2010 AE Senior Thesis

Final Report

Structural Redesign of University Medical Center at Princeton

Stephen Perkins-Structural Option

Advised by Dr. Linda Hanagan

University Medical Center at Princeton

Plainsboro, NJ

General Statistics

Location: Plainsboro, NJ
Owner: Princeton Health Care System
Size: 800,000 sq. ft.
Cost: $250 million
Occupancy: Mixed use
Construction Manager: Turner Construction
Dates of Construction: Aug 2007-Sep 2010

Structural

Structural Engineer: O'Donnell & Naccarato
Civil Engineer: French & Parrello Associates

Composite floor system:
3 1/4" lightweight concrete over 3", 20 Ga. composite metal deck
Structural steel framing system:
W-shape is typical shape for beams, columns
Lateral force resisting system:
Moment and braced frames handle lateral loads. HSS shape used for diagonal bracing.
Foundation:
Loads are transferred from steel columns to concrete piers and into concrete spread footings.
Large retaining walls exist along much of the building perimeter.
Tension only mini piles support footings at braced frame column locations.

Architectural

Architect: RMJM Hillier & HOK (Joint venture)

Scope:
6 story New Hospital
2 story Diagnostic and Treatment Facility
2 story Central Utility Plant
Layout:
Hospital is divided into eight different "Centers of Care" which allows specialized care while also providing comprehensive services.
Facade:
An insulated glass facade rises 92' on the southern face of the building providing daylight into nearly all of the 269 patient rooms. Other facade materials include: brick veneer, translucent fiberglass, metal panels, and aluminum window mullions.

MEP

MEP Engineer: Syska & Hennessy

Mechanical:
Combined variable and constant air control servicing a multitude of zones.
Fin tube radiation heating in main lobby.
Shell and Tube heat exchangers in basement and rooftop penthouse.
Electrical:
Serviced from (2) 13.2 kV feeders.
3333 kVA Dry-Type transformer steps to 277/480V.
Diesel fuel generators provide emergency power.
Lighting:
Most spaces utilize low voltage fluorescent lamp fixtures.

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Executive Summary

The New Hospital of the University Medical Center at Princeton is a six-story facility which rises 106'-0 above grade and is the centerpiece of an entire medical complex currently under construction in Plainsboro, NJ. The current structural system of the hospital is steel framing with a composite beam floor diaphragm. Lateral forces are resisted by eighteen braces frames spread throughout the building and two long moment frames on both the north and south exterior faces. Spread footings are located underneath each steel column to carry the loads to the ground.

The aim of this thesis is to eliminate net tension forces found at the base of the braced frames due to lateral loads. By redesigning the structure in concrete, the increase in building weight should provide enough additional compressive force to negate the tension at the footings. This would eliminate the need for tension-only mini piles to anchor the spread footings to bedrock.

Being that this facility is a hospital which contains sensitive equipment, the second goal of this thesis is to redesign the floor system with the intention of meeting particular vibration standards for sensitive areas including operating rooms, MRI rooms, and labs.

The structural system of the New Hospital was modeled, analyzed, and designed in RAM Structural System. The eighteen braced frames of the original lateral design were replaced with thirteen concrete shear walls placed at similar locations in the building. Even with the significant increase in building mass, wind forces still controlled the design in each of the principal directions. Columns sized at 24" square extend the first four stories of the building and are tapered to 20" square for the remainder of the structure's height. In order to avoid disruption to the floor plan, the column grid was preserved from the original design. Concrete moment frames replace the steel moment frames on the north and south facades of the hospital. The frames are designed to participate more in the east-west lateral force resisting system as opposed to the moment frames of the original design.

An 8" two-way flat slab was designed using RAM Concept and is found on the 1st and 2nd floors. This floor system just meets the 4000μin/s vibration velocity requirement for areas with sensitive equipment. The thickness of the slab reduces to 7" for the remaining floors in order to meet punching shear requirements. These slabs easily meet the standards for human perception of vibration due to walking.

Redesigning the structure in concrete has significant impacts on the architecture of the hospital. A Revit model of the hospital was created in order to investigate the interaction of the concrete columns with the prominent glass curtain wall on the south façade. The thicker concrete columns are successful at providing vertical breaks to the strong horizontal spandrel panels located at the floor levels. However on the interior side of the lobby, these same columns squeeze the space and at times produce over boding shadows on the lobby floor.

A cost investigation of the two structural systems concluded that the steel system is less expensive but this calculation did not include the additional foundation costs of the original steel system. A schedule analysis determined that the original steel design will be built in a timelier manner than the redesigned concrete structure.

Acknowledgements

Buildings are not designed and built by a single individual. Rather, it takes teams of professionals across numerous disciplines in order to complete the design and construction of great buildings. While my name is at the top of this report, it is unfathomable to think that I could have completed this project on my own without the help and advice of many others.

A special thank you is reserved for the Princeton Health Care System for allowing me to use the University Medical Center at Princeton for my thesis study.

Another special thank you to Turner Construction Company, especially Chris Auer and Miles Cava, for providing me with all of the necessary documents and reports needed to complete this project.

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I would like to thank my parents for their constant support and love they have shown for me throughout my entire student life.

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Introduction

The University Medical Center at Princeton (UMCP) is a new state-of-the-art medical facility currently under construction in Plainsboro, NJ. The project consists of a Central Utility Plant, a Diagnostic and Treatment Center (D&T) and a New Hospital. The site already has an existing building (Building #2) and it will be connected to the north side of the New Hospital as part of the project. The Medical Office Building (MOB) is only proposed at this time. The 800,000 square foot complex is set to be complete by the summer of 2012.

For the purposes of this particular thesis project, only the New Hospital will be considered (see Figure 1 below). This is the tallest portion of the complex at 91'-0" from grade to roof with a 14'-0" metal panel system above for a total height of 105'-0" above grade. The hospital is designed for a future four-story addition which extends the overall height above grade to 147'-0".

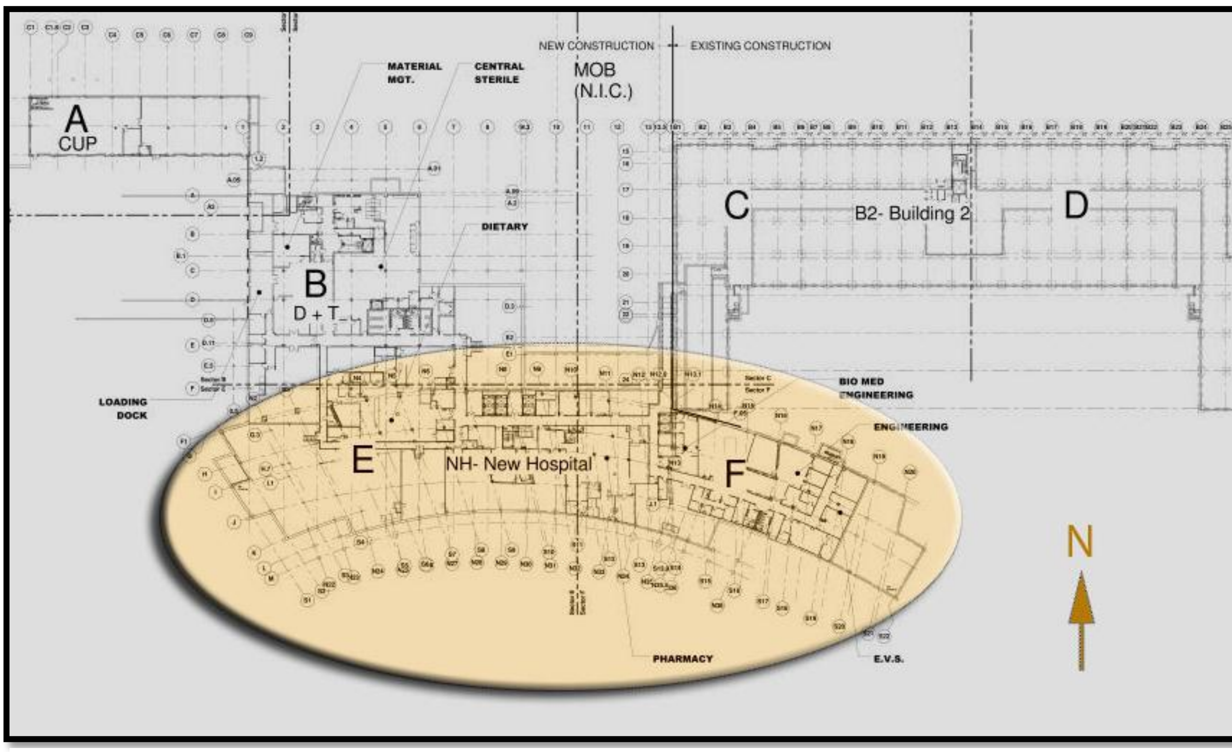


Figure 1: Overall plan of UMCP. The New Hospital is identified as the focus of this project.

Architectural Overview

The New Hospital is part of a medical campus which sits on an open site near Princeton University in Plainsboro, NJ. The hospital's long, curved footprint provides an appealing character to a facility which is mostly defined by rectangular forms. The broad curves extend outward almost as if it is welcoming visitors with open arms-an important expression for any hospital to make since a fair amount of visitors are anxious, fearful, and uncomfortable upon entry.

Figure 2: Evening view of south facade looking west

Along the entire length of the curved south façade is a 92' high curtain wall of clear, insulated glass which provides a great deal of natural daylight into the main lobby as well as all patient rooms on the south side of the hospital. To control excessive cooling loads during the warmer months, aluminum sunshades are attached at spandrel areas to provide appropriate shading from the sun. These sunshades emphasize the long, horizontal façade but are contrasted nicely by curtain wall offsets which run the entire height of the building and provide a break in the sunshade at four locations along the length of the façade.

Figure 3: Rendering on afternoon of summer solstice looking northeast.

The combination of horizontal sunshades, vertical curtain wall offsets, and a slightly off-center two-story glass enclosed lobby gives the southern façade of the hospital a unique and visually appealing feel which is not indicative of most medical facilities.

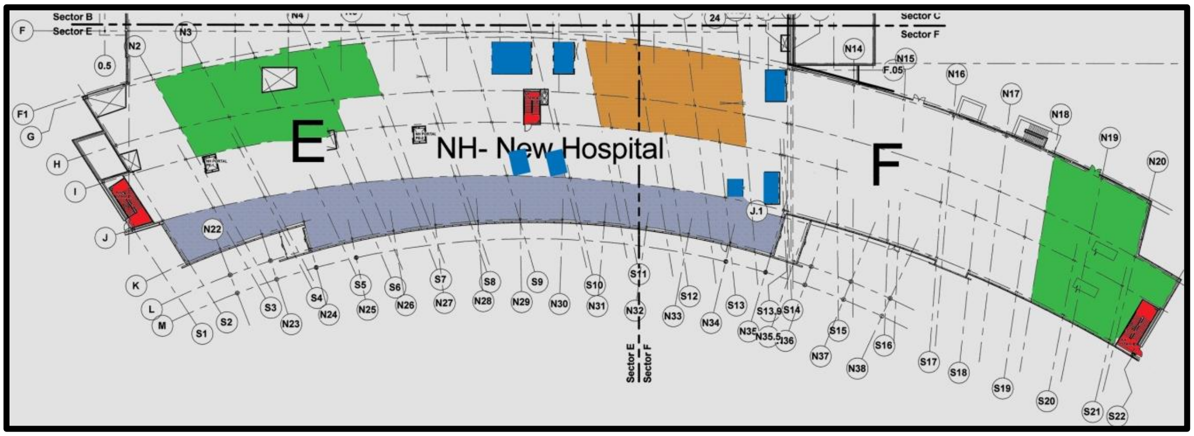


Figure 4: 1st Floor plan. Main lobby shown in purple, café shown in orange, main stairwells in red, elevators in blue, vibration sensitive areas in green.

The floor plan of the hospital follows the overall form of the building. The first floor is mostly public in the center of the plan with large lobbies, waiting areas, and a café located on the north side. The remainder of the first floor and part of the second floor is reserved for nursing facilities, examination rooms, and outpatient services.

Private patient rooms are located on remainder of the second floor all the way to the sixth floor. As mentioned earlier, the patient rooms are positioned on the exterior northern and southern faces to provide comforting views and better daylight for the patients.

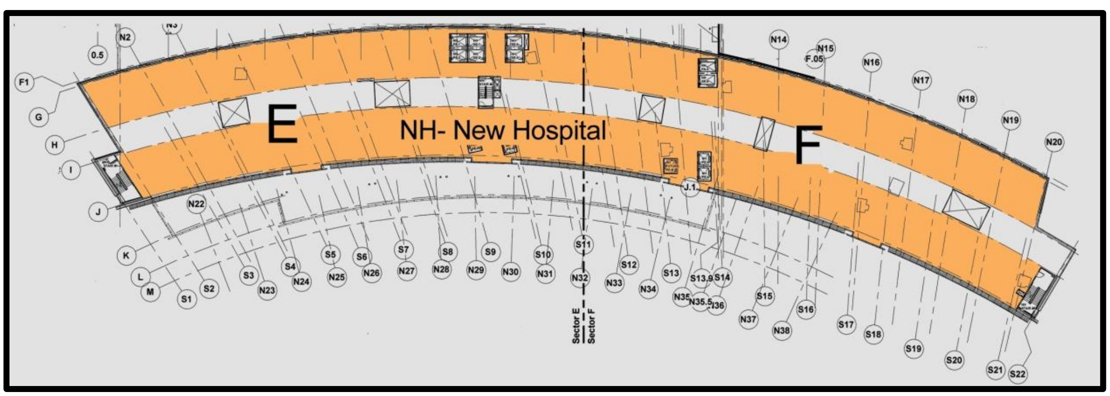


Figure 5: Typical floor plan. Patient rooms located on facades, shown in orange.

Most of the vertical transportation is centrally located with elevator lobbies and a main staircase at the center of the E-W axis. Two additional staircases are located at either end of the facility in order to meet fire code provisions as well as to provide added convenience of movement throughout the building.

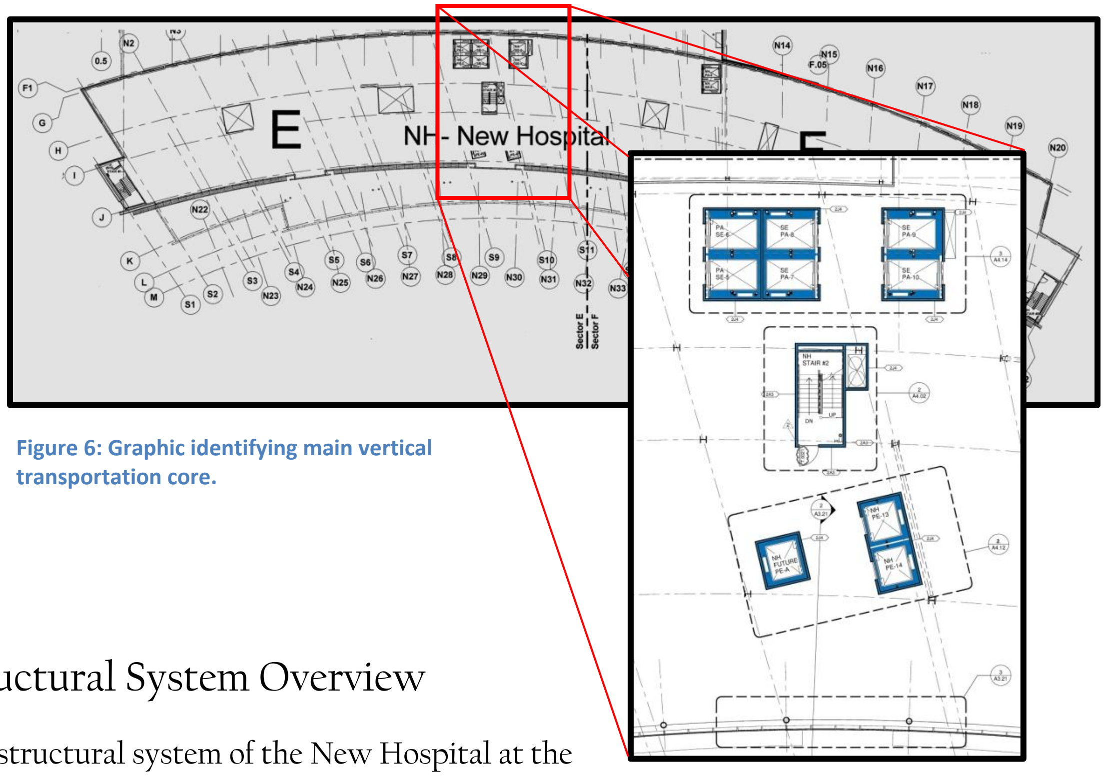


Figure 6: Graphic identifying main vertical transportation core.

Structural System Overview

The structural system of the New Hospital at the University Medical Center was designed by O'Donnell & Naccarato Structural Engineers using a Load Resistance Factor Design approach. It is a structural steel building with a composite floor diaphragm. Braced frames run in both directions and there are two long moment frames spanning the entire length of the building on both the south and north facades as seen below in Figure 2. Both the braced and moment frames are the building's main resistance to lateral load.

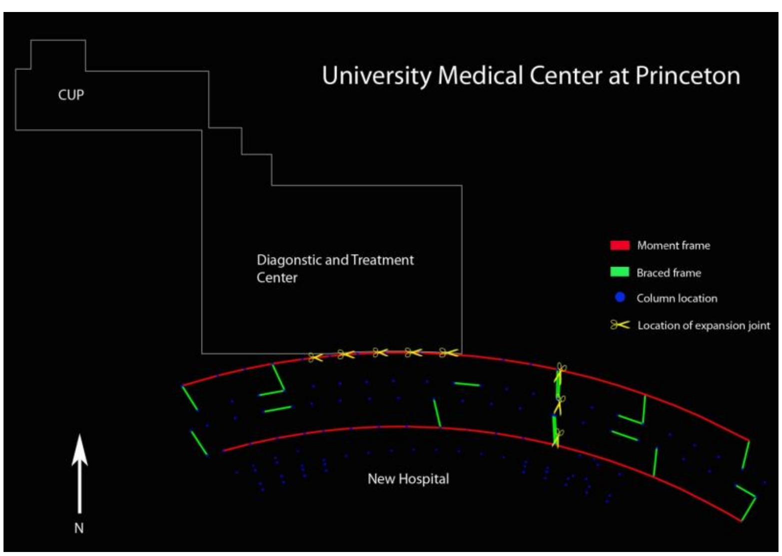


Figure 7: Layout of lateral force resisting system consisting of braced and moment frames. Expansion joints are shown.

Due to the great length of the building in the west-east direction, an expansion joint was placed at a distance from the western façade roughly equal to 2/3 of the total building length. This effectively splits the building into two different structures which behave on their own.

Foundation

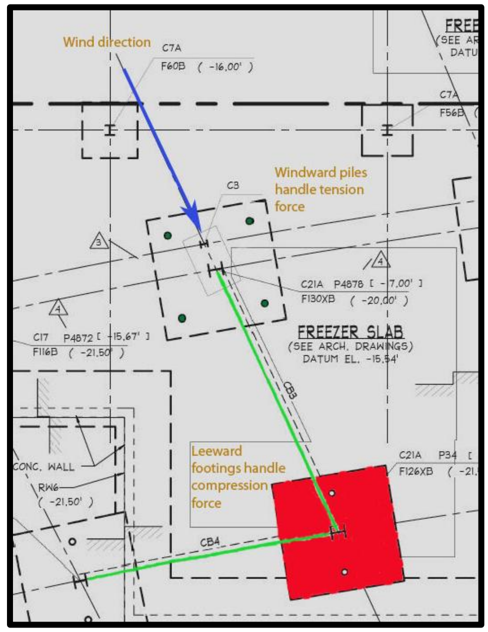


Figure 8: Detail of concrete pier connection to footing. Loads transmitted from column through pier to footing.

Concrete piers with sizes anywhere from 18" x 18" to 48" x 78" are attached to the base of the steel columns and transmit vertical load from the superstructure to the concrete spread footings. The size of these footings varies from as small as 3'-0" x 3'-0" x 14" to as large as 21' x 21' x 50". All footings supporting braced frame columns have mini-piles attached at their base in order to handle high tension forces resulting from lateral loading. These piles extend to decomposed bedrock (8'-30' deep). The top of all exterior footings are at a minimum depth of 42" below grade.

The floor at the base level is concrete slab-on-grade with thicknesses from 4"-12".

Huge concrete retaining walls with footings up to 17'-0" wide trace the perimeter of the foundation system.

Superstructure

The structural steel provides both gravity and lateral load resistance for the building. Columns are typically W14 while beams and girders range from W12-W27 shapes. Rectangular HSS shapes are used for the diagonal members in the braced frames and round HSS columns support the massive glass façade on the south face of the hospital. The HSS columns are intentionally exposed for architectural purposes. The floor layout is uniform and has a typical bay size of 30' x 30'.

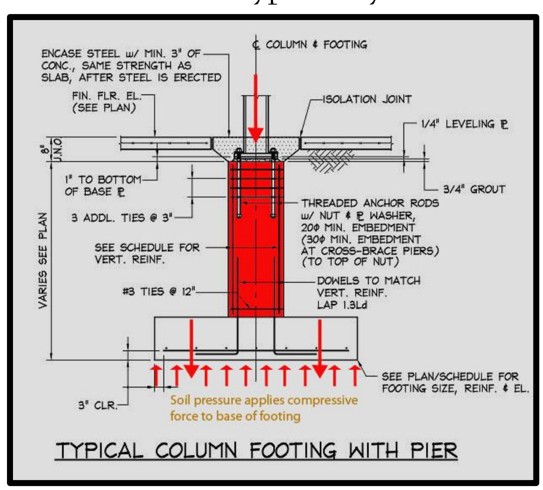


Figure 9: Compression and tension sides of braced frame. Red footing is in compression.

The floor system spanning over the main area of the building is composite construction. Typically, the concrete slab is 3-1/4" lightweight concrete poured over a 3" composite metal deck. In certain mechanical and roof areas, the floor system switches to a 6-1/2" normal weight concrete due to higher loads in those areas.

The composite floor is considered to act as a rigid diaphragm and therefore able to transmit lateral forces from the façade to the braced and moment frames.

Figure 10: Typical framing plan, west wing of hospital. Typical 30’x30’ bay and typical 30’x18’ bay are highlighted.

Figure 11: Typical 30'x30' bay.

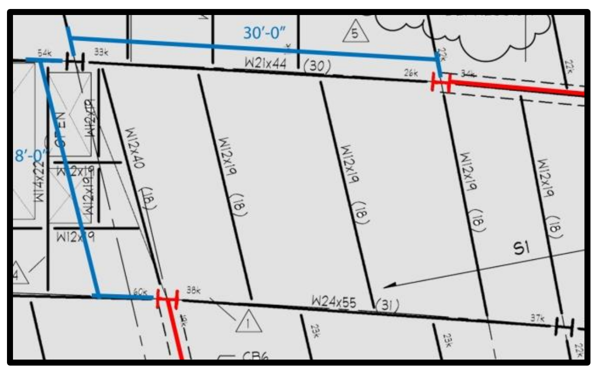


Figure 12: Typical 30'x18' span.

Lateral System

The primary components of the lateral force resisting system in the New Hospital are braced and moment frames. On the western wing of the facility, there are six braced frames running in the N-S direction. In the W-E direction, there are three braced frames and two long moment frames. The eastern wing has a similar layout with six braced frames in the N-S and three in the W-E as well as two moment frames in the W-E.

When lateral forces such as wind are applied to the hospital, the building façade is the first structural element to experience the forces. As shown in Fig. 3 below, a force resulting from wind pressure on the building façade strikes the glass curtain wall which deflects and develops stresses throughout its length. A mechanical connection between the façade and the floor diaphragm is located at every level and provides a load path from the curtain wall to the floor via the steel angle and headed stud. Once the force is received into the floor diaphragm from the steel angle connection, it is then distributed to all other structural elements attached to that particular floor diaphragm. Since the floor system is composite with both concrete and steel working together, it is considered to act as rigid diaphragm. That is to say that the composite floor system is stiff enough to induce equal lateral displacement of all attached structural elements.

By assuming a rigid diaphragm, the distribution of forces to all structural elements tied to this diaphragm is based upon relative stiffness of each member and frame. The stiffer frames receive more force than those frames which are less stiff.

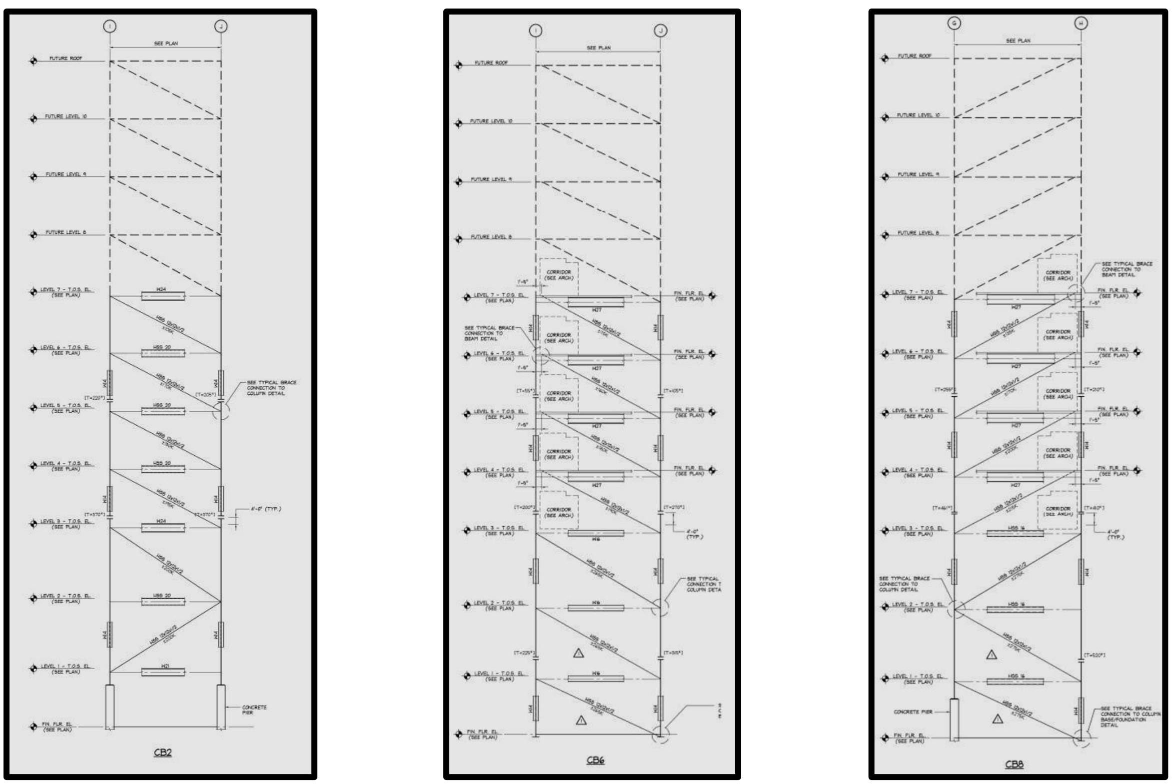


Figure 13: Elevations of braced frame #2, #6, #8

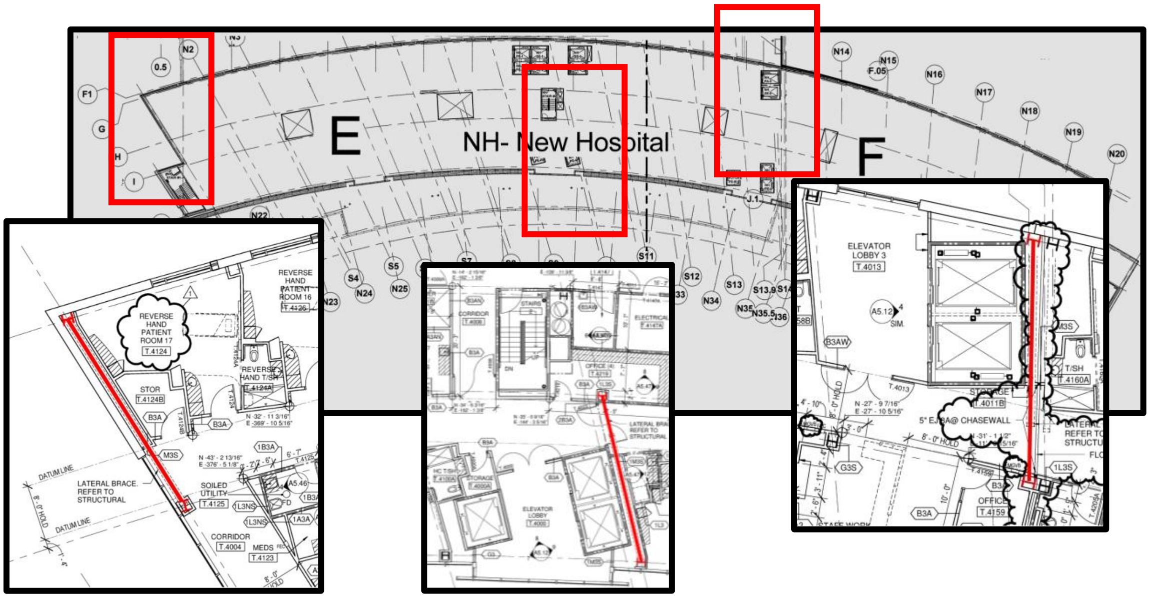


Figure 14: Location of frames #2, #6, #8 and close-up view of braced frame interaction with floor plan.

