to support the resulting load.

Disadvantages:
Unlike concrete floor systems, the steel beams and girders supporting the deck will require fireproofing. The steel deck with steel beams and girders will require an increase in labor and cost for welding. Also, even though the individual steel members may be shallow, the overall system can be much deeper than concrete floor systems. Additionally, Orlando is not a pro-steel city, so the cost of materials and skilled labor will be much more expensive than concrete.

One-Way Slab with Continuous T-beams

Descriptions:
The one-way slab with continuous T-beams is a cast-in-place concrete system. Wide beams are used to transfer the loads to the columns because there are no intermediate beams traversing the other direction of the slab. Figure 14 shows a typical one-way slab with continuous T-beams below.
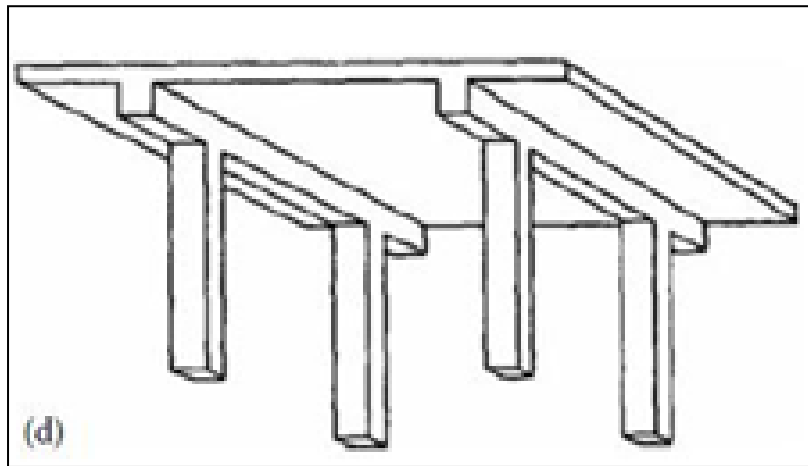

Figure 14 - One-Way Slab with Continuous T-beams. Courtesy engcastle.com.

 This alternate floor system is designed using a 9" slab spanning between the wide beams. The reinforcement in the slab is #5 bars spaced at 12" O.C. The beams are designed to be 9' wide and 10" deep. The top reinforcement in the beam consists of (34) #5 bars, while the bottom reinforcement is designed with (24) #7 bars. Hand calculations can be found in Appendix F.

Advantages:
The one-way slab with continuous T-beams provides larger bay spacing, which gives wider column spacing in the building layout. This alternate floor system is also used with progressive collapse systems, which might be considered as a potential thesis depth study. Also, as stated in the two-way system, Orlando is a pro-concrete city, so the cost of labor and materials for concrete is much lower than steel.

Disadvantages:

The cast-in-place concrete system will require more complicated formwork than most other concrete systems, which results in an increase in cost. Also, there is a longer lead time for the floor system because of the detailed forming process. Additionally, this concrete system is heavier than a steel system, which will result in larger columns and foundation system to support the weight.

Comparison of Floor Systems:

Table 5 shows the various categories used to rate the existing and alternate floor systems. R.S. Means 2009 is used to estimate the cost of each system. To more accurately understand why the existing system was chosen, the 2009 edition is used because construction began that year as to see prices when decisions were being made. A location factor for Orlando, Florida has been applied to the cost estimates, which are based off of total cost of material and installation. Differences between the R.S. Means' system and the actual floor system are discussed in Appendix G with the individual tables from R.S.Means. A discussion follows Table 5 to explain assumptions and factors that went into determining each category.

Floor System Comparison				
Design Concern	Existing *Two-Way Flat Slab with Drop Panels*	Alternative I *Pre-Cast Hollow Core Planks on Steel Beams*	Alternative II *Steel Deck with Steel Beams and Girders*	Alternative III *One-Way Slab with Continuous T-Beams*
Slab Depth	12"	10"	3.5"	9"
System Depth	18.25"	34.1"	53.8"	19"
Beam Deflection (D+L)	0.90" (slab deflection)	1.43"	1.42"	1.38"
System Cost	$17.18/S.F.	$13.46/S.F.	$18.02/S.F.	$20.53
System Weight	220.5 psf	93 psf	55.3 psf	237.5 psf
Fire Protection	Inherent	Spray-On	Spray-On	Inherent
Formwork	Yes	No	No	Yes
Lateral System Alterations	No	Yes	Yes	No
Foundation Alterations	No	No	No	No
Feasibility	Yes	No	No	Yes

Table 5 - Floor System Comparison

Slab Depth/System Depth

All the floor systems, except for the steel deck, have similar slab depths. The steel deck system slab has the smallest slab depth, but it more than surpasses each system with a 53.8" total depth. The existing two-way flat slab with drop panels and alternate one-way slab with continuous T-beams provide the shallowest system depths, making them the best floor systems in this category. These systems also allow the MEP to be connected directly to the floor assembly, instead of having to be hung from the steel deck system.

Beam Deflection (D+L)

The beam deflections of each of the three alternate systems have negligible variance. Each of the systems meet the code deflection requirements for live load and total load deflection. The two-way flat slab system's slab deflection is about half of the beam deflections, but these two deflections are not comparable because they are two different components. It is extremely important for a hospital to choose a floor system with the least amount of deflection due to the precision required for many of the medical machines directly mounted to the floor.

System Cost

R.S. Means 2009 Assemblies with a location factor for Orlando, Florida is used to roughly estimate the cost of each system. Most of the systems are not exactly found in R.S. Means, so a system that is similar to the actual floor system is used instead. A discussion of the individual cost/S.F. values can be found in Appendix G.

System Weight

The weight of the floor system has a direct affect on the column and foundation designs. A heavier system will require larger columns and an increase in the foundation system, which will result in an increase in cost for additional building materials. NCHTNF's existing two-way flat slab with drop panels is one of the heavier floor system options. So, if the either the pre-cast hollow core planks or the steel deck is used instead, the size of the columns and foundation might be able to be reduced.

Fire Protection

The code requires all structural systems to have a 2-hour fire rating. Since the two-way flat slab and one-way continuous T-beam systems are concrete, they inherently provide this required 2-hour fire rating. The steel deck and pre-cast hollow core systems will require fire proofing for the exposed supporting steel to attain the 2-hour fire rating.

Formwork

Formwork is only necessary for the two cast-in-place concrete floor systems. The cost of labor and materials for the formwork will need to be taken into account when comparing the costs of the floor systems.

Lateral System Alterations

The calculations for these floor systems only take gravity loads into account, so additional calculations will be required for a detailed analysis of the affect each floor system has on the lateral system. In general, the existing system is designed for the two-way flat slab, so the one-way system can probably use the same lateral system due to the similarity in stiffness. The pre-cast hollow core and steel systems may require an increase in lateral system because they are less stiff than the existing two-way flat slab system.

Foundation Alterations

All of the studied floor systems can use the existing column layout, except for the pre-cast hollow core system. The pre-cast hollow core system is based on 4' modules, so the typical bay size will need to be resized from 30'x30' to 32'x32'. This will require changes in the foundation layout due to the movement of column placement. The two-way and one-way systems will most likely require the same foundation system, but the steel deck can probably be constructed with a smaller foundation system.

Conclusion:

After studying Table 5's results, the feasibility of each floor system needs to be taken into consideration as well. The steel deck with steel beams and girders is ruled out due to Orlando being a pro-concrete city. The skilled labor and materials are not readily available and will be much more expensive to construct a steel design. The cost of the system is a little more expensive than the existing system, and this does not take Orlando's concrete preference into account, which will only increase the cost/S.F. The steel deck system is an additional 35.5" deeper than the existing system, yet another drawback.

The pre-cast hollow core system is cheaper than the existing system, but it lacks in constructability. The 4' modules will require the bay sizes to move from 30'x30' to 32'x32'. Also, the pre-cast hollow core system will need to be sawcut to fit the curved curtain wall, seeing as the pre-cast shapes are only rectangular. Also, similar to the steel deck system, the pre-cast hollow core system is an additional 16" deeper than the existing system.

Even though the two-cast-in place concrete systems are the heaviest and most expensive systems, they are the most feasible designs. Each system depth has negligible difference as well as no difference between the fire proofing and formwork requirements when compared to each other. Without lateral analysis it is difficult to determine which system is more beneficial. In conclusion, both of these systems appear to be equally adequate.

Technical Report III will focus on analyzing lateral systems and confirming the conclusions found in this report.

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

Floor	Elevation	z	k_z	q_z	q_h	Windward(psf)	Leeward (psf)	Trib. Area (ft^2)	Force (k)
					North - South Clinic (MWFRS)				
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

Floor	Elevation	z	k_z	q_z	q_h	Windward(psf)	Leeward (psf)	Trib. Area (ft^2)	Force (k)
					East - West Clinic (MWFRS)				
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

$T_a = C_t h_n^x$ $C_t = 0.016$ (Table 12.8-2)
$T_a = (0.016)(135')^{0.9} = 1.32$ $x = 0.9$ (Table 12.8-2)
$1/T_a = f = 0.756 < 1.0$ $h_n = 135'$
∴ Building is not rigid, calculate G_f

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

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

$I_{\bar{z}} = c(33/\bar{z})^{1/6}$ $c = 0.2$ (Table 6-2)
$I_{\bar{z}} = 0.2(33/81)^{1/6}$ $\bar{z} = 0.6h$
$I_{\bar{z}} = 0.1722$ $\bar{z} = 0.6(135') = 81$

$g_Q = g_v = 3.4$

$Q = \sqrt{1 / \left[1 + 0.63 \left(\frac{B+h}{L_{\bar{z}}} \right)^{0.63} \right]}$
   $B_{NS} = 465'$
   $B_{EW} = 300'$
   $L_{\bar{z}} = \ell(\bar{z}/33)^{\bar{\epsilon}}$ $\ell = 500'$ (Table 6-2)
   $L_{\bar{z}} = 500'(81/33)^{1/5} = 598.36$ $\bar{\epsilon} = 1/5$ (Table 6-2)
$Q_{NS} = \sqrt{1 / \left[1 + 0.63 (465 + 135 / 598.36)^{0.63} \right]} = 0.783$
$Q_{EW} = \sqrt{1 / \left[1 + 0.63 (300 + 135 / 598.36)^{0.63} \right]} = 0.812$

$g_R = \sqrt{2 \ln(3,600 n_1)} + 0.577 / \sqrt{2 \ln(3,600 n_1)}$ $n_1 = f = 0.756$
$g_R = \sqrt{2 \ln(3,600 (0.756))} + 0.577 / \sqrt{2 \ln(3600 (0.756))}$
$g_R = 4.122$

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

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

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

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

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

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

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

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

$\quad N_1 = 0.756 (598.36) / 165.376$

$\quad N_1 = 2.74$

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

$R_n = 0.074$

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

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

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

$\quad \eta = 4.6 (0.756)(135) / 165.376$

$\quad \eta = 2.84$

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

$R_h = 0.290$

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

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

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

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

$\quad \eta = 9.781$

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

$R_{B_{NS}} = 0.097$

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

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

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

$\quad \eta = 6.309$

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

$R_{B_{EW}} = 0.146$

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

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

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

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

$\quad \eta = 21.122$

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

$R_{L_{NS}} = 0.044$

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

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

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

$\quad \eta = 32.736$

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

$R_{L_{EW}} = 0.030$

$$R_{NS} = \sqrt{(1/\beta)\, R_n R_h R_B \,(0.53 + 0.47 R_L)}$$
$$R_{NS} = \sqrt{(0.074)(0.290)(0.097)(0.53 + 0.47(0.040))}$$
$$R_{NS} = 0.034$$
$$R_{EW} = \sqrt{(1/\beta)\, R_n R_h R_B \,(0.53 + 0.47 R_L)}$$
$$R_{EW} = \sqrt{(0.074)(0.290)(0.146)(0.53 + 0.47(0.030))}$$
$$R_{EW} = 0.041$$

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

Enclosed flexible building
    ↳ nonoperable windows ∴ enclosed

$$p = q\, G_f\, C_p - q_i\, (G C_{pi})$$
    $q = q_{\bar{z}}$ for windward walls
    $q = q_h$ for leeward walls
    $C_p = 0.8$ (windward walls)   (fig. 6-6)
    $C_p = -0.5$ (leeward walls)   (fig. 6-6)
    $q_i = q_{\bar{z}}$
    $G C_{pi} = \pm 0.18$ (fig. 6-5)
    $q_{\bar{z}} = 0.00256\, k_z\, k_{zt}\, k_d\, V^2\, I$
       k_z = Table 6.3 (varies w/ height)
       $k_{zt} = 1.0$
       $k_d = 0.85$ ⎤ see pg.1 of wind calcs for data location
       $V^2 = 110\,mph$
       $I = 1.15$ ⎦

*** 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

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

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/i |

11.4 Seismic ground motion

    Site class D (firm soil) according to geotech report

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

    $S_{ms} = F_a S_s$ $F_a = 1.6$ (Table 11.4-1)
    $S_{ms} = (1.6)(0.096) = 0.15$ $F_v = 2.4$ (Table 11.4-2)
    $S_{m1} = F_v S_1$
    $S_{m1} = (2.4)(0.038) = 0.09$

    $S_{DS} = 2/3 \, S_{ms}$
    $S_{DS} = 2/3 (0.15) = 0.10$
    $S_{D1} = 2/3 \, S_{m1}$
    $S_{D1} = 2/3 (0.09) = 0.06$

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

                             → 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
                                → see bottom note
    $V = C_s W$ $W = 69,485^k$ (calculated using
                                        spreadsheet)
    $C_s = S_{DS}/(R/I)$ for $T \leq T_L$ $R = 5$ (ordinary reinforced
    $T = 0.7565 \leq T_L = 8s$ ✓   concrete shear walls)
       ↳ calculated for wind calcs   (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: Existing Two-Way Flat Slab with Drop Panels
C.1 Hand Calculations
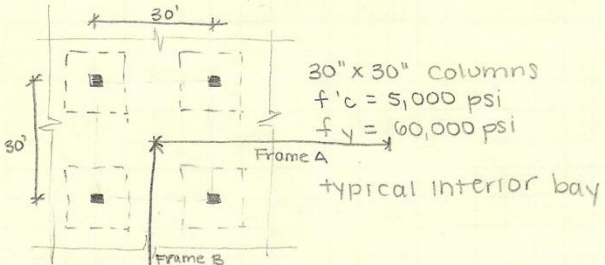
Caitlin Behm   AE Senior Thesis   Gr Floor System Checks   1/5