Once the proportional amount of force reaches the braced frame, it is transferred into the members of the frame. The frame is capable of handling this horizontal force because of the diagonal bracing between the columns. For this structure, the diagonal is a rectangular HSS tube which carries the force axially to the opposite corner of the panel. The tubes also resist the tendency for the frame to displace under load and provide lateral support to the columns. Figure 4 below shows how the load travels through the height of the frame and eventually to the base. It is here where the force is transmitted to the concrete pier and/or spread footing and into the ground.

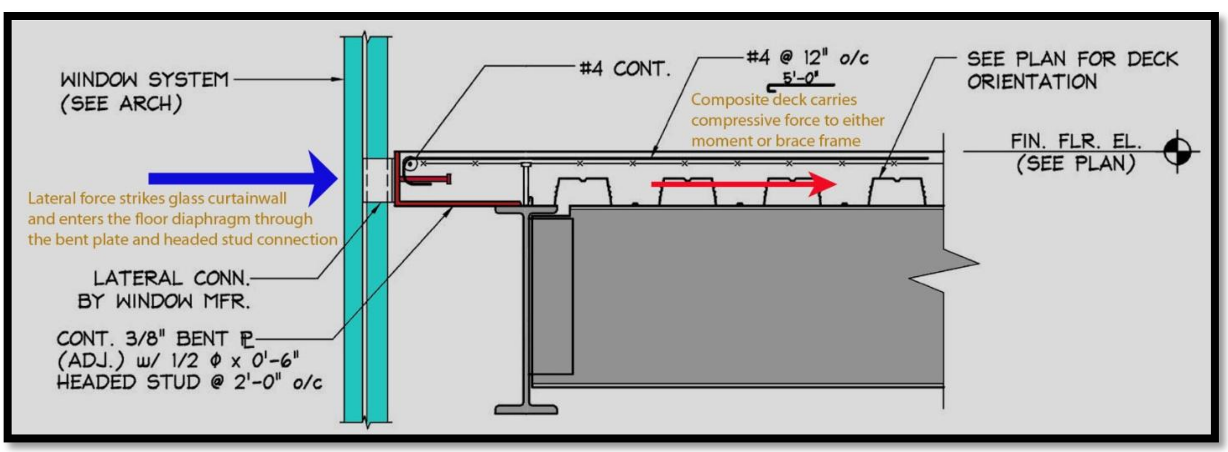


Figure 15: Diagram showing lateral load transfer from curtain wall to rigid floor diaphragm.

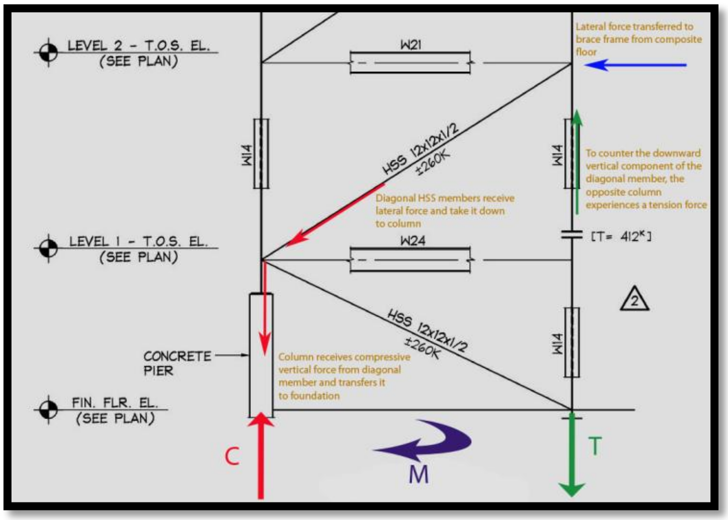


Figure 16: Lateral force is transferred from rigid diaphragm to braced frame.

In a braced frame like the one shown above, the columns on the "loading side" of the frame are in tension while the columns on the far side are in compression. This coupling of forces creates a moment that opposes the tendency of the lateral force to push the frame in a counterclockwise direction. In the case of wind blowing from the other direction, the forces in the columns will flip and the member that was once in tension would then be in compression and vice versa.

Once the force from the diaphragm is taken into the footing, it must be transmitted to the soil below. In the case of a compressive force pushing down, the footing will be driven into the ground and release that force into the soil. However if the force is a tensile one, it will try to pull the footing out of the ground.

In the original design of the New Hospital at the University Medical Center, the weight of the building was not great enough to overcome the tension force at the base of the frame. In order to avoid dramatically upsizing the spread footings underneath the frame to handle the tension, the structural engineers at O'Donnell and Naccarato designed mini-piles attached to the spread footings which anchor the frame to the bedrock located further below. These mini-piles are for tension forces only and effectively solve the overturning problems of the braced frames.

Due to the curved façade of the hospital, no frame is placed exactly perpendicular to loading. This means that while more of the frames are oriented towards the North-South direction, each braced frame participates in resisting loads from all directions. So for wind striking the building from the East, the braced frames which typically handle the load from the South help out in delivering these forces to the foundation. Also helping are the two long moment frames along the North and South facades. Moment frames do not have diagonal members but rely on the stiffness of the columns and beams to resist lateral loads. Without the diagonals, these frames are significantly less stiff than braced frames and consequently do not handle as much load. However, they do contribute to the overall lateral resisting system albeit mainly for loads acting along the East-West axis of the building.

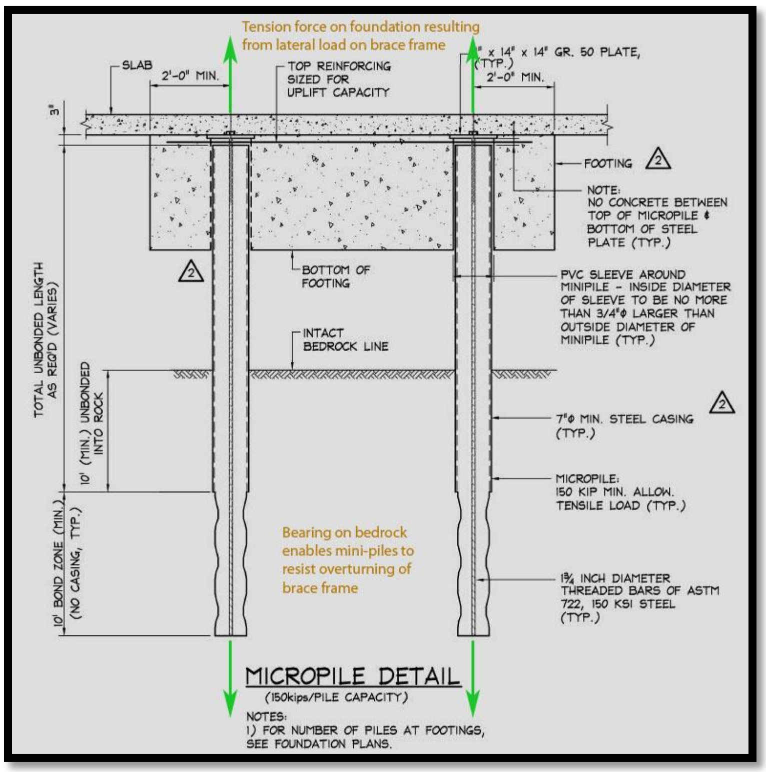


Figure 17: Mini-pile detail showing connection to spread footing.

Construction Overview

The project delivery method for the University Medical Center at Princeton is design-bid-build with a GMP contract between Princeton Healthcare System and Turner Construction. The total estimated cost of the New Hospital is roughly $115 million.

Construction of the New Hospital was set to begin in May 2009 and be completed by January 2012.

Thesis Objective

Upon analyzing the existing structural design of the New Hospital, it was discovered that design loads levied upon the structure will produce force conditions at the base of the lateral force resisting elements which require special design considerations at said locations. These considerations include a design solution which will sufficiently account for the tension force found at the base of each column in all 18 braced frames.

As mentioned earlier, all forces in a structure must eventually reach the ground and dispersed into the soil. Tension force at the ground level of a structure can be a design issue due to the fact that soil has no tensile capacity. That is to say that the interface between the building foundation and soil will not transfer any tension force from one to the other. When braced frames are used as a lateral force resisting element (as they are in the original design of the New Hospital) the windward side of the frame will always be placed in tension while the leeward side will be in compression. As those forces move down the frame, they increase until they reach the maximum at the base of the frame. The compression force in the column is not a significant concern as the concrete foundation can typically handle the transfer of that force to the soil. However if the compressive axial force at the base of the column due to gravity loading is not greater than the maximum tension force due to lateral loading, the foundation will see a net tensile force acting upon it.

There are several ways to address the issue of uplift force on a foundation. One solution is to increase the footing dimensions or design a mat foundation so that the self-weight of the footing can hold against the tension force. Another solution is to use a deep foundation with piles or caissons which will anchor the footing. This is essentially the design solution chosen by the structural engineers on the project. Tension-only piles were attached to the spread footings underneath each braced frame. These piles were anchored into bedrock which is located further below the base excavation.

This thesis project set out to investigate a different design solution. By redesigning the structure of the New Hospital in concrete rather than steel framing, it is the hope of the author that the increase in overall building weight will be great enough to overcome the tension force from lateral loading and eliminate the need for any special design consideration at the foundation level.

Another observation made during the analysis of the existing design was the vibration performance of the floor system. Since this is a hospital, spaces such as operating rooms or rooms which house sensitive equipment have stricter vibration criteria which should be met in order to achieve satisfactory building performance. The original composite beam floor system did meet generally accepted standards for vibration response due to human walking but fell short of standards for sensitive equipment. Therefore, this thesis project will also aim to improve the overall vibration performance of the floor system so that it meets generally accepted standards for sensitive equipment.

Fortunately, the two overarching goals of this project both share a common solution. Since concrete floor systems typically have better vibration performance than steel-framed floor systems, a redesign in concrete could solve the problem of tension at the base of the columns as well as allow the floor system of the hospital to meet the strict vibration standards for sensitive equipment.

Redesign Considerations

There are very good reasons as to why the structure was originally designed in steel rather than concrete. Availability of materials, speed of construction, labor costs, and architectural adaptability are all strengths of a steel design for the University Medical Center at Princeton in Plainsboro, NJ. Building design is full of many different variables which encompass several disciplines. Each situation calls for different design solutions and each project has a different set of conditions which govern design decisions. When undertaking a redesign as dramatic as changing the structural system from steel to concrete, it is important to consider all possible impacts this will have on the entire project.

Structural

Of course, a concrete redesign of a steel structure has a substantial impact on the structural design. Braced frames were the primary elements of the original lateral force resisting system. With a concrete design, those frames will be replaced with shear walls. The beams and columns in the exterior moment frames will be designed in concrete but will still act as a moment frame due to the inherent fixity of monolithic concrete construction. This can actually be an area of cost savings over the original design because it eliminates the extra labor needed to construct the steel frame moment connections.

The floor system also must be redesigned from the composite beam system of the original design. This redesign will allow for the opportunity to improve upon the vibration performance of the floor. Analysis of potential concrete floor systems yielded two viable alternatives: two-way flat slab without beams and one-way slab with beams. The two-way flat slab was chosen for this redesign because it will likely have a reduced overall floor thickness and have better vibration performance than the one-way system.

Finally, the increased weight of a concrete structure over a steel structure will create different conditions at the foundation of the building. While this extra weight is crucial to meeting the stated goal of this thesis, the original footing designs will likely not remain the same and therefore must also be redesigned.

Architectural

The redesign of the lateral system from braced frames to shear walls will have an impact on the floor plan of the New Hospital. Braces can be configured in different patterns to avoid corridors and other openings. While openings can be placed within shear walls, it makes for a more complicated design. Therefore, the placement of the walls will be important in order to minimize impacts to the floor plan. The best locations for shear walls typically are in elevator or mechanical shafts and stairwells because these are usually unobstructed spaces throughout the entire height of a building. Locations of possible shear wall locations are outlined in the figure below.

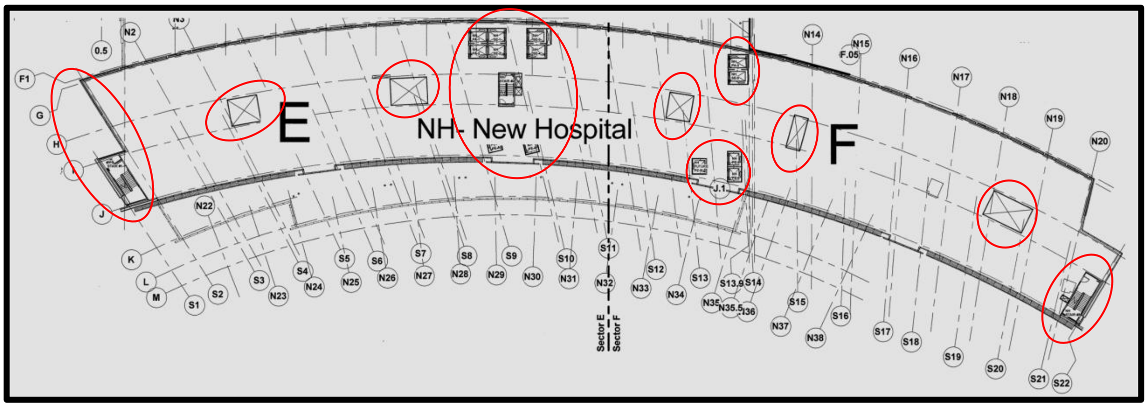


Figure 18: Overall floor plan with possible shear wall locations identified.

A concrete redesign will also affect the interaction between the structure and south façade. Currently, round HSS steel columns are designed to support the glass curtain wall on the south face of the hospital. This is the defining architectural statement of the building and the HSS shapes are intentionally exposed as an architectural feature in the lobby's interior. By redesigning the round HSS steel columns as circular concrete columns, the relationship between the structure and the façade will need to be considered.

Construction

Many of design decisions are controlled by issues surrounding constructability and cost. While a design solution may make sense from a structural or architectural perspective, it may not be able to be built at a reasonable price or within a reasonable timeframe. This is the nature of the building industry and consequently, the construction schedule and cost are typically motivating factors behind approving or turning down design ideas. This thesis project is no exception to this reality and therefore construction schedule for the New Hospital designed in concrete must be compared with schedule for the New Hospital designed in steel.

There are advantages and disadvantages for each material. Before the placement of concrete, formwork must be assembled and rebar must be laid out. Upon finishing the placing, the concrete must be allowed to reach a certain strength level before it can be expected to support floors above it. This dramatically slows down the construction process as compared to steel which can be built much quicker and has its entire strength characteristics upon assembly. However, a steel building has a much longer lead time due to fabrication and detailing of each individual member. Concrete buildings do not have nearly the lead time as steel which allows for a quicker start to construction.

A second consideration regarding construction is the overall cost of the two structural systems. As mentioned earlier, there are cost trade-offs between both materials. However, another factor is the cost of labor in particular locations. Certain areas have strong labor influences towards either steel or concrete which can dramatically affect the cost. In the Plainsboro, NJ region, many mid to large-sized buildings are built in steel. Therefore, contractors are much more familiar with steel construction and are likely better at it. Ultimately with more steel buildings there is increased competition between steel contractors which drives down prices.

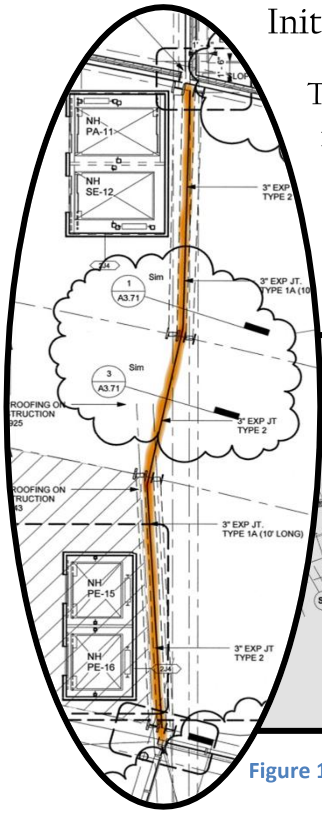
Structural Depth

Scope

The scope of this structural depth study will be the redesign of the New Hospital in concrete. The lateral force resisting system will consist primarily of specially reinforced concrete shear walls in both major axes with concrete moment frames providing additional resistance in the E-W direction. The existing steel columns will be replaced with concrete columns on the same grid so that bay sizes and column layout will not change. The existing composite beam floor system will be replaced with a two-way flat slab designed with the intention of meeting vibration criteria for sensitive equipment. The only concrete beams will be those found within the moment frames on the perimeter of the building. These beams will be designed to handle the weight of the exterior curtain wall just as the steel beams were in the original design. There will be no interior beams supporting the floor system. The foundation design, which is the overall goal of this thesis, will be square spread footings underneath each column and continuous wall footings underneath the shear walls. The intention is that the spread footings will be designed without the need for tension-only mini piles anchoring them in bedrock. There are some areas where a mat foundation might be advised in order to eliminate the congestion of continuous wall footings. The complete design of these mat foundations is outside the scope of this thesis project.

Initial Assumptions

The New Hospital is nearly 600 ft in the East-West direction which requires an expansion joint in order to control problems associated with façade movement due to loading and temperature changes. The original design locates the expansion joint at a distance about 2/3 of the overall building length measured from the westernmost façade. See figure below.



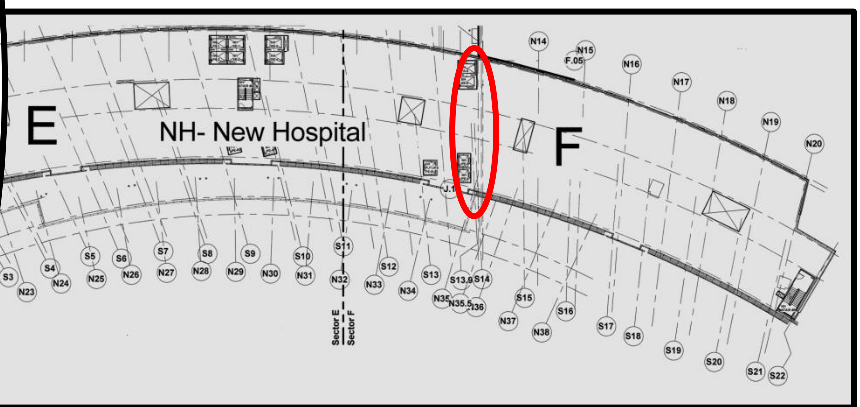


Figure 19: Location and detail of expansion joint separating New Hospital into west and east wings.

The expansion joint is detailed in the figure below. Essentially, an expansion joint splits a building into two isolated structures which act independently of one another. The floor diaphragm does not carry across the joint and the joint is not designed to for the transfer of forces from one diaphragm to the other. Indeed, the New Hospital at the University Medical Center at Princeton is two separate buildings.

For analysis and design purposes the structure was modeled with the acknowledgement of the existence of an expansion joint. This is a significant assumption because wind and seismic loading must now be calculated for two separate structures rather than just one. The details of these load determinations will be discussed later in this report.

As mentioned earlier, the scope of this thesis project is solely focused on the redesign of the New Hospital. This is a valid assumption because the structure of the hospital is not tied to any other structure of the Medical Center. The 2-story Diagnostic and Treatment (D&T building) is attached on the north side of the hospital building but is also isolated by another expansion joint. See figure below.

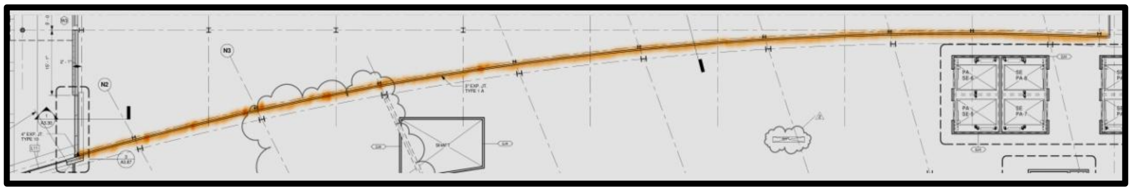


Figure 20: Expansion joint location separating New Hospital and D&T Building.

It is also important to note that the hospital is originally designed to be able to handle a four-story addition at a later date. This is an important design condition because it requires the structure to be designed not only as a six-story building but as a ten-story building as well. Adding four additional stories will increase the height and weight of the building which will change the wind, seismic, and gravity loads on the structure.

However, simply designing the hospital as a ten-story building and not considering that a certain portion of its life will be as a six-story building could affect the results of the foundation design. Without the four-story addition, the compressive load on the columns and shear walls will be reduced. It must be assured that this reduction does not result in net tension.

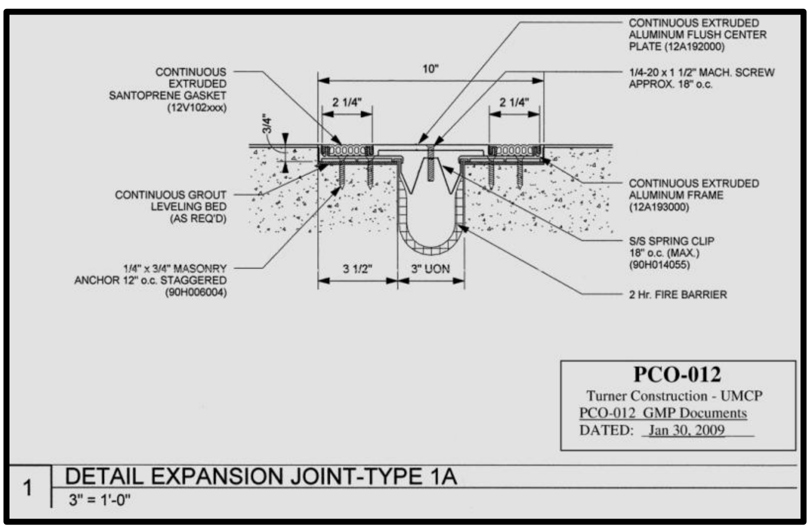


Figure 21: Cross-section detail of expansion joint.

Implementation of 3-D Computer Model

The curved form of the New Hospital at the University Medical Center makes it the centerpiece of the entire facility. The geometry of the building makes it difficult to correctly model. RAM Structural System was chosen as the platform to use for this thesis because of its interoperability with Revit Structure. Once the grid lines are created in a Revit file, they can be exported directly into RAM-effectively bypassing an extensive amount of geometry calculation.

RAM Structural System is a program which allows for analysis and design of structures. It is a powerful engineering tool but sound engineering judgment is required in order to model accurate structural behavior. The following is a list of assumptions which were used in the model of the New Hospital at UMCP.

Model Assumptions

- The two-way flat slab is considered to act as a rigid diaphragm
  - RAM Structural System will acknowledge the rigid diaphragm assumption for any slab but it is not capable of effectively designing a two-way slab. This part of the redesign was completed in RAM Concept which is a finite element program specifically used for slab design.
- The mass of the slab, walls, and columns were considered in the determination of the building period. Mass of the beams was ignored.
- The self-weight of the walls, columns, and beams are counted within the model. To insure that the self-weight of the slab was considered a surface dead load equal to the slab thickness times the density of lightweight concrete (120 pcf) was applied on both diaphragms.
- Columns are assumed to be braced against side sway by the shear walls and the slab
- Moment frame beams were modeled as fixed at column connection in order to transfer moment across the frame. In reality, the beam-column connection is not completely fixed but it behavior is closer to fixed than pinned.
- Rigid end zones were applied at all joints with a 50% reduction in order to achieve a more accurate beam length.
- P-Δ effects are considered in the model
- Walls were meshed at 3'-0" intervals in order to achieve a good balance between accuracy and analysis time.
- Walls were assumed to have no stiffness out-of-plane.
- Only slab openings of considerable size were included in the model. Typically these are openings for mechanical and elevator shafts.
- The moment of inertia for all concrete elements is as follows:

Columns = 0.7Ig

Beams = 0.35Ig

Walls = 0.35Ig

Slab = 0.25Ig

These values are for strength calculations per ACI 10.10.4.1. For serviceability, these values are modified per ACI 8.8.1.

- The strength of concrete, f'c, is as follows:

  Columns = 5 ksi

  Beams= 5 ksi

  Walls = 8 ksi

  Slab = 5 ksi

  Foundations = 3 ksi

<u>Loading Assumptions</u>

Due to the decision to model the expansion joint and separate the hospital into two individual structures, lateral loading calculations became more complicated. From this point forward, the two buildings will be referred to as the west wing and east wing.

Wind Load Calculation

Wind loading was determined for each wing of the hospital as if each was an individual building. In the N-S direction, both the windward and leeward wind pressures were calculated according to the Analytical Procedure set forth in ASCE7-05. In the E-W direction, the windward and leeward wind pressures were calculated as if the hospital were one structure. This assumption was made because the leeward side of each diaphragm is located within the building at the N-S expansion joint. Therefore, the west wing would experience windward pressure only while the east wing would see leeward pressure and vice versa. Since it is assumed that the length of the west and east facades are roughly the same, the wind loading in the E-W direction will be the same for both diaphragms and will not include leeward pressure.

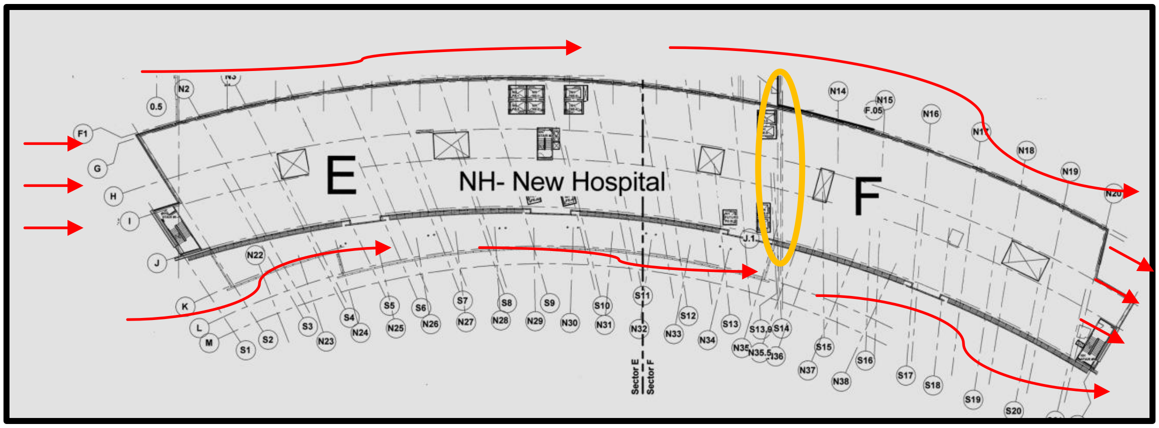


Figure 22: Wind loading in E-W direction is only windward. Leeward loading is on separate diaphragm.

Wind Load Cases

Wind does not always blow directly perpendicular to the facades of buildings. To account for a variable directionality of wind pressure, ASCE7-05 has defined four separate load cases to consider when applying wind load on a building.

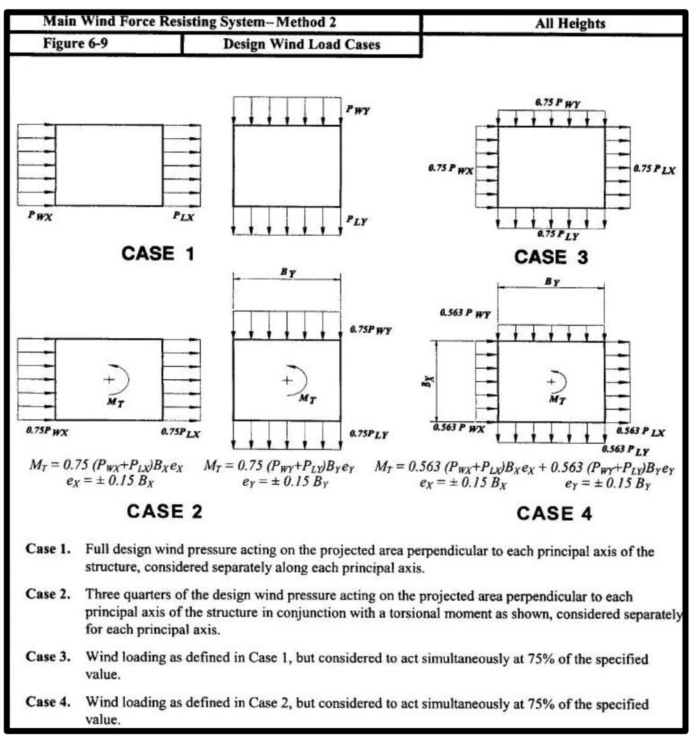


Figure 23: Wind load cases. Courtesy ASCE.

To be sure the correct wind loads were applied at the correct locations each load case was calculated and entered into the RAM model manually. Tables summarizing the loading values, angle of loading, and point of application for each of the four load cases can be found in the Appendix of this report.

Seismic Load Calculation

Similar assumptions were made for the seismic loading. Since two different structures are modeled, each structure has a unique set of periods due to different mass and stiffness values. When the moment of inertia reduction is applied to the shear walls (I=0.35Ig), the fundamental period of both structures exceeded the upper bound of CuTa set forth in Chapter 12.8.2 of ASCE7-05. Therefore, the period used to determine seismic forces is limited by CuTa

where $Ta = C_t h_n^x$ ASCE Eqn. 12.8-7

or

$Ta = (0.0019/C_w{}^{\wedge (1/2)}) * h_n$ ASCE Eqn. 12.8-9

The latter equation for Ta is permitted by the code for structures with concrete shear walls.

To determine which period to use for seismic load calculation, the fundamental period of the structure was calculated in RAM and is reported in the table below.

Period of Vibration-New Hospital								
Mode	Period	Circular Frequency	Modal Effective Mass					
			% Mass			% Sum Mass		
		(rad/s)	x	y	z	x	y	z
1	3.763	1.670	25.150	0.660	0.210	25.150	0.660	0.210
2	2.810	2.236	27.240	15.300	1.980	52.390	15.960	2.190
3	2.035	3.088	15.650	28.250	0.070	68.040	44.210	2.260
4	1.865	3.368	0.230	21.590	1.980	68.270	65.800	4.240
5	1.431	4.392	1.300	0.380	54.760	69.570	66.180	59.000
6	1.100	5.710	0.810	3.370	10.580	70.380	69.550	69.580
7	0.838	7.494	5.970	0.220	0.050	76.350	69.770	69.630
8	0.542	11.593	8.540	3.620	0.540	84.890	73.390	70.170
9	0.376	16.724	3.440	9.530	0.100	88.330	82.920	70.270
10	0.352	17.860	0.230	5.800	0.530	88.560	88.720	70.800
11	0.338	18.589	1.940	0.220	0.010	90.500	88.940	70.810
12	0.267	23.541	0.630	0.010	16.040	91.130	88.950	86.850
13	0.220	28.547	0.260	1.150	3.130	91.390	90.100	89.980
14	0.219	28.664	2.390	1.020	0.390	93.780	91.120	90.370

Figure 24: Modes of vibration for New Hospital. 90% of mass in each direction is activated within first 14 modes.

The first 14 modes of vibration are reported in order to reach a total mass participation of 90% in each direction. The period in the x-direction for each wing is significantly larger than the periods in the corresponding y and z-directions.

For the x-direction CuTa is lower than the fundamental period of both the west and east wing regardless of which Ta is used from the code. Therefore, eqn. 12.8-9 will be used to calculate Ta because it will yield lower design forces due to a higher design period.

For the y-direction, using eqn. 12.8-9 for Ta will yield a design period value which falls in-between the fundamental period values of the west wing and the east wing of the hospital. In order limit the number of load cases in RAM, Ta will be calculated using eqn. 12.8-7 which yields a design period below both fundamental periods in the y-direction. These assumptions do not give the most accurate seismic loading on the structure but the loading is conservative. The final design periods are listed in the table below.

Fundamental Period Along Principal Axes				
Direction	West Wing		East Wing	
	T	Mode	T	Mode
X	2.810	2	3.763	1
Y	2.035	3	1.865	4
Z	1.431	5	1.100	6

Figure 25: Design periods for both west and east wing of hospital.

Since both structures are 147'-0" above the ground with shear walls as the primary lateral system:

h_n = 147'

C_u = 1.68 (per ASCE7-05 Table 12.8-1)

RAM automatically calculates the weight of each wing separately and applies the seismic forces at each level per the Equivalent Lateral Force Procedure in Chapter 12.8 of ASCE7-05.

Other seismic assumptions:

- Lateral system is categorized as special reinforced concrete shear walls (ASCE7-05 Table 12.2-1)
  - Response modification coefficient, R = 6
  - Deflection Amplification factor, C_d = 5
- Horizontal irregularity Type 5: Non-parallel lateral systems (ASCE7-05 Table 12.3-1) applies to both structures

- No vertical irregularities apply to either structure
- Redundancy factor, ρ is equal to 1.0 for seismic design category C (ASCE7-05 12.3.4.1)
- Inherent and accidental torsion are accounted for within the seismic load cases of the RAM model
- The stability coefficient, Θ was not calculated for the structure. Therefore, P-Δ effects are considered

Seismic Load Cases

Due to the horizontal irregularity of the lateral force resisting system, it is required that additional seismic load cases be developed. ASCE7-05 12.5.3a states:

> "*…the most critical load effect…is permitted to be assumed to be satisfied if components and their foundations are designed for the following combination of prescribed loads: 100 percent of the forces in one direction plus 30 percent of the forces for the perpendicular direction…*"

This condition along with inherent and accidental torsion creates a significant number of seismic load cases which are to be evaluated by RAM Structural System.

Controlling Loads

In order to determine the controlling load case, base shears from each case were totaled and compared. This provides a good idea of how and where the lateral loads are acting on the structure. The controlling load cases are listed in the table below.

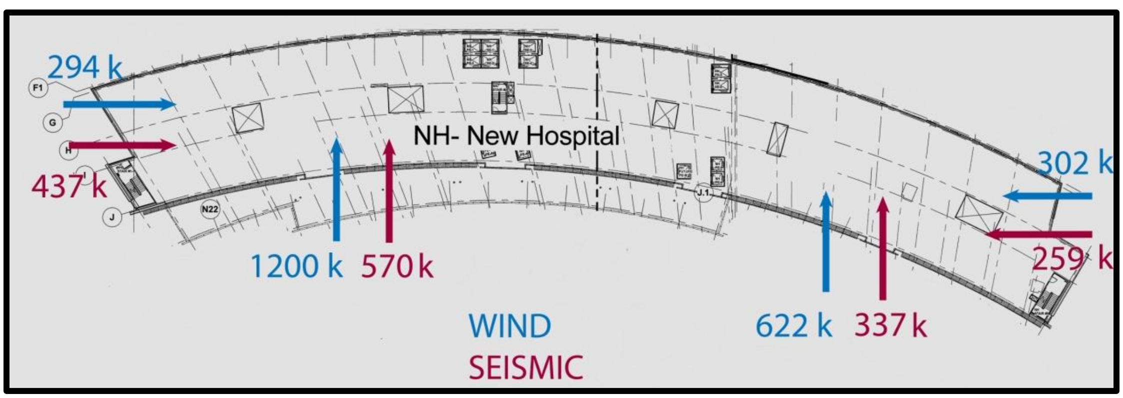


Figure 26: Wind and Seismic Base Shears for both wings of hospital.

These loads are unfactored and therefore are not the loads which will ultimately be used for design. In fact, the 1.6 multiplier on the wind load will cause wind to be the controlling base shear in all four cases.

Design Process

Slab Design

The first step in the redesign process was to determine the necessary thickness of the two-way slab in order to meet vibration criteria. As mentioned earlier, the upper floors of the hospital are mainly private patient rooms and nursing stations. While these areas of the slab should be designed for human comfort, it is probably unnecessary to enforce that these areas meet sensitive equipment requirements. The first floor of the western wing is attached to the D&T building to the north. Sensitive areas such as operating rooms are found in the D&T building. While the New Hospital and the D&T facility are separated by an expansion joint, there is a variety of machinery including x-ray machines, linear accelerators, EKG machines, and PET scan equipment which are used for operations in the D&T building but are located on the slab of the New Hospital. While the two structures are completely isolated from one another, designing the floor slab on the second floor to meet strict vibration criteria allows the owner some ability to adjust the floor plan in the future if that is desired. Therefore, the first and second floors will be designed to meet sensitive equipment criteria while the remainder of the floors will be designed to meet vibration standards for human comfort.

When designing a floor for vibration performance, there are certain factors which need to be considered. (Aalami) These include:

1. Vibration source
2. Transmission path of vibration
3. Characteristics of floor system
4. Sensitivity to vibration
5. Standard of acceptable response

Vibration Source

Vibrations are typically the result from external sources, internal sources, and machinery (Ungar). For this hospital, the focus will be on limiting the response of the floor to internal sources such as walking. Problems typically arise when floor systems and vibration sources reach a state of resonance which will dramatically amplify the vibration of the floor. The frequency of a typical footfall ranges from 2-3 Hz (Aalami).

Transmission Path

The medium by which vibration is carried from source to receiver is the transmission path. Most structural components act as a transmission path for excitations (Pavic). Certain parameters of the transmission path such as mass, modulus of elasticity, and damping will have an impact on its response to a dynamic force. Mass is defined as the weight of the floor divided by the acceleration of gravity ($=32.2 ft/s^2$). When performing a dynamic analysis, the modulus of elasticity can be increased by 25% over the static value (Aalami). Damping is a parameter which is difficult to precisely quantify. Research has shown that for a concrete floor with full-height partitions, a damping ratio of 5% is reasonable (Allen, Murray). All three of these parameters are important for determining the natural frequency of a floor system.

Floor Characteristics

The two essential characteristics of a floor system are the natural frequency and the peak acceleration (Aalami). ADAPT has published a technical note which details a simplified method for determining the natural frequency of a floor system.

$$f_n = (c * \emptyset)/a^2$$

where $c = [Eh^3/12(1-\nu^2)]*g/q$

f_n = nat. frequency [Hz]

a = span length [in.]

E = modulus of elasticity [psi]

h = slab thickness [in.]

ν = Poisson's ratio = 0.2 for concrete

g = acceleration of gravity = 32.2 ft/s^2

q = weight of slab/unit surface area [psi]

The equation for peak acceleration is a widely accepted standard which has been cited in several research articles.

$$a_p/g \leq P_o e^{-0.35fn}/\beta W$$

where a_p = peak acceleration [ft/s^2]

P_o = walking force [k]

β = damping ratio

W = effective panel weight w/ superimposed dead load [k]

Human Sensitivity to Vibration

There is no universal line in the sand which defines the difference between objectionable and non-objectionable vibration perception for humans. Acceleration of the floor in relation to its natural frequency is used to define a general range of acceptance. Research has shown that humans are more sensitive to accelerations with frequencies around 4-8 Hz (Allen, Pernica).

Design Standards for Acceptable Response

There are two different standards which need to be met for this thesis project. The first is human disturbance from vibrations due to walking. The second is disturbance of sensitive equipment from vibrations due to walking. The first criterion is defined in terms of a minimum natural frequency (Allen, Murray):

$$f_n \geq 2.86\ln(K/\beta W)$$

where K = constant

If the designed floor has a natural frequency greater than this value, then it will prevent human disturbance due to walking.

The second criterion is defined in terms of velocity. There are several criterion based upon the specific equipment that will be affected. For the New Hospital at UMCP, the vibration velocity limit is 4,000μin/sec (AISC). This is the accepted level for operating rooms, surgery, and bench microscopes with magnification up to 100x. The equation used to determine vibration velocity is given in AISC Design Guide 11 and is as follows:

$$V = U_v\Delta_p/f_n$$

Where V = vibration velocity [μin/s]

$U_v = \pi F_m f_o^2$ = constant for a particular walker and walking speed

F_m = maximum force [lb]

f_o= frequency of footstep pulse [Hz]

Δ_p = deflection due to unit load at middle of bay [in/lb]

With the criteria set, a trial slab thickness of 12" was selected and deflection calculations for the two-way slab were performed.

An accurate deflection of a two-way flat slab must consider the deflection of the panel in both directions.

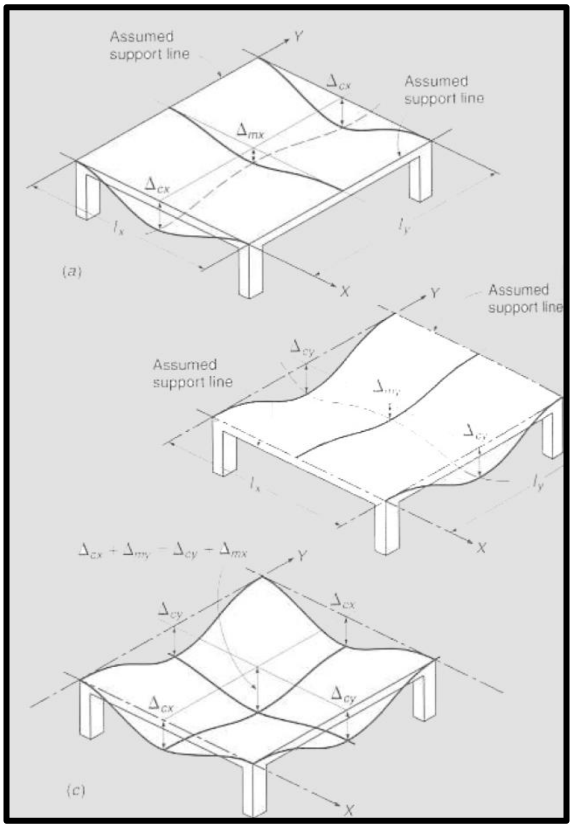


Figure 27: Two-way slab deflection. Courtesy Design of Concrete Structures.

$\Delta_T = \Delta_{Ma} + \Delta_{Cb} = \Delta_{Ca} + \Delta_{Mb}$

Drop panels and column capitals were sized in order to meet ACI code provisions. At this point, moment of inertias were calculated for different regions of the slab in order to obtain the maximum deflection value used in the vibration velocity equation stated above.

The vibration velocity of a 12" slab was found to be 881 μin/s – a significantly lower value than the stated goal of 4000 μin/s. This meant that the slab thickness could be reduced for a more efficient design. A spreadsheet was created in order to quickly investigate trial thicknesses.

An 8" two-way flat slab was calculated to have a vibration velocity of 3991 μin/s for a 185 lb person walking quickly at 100 steps/min. This is a substantial improvement over the previous composite beam floor system which, under the same design conditions, had a vibration velocity of nearly 43,000 μin/s. For the floors above the second floor, a 5.5" slab was deemed satisfactory for human perception of vibrations due to walking.

With the slab thicknesses determined, the design of the lateral force resisting system could move forward.

Lateral Force Resisting System

Before the components of the lateral system can be designed, the wind and seismic forces on those components must be determined. Once the footprint and height of the building is determined, wind forces can be calculated independently of all other design considerations. Seismic forces however are dependent upon the mass and stiffness of the structure. Once the slab thickness is defined, a reasonable estimate for the building mass can be made.

The original design assumption was to place shear walls in roughly the same locations as the originally designed braced frames. The original braced frame layout was determined to be a good design because of its balance and ability to limit overall torsion effects. If the shear walls were placed in the same locations, it is assumed that the benefits of the original system would be reflected in the redesigned system. In addition mirroring the braced frame layout would minimize impact on the original floor plan of the hospital which is one of the design considerations mentioned earlier in this report.

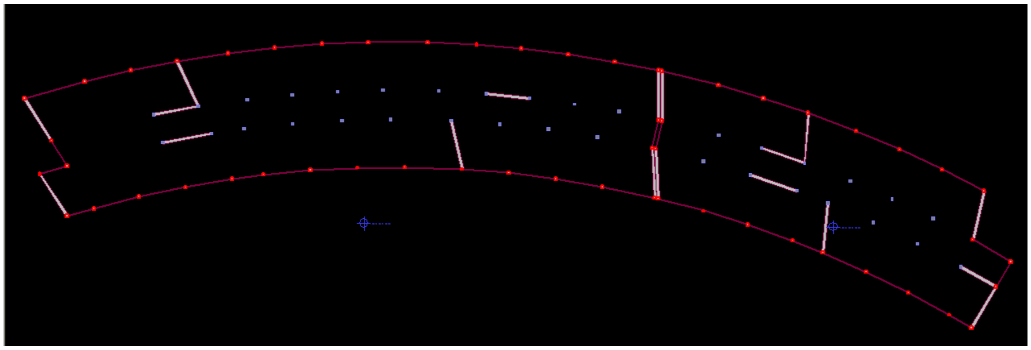


Figure 28: Initial shear wall layout. Walls placed at braced frame locations.

Once the shear walls were modeled and the slab thickness was defined, a modal analysis of the simplified structure could be performed to determine the fundamental periods in each direction. Several iterations of this process were performed in order to get a good understanding of how the structure would behave. One of the iterations is shown in the figure below.

Figure 29: Trial #34 wall layout. Shear wall groups located at elevator shafts.

Trial #34				
Wall f'c		10 ksi		
Wall thickness		24"		
Slab thickness		10"		
Design Periods				
Direction	West		East	
	Period	Mode	Period	Mode
X	1.682	1	1.314	2
Y	1.308	3	0.719	5
Z	0.941	4	0.585	6

The table to the left shows the design parameters of this iteration.

Figure 30: Design parameters for Trial #34. Fundamental period is also listed for each wing.

Under this iteration, seismic forces controlled in the x-direction and were close to controlling the y-direction as well. Since UMCP is located in New Jersey where seismic typically will not control, it seemed that the structure was far stiffer than it needed to be. Due to the fact that the code upper limit for the design period is CuTa, any fundamental period which exceeds CuTa will not change the seismic forces.

Therefore, the new approach was to "loosen up" the design so that the fundamental period would exceed CuTa. The new design is shown in the table below.

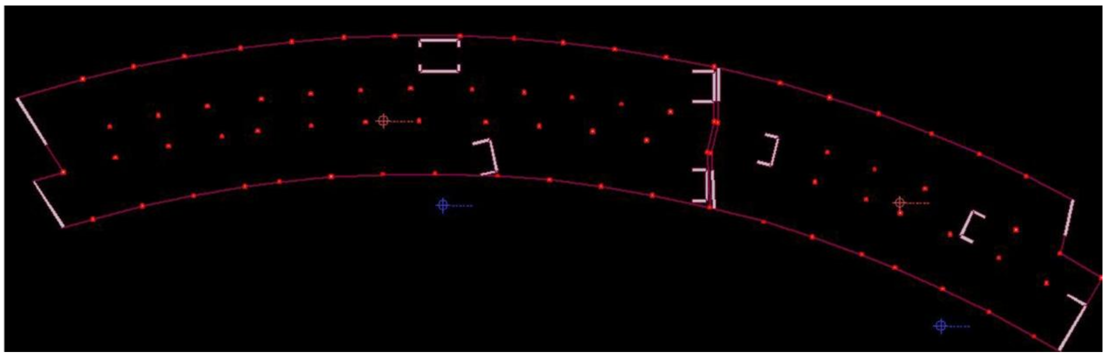


Figure 31: Final design iteration.

Final Iteration				
Wall f'c		8 ksi		
Wall thickness		12"		
Slab thickness		8"		
Design Periods				
Direction	West		East	
	Period	Mode	Period	Mode
X	2.810	2	3.763	1
Y	2.035	3	1.865	4
Z	1.431	5	1.100	6

Figure 32: Design parameters for final iteration.

Lateral Drifts

The story and overall building drifts were determined and checked against the limitations set forth in Chapter C Appendix C of ASCE7-05. The code limits building and story drift to h/400 where h is the height of the story. Other considerations include limiting story drift to no greater than 3/8" in order to prevent damage to non-structural partitions, cladding, and glazing. The code also allows for a 30% reduction in wind load due to the fact that a factored wind load is overly conservative for serviceability. Therefore, the load combination used to check wind drift is:

D + 0.5L + 0.7W (ASCE7-05 CC.1.2)

In addition to these provisions, Chapter 8.8 of the ACI code allows for an increase in member stiffness when checking serviceability. The designer is given the option of increasing the moment of inertia used in strength design by 40% or using I=0.7Ig. Therefore, when checking wind drift on the New Hospital, the walls were assigned a moment of inertia equal to 0.7Ig. For simplicity, the moment of inertia for the columns, beams, and slab were not adjusted because it was assumed that the drift of the building would be acceptable even without it. The values of the story drifts and overall building drift due to wind are listed below.

Story 10						
West Diaphragm						
	Displacements				Drift	
	x		y		x	y
Level	10	9	10	9		
Load Combo	(in)					
22	0.389	0.345	0.130	0.114	0.044	0.016
23	0.497	0.442	1.044	0.925	0.056	0.118
24	0.291	0.258	0.802	0.715	0.033	0.087
25	0.302	0.268	0.098	0.085	0.034	0.013
26	-0.074	-0.067	-0.676	-0.601	0.008	0.075
27	0.561	0.497	0.849	0.752	0.064	0.097
28	0.015	0.013	-0.518	-0.464	0.002	0.054
29	0.438	0.389	0.666	0.592	0.049	0.074

Figure 33: Wind drifts for west wing.

Story 10						
East Diaphragm						
	Displacements				Drift	
	x		y		x	y
Level	10	9	10	9		
Load Combo	(in)					
22	-1.310	-1.173	0.180	0.161	0.137	0.019
23	-0.374	-0.336	0.568	0.505	0.038	0.063
24	-0.178	-0.161	0.434	0.386	0.017	0.048
25	-1.019	-0.912	0.129	0.115	0.107	0.013
26	-0.689	-0.616	-0.298	-0.264	0.073	0.035
27	-0.502	-0.435	0.442	0.390	0.067	0.052
28	-0.655	-0.586	-0.156	-0.138	0.069	0.018
29	-0.955	-0.856	0.534	0.475	0.099	0.059

Figure 34: Wind drifts for east wing.

Allowable drift = h/600

[14' *12"/ft]/600 = **0.28"**

[147' *12"/ft]/600 = **2.94"**

Load combinations analyzed:

LC22: 1.0D + 0.5L +0.7W3

LC23: 1.0D + 0.5L +0.7W4

LC24: 1.0D + 0.5L +0.7W5

LC25: 1.0D + 0.5L +0.7W6

LC26: 1.0D + 0.5L +0.7W7

LC27: 1.0D + 0.5L +0.7W8

LC28: 1.0D + 0.5L +0.7W9

LC29: 1.0D + 0.5L +0.7W10

The wind loads used in the above combinations are the eight wind cases generated per ASCE Fig. 6-9. The drift criterion is easily satisfied. For similar drift calculations for story 7 and story 3 see the tables in appendix.

Seismic drift is handled a little differently because it is considered to be a check on strength rather than on serviceability. Therefore, the stiffness of the members is not increased as it is for the wind case. The story drifts are determined from RAM Frame and are then multiplied by the deflection amplification factor, C_d and then divided by the importance factor, I. The resulting value is then compared to the allowable story drift provided in Table 12.12-1 of ASCE7-05. The allowable story drift for the New Hospital is $0.010h_{sx}$. The table below lists all of the seismic story drifts as compared to the allowable.

Seismic Story Drifts- West Diaphragm						
	Displacements		Story Drifts			
			Ratio		Adjusted	
Load Case	E113	E117				
Story	Direction		Direction		Direction	
	x	y	x	y	x	y
	(in.)					
10	2.359	2.489	0.269	0.290	0.895	0.968
9	2.090	2.198	0.273	0.293	0.909	0.975
8	1.818	1.906	0.274	0.292	0.914	0.974
7	1.544	1.614	0.282	0.288	0.941	0.960
6	1.261	1.326	0.265	0.278	0.882	0.927
5	0.997	1.047	0.252	0.263	0.839	0.875
4	0.745	0.785	0.231	0.240	0.771	0.799
3	0.514	0.545	0.202	0.209	0.673	0.697
2	0.312	0.336	0.206	0.213	0.686	0.710
1	0.106	0.123	0.116	0.123	0.387	0.410

Allowable seismic drift = $0.010h_{sx}$

0.010*(14'*12"/ft) = **1.68"**

Figure 35: Seismic story drifts for west wing.

Seismic Story Drifts- East Diaphragm						
	Displacements		Story Drifts			
			Ratio		Adjusted	
Load Case	E114	E119				
Story	Direction		Direction		Direction	
	x	y	x	y	x	y
	(in.)					
10	4.703	1.630	0.461	0.182	1.537	0.608
9	4.242	1.447	0.479	0.186	1.597	0.620
8	3.763	1.261	0.491	0.188	1.637	0.628
7	3.272	1.073	0.500	0.189	1.667	0.629
6	2.772	0.884	0.499	0.185	1.663	0.616
5	2.273	0.700	0.460	0.177	1.533	0.589
4	1.813	0.523	0.449	0.163	1.497	0.543
3	1.364	0.360	0.420	0.142	1.400	0.473
2	0.944	0.218	0.425	0.142	1.417	0.472
1	0.519	0.076	0.209	0.076	0.697	0.254

Figure 36: Seismic story drifts for east wing.

With gravity and lateral loads analyzed and story drift within the allowable by code, the structure can be taken into the RAM design modules to confirm the design.

Organization of RAM Structural System

Before the final design is introduced, a brief explanation of RAM is necessary so that the reader has an understanding of how the program is organized.

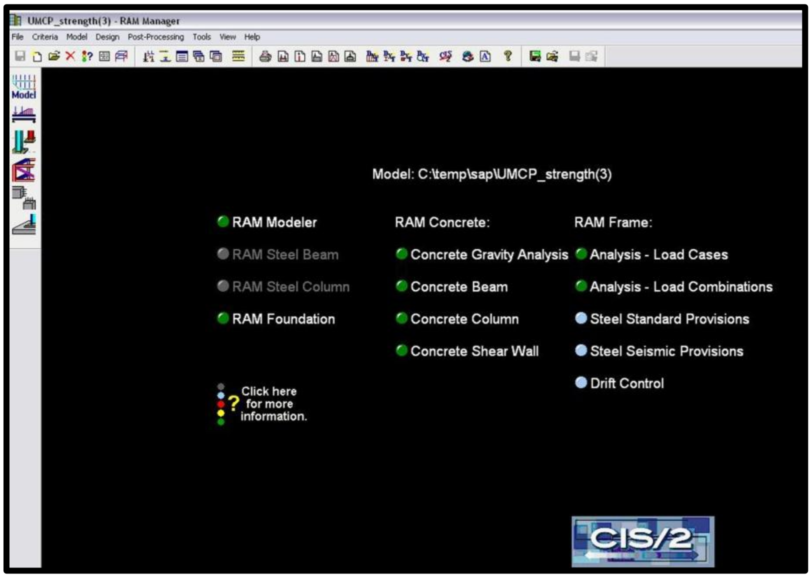


Figure 37: Screenshot of RAM Manager.

RAM Structural System is comprised of multiple modules, each performing different tasks. RAM Manager is the central hub which is where the user can access each of the different modules. The table below shows the name and purpose of each module within RAM Structural System.