ACI 318.08
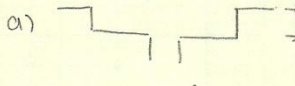
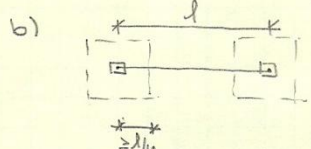Existing Two way flat Slab w/ Drop Panels
   12" NW concrete elevated two-way flat Slab w/
   6'4" drop panels

30" x 30" Columns
$f'c = 5,000$ psi
$f_y = 60,000$ psi

Frame A

typical Interior bay

Frame B

ACI 13.2.5 when reducing the amount of negative moment
reinforcement over a column or minimum required
Slab thickness, a drop panel shall
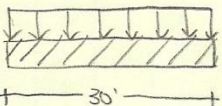
a)
   $a \geq 1/4$ adjacent slab thickness
   $a \geq 1/4 (12") = 3"$
   $a = 6 1/4"$ ✓

b)
   $\ell = 30'$
   $\ell/6 = 5' \rightarrow$ panels project out 6' ✓
   $\geq \ell/6$

∴ Drop panels → 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 (servery → storage) → see building load section

30'

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

Slab thickness
   $\ell_n/36$ (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 | Floor System | 2/5

Frame A = Frame B b/c of symmetry

$$M_0 = \frac{1}{8} W_u \, l_2 \, l_n^2 = \frac{1}{8}(0.395 \, ksf)(30')(30' - 30''/12'')^2 = 1120.2 \, ft \cdot k$$
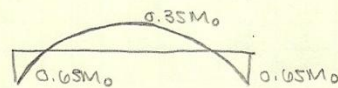
Int. Span
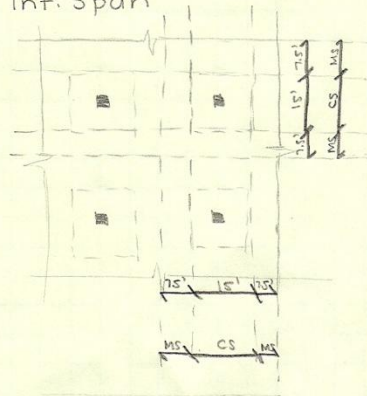
Frame A = frame B

+392.07	
-728.13	-728.16

Column Strip
$$l_2/2 = 30'/2 = 15'$$

Middle strip
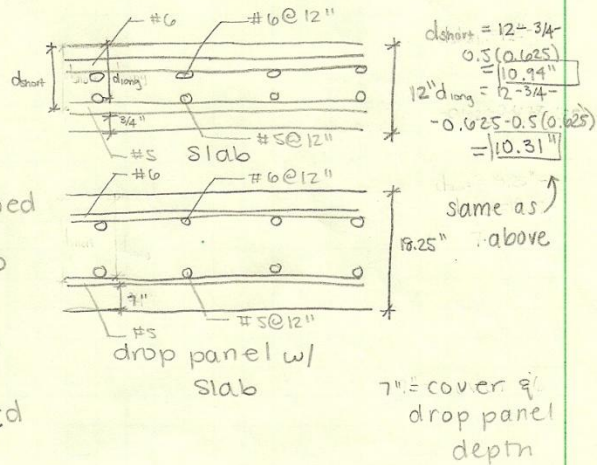$$30' - 15' = 15'/2 = 7.5' \text{ on each side of cs}$$

0.35 M₀
0.65 M₀   0.65 M₀

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

*** thickness of drop panel
below slab shall not be assumed
to be greater than ¼ The
distance from edge of drop
panel to face of column.

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

(6.25" droppanel used
in current design) ✓

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

#6   #6 @ 12"
d_{short} d_{long}
3/4"
#5   Slab   #5 @ 12"

#6   #6 @ 12"

same as
above

18.25"

7"
#5   #5 @ 12"
drop panel w/
Slab

7" = cover &
drop panel
depth

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 | Floor System | 3/5 |

Reinforcement Design & Distribution

Int. Span.

Description	Neg M.	Pos. M	(frame A = Column strip)
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 in²	5.51 in²	
ⓑ $A_{s,min} = 0.0018 bt$	5.91 in²	5.91 in²	
ⓒ N = larger A or B	8.27 in²	5.91 in²	adequate reinforcing
ⓓ N_{min} = width strip / 2t	4.93 in²	4.93 in²	exceeding these
N = (larger C or D)/0.31			values. See slab detail

Int. Span

Description	Neg M.	Pos. M	(frame A = middle strip)
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 in²	1.77 in²	
ⓑ $A_{s,min} = 0.0018 bt$	3.89 in²	3.89 in²	
ⓒ N = larger A or B	5.32 in²	3.89 in²	adequate reinforcing
ⓓ N_{min} = width strip / 2t	7.5 in²	7.5 in²	exceeding these
N = (larger C or D)/0.31			values. see slab detail.

Int. Span

Description	Neg M	Pos M	(frame B Column strip)
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 in²	5.75 in²	
ⓑ $A_{s,min} = 0.0018 bt$	5.91 in²	5.91 in²	
ⓒ N = larger A or B	8.72 in²	5.91 in²	adequate reinforcing
ⓓ N_{min} = width strip / 2t	4.93 in²	4.93 in²	exceeding these
N = (larger C or D)/0.31			values. See slab detail.

Int. Span

Description	Neg M	Pos M	(frame B middle strip)
width of strip (b)	180"	180"	
effective depth	10.31"	10.31"	
$R = M_u / bd^2$	102.67	61.48	
ρ (from Table A.5a)	0.0031	0.0010	
ⓐ $A_s = \rho bd$	5.75 in²	1.86 in²	
ⓑ $A_{s,min} = 0.0018 bt$	3.89 in²	3.89 in²	
ⓒ N = larger A or B	5.75 in²	3.89 in²	adequate reinforcing
ⓓ N_{min} = width strip / 2t	7.5 in²	7.5 in²	exceeding these
N = (larger C or b)/0.31			values. See slab detail.