RAM Modules	
RAM Modeler	Structure is modeled
	Section properties defined
	Gravity loads applied
RAM Frame	Lateral load cases defined/ analyzed
	Drift calculations
	Location of COM/ COR
	Load combinations generated
RAM Concrete	Gravity loads analyzed
	Design of beams
	Design of columns
	Design of shear walls
RAM Foundation	Design of foundation

Figure 38: Description of each RAM Module.

When designing the structural system, RAM makes assumptions regarding the loading used to confirm the design. These assumptions will be discussed in more detail later in this section.

<u>Confirmation of Loads</u>

Before the model was designed in RAM, the loads from the RAM Gravity Analysis and RAM Frame Analysis were checked for accuracy. To confirm the gravity loads, a column takedown was performed by hand and compared with the RAM Gravity loads. The seismic loads calculated in RAM Frame were also confirmed by hand using the Equivalent Lateral Force Method set forth in ASCE7-05. The wind loads were calculated by hand using the Analytical Procedure set forth in ASCE7-05 and were entered into RAM Frame manually. These loads are assumed to be correct.

With the confirmation of these loads within a reasonable percent, it can be assumed that the model is working correctly and that the forces used to check the strength of the members are accurate. Listed below are the load comparisons for gravity and seismic loading. A complete tabulated set of loads applied in RAM can be found in the appendix of this report.

Interior Column (H-N4)														
Column Below Level	Tributary Area	Tributary Area1	Tributary Area 2	Dead Load	Dead Load	Self Weight	Roof Live Load	Roof Live Load	Floor Live Load 1	Floor Live Load 2	Floor Live Load	Unfactored Column Load	RAM Column Design	% Difference
	sf	sf	sf	psf	k	k	psf	k	psf	psf	k	k	k	k
Roof	720	450	270	55	39.6	4.8	20.0	14.4	0.0	0.0	0.0	58.8	60.6	-3.0
10	1440	900	540	70	100.8	9.6			100.0	70.0	63.9	188.7	192.7	-2.1
9	2160	1350	810	70	151.2	14.4			100.0	70.0	127.8	307.8	324.7	-5.2
8	2880	1800	1080	70	201.6	19.2			100.0	70.0	191.7	426.9	456.6	-6.5
7	3600	2250	1350	70	252.0	25.0			100.0	70.0	255.6	547.0	589.5	-7.2
6	4320	2700	1620	70	302.4	30.8			100.0	70.0	319.5	667.1	722.2	-7.6
5	5040	3150	1890	70	352.8	36.6			100.0	70.0	383.4	787.2	854.9	-7.9
4	5760	3600	2160	70	403.2	45.0			100.0	70.0	447.3	909.9	990.0	-8.1
3	6480	4050	2430	70	453.6	55.8			100.0	70.0	511.2	1035.0	1129.3	-8.3
2	7200	4500	2700	80	511.2	66.0			100.0	100.0	583.2	1174.8	1274.8	-7.8
1	7920	4950	2970	80	568.8	77.1			120.0	120.0	669.6	1329.9	1435.1	-7.3

Figure 39: Column load takedown for column at grid line H-N4. Comparison with RAM forces is on the right.

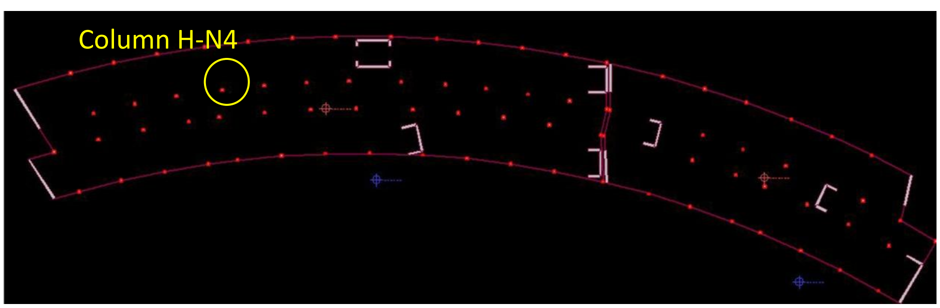


Figure 40: Plan identifying location of column at grid line H-N4.

Seismic Story Forces										
Level	Height Above Ground	Story Height	Weight	C_{vx}	Fx	Fy	RAM Fx	RAM Fy	% DIFF X	% DIFF Y
	(ft)	(ft)	(K)		(k)	(k)	(k)	(k)		
1	0	0	0	0	0	0	0	0	N/A	N/A
2	17	17	5290	0.008	5.30	10.14	5.69	10.86	6.9	6.6
3	35	18	5394	0.026	18.06	29.61	19.26	31.51	6.2	6.0
4	49	14	4539	0.038	26.68	40.70	27.62	42.03	3.4	3.2
5	63	14	4539	0.058	40.61	58.70	41.11	59.27	1.2	1.0
6	77	14	4539	0.081	56.80	78.65	57.5	79.41	1.2	1.0
7	91	14	4539	0.107	75.10	100.33	75.28	100.33	0.2	0.0
8	105	14	4539	0.136	95.41	123.60	94.69	122.38	0.8	1.0
9	119	14	4539	0.167	117.61	148.34	116.73	146.88	0.8	1.0
10	133	14	4539	0.201	141.65	174.44	140.58	172.73	0.8	1.0
Roof	147	14	3423	0.180	126.30	152.23	117.48	141.27	7.5	7.8
	Sum		45880	1.000	703.52	916.74	695.94	906.67	1.1	1.1

Figure 41: Calculated seismic forces. Comparison with RAM forces is shown on the right.

Final Design

The member sizes and reinforcement design was completed in RAM Concrete. Beams were designed in the beam module, columns in the column module, and shear walls in the wall module. The slab was designed in RAM Concept. Once those designs were completed and minor adjustments were made, the foundations were designed in RAM Foundation. The following is an explanation of each of the design modules and a summary of the design that was produced. Hand checks for the design and detailing of typical beams, columns, walls, slabs, and foundations can be found in the appendix of this report.

Slab Design

RAM Concept is a program which utilizes finite element modeling to analyze and design floor slabs and mat foundations. The following figures show the slab modeled in Concept.

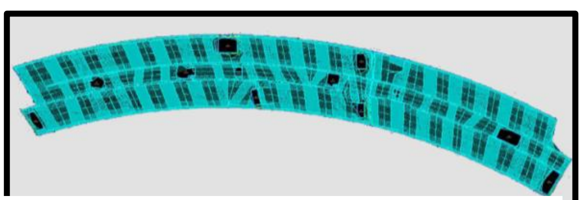


Figure 43: Design Strip Plan in Concept. Column and middle strips shown in each direction.

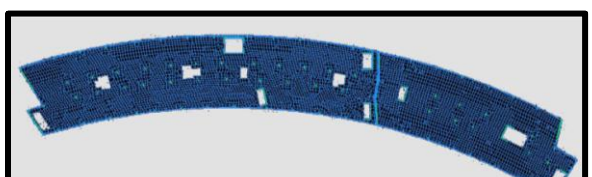


Figure 42: Element plan in Concept. Mesh was generated at 4.5'.

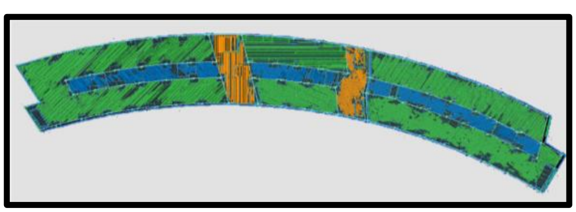


Figure 44: Slab loading. Green = 80 psf live. Orange = 100 psf live. Blue = 70 psf live.

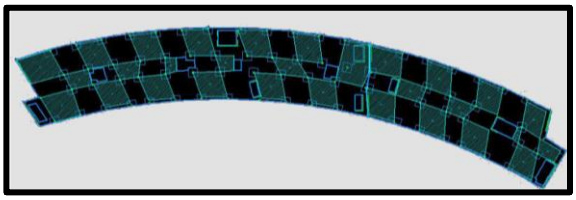


Figure 45: Load pattern 1

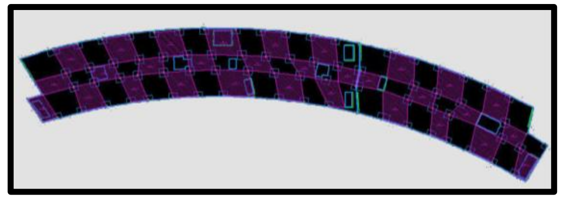


Figure 46: Load pattern 2

The program designs the slab in a way similar to the methods specified in the code. Span segments (shown above) are specified in the model and automatically generate column and middle strips which are to be designed. Live load reduction and pattern loading were applied within the Concept model whereas these conditions were not applied in RAM Structural System.

The load combinations specified in the model are as follows:

1. All Dead (Self-weight + superimposed)
2. Service (Dead + 1.0 Live)
3. 1.4 Dead
4. 1.2 Dead + 1.6 Live + 0.5 L_r
5. Long Term Deflection (3.35 Dead + 2.18 Live)

Design Assumptions

It is assumed that the slab is capable of successfully transferring lateral loads to the lateral resisting elements. Therefore, Concept was used exclusively for gravity design of the slab. Lateral deflection of the slab is assumed to be within reasonable limitations.

The pre-design assumption was that vibration would control the thickness of the slab. However, ACI Table 9.5c specifies a minimum slab thickness for control of deflections. According to this table, the minimum slab thickness for a 30'-0" span with f_y = 60 ksi is 10". This thickness is greater than what was calculated for vibration control (thickness = 8.0"). Indeed, designing a 10" slab would meet both vibration and deflection criteria easily. The drawback to this design solution is that the building weight is significantly increased by thickening the slab which would increase the seismic forces and impact both the gravity and lateral design.

According to ACI 9.5.3.4, strict adherence to Table 9.5c is not required so long as a reasonable determination of slab deflection is performed and meets the overall deflection criteria set forth in Table 9.5b.

Maximum permissible deflections:

$$L/240 = (30'*12"/ft)/240 = 1.50"$$

Under this provision, the seventh floor slab was designed with a thickness of 7" in order to meet vibration criteria for private patient rooms while also limiting the increase to the overall weight of the building.

The seventh floor slab was chosen to be modeled but in fact, this slab is typical for all floors above the second level. The first and second level slabs will be designed as 8" slabs in order to meet stricter vibration requirements which were explained earlier in this report. The roof was not designed for any specific vibration criteria and was designed to be 5.5" thick.

Design Results

The 7" slab was successfully designed in Concept to meet all code provisions. After analyzing the slab, it became clear that punching shear was the ultimate controlling condition of the slab design. Drop panels in certain slab areas had to be increased from 10'-0" dimensions to 14'-0" dimensions in order to increase the area for punching shear resistance. These same panels also had to be thickened as much as 9" below the slab in order to have enough capacity to handle punching shear. This design solution seems a bit extreme but it is considered a better solution than increasing the thickness of the entire slab which would likely lead to other design issues including seismic forces controlling in the E-W direction of the building.

The flexural capability of the slab was found to be well within acceptable limits and did not control the design of the slab.

In order to justify the use of ACI 9.5.3.4, an additional deflection check was performed by hand in order to confirm the results from Concept. The results of this calculation are listed in the table below. The full calculation can be found in the appendix of this report.

Dead Load Deflection				
Δ_{fcolD}	Δ_{fmidA}	Δ_{max}	$\Delta_{concept}$	% Diff
(in.)				
0.160	0.034	0.194	0.180	7.8

Figure 47: Slab deflection due to dead load only.

Dead + Live Load Deflection				
Δ_{fcolD}	Δ_{fmidA}	Δ_{max}	$\Delta_{concept}$	% Diff
(in.)				
0.304	0.065	0.369	0.350	5.4

Figure 48: Slab deflection due to dead + live load.

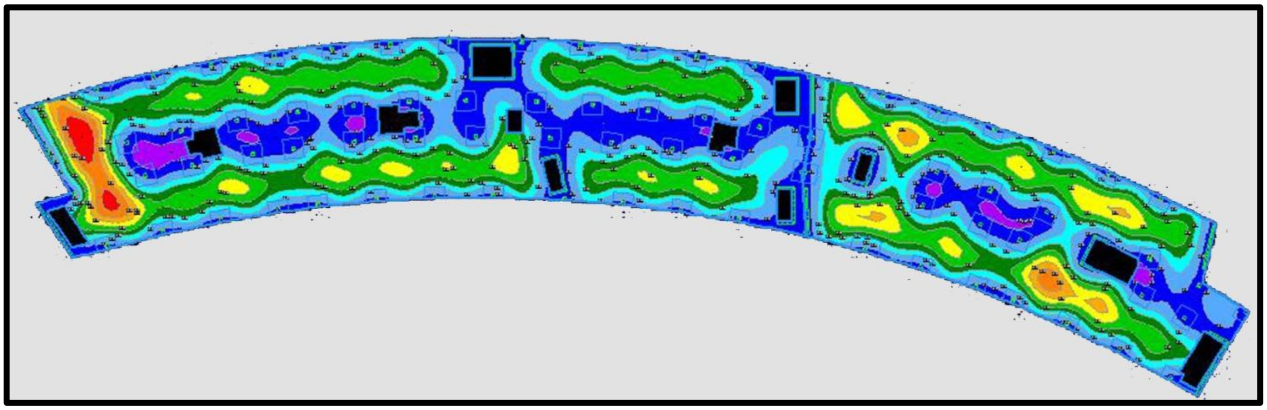


Figure 49: Deflection of floor slab under all dead + live loading. Red shows the largest deflection which was found to be within acceptable ranges.

At most columns, upsizing the drop panel dimensions was not enough to handle punching shear alone. Therefore, shear stud reinforcement was designed at the column locations shown below in order to provide the necessary capacity of the section to handle shear.

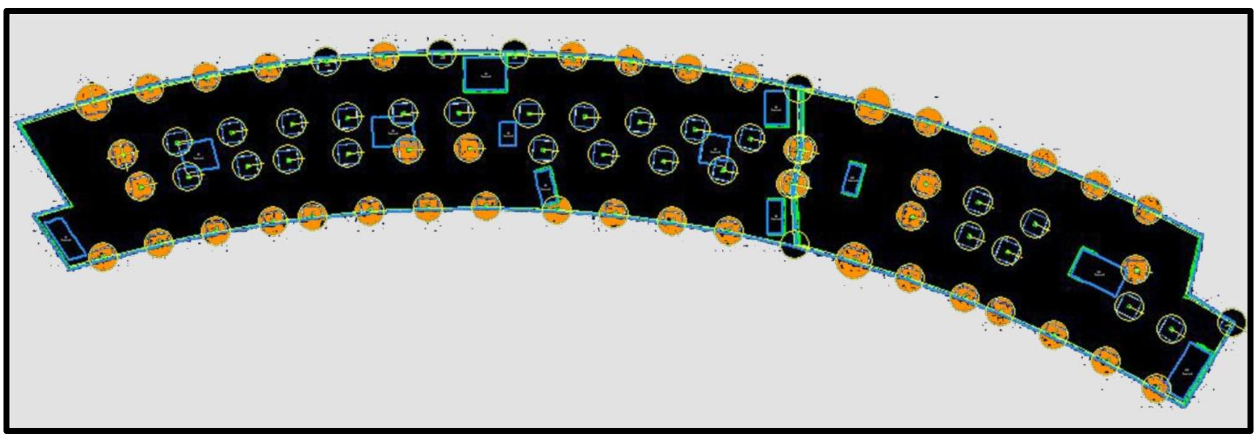


Figure 50: Design strip plan identifying locations where shear stud reinforcement was necessary to handle punching shear.

Hand calculations were performed to confirm the design provided by Concept. The entire design check can be found in the appendix of this report.

Wall Design

The concrete shear walls are the main elements of the lateral force resisting system. The walls were designed in the wall module of RAM Concrete. In order for the program to perform a design, all gravity and lateral loads must first be analyzed. As previously mentioned RAM Frame analyzes the wind and seismic forces. It then combines these forces with gravity loads from RAM Modeler to create the necessary amount of load combinations.

For UMCP, there were a total of 317 load combinations analyzed. The reason for this many combinations is that the hospital has a lateral system layout which is irregular. When these conditions exist, orthogonal effects of seismic loading must be considered (100% in one direction plus 30% in the orthogonal direction). When all eccentricities are considered, the number of load combinations expands significantly. Of course, computer software makes the analysis of all these combinations possible.

Design Assumptions

Each wall location in the plan was designed as a unique wall group. This is particularly relevant for c-shaped shear wall groups located at elevator shafts. Essentially, these walls are designed as one unit rather than three individual elements. There are a total of 13 wall groups; seven in the west wing and six in the east wing.

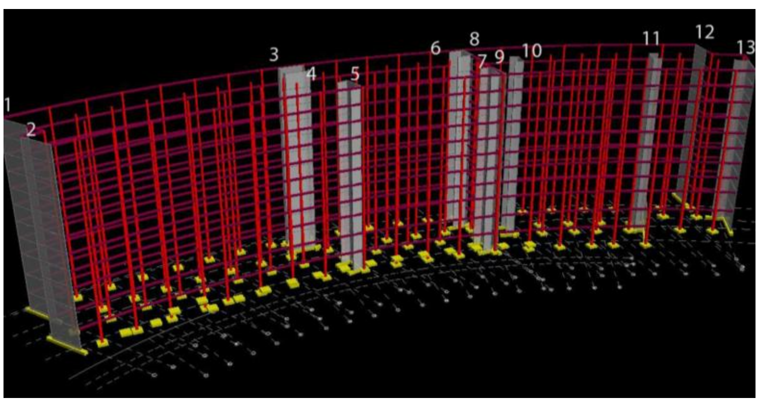


Figure 51: Perspective identifying the location of the 13 shear wall design groups.

In order to use a higher R-value in the seismic force calculation, the walls were designed as specially reinforced. This means that the reinforcement at the base of the wall is detailed in such a way so as to force hinge formation at this location. This behavior is desirable during an earthquake event because the concrete wall is actually designed to yield which dissipates more energy and lessens the forces on the structure. Hand calculations were performed in order to properly detail this behavior. These calculations can be found in the appendix.

In order to evaluate and design shear walls, RAM creates section cuts at critical wall sections and evaluates the forces at these locations. Each load combination is evaluated for each cut so it is conceivable that different combinations control at different locations along the wall.

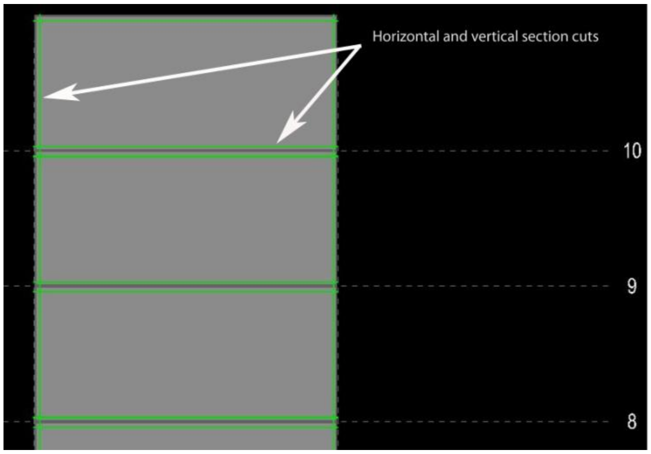
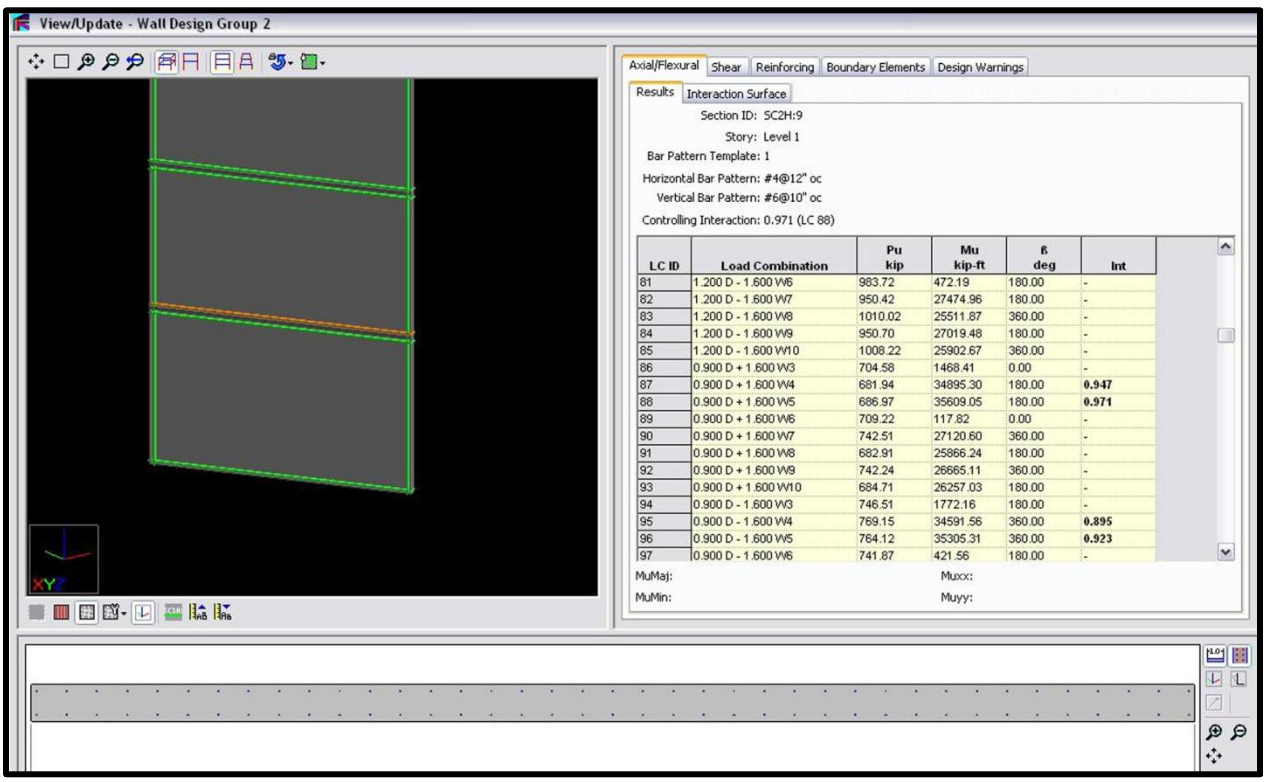


Figure 52: Elevation of shear wall with design section cuts shown in green.

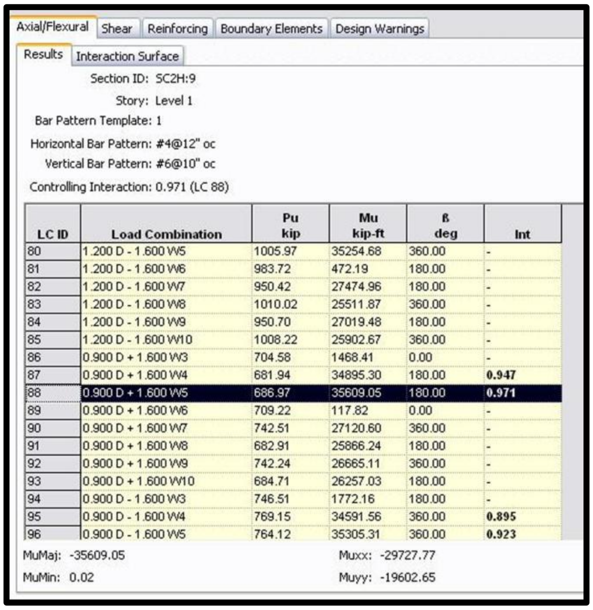
Once the program has run the design process, the user can view the results for each section cut. The images below show the results for a horizontal section cut at the 1st level of Wall Design Group 2.

Design Results



LC ID	Load Combination	Pu kip	Mu kip-ft	ß deg	Int
81	1.200 D - 1.600 W6	983.72	472.19	180.00	-
82	1.200 D - 1.600 W7	950.42	27474.96	180.00	-
83	1.200 D - 1.600 W8	1010.02	25511.87	360.00	-
84	1.200 D - 1.600 W9	950.70	27019.48	180.00	-
85	1.200 D - 1.600 W10	1008.22	25902.67	360.00	-
86	0.900 D + 1.600 W3	704.58	1468.41	0.00	-
87	0.900 D + 1.600 W4	681.94	34895.30	180.00	**0.947**
88	0.900 D + 1.600 W5	686.97	35609.05	180.00	**0.971**
89	0.900 D + 1.600 W6	709.22	117.82	0.00	-
90	0.900 D + 1.600 W7	742.51	27120.60	360.00	-
91	0.900 D + 1.600 W8	682.91	25866.24	180.00	-
92	0.900 D + 1.600 W9	742.24	26665.11	360.00	-
93	0.900 D + 1.600 W10	684.71	26257.03	180.00	-
94	0.900 D - 1.600 W3	746.51	1772.16	180.00	-
95	0.900 D - 1.600 W4	769.15	34591.56	360.00	**0.895**
96	0.900 D - 1.600 W5	764.12	35305.31	360.00	**0.923**
97	0.900 D - 1.600 W6	741.87	421.56	180.00	-

Figure 53: Design window for shear wall group #2. Data shown is for section cut identified in orange.


Axial/Flexural | Shear | Reinforcing | Boundary Elements | Design Warnings
Results | Interaction Surface

Section ID: SC2H:9
Story: Level 1
Bar Pattern Template: 1
Horizontal Bar Pattern: #4@12" oc
Vertical Bar Pattern: #6@10" oc
Controlling Interaction: 0.971 (LC 88)

LC ID	Load Combination	Pu kip	Mu kip-ft	ß deg	Int
80	1.200 D - 1.600 W5	1005.97	35254.68	360.00	-
81	1.200 D - 1.600 W6	983.72	472.19	180.00	-
82	1.200 D - 1.600 W7	950.42	27474.96	180.00	-
83	1.200 D - 1.600 W8	1010.02	25511.87	360.00	-
84	1.200 D - 1.600 W9	950.70	27019.48	180.00	-
85	1.200 D - 1.600 W10	1008.22	25902.67	360.00	-
86	0.900 D + 1.600 W3	704.58	1468.41	0.00	-
87	0.900 D + 1.600 W4	681.94	34895.30	180.00	0.947
88	0.900 D + 1.600 W5	686.97	35609.05	180.00	0.971
89	0.900 D + 1.600 W6	709.22	117.82	0.00	-
90	0.900 D + 1.600 W7	742.51	27120.60	360.00	-
91	0.900 D + 1.600 W8	682.91	25866.24	180.00	-
92	0.900 D + 1.600 W9	742.24	26665.11	360.00	-
93	0.900 D + 1.600 W10	684.71	26257.03	180.00	-
94	0.900 D - 1.600 W3	746.51	1772.16	180.00	-
95	0.900 D - 1.600 W4	769.15	34591.56	360.00	0.895
96	0.900 D - 1.600 W5	764.12	35305.31	360.00	0.923

MuMaj: -35609.05 Muxx: -29727.77
MuMin: 0.02 Muyy: -19602.65

Figure 55: Controlling load combination for section cut shown in figure 53.

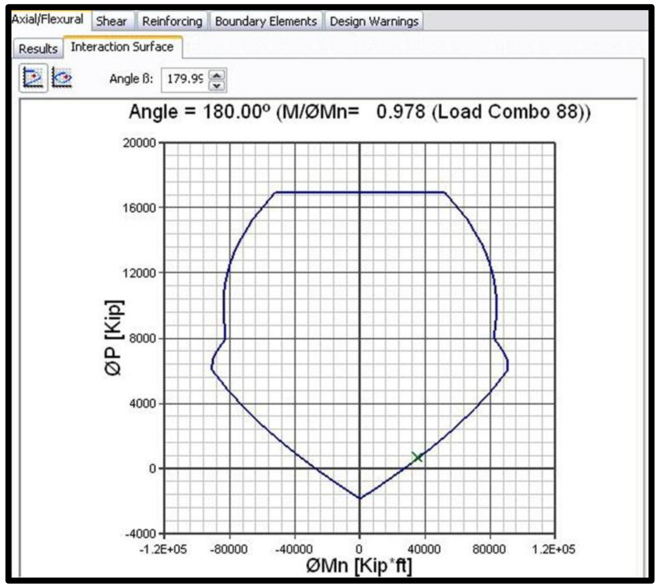


Figure 54: Interaction diagram for section cut shown in figure 53.

Column Design

The columns for the New Hospital were designed in the column module of RAM Concrete. The column sizes are reduced along the height of the structure. The bottom four stories are 24"x24" square columns. The middle three stories are 20"x20" square columns. When the addition is added, columns on those stories will be 18"x18". Three unique reinforcement patterns were assigned to each column to provide a variety of design options. The patterns are as follows:

1. 20 bars (6 x 4), min. long.- #5 max. long.- #11, transverse- #4
2. 24 bars (7 x 5), min. long.- #5 max. long.- #11, transverse- #4
3. 28 bars (8 x 6), min. long.- #5 max. long.- #11, transverse- #4

Design Assumptions

Gravity forces used to design the columns come from RAM Gravity analysis and RAM Frame analysis. The generated combinations are applied to each column and the critical combination is used for design.

As stated in the general assumptions, the columns are assumed braced against sidesway by the walls and the slab. This assumption is valid under section 10.10.1 of ACI318-08 which states:

"It shall be permitted to consider compression members braced against sidesway when bracing elements have a total stiffness…of at least 12 times the gross stiffness of the columns in that story."

Based upon the moment of inertia of the slab and the shear walls as compared to the columns, it can be assumed that this condition is satisfied and the columns can be considered braced against sidesway.

Slenderness is considered in the design of all columns with a K value equal to 1. This is a conservative assumption for a non-sway frame. Even with this assumption, all columns were found to be non-slender.

A magnification of moments for non-sway frames was applied to each column within the model.

Design Results

The design output for columns is similar to the output for the walls. The controlling design case is given for the longitudinal reinforcement design with the final reinforcement pattern chosen from among the group of bar patterns which were assigned previously.

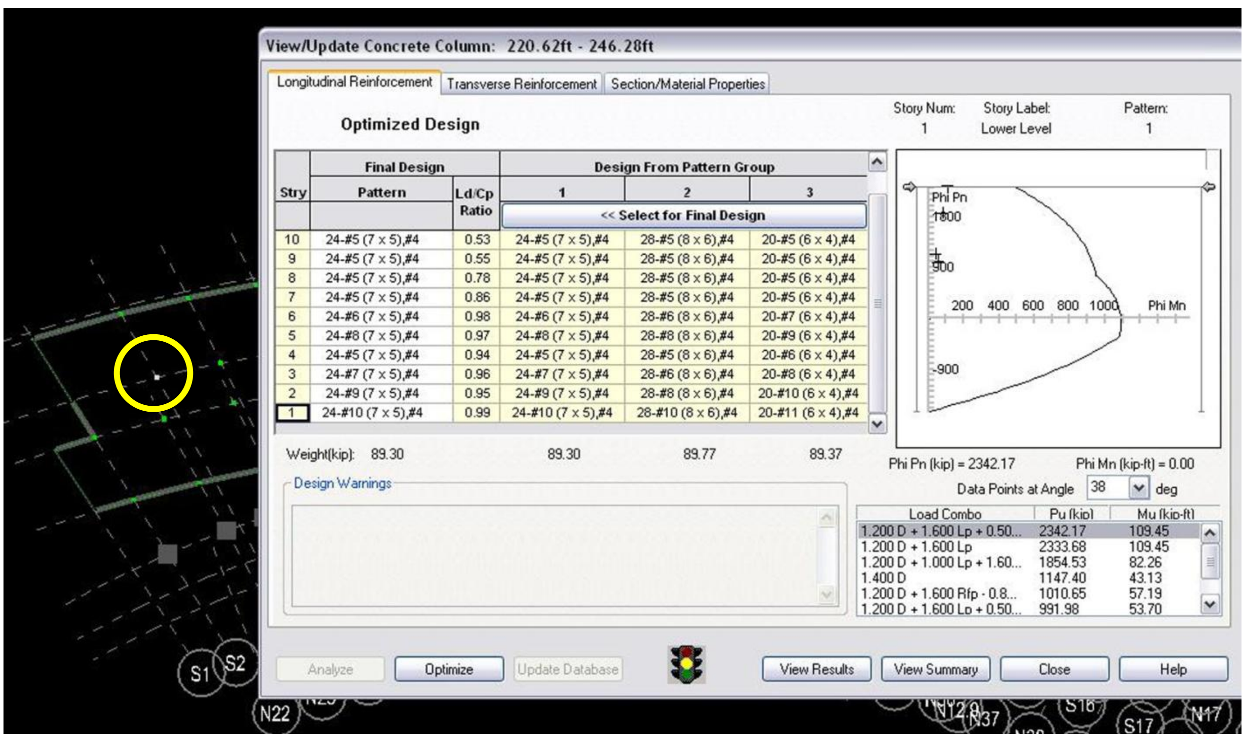


Figure 56: Design window for critical column highlighted in plan above.

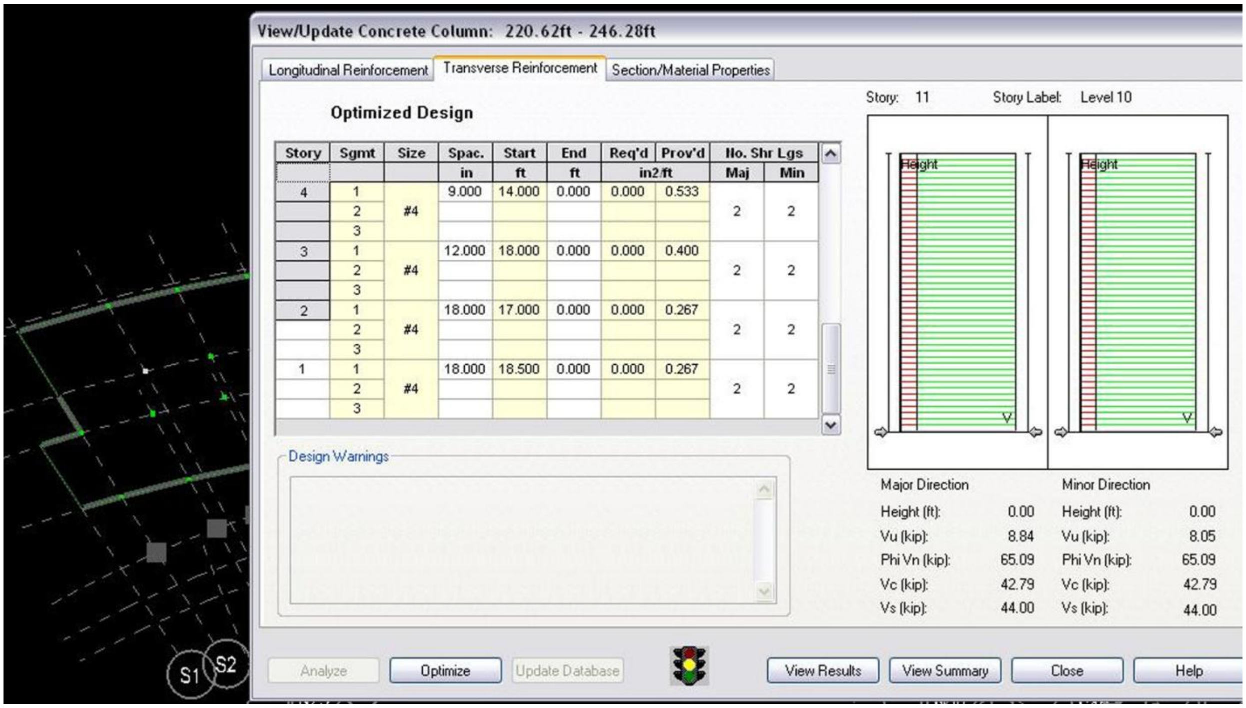


Figure 57: Design window for transverse reinforcement of column.

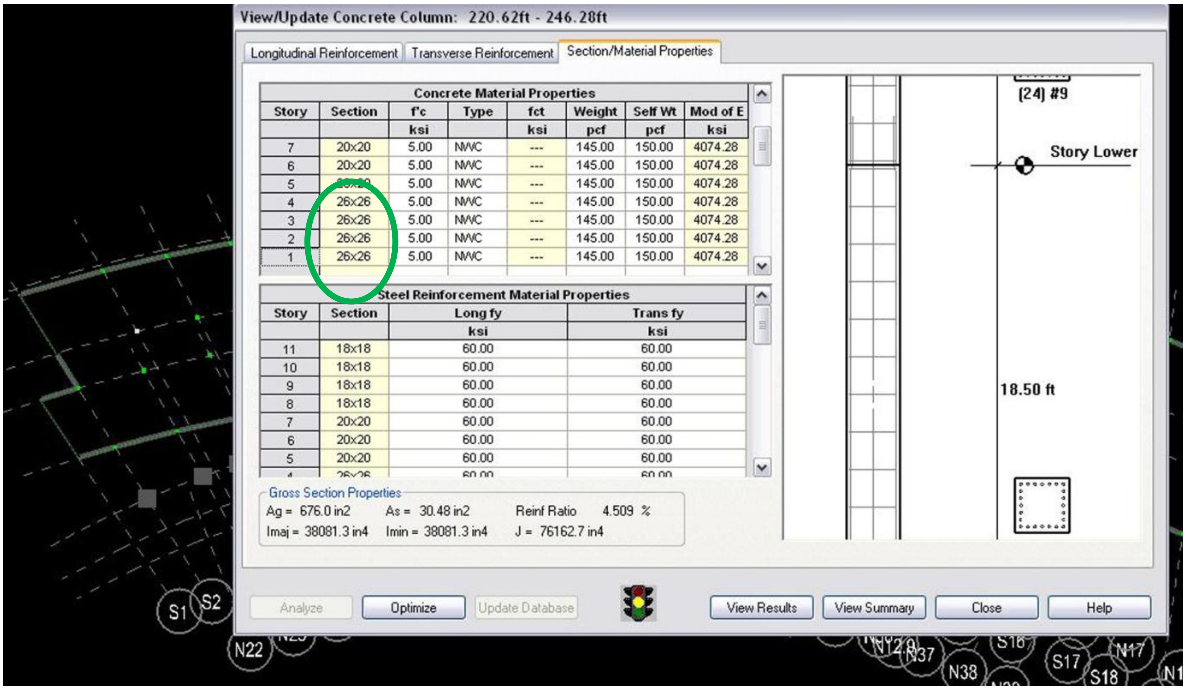


Figure 58: Design window identifying section properties and material strengths. Note that column had to be upsized to 26"x26" for first four stories.

The column highlighted above has a higher tributary area than other columns on the floor plan due to an enlarged bay on the western end of the building. This additional load required the column to be upsized to 26"x26" from 24"x24" which is typical on those stories. This is the only case where the column size deviates from the typical layout.

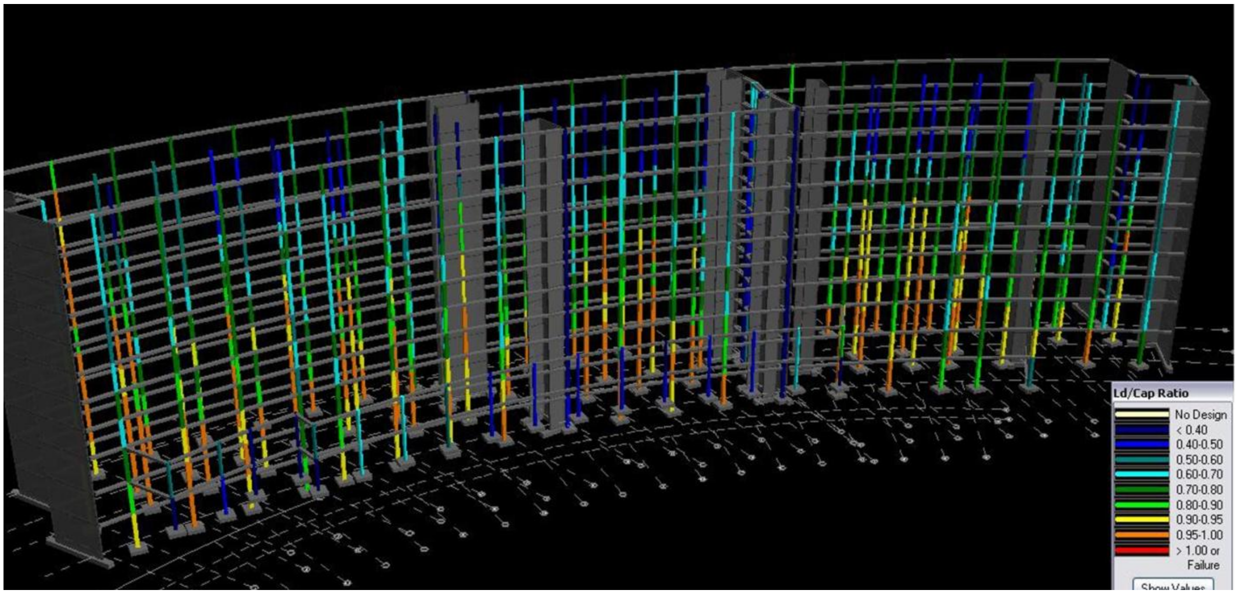


Figure 59: Perspective showing all column designs. Colors indicate interaction diagram values.

The image above shows the interaction values for all of the designed columns. Any column which fails the interaction diagram or code reinforcement provisions appears in red. All other colors indicate a successful design.

Beam Design

The concrete beams on the perimeter of the building are a part of the moment frames which help resist lateral loading in the x-direction. However, the primary responsibility of the beam is to carry the weight of the curtain wall. It is assumed that the flexure and shear capacity of the 18"x20" beam will be satisfactory. To insure that the façade will not experience failure due to excessive displacements, the deflection of the beam will be checked.

Design Assumptions

The perimeter beams are considered to be continuous members. In order to design to this assumption beam lines are assigned around the exterior of the building. When the design is completed, the output will group the individual beams in each beam line as one entire beam. However, the reinforcement design can be adjusted at each span and column location.

Deflection criteria are as follows:

> Live load: $\Delta \leq L_n/360$
>
> Long term deflection + live load: $\Delta \leq L_n/240$

Beams will not be designed with any camber.

Design Results

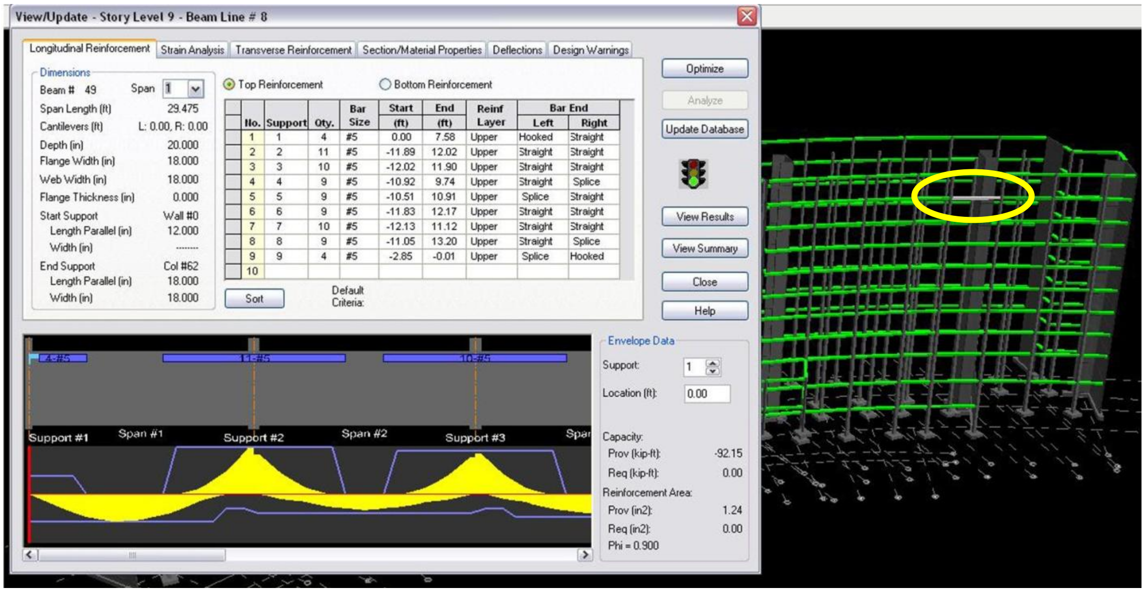


Figure 60: Design window for beam highlighted in perspective. Longitudinal design data is shown on the left.

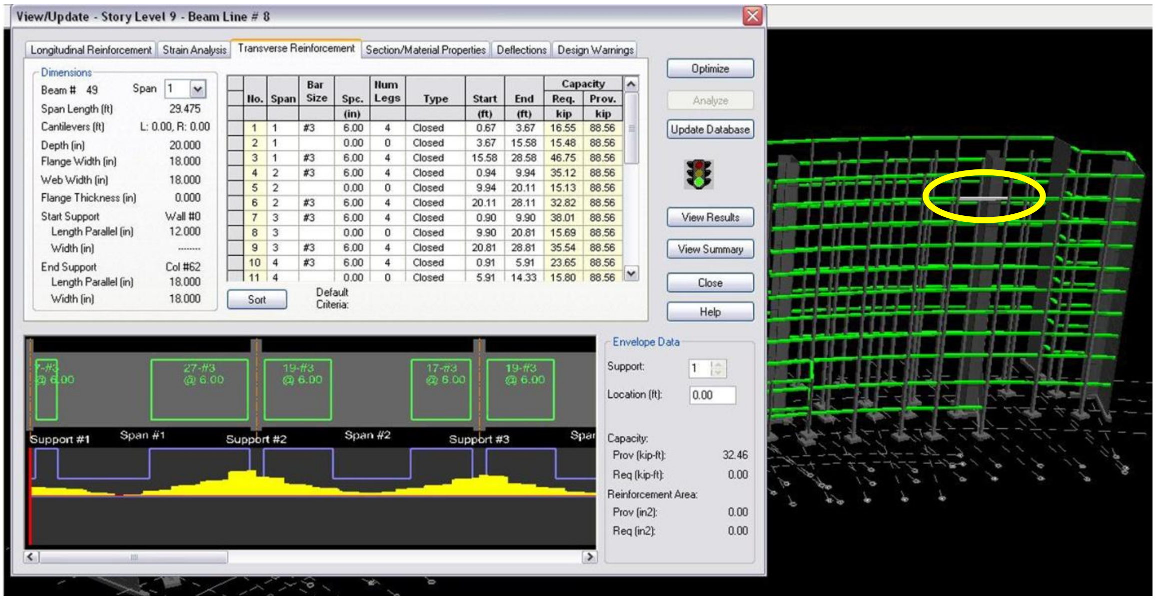


Figure 61: Design window for beam highlighted in perspective. Transverse design data is shown on the left.

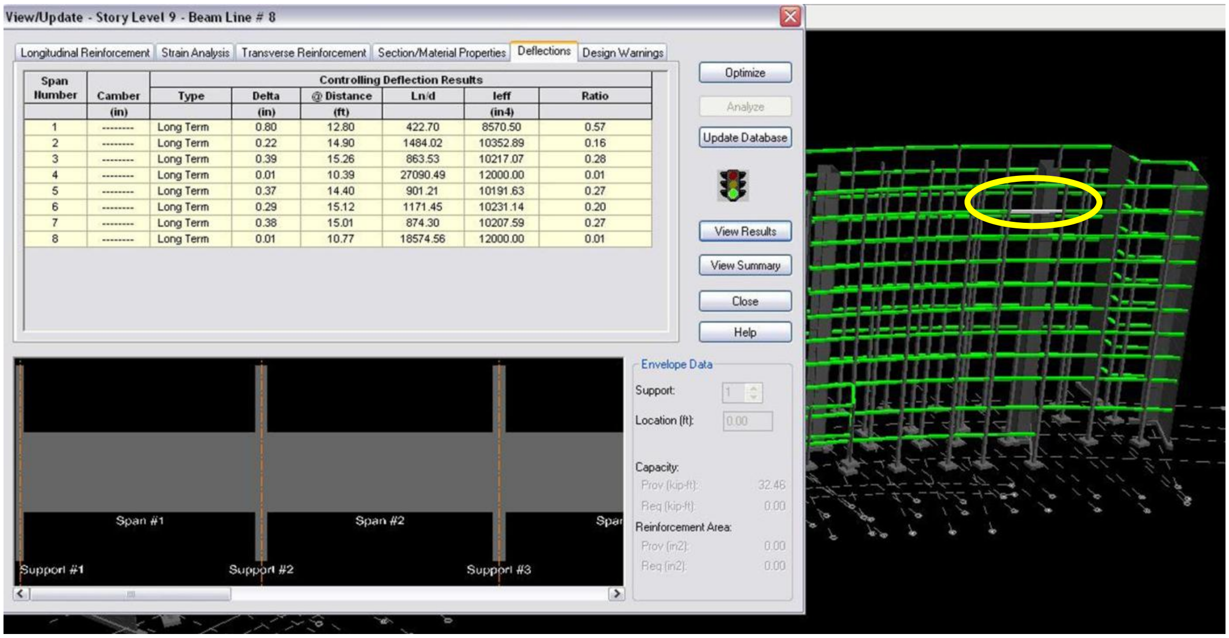


Figure 62: Design window for beam highlighted in perspective. Deflection design data is shown on the left.

The design output above shows that the beam clearly meets flexure, shear, and deflection requirements. The deflection of the beam was checked by hand and can be found in the appendix.

Foundation Design

The overall goal of this thesis was to explore a design alternative in an attempt to simplify the design of the foundation. After analyzing and designing the column and shear walls, it is clear that the increase in building weight resulted in a compressive force with a magnitude large enough to counter the tension force due to lateral loading. In order to prove this assertion, RAM models (6-story and 10-story) of the original steel design were created and analyzed. Perspective views of these models are shown below.

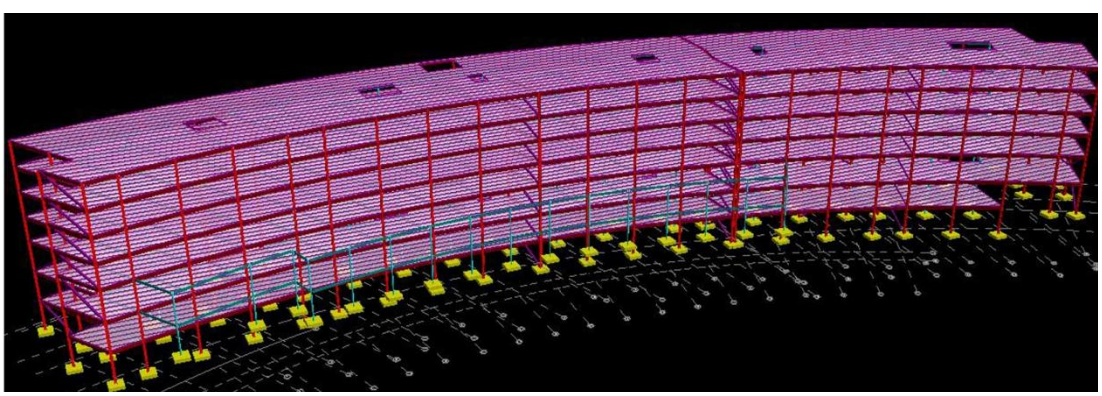


Figure 63: RAM model of six-story steel design.

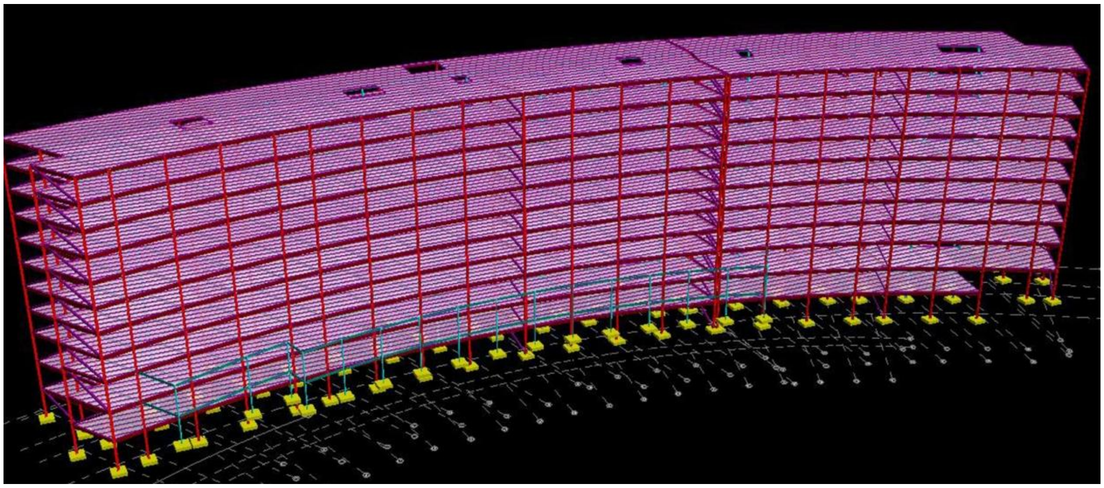


Figure 64: RAM model of 10-story steel design.

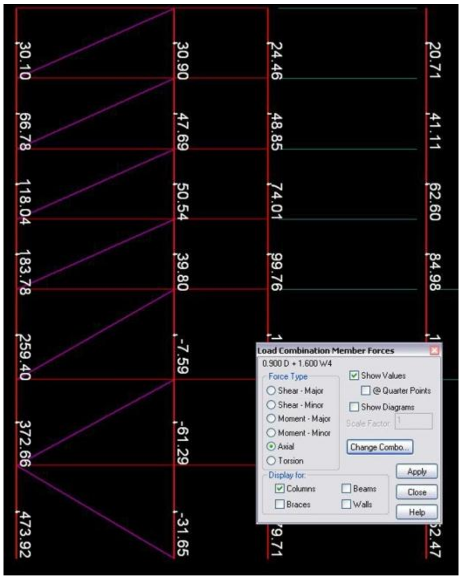


Figure 66: Braced frame #1 elevation for six-story steel building. Axial forces are shown.

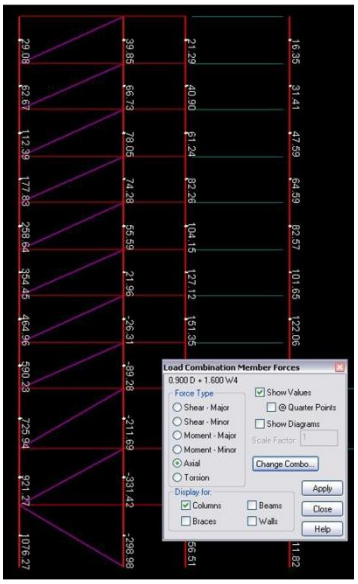


Figure 65: Braced frame #1 elevation for 10-story steel building. Axial forces are shown.

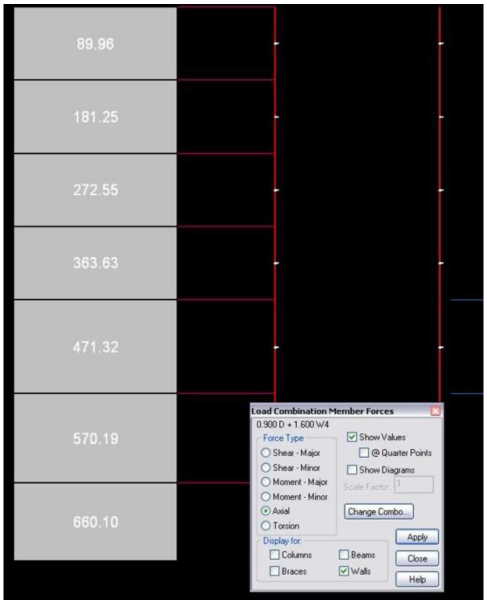


Figure 68: Shear wall design group #2 for six-story concrete building. Axial forces are shown.