Caitlin Behm | AE Senior Thesis | Floor Systems | 4/5

wide beam action

drop panel = 12' × 12'

$d_2 = d_{avg} = 10.63"$

$d_1 = 10.63" + 72" - 15" = 67.63"$

$\pi d^2/4 = a^2$ $\pi d^2/4 = a^2$

$\sqrt{\pi d^2/4} = a$ $\sqrt{\pi d^2/4} = a$
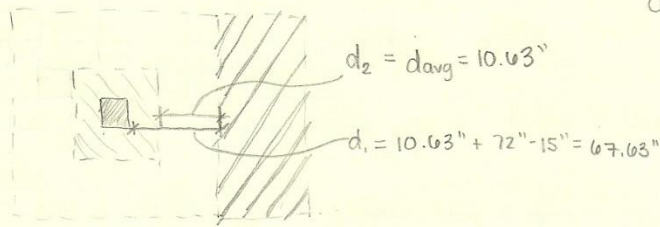
$\sqrt{\pi(12.5)^2/4} = 10.63'$ $\sqrt{\pi(12.03)^2/4} = 10.63'$

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

$= 8.80'$ $= 8.80'$

$V_u = W_u \times 8.80 \times l_z$ $V_u = W_u \times 8.80 \times l_z$

$V_u = (0.395)(8.80)(30)$ $V_u = (0.395)(8.80)(30)$

$V_u = 104.28 K$ $V_u = 104.28 K$

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

$\phi V_n = 324.71 K > V_u$ ✓

same in other direction b/c of symmetry.

punching shear

edge of drop panel

$d_1/2$ $d_2/2$

d_1 d_2
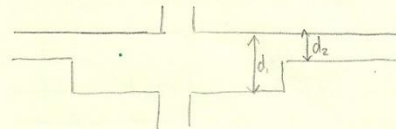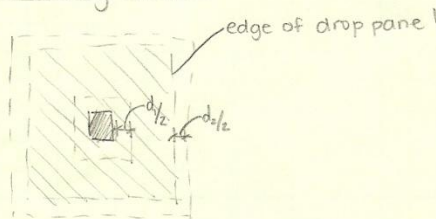
$b_o = 2(d_1/2) + col. width$ $b_o = 2(d_2/2) + col width$

$b_o = 2(18.25/2) + 30 = 48.25$ $b_o = 2(12/2) + 30 = 42$

$4(48.25) = 193"$ (perimeter) $4(42) = 168"$ (perimeter)

$V_c = 4\sqrt{f'c} \, b_o d$ $V_c = 4\sqrt{f'c} \, b_o d$

$V_c = 4\sqrt{5000}(193)(18.25)(1/1000)$ $V_c = 4\sqrt{5000}(193)(12)(1/1000)$

$V_c = 996.24 K$ $V_c = 655.06 K$

$V_u = W_u \cdot Area$ $V_u = W_u \cdot Area$

$V_u = (0.395)(48.25^2)$ $V_u = (0.395)(42^2)$
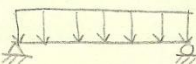
$V_u = 919.58 K$ $V_u = 696.78 K$

$V_u < V_c$ NG → $V_u > V_c$ ✓

| Caitlin Behm | AE Senior Thesis | Floor System | 5/5 |

Deflection Check:
  Immediate deflection due to total dead load (column strip)

$$I_g = [(15')(12''/1') \times (12'')^3]/12 = 25920 \text{ in}^4$$
$$E_c = 57000\sqrt{5000} = 4030.5 \text{ ksi}$$

Slab self weight = $(12''/12)(145 \text{pcf}) = 145 \text{psf}$
SDL = 12 psf
Total Dead Load = 167 psf

$$\omega_b = (167 \text{psf})(30')(0.675) = 3.4 \text{klf}$$

$$\Delta_D (\text{max}) = 0.0026 \, \omega_D \ell^4 / EI$$
$$\Delta_D (\text{max}) = 0.0026 (3.4 \text{klf})(1/12)(360)^4 / (4030.5 \text{ksi})(25920 \text{in}^4)$$
$$\Delta_D (\text{max}) = 0.12''$$

Immediate deflection due to dead load (middle strip)

$$\omega_D = (167 \text{psf})(30')(0.325) = 1.7 \text{klf}$$

$$\Delta_D (\text{max}) = 0.0026 \, \omega_D \ell^4 / EI$$
$$\Delta_D (\text{max}) = 0.0026 (1.7 \text{klf})(1/12)(360)^4 / (4030.5 \text{ksi})(25920 \text{in}^4)$$
$$\Delta_D (\text{max}) = 0.06''$$

$$\Delta_D (\text{max, total}) = 0.12'' + 0.06'' = 0.18'' < \ell/480 = 0.75'' \checkmark$$

Immediate deflection due to live load (column strip)

$$\omega_{LL} = (76 \text{psf})(30')(0.675) = 1.5 \text{klf}$$

$$\Delta_L (\text{max}) = 0.0048 \, \omega_D \ell^4 / EI$$
$$\Delta_L (\text{max}) = 0.0048 (1.5 \text{klf})(1/12)(360)^4 / (4030.5 \text{ksi})(25920 \text{in}^4)$$
$$\Delta_L (\text{max}) = 0.10''$$

Immediate deflection due to live load (middle strip)

$$\omega_{LL} = (76 \text{psf})(30')(0.325) = 0.7 \text{klf}$$

$$\Delta_L (\text{max}) = 0.0048 \, \omega_D \ell^4 / EI$$
$$\Delta_L (\text{max}) = 0.0048 (0.7 \text{klf})(1/12)(360)^4 / (4030.5 \text{ksi})(25920 \text{in}^4)$$
$$\Delta_L (\text{max}) = 0.05''$$

$$\Delta_L (\text{max, total}) = 0.10'' + 0.05'' = 0.15'' < \ell/360 = 1'' \checkmark$$

$$\Delta_{tot} = 0.75'' + 0.15'' = 0.9''$$
          ↖ total deflection due to dead &
              live loads.

Appendix D: Alternate Pre-Cast Hollow Core Planks on Steel Beams
D.1 Hand Calculations

| Caitlin Behm | AE Senior Thesis | Floor System | 1/1 |

Pre-cast Hollow Core Planks on Steel Beams

Typical Bay ⇒ 30' × 30'

Safe Superimposed Service Load
  LL + SDL = 125 psf + 12 psf = 137 psf
  * Assumption plank & topping self weight taken into account in table
  Use Nitterhouse 10" × 4'-0" (due to span limitations)

Hollow core plank w/ 2" topping - 2 hr fire rating
  30' span use 7 - ½" φ strands
  Safe superimposed service load = 162 psf > 137 psf ✓

Note: $f'c = 6,000$ psi
  NWC = 150 pcf

Plank & topping self weight
  = 68 psf + 25 psf = 93 psf *** values from Nitterhouse specs ***

Design girder running ⊥ to planks
  SDL = 12 psf
  DL = 93 psf $LL_R = 125 (0.25 + {}^{15}/\sqrt{2(30)(30)})$
  LL = 125 psf $LL_R = 76$ psf

  $W_u = 1.2 (12\text{ psf} + 93\text{ psf})(30') + 1.6 (76\text{ psf})(30') = 7.43$ klf

  $M_u = W_u \ell^2/8 = 7.43\text{ klf}(30')^2/8 = 835.65$ k·ft

  $V_u = W_u \ell/2 = 7.43\text{ klf}(30')/2 = 111.45$ K

From Table 3.10 in AISC steel manual: use W24 × 84
  $\phi M_n = 840$ ft·K > 835.65 ft·K
From Table 3.2 in AISC steel manual
  $\phi V_n = 340^K > 111$

Check deflection
  construction Δ: DL = (93 psf + 12 psf + 76 psf)(30') = 5.4 klf
  $\Delta = 5/384 [(5.4)(30')^4 (1728)/(29000)(2370 \text{ in4})] = 1.43"$
  $L/240 = 30(12)/240 = 1.5" > 1.43"$ ✓

  Live Load Δ:
  $\Delta = 5/384 [(0.076)(30)(30')^4 (1728)/(29000)(3610)] = 0.40"$
 $L/360 = 30(12)/360 = 1" > 0.40"$ ✓