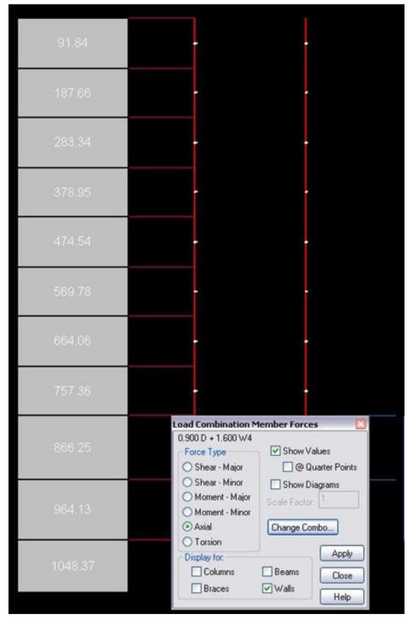


Figure 67: Shear wall design group #2 for 10-story concrete building. Axial forces are shown.

For a quick and direct comparison, braced frame #1 (steel) and wall group #1 (concrete) were highlighted to evaluate the difference in axial force at the foundation. These elements are primarily oriented in the N-S direction and share the same floor plan location. The combination chosen for the comparison was 0.9D + 1.6W because this is typically the controlling case for uplift.

Both steel frames have tension at the base of the windward column which is expected since this is the original design. Conversely, both concrete shear walls experience axial compression for the entire height.

The force envelope was checked for each member in both of the concrete models and the minimum axial force for each wall group was found to remain as a compressive force.

Design Assumptions

The geotechnical report provided by the engineers at French & Parrello identified the soil on site as coarse to fine sand, silt, and clay. Highly decomposed sandstone and shale, fractured sandstone and shale bedrock, and highly decomposed bedrock were also found at depths ranging from 8' to 30' below ground level. Groundwater was found approximately 15-20 feet below the surface and is not expected to be encountered during the excavation.

The design soil bearing pressure is equal to 4000 psf.

In order to satisfy the requirements for frost protection set forth in the International Building Code, the top of all foundations will be no less than 42 inches below grade.

The unit weight of the soil, γ, is given as 120 pcf.

All foundations are designed under the assumption that no additional loads are located within the vicinity of the foundation.

Moment due to shear at the base of the frame column will not be included in the design of the footing. It is assumed that the slab-on-grade will handle this shear force.

The self-weight of the footing is included in the check of soil stress.

The safety factor used for uplift is equal to 1.1.

Design Results

The size of the spread footings varied across the base of the building. Interior columns required footing sizes ranging from 6'x6' square to 7'x7' square. Exterior columns which are part of the moment frames needed footings which were typically 10'x10' with some variation in specific areas.

Sizes for the continuous wall footings were also determined. The walls without any returns (straight line shape) had typical widths of around 15' (7.5' on each side of the wall). These designs seem reasonable and were confirmed with hand calculations which can be found in the appendix. However, most of the shear walls are grouped as C-shapes rather than single walls. While RAM Concrete acknowledges wall groups and designs the shear walls accordingly, RAM Foundation does not recognize the wall group and views each wall individually. When viewed as a group, the shear walls experience no net tension. But when viewed separately, certain load combinations create net tension forces in the smaller return walls. For an acceptable design in RAM Foundation, the wall footings have to be upsized considerably which causes a significant amount of foundation overlap.

The expansion joint area is going to be congested at the foundation level regardless of the footing sizes. The original foundation design included a mat foundation at this location so it is considered to be a worthwhile design solution for this case.

For the sake of convenience and constructability, mat foundations should be located underneath all C-shaped shear walls. The complete design of these foundations was not included in the proposal for this thesis project. Had more time been available, a complete design would have been explored in greater depth.

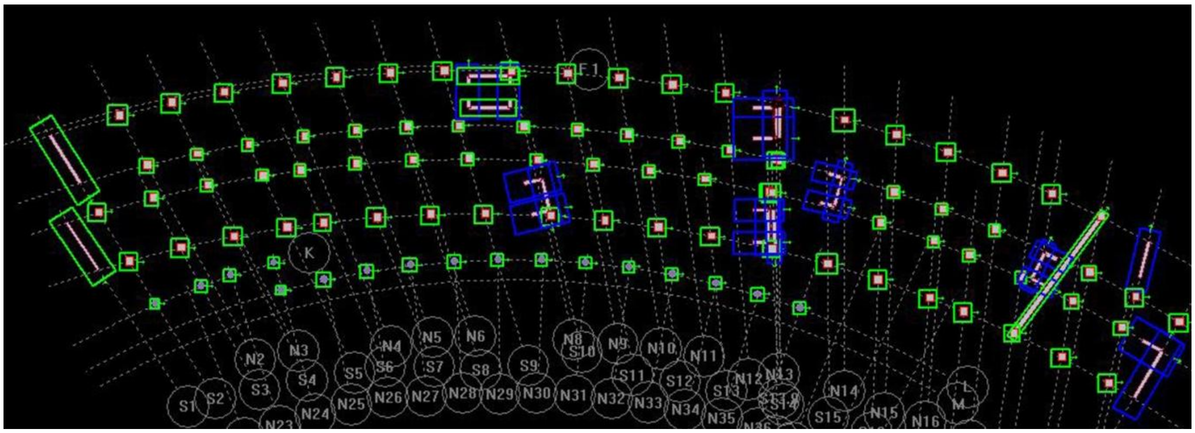


Figure 69: Foundation plan showing designs. Green indicates acceptable design. Blue indicates that design was modified by user to become acceptable.

The design of a typical spread footing and continuous wall footing are displayed in the images below.

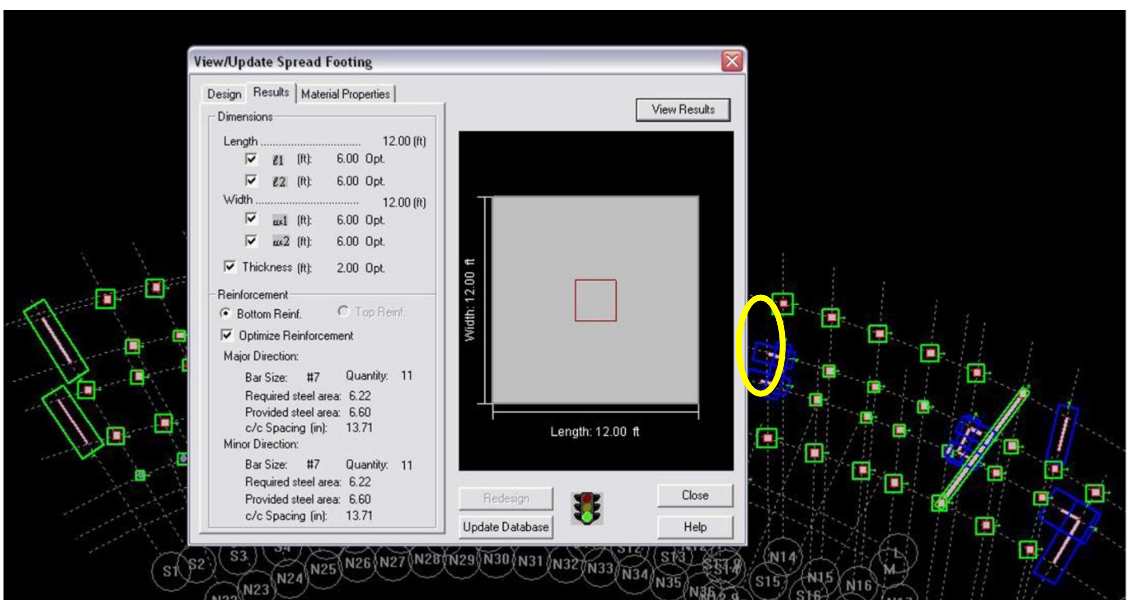


Figure 70: Design window for critical spread footing.

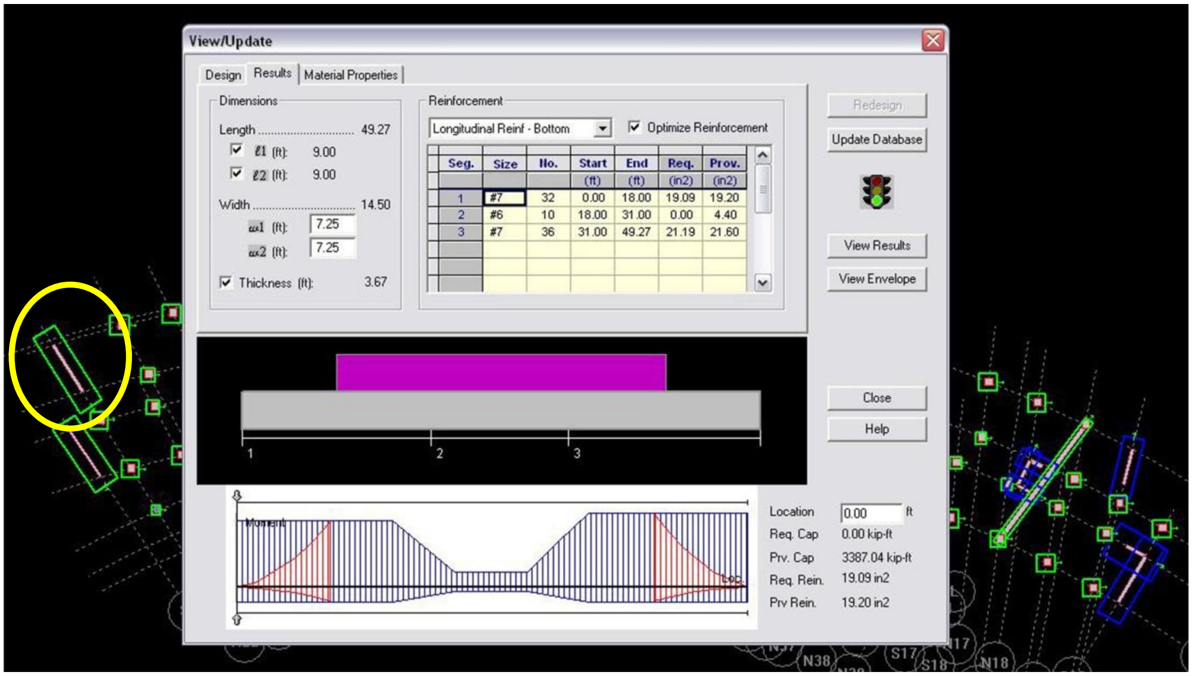


Figure 71: Design window for continuous wall footing.

Depth Summary

The goals established at the outset of this thesis project were to eliminate the net tension forces at the foundation and to improve the vibration response of the floor in order to achieve superior building performance. The proposed solution to accomplish these goals was to redesign the University Medical Center at Princeton as a concrete structure.

The key aspect of the concrete redesign is the change made to the lateral force resisting system. The steel braced frames were replaced with Slaconcrete shear walls which were properly designed to handle all necessary loads on the structure without any disruption to the original floor plan. In this regard, the redesign can be considered a success.

In terms of reaching the overall goal of eliminating net tension, the design again can be considered successful in that each wall group has a resulting compressive axial force at its base. However, the wall group assumption used in RAM Concrete to design the shear walls does not translate to the same assumption used in RAM Foundation for design of the footings. Therefore, the wall footings supporting a majority of the shear walls had to be upsized to the point where it made more sense to have mat foundations supporting these walls instead of continuous wall footings.

Due to time constraints, the complete design of these mat foundations was not completed. It is important to note that the overall proposal of this redesign was to create a simplified design condition at the foundation level. The proposal did not include a full investigation of mat foundations against tension-only mini piles. Therefore while the simplified design condition was achieved, other factors of the design created a new scenario (mat foundations) which now must be compared to the original design solution in order to determine its viability. Given more time, this comparison would have been explored as a complementary addition to the overall project.

As for the goal of improved vibration performance, it can be asserted with a good deal of confidence that the 7" and 8" two-way flat slab designed for this hospital provides a marked improvement in vibration response over the original composite beam floor system. Therefore, this goal was achieved.

Architectural Breadth

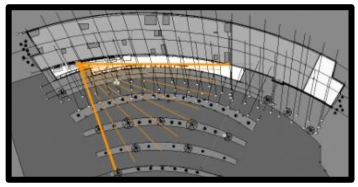
A redesign of the New Hospital as a concrete structure impacts other building features besides the structural system. This hospital is the centerpiece of a facility which is undergoing a major addition. Upon completion of this project, the Princeton Health Care System will be considered among the elite of all providers of health services. The University Medical Center at Princeton will be a destination point not only for patients seeking high-quality medical care but also for the finest specialists, surgeons, diagnosticians, and nurses who can deliver a high level of medical care. This will be a facility that will be highly visible to a great number of people.

The south façade of this hospital provides the first impression for any person about to enter this facility. The original design called for exposed circular HSS shapes to support this façade. By redesigning the structure with concrete, these shapes will be removed and the façade will be supported by concrete columns instead. The goal of this breadth is to evaluate the impacts of the structural redesign on the appearance of the façade from the exterior as well as the interaction between the concrete columns and the interior lobby space.

In order to perform this analysis, two Revit models of the New Hospital were created; one with the original steel design and the other with the concrete redesign. Renderings of the main lobby (both exterior and interior) were produced and these images will provide the basis for the analysis.

Main Lobby: Summer Solstice

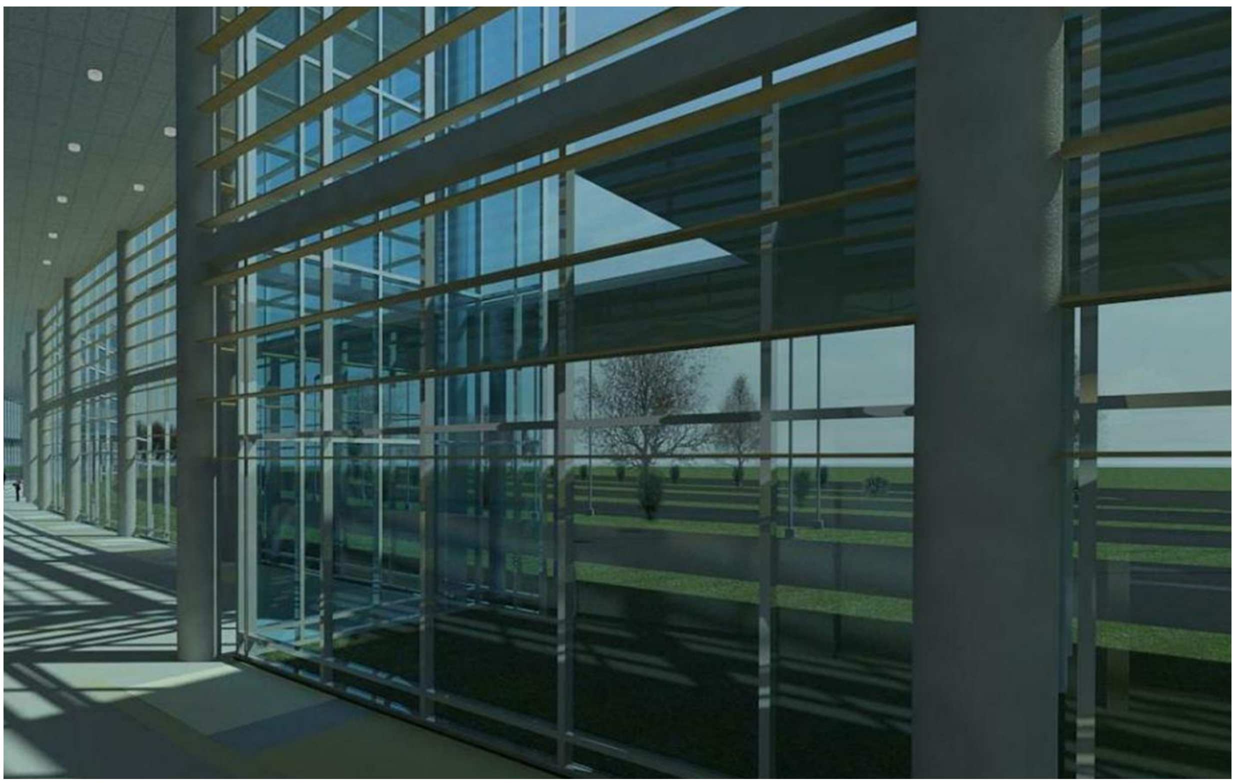
Original design in steel



The rendering above shows a view from the main lobby looking east at midday of the summer solstice. The glass curtain wall is supported by circular HSS columns and rectangular HSS beams. Since the steel in the lobby is exposed, it is fireproofed with a layer intumescent paint. The glossiness of the paint is visible, especially in the column near the center of the image. This appearance is similar to the aluminum mullions which are reflective as well. The slenderness of the steel columns helps the façade keep a consistency throughout its length. The pattern of the mullions is only slightly interrupted by columns which are thin enough to blend with the mullion pattern effortlessly.

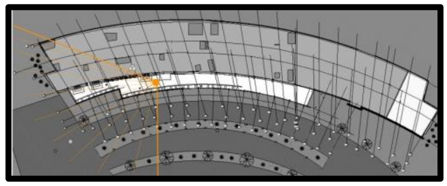
The sun's position at midday of the summer solstice is high in the sky which is the reason for the lack of daylight in the lobby. When the angle of the sun drops later in the year, the horizontal sunshades spanning from column-to-column will contribute more to the shading of the lobby.

Redesign in concrete

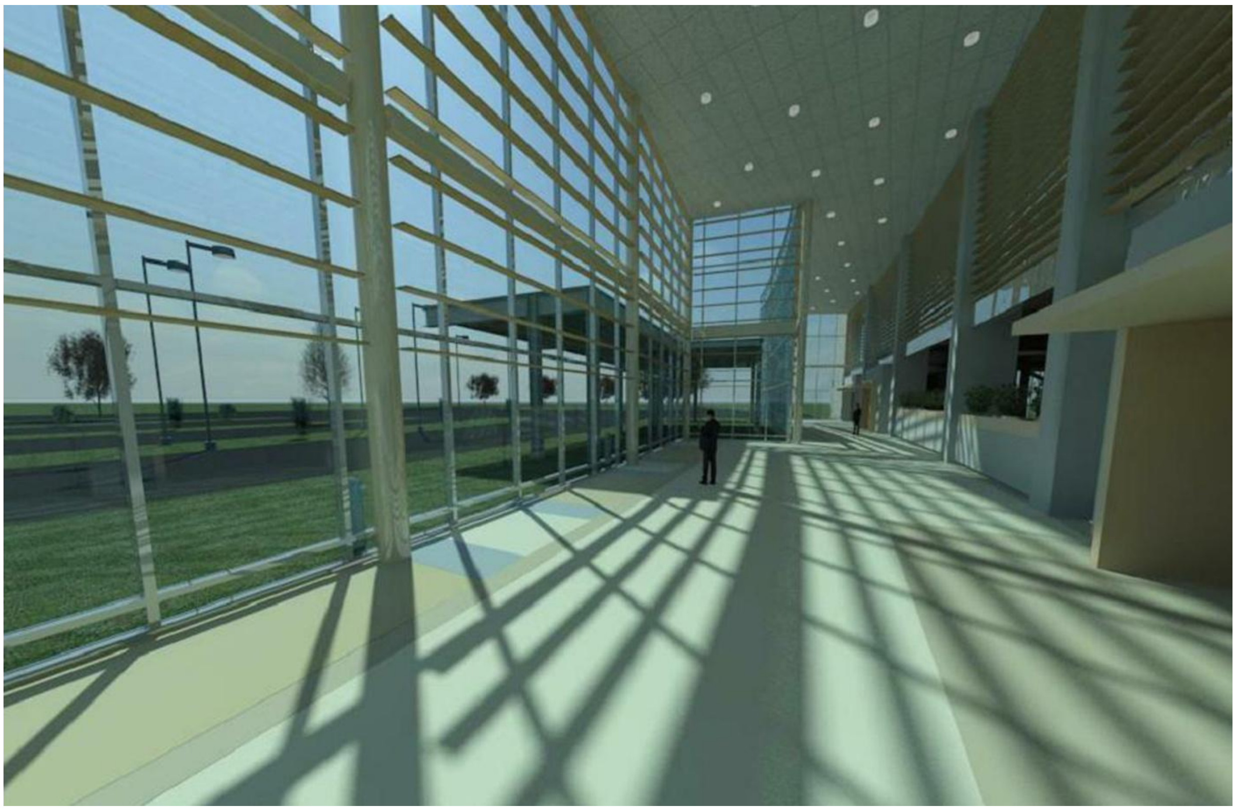


This is a similar rendering except with concrete framing instead of the HSS shapes. The obvious distinction between the two is the thickness of the concrete columns as opposed to the slender steel shapes. This creates a different sort of interaction between the circular concrete columns and the glass curtain wall. The thickness of the concrete structure provides bold boundaries for the curtain wall. The columns and beams clearly partition the glass into similarly shaped rectangles all the way down the façade. The concrete clearly isolates itself as structure whereas the steel blended with the mullions to create the illusion of a free-standing glass curtain wall.

Main Lobby: Spring Equinox



Original design in steel



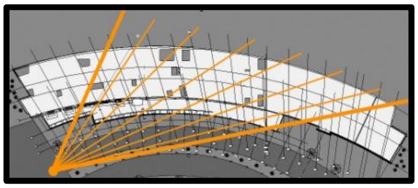
The rendering above is a view looking west from the middle of the lobby. Entrances to the different treatment facilities within the hospital are located along the wall on the right. The lobby serves as a gathering space and concourse for visitors to navigate the hospital. As shown in this rendering, the space has an open feel even though dimensionally it is long and narrow. The lobby also gets a considerable amount of daylight depending on the angle of the sun which is the reason for the interior sunshades on the glass curtain wall as well as on the second floor curtain wall which sits above the portal entrances. Again, the steel structure blends in nicely with the aluminum mullions.

Redesign in concrete

A similar rendering is shown above with concrete framing. There are two significant impacts which the concrete structure has on this lobby. First, the larger concrete shapes cast broader shadows on the floor as opposed to the steel framing. These shadows provide a constant reminder of the structure's presence. The other significant impact is the feeling of a reduced space. The line of thick columns down the glass façade mirror the thick vertical shapes on the interior wall. Together, these rows of columns squeeze the space as opposed to the steel which opens it up. In a way, this is a positive impact in that the structure responds to the form of the building. The lobby is in fact narrow and the structure reflects that. However in combination with the shadows, the concrete framing can create an intimidating feeling for patrons entering the hospital which is not the best expression for a hospital.

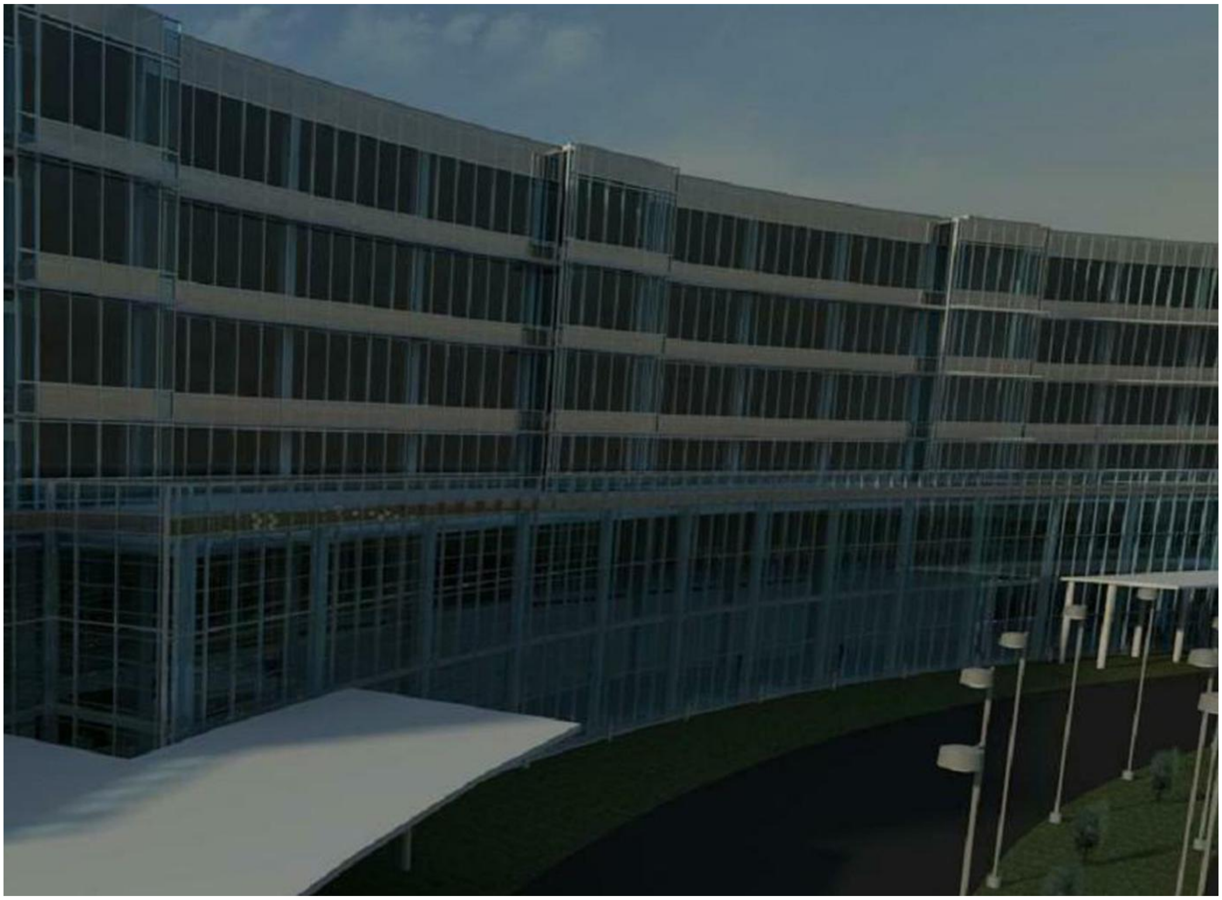
Exterior View: Mid-afternoon

Original design in steel



The rendering shown above is an exterior view of the south façade looking northeast. The length of the façade is very prominent and is further accentuated by the opaque spandrel panels at each floor level. These panels form bands which stratify the façade and further emphasize horizontal movement. The steel columns along with the four curtain wall extrusions provide some vertical break to the lateral flow. However, it is not enough to draw the eye's attention away from the banded feel of the curtain wall.

Redesign in concrete



A similar view as the one shown above except zoomed in closer. The concrete frame is more successful at breaking the thick spandrel bands at the floor levels. The ratio of thickness between the spandrels and the columns is nearly 1:1 which actually gives the facade a “checkerboard” character rather than the long, extended feel of the steel frame. The visibility of the structure is more apparent with the concrete design and makes the building appear sturdier. From this view, the enclosure of the main lobby is clearly partitioned in an even and orderly manner by the circular columns.

Summary

The goal of this breadth study was to evaluate the impact of the concrete redesign on the south façade of the hospital. Certainly, the concrete structure interacts with the glass much differently than the steel structure. The strength of the steel lies in its ability to blend with the aluminum mullions of the curtain wall as if the steel is in fact part of the curtain wall. This creates an open feel in a geometrically narrow lobby which is a comforting feel for the visitors. On the exterior, the steel structure does not provide enough vertical breaks to the façade thereby allowing the spandrel panels and the natural shape of the building to dominate the appearance.

The strength of the concrete frame is that it does provide a vertical complement to the horizontal banding of the façade. This interaction creates an interesting appearance and partitions the curtain wall in a symmetrical and logical manner. However the boldness of the columns from the exterior compromises their effectiveness on the interior lobby space. The already narrow lobby is crowded by the bigger columns and at certain times of the day, the shadows can tower over the space.

Due to the differences in the curtain wall interaction between steel and concrete, it cannot simply be assumed that a concrete redesign will automatically agree with the original intentions of the architect and the owner. This is not necessarily detrimental to the proposed redesign. The concrete system does in fact have architectural strengths when compared with the steel. However both systems would have to be evaluated more closely so that the structural design can effectively meet the architectural desires of the owner. Due to the fact that the concrete structure provides some architectural benefits, it is asserted that within the context of this thesis proposal a concrete redesign is architecturally sufficient.

Construction Management Breadth

While a concrete redesign has significant impacts on the structure and architecture of the New Hospital, it also dramatically affects how the overall project is organized and paid for. The cost and planning of a steel building versus a concrete building can be substantially different. Therefore, it is necessary to evaluate those differences in order to determine the viability of one design compared to the other.