  W24 × 84

Prestressed Concrete
10"x4'-0" Hollow Core Plank
2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 327 in.^{2} Precast b_w = 13.13 in.
I_c = 5102 in.^{4} Precast S_{bcp} = 824 in.^{3}
Y_{bcp} = 6.19 in.   Topping S_{tct} = 1242 in.^{3}
Y_{tcp} = 3.81 in.   Precast S_{tcp} = 1340 in.^{3}
Y_{tcp} = 5.81 in.   Precast Wt. = 272 PLF
                Precast Wt. = 68.00 PSF

DESIGN DATA

1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø and 0.6"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
    6-1/2"Ø, 270K = 168.1 k-ft at 60% jacking force
    7-1/2"Ø, 270K = 191.7 k-ft at 60% jacking force
7. Maximum bottom tensile stress is $10\sqrt{f'c}$ = 775 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI.   Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span.   A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99.   Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.
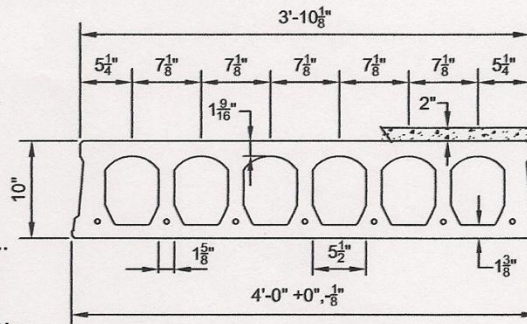
SAFE SUPERIMPOSED SERVICE LOADS		IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)																	
Strand Pattern		SPAN (FEET)																	
		26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44
6 - 1/2"ø	LOAD (PSF)	202	181	161	144	128	114	101	90	79	69	60	52	45	38					
7 - 1/2"ø	LOAD (PSF)	246	222	200	180	162	146	131	118	105	94	84	74	66	58					

NITTERHOUSE
CONCRETE N PRODUCTS

2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17202-9203
717-267-4505   Fax 717-267-4518

11/03/08

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths.   The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

10F2.0T

Appendix E: Alternate Steel Deck with Steel Beams and Girders
E.1 Hand Calculations

Caitlin Behm | AE Senior Thesis | Floor System | 1/2

Steel Deck w/ Steel Beams & Girders

|← 10' →|← 10' →|← 10' →| 30'

Loads:
SDL = 12 psf
DL = 93 psf
LL = 125 psf
Slab = 38 psf

$W_{LR} = 125(0.25 + 15/\sqrt{2(10)(30)})$
$W_{LR} = 107.8$ psf ⇒ 108 psf

1.5VLR18 w/ t = 2"   (SDI maximum unshored 3-span (10
10'-8" > 10'-0" ✓

Beam Design   ┌ bm self wt. allowance
$W_U = 1.2(93 + 12 + 38 + 5) + 1.6(108) = 350.4$ psf
$W_U = 3.50$ klf

$M_U = W_U \ell^2/8 = (3.50 \text{ klf})(30')^2/8 = 394.2$ k·ft

using Z_x Table:
$M_U \leq \phi M_{px}$ W21×48 $\phi_b M_{px} = 398$ k·ft > 394.2 k·ft
$V_U \leq \phi V_{nx}$ W21×48 $\phi_v V_{nx} = 217$ k > 105 k

Check deflection
construction Δ:
$\Delta = 5/384 [(2.56)(30')^4(1728)/(29000)(959)] = 1.67"$
$L/240 = (30 \times 12)/240 = 1.50" < 1.67"$ NG

$I_{req'd} = 959(1.67/1.50) = 1067.7$ Try W21×55

$\Delta = 5/384 [(2.56)(30')^4(1728)/(29000)(1140)] = 1.41"$
$L/240 = (30 \times 12)/240 = 1.50" > 1.41"$ ✓

Live Load Δ:
$\Delta = 5/384 [(0.108)(10)(30')^4(1728)/(29000)(1140)] = 0.60"$
$L/360 = (30 \times 12)/360 = 1.0" > 0.60"$ ✓

Check beam weight assumption:
i. $W_{beam} = 55/10' = 5.5$ psf > 5 psf

Check construction Δ w/ actual beam weight:
$\Delta = 5/384 [(2.57)(30')^4(1728)/(29000)(1140)] = 1.42"$
$L/240 = (30 \times 12)/240 = 1.50" > 1.42"$ ✓

Nemours Children's Hospital as a part of The Nemours Foundation
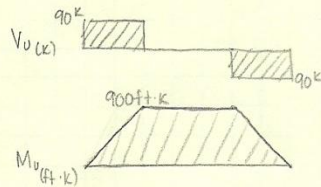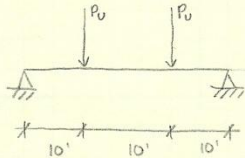
| Caitlin Behm | AE Senior Thesis | Floor System | 2/2 |

Girder Design

$L = 125 (0.25 + 15/\sqrt{2(30 \times 30)}) = 76 \text{ psf}$

$W_U = 1.2(93 + 12 + 38 + 5.5) + 1.6(76) = 299.8 \text{ psf}$

$P_U = 299.8 \text{ psf} (10')(30') = 89.94^K \approx 90^K$

Using Z_x Table:

$M_U \leq \phi M_{px}$ W27x84 $\phi_b M_{px} = 915 \text{ ft} \cdot k > 900 \text{ ft} \cdot k$

$V_U \leq \phi V_{nx}$ W27x84 $\phi V_{nx} = 369^K > 90^K$

Check deflection

Live load Δ:

$P_{LL} = 76 \text{ psf} (10')(30') = 22.8^K$

$\Delta = (22.8^K)(30')^3(1728)/(28(29000)(2850 \text{ in}^4)) = 0.46"$

$L/360 = (30 \times 12)/360 = 1.0" > 0.46" \checkmark$

Construction Δ:

$P_T = 22.8^K + 68^K$

$\Delta = (90.8^K)(30')^3(1728)/(28(29000)(2850 \text{ in}^4)) = 1.83"$

$L/240 = (30 \times 12)/240 = 1.5" < 1.83"$ NG

$I_{req'd} = 2850(1.83/1.50) = 3477 \text{ in}^4$ Try W30x90

Live load Δ:

$P_{LL} = 76 \text{ psf} (10')(30') = 22.8^K$

$\Delta = (22.8^K)(30')^3(1728)/(28(29000)(3610 \text{ in}^4)) = 0.36"$

$L/360 = (30 \times 12)/360 = 1.0" > 0.36" \checkmark$

Construction Δ:
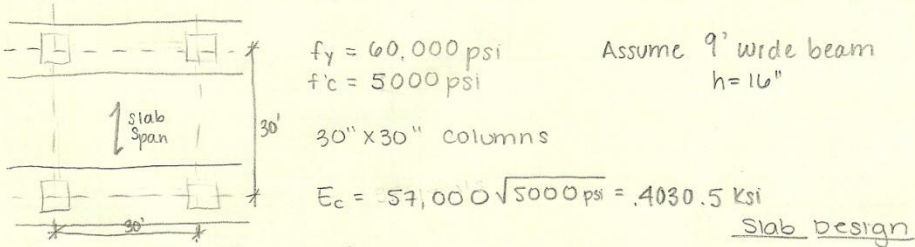
$P_T = 22.8^K + 68^K$

$\Delta = (90.8^K)(30')^3(1728)/(28(29000)(3610 \text{ in}^4)) = 1.44"$

$L/240 = (30 \times 12)/240 = 1.5" > 1.44" \checkmark$

Appendix F: Alternate One-Way Slab with Continuous T-Beams
F.1 Hand Calculations

Caitlin Behm | AE Senior Thesis | Floor System | 1/6

One way slab w/ Continuous T-beams

$f_y = 60,000$ psi   Assume 9' wide beam
$f'c = 5000$ psi $h = 16"$

slab span   30'   30"x30" columns

$E_c = 57,000 \sqrt{5000\,psi} = 4030.5$ ksi

Slab Design

From ACI 318-08 Table 9.5a
solid one-way slabs, both ends continuous
$h \geq \ell/28 = (21')(12"/ft)/28 = 9"$ slab depth

Factored Loads
$LL = 125$ psf $LLR = 125(0.25 + 15/\sqrt{2(30)(30)})$
$SDL = 12$ psf $LLR = 76$ psf
$DL = (18/12)(150\,pcf) = 225$ psf

unit method

$W_u = 1.2(12\,psf + 225\,psf) + 1.6(76\,psf) = 406$ psf (1ft) = 406 plf
$M_u = W_u \ell^2/11 = (406\,plf)(21')^2/11 = 16.3$ k·ft/ft (Negative Moment)
$M_u = W_u \ell^2/16 = (406\,plf)(21')^2/16 = 11.2$ k·ft/ft (Positive Moment)
Minimum reinforcement for shrinkage & temperature
$0.0018 A_g$
$= 0.0018(9")(12"/ft) = 0.194$ in²/ft

Try #5 $A_s = 0.31$ in² $d = 9" - 3/4" - \frac{1}{2}(0.625) = 7.94"$

$a = A_s f_y/(0.85 f'c\, b) = (0.31\,in²)(60\,ksi)/(0.85)(5\,ksi)(12"/ft) = 0.365$ in
$c = 0.365\,in/(0.80) = 0.46" < 2.98" \therefore$ Tension controlled, $\phi = 0.9$

$\phi M_n = A_s f_y (d - a/2)$
$= 0.9(0.31\,in²)(60\,ksi)(7.94" - 0.365/2)$
$= 129.86$ ft·k/ft

$M_u = 16.3$ k·ft < 129.86 k·ft ✓ (negative moment)
$M_u = 11.2$ k·ft < 129.86 k·ft ✓ (positive moment)

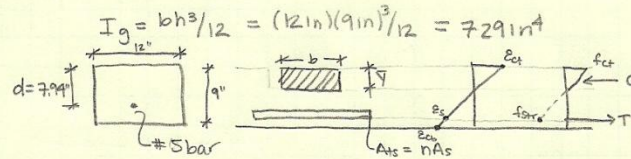Caitlin Behm | AE Senior Thesis | Floor System | 2/6

Deflection:

$$I_e = (M_{cr}/M_a)^3 I_g + [1 - (M_{cr}/M_a)^3] I_{cr}$$

$$I_g = bh^3/12 = (12in)(9in)^3/12 = 729 in^4$$

$$\bar{y}: b\bar{y}^2/2 + nA_s\bar{y} - nA_s d$$
$$= 12\bar{y}^2/2 + 7.2(0.31)\bar{y} - 7.2(0.31)(7.94)$$
$$= 6\bar{y}^2 + 2.23\bar{y} - 17.72$$
$$= 1.54 in$$

$$n = \mathcal{E}_s/\mathcal{E}_c = 29000/4030.5 = 7.20$$

$$I_{cr} = (12)(1.54)^3/12 + 12(1.54)(1.54/2)^2 + 7.20(0.31)(7.94 - 1.54)^2$$
$$I_{cr} = 3.65 + 10.96 + 91.42 = 106.03 in^4$$

$$M_{cr} = f_r I_g / y_t = [7.5\sqrt{5000}(729)/(4.5)] = 7.16 ft\cdot k$$
$$M_a = 11.2 ft\cdot k \text{ (positive moment)}$$

$$I_e = (11.2/7.16)^3(729) + [1 - (11.2/7.16)^3](106.03) = 2490.45 in^4$$

Live load Δ:

$$5w_L l^4/385 E_c I_e = 5(0.076 klf)(21')^4(1728)/385(4030.5 ksi)(2490.45 in^4)$$
$$= 0.03 in$$
$$l/360 = (21')(12''/1)/360 = 0.70 in > 0.03 in \checkmark$$

Total load Δ:

$$5w_L l^4/385 E_c I_e = 5(0.313 klf)(21')^4(1728)/385(4030.5 ksi)(2490.45 in^4)$$
$$= 0.14 in$$
$$l/240 = (21')(12''/1)/240 = 1.05 in > 0.14 in \checkmark$$

Beam Design
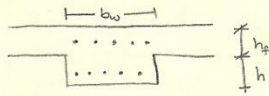
Loading on beam

$W_{slab} = (150 \, pcf)(9")(1/12)(21') = 2362.5 \, plf$

$W_{beam} = (150 \, pcf)(19")(1/12)(9') = 2137.5 \, plf$

$SDL = (12 \, psf)(30') = 600 \, plf$

$LL = (76 \, psf)(30') = 2280 \, plf$

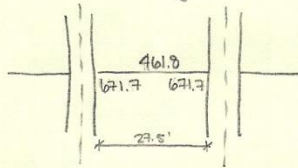$W_u = 1.2(2.36 + 2.14 + 0.6) + 1.6(2.28) = 9.77 \, klf$

ACI moment coefficients

Negative Moment (Int. Support)

$W_L \ell_n^2 / 11 = (9.77 \, klf)(27.5)^2 / 11 = 671.7 \, k \cdot ft$

Positive Moment (Int. Support)

$W_L \ell_n^2 / 16 = (9.77 \, klf)(27.5)^2 / 16 = 461.8 \, k \cdot ft$

Moment Diagram (Int. span)

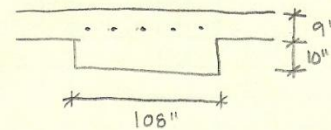Top Reinforcement (int span / int support)

$A_s \approx M_u / 4d$ (34) #5 bars

$\approx 671.7 / (4 \cdot 16.75)$ $A_s = 10.54 \, in^2$

$\approx 10.02 \, in^2$

$b_{eff} = \begin{cases} 1/4\ell = (1/4(27.5')(12)) = 82.5 \leftarrow controls \\ 16h_f + b_w = 16(9) + 108 = 252 \\ b_w + \ell_n = 108 + 27.5(12) = 432 \end{cases}$

$d = 19 - 1.5 - 0.375 - 0.625 = 16.75"$

$a = A_s f_y / (0.85 f'_c b) = (10.54)(60) / ((0.85)(5)(82.5)) = 1.80"$

$c = a/\beta = 1.80/0.80 = 2.25" < 0.375d = 0.375(16.75) = 6.28"$

$\therefore$ tension controlled, $\phi = 0.9$

$\phi M_n = \phi A_s f_y (d - a/2)$

$= 0.9(10.54)(60)(16.75 - 1.80/2) = 751.77 \, ft \cdot k > 671.7 \, ft \cdot k$ ✓