Cost

Estimating the cost of any building project is a detailed and complicated exercise. For the purposes of this project, the process has been drastically simplified in order to provide a reasonable conclusion.

This particular analysis will focus on the cost of the original structural system versus the cost of the redesigned structural system. The overall totals include the cost of materials, labor, and equipment for columns, framing (beams), floor slab, and the newly designed shear walls.

Since mat foundations were not detailed in the redesign, an accurate cost estimation of the redesigned foundation would likely be inaccurate. Estimating the real cost of the original foundation would also yield inaccurate results due to the fact that the tension-only mini piles would not be included. Therefore, foundation costs are not calculated.

Formwork costs for this assessment are based on one-time use. This is a conservative assumption.

The final assumption is that reinforcement will not be included in the slab cost calculation because a simplified material takeoff could not be obtained from RAM Concept. It is acknowledged that this will reduce the cost of the redesigned structure and will be considered in the final comparison.

A Revit model of the New Hospital was created in order to assemble complete material schedules and perform these cost analyses with greater ease.

The tables below display the cost calculation for the original steel structure.

Floor/Roof Schedule				RS Means 2010								
Family and Type	Level	Volume (cu. ft.)	Area (sq. ft.)	Units	Deck			Units	Concrete			Total Cost
					Material	Labor	Equipment		Material	Labor	Equipment	
Floor: Lobby Floor	Level 1	25331.35	41928	SF	$ 1.50	$ 0.51	$ 0.05	CY	$ 109.00	$ 11.75	$ 35.25	$ 232,730.59
Floor: LW Concrete on Metal Deck	Level 2	24262.65	46584	SF	$ 1.50	$ 0.51	$ 0.05	CY	$ 109.00	$ 11.75	$ 35.25	$ 236,147.24
Floor: LW Concrete on Metal Deck	Level 3	23960.23	46004	SF	$ 1.50	$ 0.51	$ 0.05	CY	$ 109.00	$ 11.75	$ 35.25	$ 233,205.12
Floor: LW Concrete on Metal Deck	Level 4	24138.17	46345	SF	$ 1.50	$ 0.51	$ 0.05	CY	$ 109.00	$ 11.75	$ 35.25	$ 234,935.68
Floor: LW Concrete on Metal Deck	Level 5	24137.61	46344	SF	$ 1.50	$ 0.51	$ 0.05	CY	$ 109.00	$ 11.75	$ 35.25	$ 234,930.39
Floor: LW Concrete on Metal Deck	Level 6	24032.28	46142	SF	$ 1.50	$ 0.51	$ 0.05	CY	$ 109.00	$ 11.75	$ 35.25	$ 233,905.69
Basic Roof: S1	Level 3	4793.83	9204	SF	$ 1.37	$ 0.38	$ 0.04	CY	$ 109.00	$ 11.75	$ 35.25	$ 44,172.84
Basic Roof: S2	T/Parapet	40050.72	50590	SF	$ 1.37	$ 0.38	$ 0.04	CY	$ 109.00	$ 11.75	$ 35.25	$ 321,960.26
Totals		**190707**	**333141**									**$ 1,771,987.82**

Figure 72: Cost of floor deck/concrete for steel design.

Structural Column Schedule			RS Means 2010				
Family and Type	Length (ft.)	Count	Units	Material	Labor	Equipment	Total Cost
HSS-Column: HSS14X0.500	753	20	Each	$ 1,300.00	$ 57.00	$ 35.50	$ 27,850.00
W-Wide Flange-Column: W12X72	407	22	LF	$ 105.00	$ 2.60	$ 1.63	$ 44,456.61
W-Wide Flange-Column: W14X90	1366.5	53	LF	$ 145.00	$ 2.66	$ 1.67	$ 204,059.45
W-Wide Flange-Column: W14X99	1369.5	49	LF	$ 145.00	$ 2.66	$ 1.67	$ 204,507.44
W-Wide Flange-Column: W14X109	616	22	LF	$ 145.00	$ 2.66	$ 1.67	$ 91,987.28
W-Wide Flange-Column: W14X120	590.16	18	LF	$ 145.00	$ 2.66	$ 1.67	$ 88,128.59
W-Wide Flange-Column: W14X132	1894.7	66	LF	$ 145.00	$ 2.66	$ 1.67	$ 282,935.55
W-Wide Flange-Column: W14X145	1868.36	62	LF	$ 213.00	$ 2.80	$ 1.76	$ 406,480.40
W-Wide Flange-Column: W14X159	435.49	13	LF	$ 213.00	$ 2.80	$ 1.76	$ 94,745.20
W-Wide Flange-Column: W14X176	927.66	25	LF	$ 213.00	$ 2.80	$ 1.76	$ 201,821.71
W-Wide Flange-Column: W14X193	38.67	2	LF	$ 213.00	$ 2.80	$ 1.76	$ 8,413.05
W-Wide Flange-Column: W14X311	593.67	23	LF	$ 213.00	$ 2.80	$ 1.76	$ 129,158.85
W-Wide Flange-Column: W14X342	931.34	34	LF	$ 213.00	$ 2.80	$ 1.76	$ 202,622.33
Totals	12357	469					$1,987,166.45

Figure 73: Cost of steel columns.

Structural Framing Schedule			RS Means 2010				
Family and Type	Count	Length (ft.)	Units	Material	Labor	Equipment	Total Cost
HSS8X8X1/4	3	86	Each	645	51	32	$ 2,184.00
HSS8X8X5/16	6	192	Each	645	51	32	$ 4,368.00
HSS8X8X.375	2	46	Each	645	51	32	$ 1,456.00
HSS10X4X3/8	34	443	Each	645	51	32	$ 24,752.00
HSS10X8X1/2	9	238	Each	645	51	32	$ 6,552.00
HSS10X8X3/8	20	528	Each	645	51	32	$ 14,560.00
HSS10X8X5/16	3	93	Each	645	51	32	$ 2,184.00
HSS10X10X1/2	7	222	Each	645	51	32	$ 5,096.00
HSS10X10X3/8	8	257	Each	645	51	32	$ 5,824.00
HSS12X4X3/8	1	24	Each	645	51	32	$ 728.00
HSS12X8X1/2	4	126	Each	645	51	32	$ 2,912.00
HSS12X8X3/8	6	192	Each	645	51	32	$ 4,368.00
HSS12X10X1/2	10	327	Each	645	51	32	$ 7,280.00
HSS12X10X3/8	20	671	Each	645	51	32	$ 14,560.00
HSS12X12X1/2	46	1498	Each	645	51	32	$ 33,488.00
HSS14X4X3/8	2	38	Each	645	51	32	$ 1,456.00
HSS14X6X3/8	1	19	Each	645	51	32	$ 728.00
HSS16X8X3/8	10	166	Each	645	51	32	$ 7,280.00
HSS16X8X5/16	3	94	Each	645	51	32	$ 2,184.00
HSS20X8X3/8	2	33	Each	645	51	32	$ 1,456.00
HSS20X12X1/2	10	291	Each	645	51	32	$ 7,280.00
W-Wide Flange: W8X10	182	2417	LF	$12.10	$4.26	$2.68	$ 46,019.68
W-Wide Flange: W8X40	30	822	LF	$58.00	$4.64	$2.92	$ 53,890.32
W-Wide Flange: W12X14	15	21	LF	$19.35	$2.90	$1.83	$ 505.68
W-Wide Flange: W12X19	620	9645	LF	$26.50	$2.90	$1.83	$ 301,213.35
W-Wide Flange: W12X26	6	57	LF	$31.50	$2.90	$1.83	$ 2,065.11
W-Wide Flange: W12X35	40	538	LF	$42.50	$3.15	$1.98	$ 25,624.94
W-Wide Flange: W12X40	5	94	LF	$60.50	$3.41	$2.14	$ 6,208.70
W-Wide Flange: W14X22	125	2411	LF	$31.50	$2.58	$1.62	$ 86,072.70
W-Wide Flange: W16X26	696	20067	LF	$31.50	$2.55	$1.61	$ 715,589.22
W-Wide Flange: W16X31	64	1857	LF	$37.50	$2.84	$1.79	$ 78,235.41
W-Wide Flange: W18X35	164	4846	LF	$42.50	$3.85	$1.83	$ 233,480.28
W-Wide Flange: W18X40	35	972	LF	$48.50	$3.85	$1.83	$ 52,662.96
W-Wide Flange: W21X44	194	5349	LF	$53.00	$3.47	$1.65	$ 310,883.88
W-Wide Flange: W21X50	38	1152	LF	$60.50	$3.47	$1.65	$ 75,594.24
W-Wide Flange: W21X55	2	63	LF	$75.00	$3.57	$1.69	$ 5,056.38
W-Wide Flange: W24X55	206	6104	LF	$66.50	$3.33	$1.58	$ 435,886.64
W-Wide Flange: W24X62	10	336	LF	$75.00	$3.33	$1.58	$ 26,849.76
W-Wide Flange: W24X68	56	1682	LF	$82.50	$3.33	$1.58	$ 147,023.62
W-Wide Flange: W24X76	22	722	LF	$92.00	$3.33	$1.58	$ 69,969.02
W-Wide Flange: W24X104	20	228	LF	$126.00	$3.52	$1.67	$ 29,911.32
W-Wide Flange: W27X84	64	2080	LF	$102.00	$3.11	$1.47	$ 221,686.40
W-Wide Flange: W30X90	3	97	LF	$120.00	$3.08	$1.46	$ 12,080.38
W-Wide Flange: W30X99	8	300	LF	$120.00	$3.08	$1.46	$ 37,362.00
W-Wide Flange: W30X108	3	103	LF	$131.00	$3.08	$1.46	$ 13,960.62
W-Wide Flange: W30X116	4	136	LF	$140.00	$3.19	$1.51	$ 19,679.20
W-Wide Flange: W33X118	2	66	LF	$143.00	$3.14	$1.49	$ 9,743.58
W-Wide Flange: W33X130	2	69	LF	$157.00	$3.26	$1.55	$ 11,164.89
W-Wide Flange: W33X141	2	74	LF	$171.00	$3.26	$1.55	$ 13,009.94
W-Wide Flange: W36X302	1	37	LF	$365.00	$3.57	$1.69	$ 13,699.62
W-Wide Flange: W36X652	1	31	LF	$365.00	$3.57	$1.69	$ 11,478.06
Totals	2827	67995					$ 3,217,303.90

Figure 74: Cost of steel framing.

The material schedules for the redesigned concrete system were obtained from RAM takeoff reports.

The tables below display the cost calculation for the redesigned concrete structure.

Floor/Roof Schedule				RS Means 2010								
Family and Type	Level	Volume (cu. ft.)	Area (sq. ft.)	Units	Formwork Material	Formwork Labor	Formwork Equipment	Units	Concrete Material	Concrete Labor	Concrete Equipment	Total Cost
8" Two-Way Flat Slab	Level 1	27952	41928	SF	$ 4.43	$ 4.33	$ -	CY	$109.00	$ 11.75	$ 35.25	$ 528,789.72
8" Two-Way Flat Slab	Level 2	31056	46584	SF	$ 4.43	$ 4.33	$ -	CY	$109.00	$ 11.75	$ 35.25	$ 587,510.51
7" Two-Way Flat Slab	Level 3	26836	46004	SF	$ 4.43	$ 4.33	$ -	CY	$109.00	$ 11.75	$ 35.25	$ 558,045.56
7" Two-Way Flat Slab	Level 4	27035	46345	SF	$ 4.43	$ 4.33	$ -	CY	$109.00	$ 11.75	$ 35.25	$ 562,182.01
7" Two-Way Flat Slab	Level 5	27034	46344	SF	$ 4.43	$ 4.33	$ -	CY	$109.00	$ 11.75	$ 35.25	$ 562,169.88
7" Two-Way Flat Slab	Level 6	26916	46142	SF	$ 4.43	$ 4.33	$ -	CY	$109.00	$ 11.75	$ 35.25	$ 559,719.55
7" Two-Way Flat Slab	Level 3	5369	9204	SF	$ 4.43	$ 4.33	$ -	CY	$ 1.37	$ 0.38	$ 0.04	$ 80,982.98
7" Two-Way Flat Slab	T/Parapet	29511	50590	SF	$ 4.43	$ 4.33	$ -	CY	$ 1.37	$ 0.38	$ 0.04	$ 445,124.86
Totals		201708	333141									$ 3,884,525.08

Figure 75: Cost of concrete floor slab.

Structural Column Schedule					RS Means 2010														
					Formwork				Concrete				Reinforcing						
Family and Type	Length (ft.)	Area (sq. ft)	Volume (CY)	Count	Units	Material	Labor	Equipment	Units	Material	Labor	Equipment	Type	Weight (tons)	Units	Material	Labor	Equipment	Total Cost
20"x20" square	3318	658.9	341.35	237	SF	$2.28	$6.65	$ -	CY	$ 109.00	$ 37.00	$ 18.45	#7 and below	174.00	Ton	$ 800.00	$1,000.00	$ -	$ 566,906.11
24"x24" square	5092	1204	754.44	301	SF	$2.28	$6.65	$ -	CY	$ 109.00	$ 37.00	$ 18.45	#8 and above	38.30	Ton	$ 800.00	$ 650.00	$ -	$ 543,375.14
22" dia. circular	599	12920	58.52	34	LF	$ 8.15	$9.70	$ -	CY	$ 109.00	$ 37.00	$ 18.45							$ 20,315.76
Totals	12357			469															$ 1,130,597.02

Figure 77: Cost of concrete columns.

Structural Beam Schedule				RS Means 2010														
				Formwork				Concrete				Reinforcing						
Type	Length (ft.)	Area (sq. ft)	Volume (CY)	Units	Material	Labor	Equipment	Units	Material	Labor	Equipment	Type	Weight (tons)	Units	Material	Labor	Equipment	Total Cost
16" x 20"	1432	6207	117.9	SF	$ 3.43	$ 8.65	$ -	CY	$109.00	$ 39.50	$ 19.85	#7 and below	80.00	Ton	$ 800.00	$ 935.00	$ -	$ 233,608.89
18"x20"	8366	39039	774.59	SF	$ 3.43	$ 8.65	$ -	CY	$109.00	$ 39.50	$ 19.85							$ 602,021.53
																		$ 835,630.42

Figure 76: Cost of concrete beams.

Shear Wall Schedule				RS Means 2010														
				Formwork				Concrete				Reinforcing						
Type	Length (ft.)	Area (sq. ft)	Volume (CY)	Units	Material	Labor	Equipment	Units	Material	Labor	Equipment	Type	Weight (tons)	Units	Material	Labor	Equipment	Total Cost
12", 8ksi	1432	131922	2443	SF	$ 3.43	$ 8.65	$ -	CY	$109.00	$ 39.50	$ 19.85	#7 and below	98.00	Ton	$ 800.00	$ 935.00	$ -	$ 2,174,926.81
																		$ 2,174,926.81

Figure 78: Cost of concrete shear walls.

Structural System Cost		
Original Steel Design		
Slabs	$	1,771,988
Columns	$	1,987,166
Framing	$	3,217,304
Total	$	6,976,458

Figure 79: Overall structural system cost of steel design.

Structural System Cost		
Concrete Redesign		
Slabs	$	3,884,525
Columns	$	1,130,597
Framing	$	835,630
Walls	$	2,174,927
Total	$	8,025,679

Figure 80: Overall structural system cost of concrete design.

Based upon this cost analysis, the steel structural system is less costly. Of course, this does not nearly include all of the costs involved with the superstructure of a project. The foundation system of the two designs could change these numbers drastically, especially if mini-piles are not used. Expensive labor activities such as moment connection detailing are not considered nor are other structural elements such as shear studs. Lastly, location has an impact when discussing project cost. In the case of UMCP, a main reason why steel was chosen for the design is due to the fact that most of the buildings in that area are built with steel. This creates a competitive environment for steel contractors and consequently drives down prices. Overall, a $1 million difference is not very substantial considering all of the variables left out.

However, this investigation accomplished what it set out to achieve and that is to provide a basic comparison between the two systems. Upon completion of this analysis, it is determined that the two designs are very comparable.

Schedule

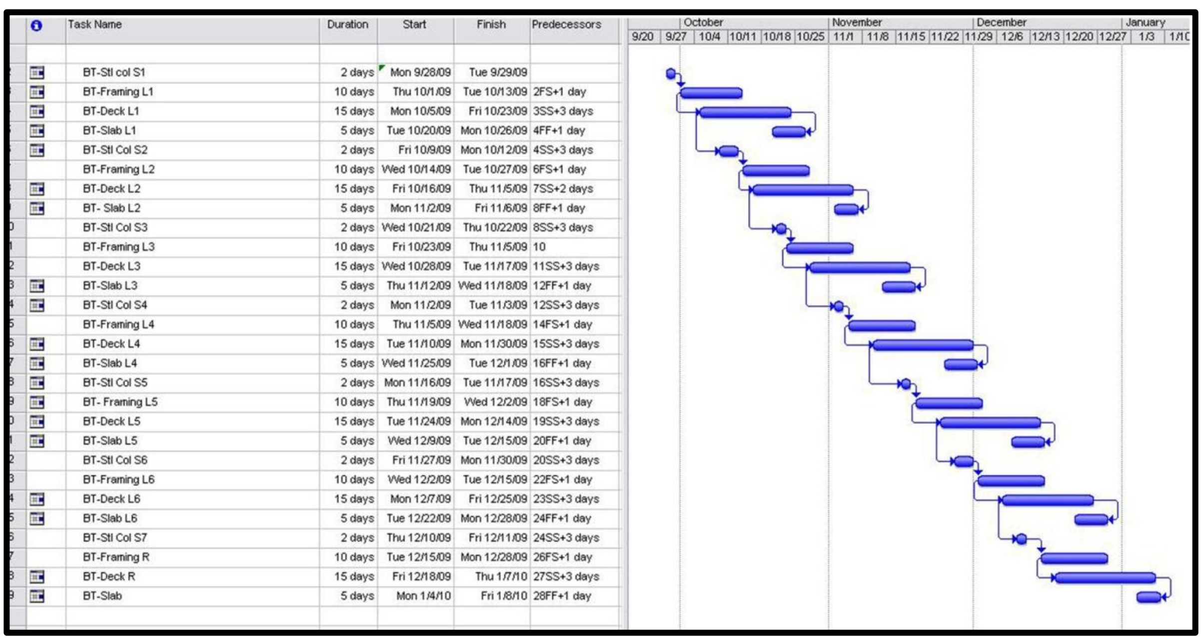
The other issue considered in this breadth is the effect of a concrete design on the construction schedule. For the purposes of this study, a schedule for each structural system was created so a fair comparison could be performed. These schedules were created without regard to any other building system or construction task. The goal was to determine which system would require a longer construction time if built independently from the building.

Of course, this is not a realistic assumption. Construction projects involve a high amount of variability as well as coordination between trades. No building system is built in a linear fashion without affecting the other systems. The idea here is to get a general idea of how much longer a concrete system would take to construct as opposed to a steel system.

Assumptions

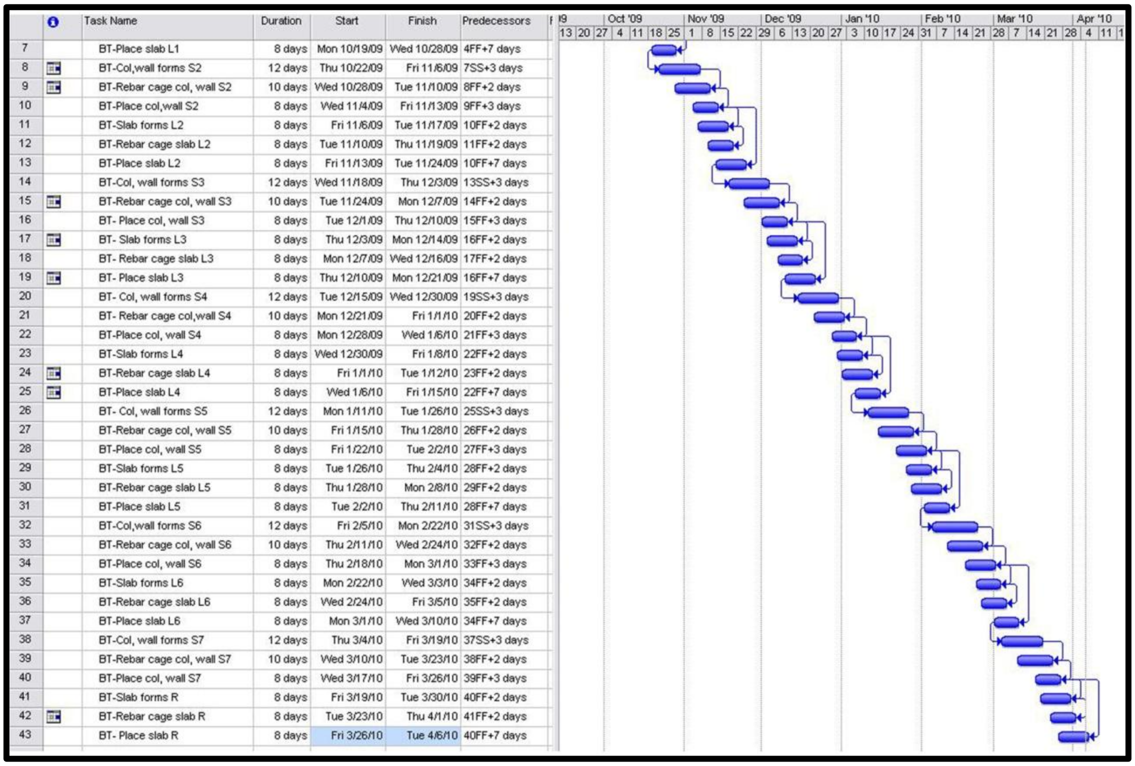
- The New Hospital is divided into four construction regions from west to east. Instead of performing the construction in two separate phases, the schedule assumes that the entire hospital is being built at once.
- It is assumed that approximately 950 linear feet of steel column can be erected in one day. The linear footage of all steel columns with just over 12,000 linear feet of steel column for the entire facility, it is assumed that an entire level of columns can be erected in two days.
- It is assumed that approximately 1000 linear feet of steel beam can be erected in one day. This means that steel framing for each floor will take around 10 days.
- Shear stud attachment is included in the construction time for the metal deck.
- Concrete slabs are not poured on top of concrete columns until 7 days after the concrete placement n the column. This allows enough time for the concrete in the column to gain the strength necessary to support the slab.
- It is assumed that 300 cubic yards of concrete are placed in one day.
- There was no consideration for holidays or weather conditions.

Below are the schedules for the steel and concrete structures.



Task Name	Duration	Start	Finish	Predecessors
BT-Stl col S1	2 days	Mon 9/28/09	Tue 9/29/09	
BT-Framing L1	10 days	Thu 10/1/09	Tue 10/13/09	2FS+1 day
BT-Deck L1	15 days	Mon 10/5/09	Fri 10/23/09	3SS+3 days
BT-Slab L1	5 days	Tue 10/20/09	Mon 10/26/09	4FF+1 day
BT-Stl Col S2	2 days	Fri 10/9/09	Mon 10/12/09	4SS+3 days
BT-Framing L2	10 days	Wed 10/14/09	Tue 10/27/09	6FS+1 day
BT-Deck L2	15 days	Fri 10/16/09	Thu 11/5/09	7SS+2 days
BT- Slab L2	5 days	Mon 11/2/09	Fri 11/6/09	8FF+1 day
BT-Stl Col S3	2 days	Wed 10/21/09	Thu 10/22/09	8SS+3 days
BT-Framing L3	10 days	Fri 10/23/09	Thu 11/5/09	10
BT-Deck L3	15 days	Wed 10/28/09	Tue 11/17/09	11SS+3 days
BT-Slab L3	5 days	Thu 11/12/09	Wed 11/18/09	12FF+1 day
BT-Stl Col S4	2 days	Mon 11/2/09	Tue 11/3/09	12SS+3 days
BT-Framing L4	10 days	Thu 11/5/09	Wed 11/18/09	14FS+1 day
BT-Deck L4	15 days	Tue 11/10/09	Mon 11/30/09	15SS+3 days
BT-Slab L4	5 days	Wed 11/25/09	Tue 12/1/09	16FF+1 day
BT-Stl Col S5	2 days	Mon 11/16/09	Tue 11/17/09	16SS+3 days
BT- Framing L5	10 days	Thu 11/19/09	Wed 12/2/09	18FS+1 day
BT-Deck L5	15 days	Tue 11/24/09	Mon 12/14/09	19SS+3 days
BT-Slab L5	5 days	Wed 12/9/09	Tue 12/15/09	20FF+1 day
BT-Stl Col S6	2 days	Fri 11/27/09	Mon 11/30/09	20SS+3 days
BT-Framing L6	10 days	Wed 12/2/09	Tue 12/15/09	22FS+1 day
BT-Deck L6	15 days	Mon 12/7/09	Fri 12/25/09	23SS+3 days
BT-Slab L6	5 days	Tue 12/22/09	Mon 12/28/09	24FF+1 day
BT-Stl Col S7	2 days	Thu 12/10/09	Fri 12/11/09	24SS+3 days
BT-Framing R	10 days	Tue 12/15/09	Mon 12/28/09	26FS+1 day
BT-Deck R	15 days	Fri 12/18/09	Thu 1/7/10	27SS+3 days
BT-Slab	5 days	Mon 1/4/10	Fri 1/8/10	28FF+1 day

Figure 81: Construction schedule of structural system for original design.



		Task Name	Duration	Start	Finish	Predecessors
7		BT-Place slab L1	8 days	Mon 10/19/09	Wed 10/28/09	4FF+7 days
8		BT-Col,wall forms S2	12 days	Thu 10/22/09	Fri 11/6/09	7SS+3 days
9		BT-Rebar cage col, wall S2	10 days	Wed 10/28/09	Tue 11/10/09	8FF+2 days
10		BT-Place col,wall S2	8 days	Wed 11/4/09	Fri 11/13/09	9FF+3 days
11		BT-Slab forms L2	8 days	Fri 11/6/09	Tue 11/17/09	10FF+2 days
12		BT-Rebar cage slab L2	8 days	Tue 11/10/09	Thu 11/19/09	11FF+2 days
13		BT-Place slab L2	8 days	Fri 11/13/09	Tue 11/24/09	10FF+7 days
14		BT-Col, wall forms S3	12 days	Wed 11/18/09	Thu 12/3/09	13SS+3 days
15		BT-Rebar cage col, wall S3	10 days	Tue 11/24/09	Mon 12/7/09	14FF+2 days
16		BT- Place col, wall S3	8 days	Tue 12/1/09	Thu 12/10/09	15FF+3 days
17		BT- Slab forms L3	8 days	Thu 12/3/09	Mon 12/14/09	16FF+2 days
18		BT- Rebar cage slab L3	8 days	Mon 12/7/09	Wed 12/16/09	17FF+2 days
19		BT- Place slab L3	8 days	Thu 12/10/09	Mon 12/21/09	16FF+7 days
20		BT- Col, wall forms S4	12 days	Tue 12/15/09	Wed 12/30/09	19SS+3 days
21		BT- Rebar cage col,wall S4	10 days	Mon 12/21/09	Fri 1/1/10	20FF+2 days
22		BT-Place col, wall S4	8 days	Mon 12/28/09	Wed 1/6/10	21FF+3 days
23		BT-Slab forms L4	8 days	Wed 12/30/09	Fri 1/8/10	22FF+2 days
24		BT-Rebar cage slab L4	8 days	Fri 1/1/10	Tue 1/12/10	23FF+2 days
25		BT-Place slab L4	8 days	Wed 1/6/10	Fri 1/15/10	22FF+7 days
26		BT- Col, wall forms S5	12 days	Mon 1/11/10	Tue 1/26/10	25SS+3 days
27		BT-Rebar cage col, wall S5	10 days	Fri 1/15/10	Thu 1/28/10	26FF+2 days
28		BT-Place col, wall S5	8 days	Fri 1/22/10	Tue 2/2/10	27FF+3 days
29		BT-Slab forms L5	8 days	Tue 1/26/10	Thu 2/4/10	28FF+2 days
30		BT-Rebar cage slab L5	8 days	Thu 1/28/10	Mon 2/8/10	29FF+2 days
31		BT-Place slab L5	8 days	Tue 2/2/10	Thu 2/11/10	28FF+7 days
32		BT-Col,wall forms S6	12 days	Fri 2/5/10	Mon 2/22/10	31SS+3 days
33		BT-Rebar cage col, wall S6	10 days	Thu 2/11/10	Wed 2/24/10	32FF+2 days
34		BT-Place col, wall S6	8 days	Thu 2/18/10	Mon 3/1/10	33FF+3 days
35		BT-Slab forms L6	8 days	Mon 2/22/10	Wed 3/3/10	34FF+2 days
36		BT-Rebar cage slab L6	8 days	Wed 2/24/10	Fri 3/5/10	35FF+2 days
37		BT-Place slab L6	8 days	Mon 3/1/10	Wed 3/10/10	34FF+7 days
38		BT-Col, wall forms S7	12 days	Thu 3/4/10	Fri 3/19/10	37SS+3 days
39		BT-Rebar cage col, wall S7	10 days	Wed 3/10/10	Tue 3/23/10	38FF+2 days
40		BT-Place col, wall S7	8 days	Wed 3/17/10	Fri 3/26/10	39FF+3 days
41		BT-Slab forms R	8 days	Fri 3/19/10	Tue 3/30/10	40FF+2 days
42		BT-Rebar cage slab R	8 days	Tue 3/23/10	Thu 4/1/10	41FF+2 days
43		BT- Place slab R	8 days	Fri 3/26/10	Tue 4/6/10	40FF+7 days

Figure 82: Construction schedule for redesigned structural system.