Use (34) #5 bars for top reinforcing
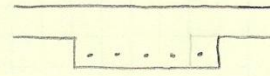
Bottom Reinforcement (Int. Span)

$A_s \approx M_u / 4d =$

$\approx 461.8/(4 \cdot 16.75)$ (24) #5 bars

$\approx 6.89 \text{ in}^2$ $A_s = 7.44 \text{ in}^2$

$a = A_s f_y / (0.85 f'_c b) = 7.44 (60) / (0.85 (5)(82.5)) = 1.27"$

$c = a/\beta = 1.27"/0.80 = 1.59" < 6.28"$ $\therefore$ Tension controlled, $\phi = 0.9$

$\phi M_n = \phi A_s f_y (d - a/2)$

$= (0.9)(7.44)(60)(16.75 - 1.27/2) = 539.53 \text{ ft·k} > 461.8 \text{ ft·k}$ ✓

Check shear capacity:

$V_u = W_u \ell_n /2 = (9.77 \text{ klf})(27.5)/2 = 134.3 \text{ k}$

$\phi V_c = \phi 2 \sqrt{f'_c} \, b_w d = 0.75(2)\sqrt{5000}(108)(16.75) = 191.9 \text{ k}$

Try #4 bars at $S \leq d/2 = 16.75/2 = 8 \text{ in}$

$A_v = 0.4 \text{ in}^2$

$V_{s\,min} = A_v f_y d /s = (0.4)(60)(16.75)/8 = 50.25 \text{ k}$

Check spacing assumption

$V_s = 50.25 \text{ k} \leq 4\sqrt{f'_c} \, b_w d = 4\sqrt{5000}(108)(16.75) = 511.7 \text{ k}$ ✓

Check min. steel requirements:

$A_{v,min} = \begin{cases} 50 \, b_w S /f_y = 50(108)(8)/60,000 = 0.72 \text{ in}^2 \\ 0.75 \sqrt{f'_c} \, b_w d /f_y = 0.75\sqrt{5000}(108)(16.75)/60,000 = 1.59 \text{ in}^2 \end{cases}$

$0.72 \text{ in}^2 > 0.4 \text{ in}^2$ NG

Try #8 bars at $S \leq d/2 = 16.75/2 = 8 \text{ in}$

$A_v = 0.79 \text{ in}^2$

$V_{s\,min} = A_v f_y d /s = (0.79)(60)(16.75)/8 = 99.24 \text{ k}$

Check Spacing assumption

$V_s = 99.24 \text{ k} \leq 511.7 \text{ k}$ ✓

Check min. steel requirements:

$A_{v,min} = \begin{cases} 0.72 \text{ in}^2 \\ 1.59 \text{ in}^2 \end{cases}$

$0.72 \text{ in}^2 < 0.79 \text{ in}^2$ ✓

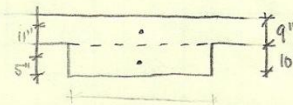$\phi V_n = \phi V_c + \phi V_{s\,min} = 191.9 \text{ k} + 0.75(99.24) = 266.33 \text{ k} > 134.3 \text{ k}$ ✓

Use #8 bars at 8" spacing

Deflection:

$I_e = (M_{cr}/M_a)^3 I_g + [1 - (M_{cr}/M_a)^3] I_{cr}$ $I_g = I_s + I_b$

$\bar{y} = \dfrac{\Sigma A d_y}{\Sigma A} = \dfrac{(10")(108)(5") + (9")(82.5")(16)}{(10)(108) + (9)(82.5)}$

$= 9.48"$

$$I_s = bh^3/12 + Ay^2 = (82.5)(9)^3/12 + (82.5)(9)(16.75-9.48) = 9852.98 \text{ in}^4$$
$$I_b = (108)(10)^3/12 + (108)(10)(9.48-5) = 13838.4 \text{ in}^4$$

$$I_g = 9852.98 + 13838.4 = 23691.38 \text{ in}^4$$

$$\bar{y} : \frac{b\bar{y}^2}{2} + nA_s\bar{y} - nA_s d \qquad\qquad n = \frac{\varepsilon_s}{\varepsilon_c} = 7.20$$

$$= \frac{(82.5)}{2}\bar{y}^2 + 7.2(7.44)\bar{y} - 7.2(7.44)(16.75) \qquad A_s = 7.44 \text{ in}^2$$
$$\qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad \angle(24)\#5$$

$$= 41.25\bar{y}^2 + 53.57\bar{y} - 897.26$$

$$\bar{y} = 4.06"$$

$$I_{cr} = (82.5)(4.06)^3/12 + (82.5)(4.06)(4.06/2)^2 + 7.2(7.44)(16.75-4.06)^2$$
$$I_{cr} = 10466.78 \text{ in}^4$$

$$M_{cr} = f_r I_g / y_t = \left[7.5\sqrt{5000}\,(23691.38)/9.48 \right] = 110.45 \text{ ft·k}$$
$$M_a = 461.8 \text{ ft·k}$$

$$I_e = (110.45/461.8)(23691.38) + \left[1 - (110.45/461.8)\right](10466.78)$$
$$I_e = 13629.74 \text{ in}^4$$

Live Load Δ Deflection
$$5w_{LL}\,l^4/385 E_c I_e = 5(2.28)(27.5)^4(1728)/(385)(4030.5)(13629.74)$$
$$= 0.53 \text{ in}$$
$$l/360 = (27.5)(12)/360 = 0.92" > 0.53" \checkmark$$

Total Load Δ
$$5w_{LL}\,l^4/385 E_c I_e = 5(7.37)(27.5)^4(1728)/(385)(4030.5)(13629.74)$$
$$= 1.72"$$
$$l/240 = (27.5)(12)/240 = 1.375" < 1.72" \text{ NG}$$

$$I_{req'd} = 13629.74\,(1.72/1.375) = 17049.57 \text{ in}^4$$

$$\therefore I_{cr} \geq 14961.66 \text{ in}^4$$

$$\bar{y} : \frac{b\bar{y}^2}{2} + nA_s\bar{y} - nA_s d \qquad\qquad \text{Try } 24 \#7$$
$$\qquad\qquad\qquad\qquad\qquad\qquad\qquad\qquad \angle A_s = 14.4 \text{ in}^2$$

$$= 41.25y^2 + 103.68\bar{y} - 1736.64$$
$$= \bar{y} = 5.35$$

$$I_{cr} = 17685.33 \text{ in}^4$$
$$I_e = 19121.81 \text{ in}^4$$

Live Load Δ
$$5(2.28)(27.5)^4(1728)/(385)(4030.5)(19121.81) = 0.37" < 0.92" \checkmark$$
Total Load Δ
$$5(7.37)(27.5)^4(1728)/(385)(4030.5)(19121.81) = 1.22" < 1.375" \checkmark$$

Need to recheck moment capacity for beam bottom reinforcement since bars changed from (24)#5 to (24)#7 to control deflection

(24) #7 $A_s = 14.4 \, in^2$

$a = A_s f_y / 0.85 f'_c b = (14.4)(60) / 0.85(5)(82.5) = 2.46''$

$c = a/\beta = 2.46/0.80 = 3.08'' < 0.375d = 0.375(16.75) = 5.98''$

∴ tension controlled, $\phi = 0.9$

$\phi M_n = \phi A_s f_y (d - a/2)$
$(0.9)(14.4)(60)(16.75 - 2.46/2) = 1005.7 \, ft \cdot K > 461.8 \, ft \cdot K$ ✓

Appendix G: R.S. Means 2009 Details

G.1 Two-Way Flat Slab with Drop Panels

Assembly B10102229000		Based on National Average Costs				
Flat slab, concrete, with drop panels, 12" slab/9" panel, 16" column, 35'x35' bay, 40 PSF superimposed load, 202 PSF total load						
Description	**Quantity**	**Unit**	**Material**	**Installation**	**Total**	
C.I.P. concrete forms, beams and girders, exterior spandrel, plywood, 12" wide, 4 use...	0.03500	SFCA	0.05	0.25	0.30	
C.I.P. concrete forms, elevated slab, flat slab with drop panels, to 15' high, 4 use, incl...	0.99900	S.F.	1.71	4.14	5.84	
Reinforcing steel, in place, elevated slabs, #4 to #7, A615, grade 60, incl labor for acc...	3.84600	Lb.	3.42	1.27	4.69	
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	1.07400	C.F.	4.41	0.00	4.41	
Structural concrete, placing, elevated slab, pumped, 6" to 10" thick, includes vibratin...	1.07400	C.F.	0.00	1.13	1.13	
Concrete finishing, floors, manual screed, bull float, machine float & steel trowel (wal...	1.00000	S.F.	0.00	0.67	0.67	
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.06	0.07	0.13	
Total			**$9.65**	**$7.53**	**$17.18**	

Figure G.1.1 - R.S. Means 2009 Two-Way Flat Slab Cost Estimate. Courtesy meanscostworks.com

A 35'x35' bay is used instead of a 30'x30' because the depth of the slab and drop panels is more accurately represented in the 35'x35' case. The loads are slightly underestimated in the R.S. Means estimate too.

G.2 Pre-Cast Hollow Core Planks on Steel Beams

Assembly B10102303600		Based on National Average Costs				
Precast concrete plank, 2" topping, 10" total thickness, 30' span, 100 PSF superimposed load, 180 PSF total load						
Description	**Quantity**	**Unit**	**Material**	**Installation**	**Total**	
C.I.P. concrete forms, elevated slab, edge forms, to 6" high, 4 use, includes shoring,...	0.10000	L.F.	0.02	0.38	0.40	
Welded wire fabric, sheets, 6 x 6 - W1.4 x W1.4 (10 x 10) 121 lb. per C.S.F., A185	0.01000	C.S.F.	0.20	0.34	0.53	
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	0.17000	C.F.	0.70	0.00	0.70	
Structural concrete, placing, elevated slab, pumped, less than 6" thick, includes vibrat...	0.17000	C.F.	0.00	0.24	0.24	
Concrete finishing, floors, manual screed, bull float, manual float, manual steel trowel	1.00000	S.F.	0.00	1.02	1.02	
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.06	0.08	0.14	
Precast slab, roof/floor members, grouted, hollow, 8" thick, prestressed	1.00000	S.F.	8.05	2.35	10.40	
Total			**$9.05**	**$4.41**	**$13.46**	

Figure G.2.1 - R.S. Means 2009 Pre-cast Hollow Core Plank Cost Estimate. Courtesy meanscostworks.com

This pre-cast concrete system is the closest assembly R.S. Means had to the pre-cast hollow core system. Similar to the flat slab system, the loads are underestimated when compared to the alternate system.

G.3 Steel Deck on Steel Beams and Girders

Assembly B10102601570			Based on National Average Costs		
Floor, cellular composite metal deck, concrete slab, 20 ga deck/20 ga plate, 10' span, 5" deep, 200 PSF superimposed load, 250 PSF total load					
Description	**Quantity**	**Unit**	**Material**	**Installation**	**Total**
C.I.P. concrete forms, elevated slab, edge forms, to 6" high, 4 use, includes shoring,...	0.05000	L.F.	0.01	0.19	0.20
Welded wire fabric, sheets, 6 x 6 - W1.4 x W1.4 (10 x 10) 121 lb. per C.S.F., A185	0.01100	C.S.F.	0.22	0.37	0.59
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	0.00900	C.Y.	1.00	0.00	1.00
Structural concrete, placing, elevated slab, pumped, less than 6" thick, includes vibrat...	0.00900	C.Y.	0.00	0.27	0.27
Concrete finishing, floors, manual screed, bull float, machine float & steel trowel (wal...	1.00000	S.F.	0.00	0.78	0.78
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.06	0.08	0.14
Metal decking, steel, cellular units, galvanized, over 15 Sq, 3" D, 20-20 ga	1.05000	S.F.	12.97	2.08	15.05
Total			**$14.25**	**$3.77**	**$18.02**

Figure G.3.1 - R.S. Means 2009 Steel Deck Cost Estimate. Courtesy meanscostworks.com

This deck system uses a 20-gage deck, when the alternate floor system uses an 18-gage deck. The slabs have a slight difference where the R.S. Means system has a 5" slab while the alternate system has a 3.5" slab. The 10'-0" span matches the minimum required span of the actual system, but the loads are underestimated when compared to the alternate system.

G.4 One-Way Slab with Continuous T-Beams

Assembly B10102198000			Based on National Average Costs		
Cast-in-place concrete beam and slab, 9" slab, one way, 16" column, 35'x35' bay, 40 PSF superimposed load, 169 PSF total load					
Description	**Quantity**	**Unit**	**Material**	**Installation**	**Total**
C.I.P. concrete forms, beams and girders, exterior spandrel, plywood, 12" wide, 4 use...	0.13300	SFCA	0.18	1.24	1.42
C.I.P. concrete forms, beams and girders, interior, plywood, 12" wide, 4 use, includes...	0.26800	SFCA	0.41	2.06	2.48
C.I.P. concrete forms, elevated slab, flat plate, plywood, to 15' high, 4 use, includes s...	0.88300	S.F.	1.43	4.55	5.98
Reinforcing steel, in place, elevated slabs, #4 to #7, A615, grade 60, incl labor for acc...	3.91200	Lb.	3.48	1.56	5.05
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	0.88000	C.F.	3.62	0.00	3.62
Structural concrete, placing, elevated slab, pumped, 6" to 10" thick, includes vibratin...	0.88000	C.F.	0.00	1.06	1.06
Concrete finishing, floors, manual screed, bull float, machine float & steel trowel (wal...	1.00000	S.F.	0.00	0.78	0.78
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.06	0.08	0.14
Total			**$9.20**	**$11.33**	**$20.53**

Figure G.4.1 - R.S. Means 2009 One-Way with Continuous T-Beams Cost Estimate. Courtesy meanscostworks.com

Similar to the two-way system, a 35'x35' bay states system requirements closer to the alternate system rather than the 30'x30', the actual bay size. The R.S. Means system has a 9" slab, but the loads are underestimated when compared to the alternate system.