According to these schedules, the steel structure (in theory) could be erected in 102 days. The concrete structure would take 189 days. Steel will typically be built quicker than concrete so this result is not of the ordinary. However, it should be noted that the lead time for steel construction is significantly larger than for concrete. This is due to the specific fabrication process for each member of the structure.

It is not known whether there was a particular time constraint on when this hospital should be completed. If time was not a great concern, than the additional time needed to build a concrete building would not be a significant issues. On the other hand if there was a pressing need for this hospital to be built within a particular timeframe, steel would be the likely choice for the structural system.

Summary

When considering the construction issues surrounding these two systems, it appears that the steel structure is more ideal. From a cost perspective there is a slight advantage to building with steel. However, there are significant additional costs related to the steel structure which were not considered in this evaluation. Under a more detailed cost analysis, it is conceivable that a concrete structure could be viable economical alternative.

In terms of scheduling, steel typically outperforms concrete and UMCP is no exception to the rule. If construction time was an issue for this project, steel is the logical choice for the structural system.

MAE Requirement

The vibration research and analysis of the steel and concrete floor systems is representative of master's level work which is a requirement for this particular thesis project. The implementation of the RAM 3-D model to analyze and design the New Hospital of the University Medical Center is also considered sufficient for the MAE requirement.

Final Summary

The aim of this project was to investigate whether a redesign of the UMCP structural system as a concrete structure will eliminate the need for mini-piles underneath the spread footings. Based upon findings stated above it appears that the additional weight of a concrete structure is enough to counteract the tension force in lateral resisting elements of the structure. Without net tension, the spread footings do not need to be anchored into bedrock with mini-piles. This is likely to result in substantial excavation and foundation cost savings.

The secondary goal of this project was to improve the vibration response of the floor system. This was accomplished with an 8" two-way flat slab which meets the 4000 μin/s sensitive equipment criteria for operating rooms.

Other considerations regarding a concrete redesign are not as promising. It was determined that construction time for the redesign is substantially longer than the original. A simple cost analysis concluded that a steel system was less expensive compared with the redesign. However, those results do not consider the additional foundation costs of the mini-piles.

Architecturally, the concrete structure interacts with the façade in a much different way than the original steel design. This could become a concern depending upon the desires of the owner.

Appendix A

Case 1-X					
Diaphragm 1					
Level	X	Y	Angle	x	y
10	17.41	0.00	0		
9	34.34	0.00	0		
8	33.32	0.00	0		
7	32.21	0.00	0		
6	31.00	0.00	0		
5	29.66	0.00	0		
4	28.15	0.00	0		
3	26.39	0.00	0		
2	27.58	0.00	0		
1	26.28	0.00	0		

Case 1-X					
Diaphragm 2					
Level	X	Y	Angle	x	y
10	-17.41	0.00	0		
9	-34.34	0.00	0		
8	-33.32	0.00	0		
7	-32.21	0.00	0		
6	-31.00	0.00	0		
5	-29.66	0.00	0		
4	-28.15	0.00	0		
3	-26.39	0.00	0		
2	-27.58	0.00	0		
1	-26.28	0.00	0		

Case 1-Y					
Diaphragm 1					
Level	X	Y	Angle	x	y
10	0	66.19	90		
9	0	131.24	90		
8	0	128.85	90		
7	0	126.27	90		
6	0	123.44	90		
5	0	120.30	90		
4	0	116.76	90		
3	0	112.66	90		
2	0	122.71	90		
1	0	125.11	90		

Case 1-Y			
Diaphragm 2			
Level	X	Y	Angle
10	0	34.60	90
9	0	68.61	90
8	0	67.36	90
7	0	66.01	90
6	0	64.53	90
5	0	62.89	90
4	0	61.04	90
3	0	58.89	90
2	0	64.15	90
1	0	65.40	90

Case 2-X					
Diaphragm 1					
Level	X	Y	Angle	x	y
10	13.06	0.00	0		24
9	25.75	0.00	0		24
8	24.99	0.00	0		24
7	24.16	0.00	0		24
6	23.25	0.00	0		24
5	22.25	0.00	0		24
4	21.11	0.00	0		24
3	19.79	0.00	0		24
2	20.69	0.00	0		24
1	19.71	0.00	0		24

Case 2-X					
Diaphragm 2					
Level	X	Y	Angle	x	y
10	-13.06	0.00	0		24
9	-25.75	0.00	0		24
8	-24.99	0.00	0		24
7	-24.16	0.00	0		24
6	-23.25	0.00	0		24
5	-22.25	0.00	0		24
4	-21.11	0.00	0		24
3	-19.79	0.00	0		24
2	-20.69	0.00	0		24
1	-19.71	0.00	0		24

Case 2-Y					
Diaphragm 1					
Level	X	Y	Angle	x	y
10	0.00	49.65	90	60	
9	0.00	98.43	90	60	
8	0.00	96.64	90	60	
7	0.00	94.70	90	60	
6	0.00	92.58	90	60	
5	0.00	90.23	90	60	
4	0.00	87.57	90	60	
3	0.00	84.49	90	60	
2	0.00	92.03	90	60	
1	0.00	93.83	90	60	

Case 2-Y				
Diaphragm 2				
Level	X	Y	Angle	x
10	0.00	25.95	90	-30
9	0.00	51.46	90	-30
8	0.00	50.52	90	-30
7	0.00	49.51	90	-30
6	0.00	48.40	90	-30
5	0.00	47.17	90	-30
4	0.00	45.78	90	-30
3	0.00	44.17	90	-30
2	0.00	48.11	90	-30
1	0.00	49.05	90	-30

Case 3-X+Y						
Diaphragm 1						
Level	X	Y	Result	Angle	x	y
10	13.06	49.65	51.34	75.3		
9	25.75	98.43	101.74	75.3		
8	24.99	96.64	99.82	75.5		
7	24.16	94.70	97.73	75.7		
6	23.25	92.58	95.45	75.9		
5	22.25	90.23	92.93	76.1		
4	21.11	87.57	90.08	76.4		
3	19.79	84.49	86.78	76.8		
2	20.69	92.03	94.33	77.3		
1	19.71	93.83	95.88	78.1		

Case 3-X+Y						
Diaphragm 2						
Level	X	Y	Result	Angle	x	y
10	-13.06	25.95	29.05	116.7		
9	-25.75	51.46	57.54	116.6		
8	-24.99	50.52	56.36	116.3		
7	-24.16	49.51	55.09	116.0		
6	-23.25	48.40	53.69	115.7		
5	-22.25	47.17	52.15	115.3		
4	-21.11	45.78	50.41	114.8		
3	-19.79	44.17	48.40	114.1		
2	-20.69	48.11	52.37	113.3		
1	-19.71	49.05	52.86	111.9		

Case 3-X-Y						
Diaphragm 1						
Level	X	Y	Result	Angle	x	y
10	13.06	-49.65	51.34	-75.3		
9	25.75	-98.43	101.74	-75.3		
8	24.99	-96.64	99.82	-75.5		
7	24.16	-94.70	97.73	-75.7		
6	23.25	-92.58	95.45	-75.9		
5	22.25	-90.23	92.93	-76.1		
4	21.11	-87.57	90.08	-76.4		
3	19.79	-84.49	86.78	-76.8		
2	20.69	-92.03	94.33	-77.3		
1	19.71	-93.83	95.88	-78.1		

Case 3-X-Y					
Diaphragm 2					
Level	X	Y	Result	Angle	x
10	-13.06	-25.95	29.05	243.3	
9	-25.75	-51.46	57.54	243.4	
8	-24.99	-50.52	56.36	243.7	
7	-24.16	-49.51	55.09	244.0	
6	-23.25	-48.40	53.69	244.3	
5	-22.25	-47.17	52.15	244.7	
4	-21.11	-45.78	50.41	245.2	
3	-19.79	-44.17	48.40	245.9	
2	-20.69	-48.11	52.37	246.7	
1	-19.71	-49.05	52.86	248.1	

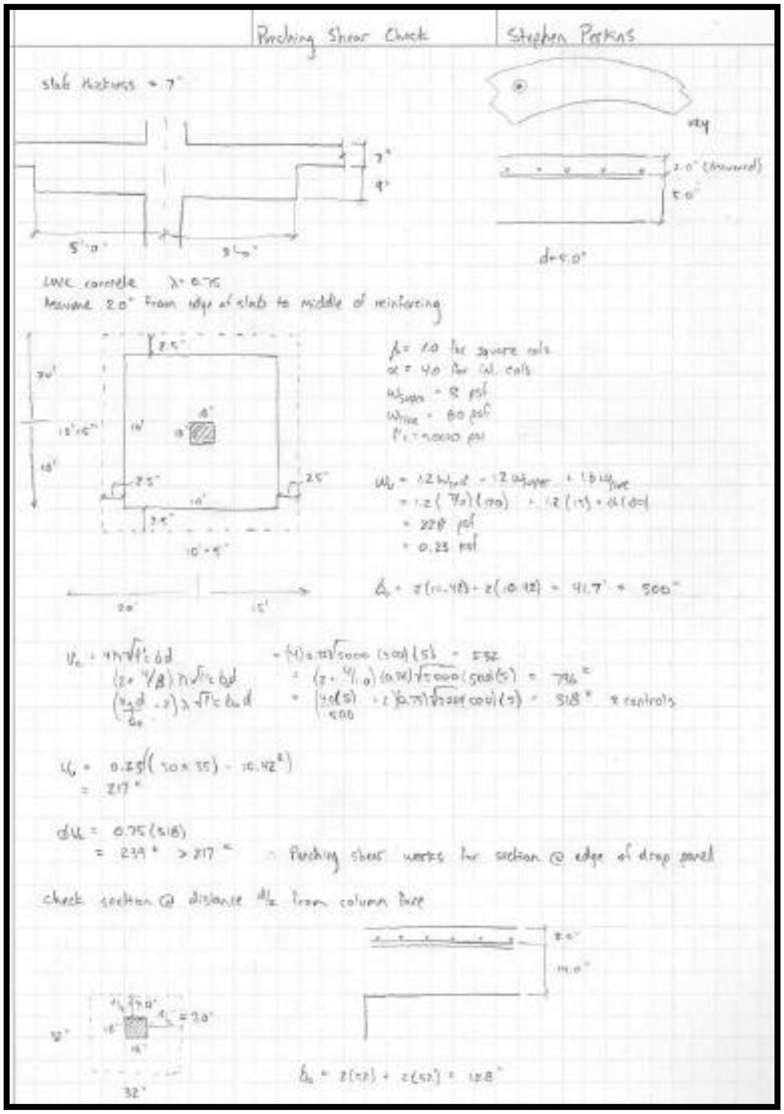
Case 4-X+Y						
Diaphragm 1						
Level	X	Y	Result	Angle	x	y
10	9.80	37.26	38.53	75.3	60	24
9	19.33	73.89	76.38	75.3	60	24
8	18.76	72.54	74.93	75.5	60	24
7	18.13	71.09	73.37	75.7	60	24
6	17.45	69.50	71.65	75.9	60	24
5	16.70	67.73	69.76	76.1	60	24
4	15.85	65.74	67.62	76.4	60	24
3	14.86	63.43	65.14	76.8	60	24
2	15.53	69.09	70.81	77.3	60	24
1	14.80	70.44	71.97	78.1	60	24

Case 4-X+Y						
Diaphragm 2						
Level	X	Y	Result	Angle	x	y
10	-9.80	25.95	27.74	110.7	-30	24
9	-19.33	51.46	54.97	110.6	-30	24
8	-18.76	50.52	53.89	110.4	-30	24
7	-18.13	49.51	52.73	110.1	-30	24
6	-17.45	48.40	51.45	109.8	-30	24
5	-16.70	47.17	50.04	109.5	-30	24
4	-15.85	45.78	48.45	109.1	-30	24
3	-14.86	44.17	46.60	108.6	-30	24
2	-15.53	48.11	50.55	107.9	-30	24
1	-14.80	49.05	51.23	106.8	-30	24

Case 4-X-Y						
Diaphragm 1						
Level	X	Y	Result	Angle	x	y
10	9.80	-37.26	38.53	-75.3	60	24
9	19.33	-73.89	76.38	-75.3	60	24
8	18.76	-72.54	74.93	-75.5	60	24
7	18.13	-71.09	73.37	-75.7	60	24
6	17.45	-69.50	71.65	-75.9	60	24
5	16.70	-67.73	69.76	-76.1	60	24
4	15.85	-65.74	67.62	-76.4	60	24
3	14.86	-63.43	65.14	-76.8	60	24
2	15.53	-69.09	70.81	-77.3	60	24
1	14.80	-70.44	71.97	-78.1	60	24

Case 4-X-Y					
Diaphragm 2					
Level	X	Y	Result	Angle	x
10	-9.80	-14.61	17.59	236.1	-30
9	-19.33	-28.97	34.83	236.3	-30
8	-18.76	-28.44	34.07	236.6	-30
7	-18.13	-27.87	33.25	237.0	-30
6	-17.45	-27.25	32.36	237.4	-30
5	-16.70	-26.56	31.37	237.8	-30
4	-15.85	-25.77	30.26	238.4	-30
3	-14.86	-24.87	28.97	239.1	-30
2	-15.53	-27.09	31.22	240.2	-30
1	-14.80	-27.62	31.33	241.8	-30

Appendix B



Punching Shear Check | Stephen Perkins

slab thickness = 7"

7"
9"
5'-0"
5'-0"
key
2.0" (assumed)
5.0"
d = 5.0"

LWC concrete $\lambda = 0.75$
Assume 2.0" from edge of slab to middle of reinforcing

2.5"
20'
15'-15"
10'
18"
18"
15'
2.5"
10'
2.5"
2.5"
10'+5"
20'
15'

$\beta = 1.0$ for square cols
$\alpha = 4.0$ for int. cols
$w_{super} = 8$ psf
$w_{live} = 80$ psf
$f'_c = 5000$ psi

$w_u = 1.2 w_{self} + 1.2 w_{super} + 1.6 w_{live}$
$= 1.2(7/12)(110) + 1.2(17) + 1.6(80)$
$= 228$ psf
$= 0.23$ ksf

$b_o = 2(10.42) + 2(10.42) = 41.7' = 500"$

$V_c = 4\lambda\sqrt{f'_c}\, b d$ $= (4)(0.75)\sqrt{5000}(500)(5) = 532$
$(2 + 4/\beta)\lambda\sqrt{f'_c}\, b d$ $= (2 + 4/1.0)(0.75)\sqrt{5000}(500)(5) = 796^k$
$\left(\frac{\alpha_s d}{b_o} + 2\right)\lambda\sqrt{f'_c}\, b_o d$ $= \left(\frac{40(5)}{500} + 2\right)(0.75)\sqrt{5000}(500)(5) = 318^k$ * controls

$V_u = 0.23\left((30 \times 35) - 10.42^2\right)$
$= 217^k$

$\phi V_c = 0.75(318)$
$= 239^k > 217^k$ ∴ Punching shear works for section @ edge of drop panel

check section @ distance d/2 from column face

2.0"
14.0"
d/2 = 7.0"
d/2 = 7.0"
18"
18"
32"
32"

$b_o = 2(32) + 2(32) = 128"$

$V_c = 4\lambda\sqrt{f'_c}\,b_o d = 4(0.75)\sqrt{5000}\,(128)(7) = 190^k$ ∗ controls

$(2 + 4/\beta)\lambda\sqrt{f'_c}\,b_o d = (2 + 4/1.0)\,0.75\sqrt{5000}\,(28)(7) = 285^k$

$\left(\frac{\alpha_s d}{b_o} + 2\right)\lambda\sqrt{f'_c}\,b_o d = \left(\frac{40(7)}{128} + 2\right)0.75\sqrt{5000}\,(28)(7) = 199^k$

$V_u = 0.23(30 \times 35) - 2.57^k) = 240^k$

$\phi V_c = 0.75(190)$

$= 145^k < 240^k$ ∴ shear stud reinforcement is req.

Recalculate V_c and ϕV_n according to ACI 11.11.5.1

$V_c = 3\lambda\sqrt{f'_c}\,b_o d = 143^k$

$V_s + V_c \geq V_u/\phi \rightarrow V_s \geq 240/0.75 - 143 = 177^k$

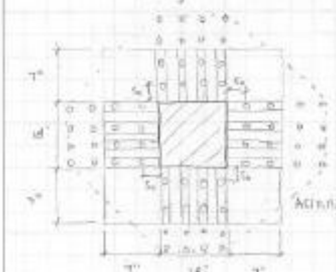
$\phi V_n = \phi 8\sqrt{f'_c}\,b_o d = 0.75(8)\sqrt{5000}\,(28)(7) = 286^k > 240^k$ ✓ ok

Try (16) stud rails each w/ (4) 3/8 dia. studs w/ 1.2" dia. heads.

$f_{yt} = 60$ ksi

$A_b = 0.11$ in²

∗ Assume spacing to first stud row is less than $d/2$ (say 2.0")



16 rails w/ 2 studs on each within $d/2$

$16(2) = 32$ studs

$A_b = 0.11$ in²

$32(0.11) = 3.52$ in² × 60 ksi = $211^k > 177^k$ ✓ ok

V_u @ $d/2 = 240^k$

$= 240/b_o d = 240/128(7) = 268$ psi

ACI 11.11.5.2 $V_{u,max} = \phi 6\sqrt{f'_c} = 0.75(6)\sqrt{5000} = 318$ psi

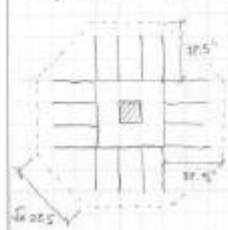
∴ Allowed to space subsequent studs @ 0.75d from initial stud

$0.75(14) = 10.5"$

Initial spacing = 2.0"

Subsequent spacing = 4.5"

Length of rail = 2.0 + 3(4.5) = 15.5" + d/2 = 15.5 + 14/2 = 22.5"



$b_o = 4\left[\sqrt{2}\,(22.5) + 18\right] = 199"$

area = $4(22.5^2)/2 + 4(22.5 \times 18) + (18 \times 18) = 20.5$ ft²

Shear @ critical section edge

$V_u = 0.23(30(35) - 20.5) = 237^k$

$= 237/b_o d = 237/(199)(7) = 170$ psi

ACI 11.11.5.4 $\phi V_c = \phi 2\lambda\sqrt{f'_c} = 0.75(2)(0.75)\sqrt{5000} = 79.5$ psi < 170 psi

∴ need more studs

Try (14) studs @ 4.5" spacing

Length of rail = 2 + 13(4.5) + 7 = 67.5"

$b_o = 4(\sqrt{2}\,(67.5) + 18) = 454"$

area = $4(67.5^2)/2 + 4(67.5)(18) + 18 \times 18 = 99.5$ ft²

Shear @ critical section edge

$V_u = 0.23(30(35) - 99.5) = 219$ k

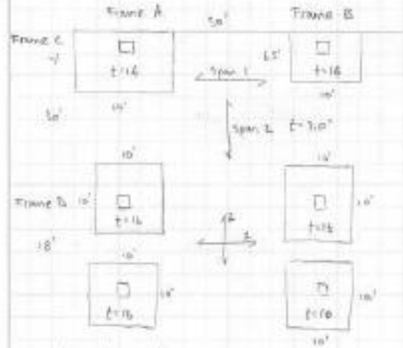
$= 219/b_o d = 219/(454)(7) = 69$ psi ✓

$\phi V_c = 0.75(2)(0.75)\sqrt{5000} = 79.5$ psi > 69 psi

∴ use 16 stud rails w/ (14) 3/8" headed shear studs per rail

Initial stud @ 2.0" and 14 @ 4.5"

Two-Way Slab Deflection



$\Delta_{max} = \Delta_{cx} + \Delta_{mx} + \Delta_{cx} + \Delta_{cy}$

Assume panel is fully fixed against rotation and vertical displacement

$$\Delta_{ref} = \frac{wl^4}{384 E_c I_{frame}}$$

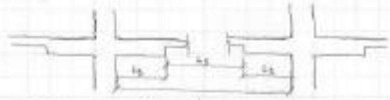
where I_{frame} = mom. of inertia of full width panel

Variation of moment, slab dimensions accounted for with M/EI for respective strips

$$\Delta_{col} = \Delta_{ref} \frac{M_{col}}{M_{frame}} \frac{E_c I_{frame}}{E_c I_{col}}$$

$$\Delta_{mid} = \Delta_{ref} \frac{M_{mid}}{M_{frame}} \frac{E_c I_{frame}}{E_c I_{mid}}$$

col = column strip
mid = middle strip



* Must account for drop panel thickness

$$I_{avg} = 2\,\frac{l_a}{l}\,I_a + \frac{l_s}{l}\,I_s$$

slab thickness = 7.0"
* Assume all drop panels = 9.0" below slab

* For simplicity, assume w_{live} = 80 psf for all spans

Strip width - Frame A, B
column strip = 0.25(30×12) = 90"
middle strip = 30(12) - 90 = 270"

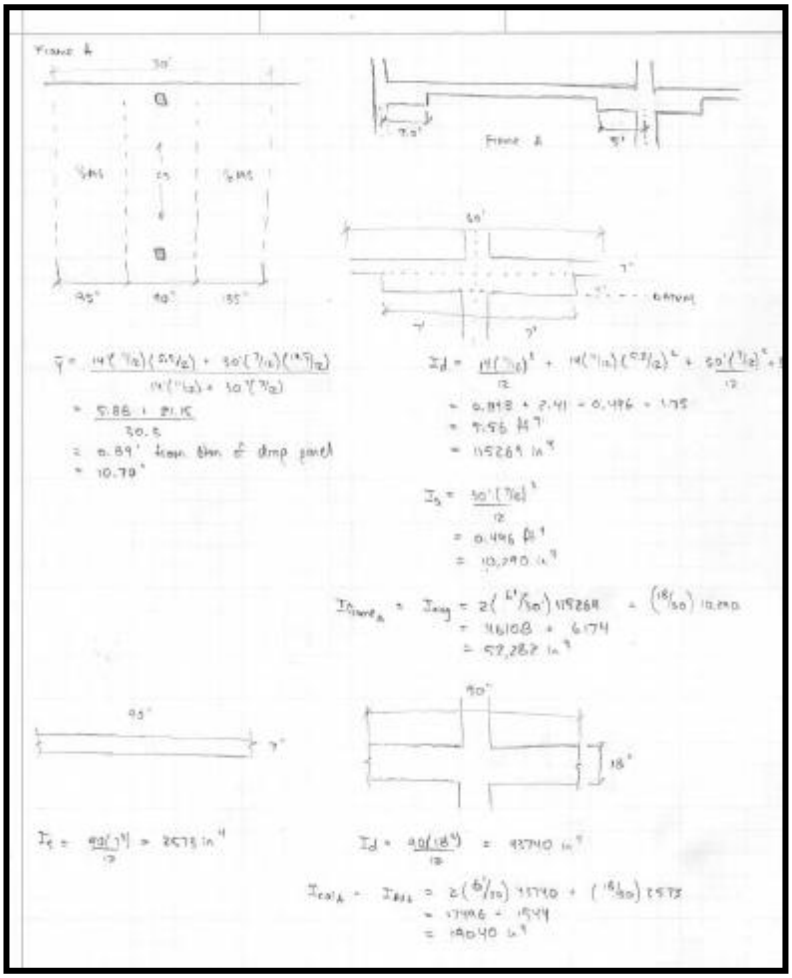
- Frame C
column strip = 0.25(30×12)/2 = 45"
middle strip = 171"

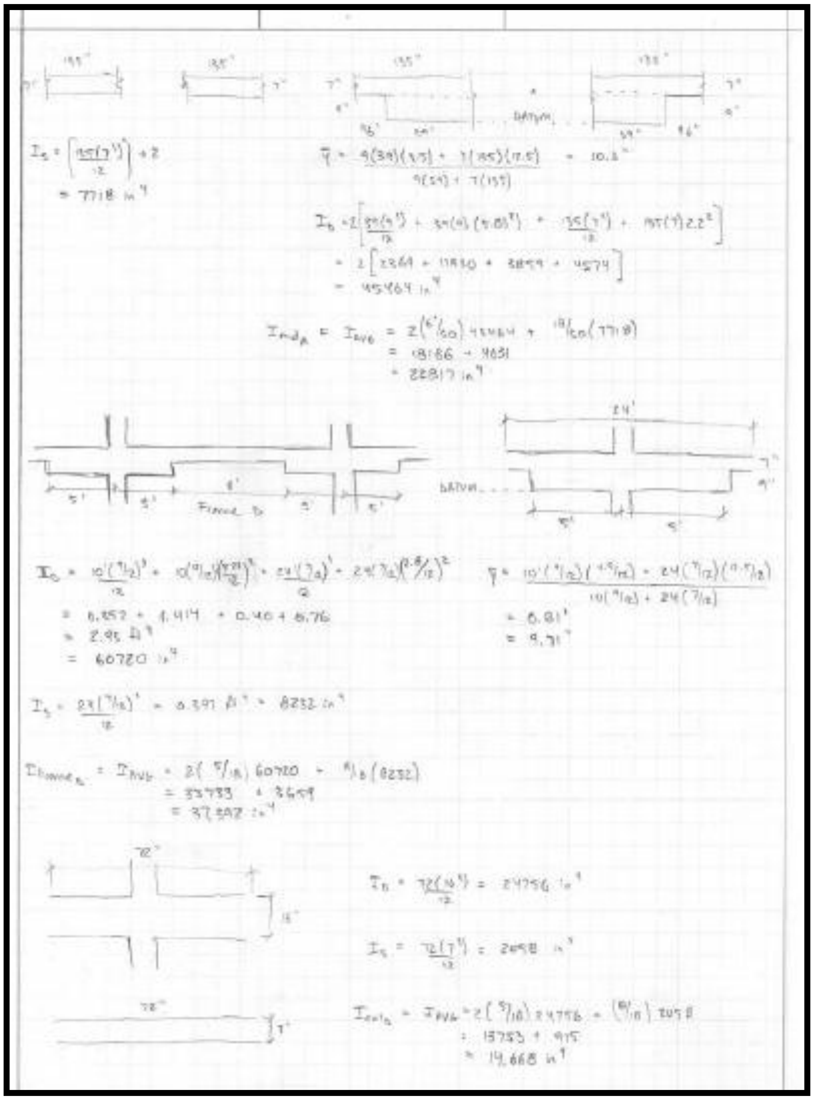
- Frame D
column strip = 0.25((17+18)+2) = 72"
middle strip = 216"

Loading

$w_D = (7/12)\,120 = 88$ psf

$E_c = w^{1.5}\, 33\sqrt{f'_c}$
$= 120^{1.5}(33)\sqrt{4000}$
$= 3.07e^6$ psi





$I_s = \left[\frac{135(7^3)}{12}\right] \div 2 = 7718\ in^4$

$\bar{y} = \frac{9(39)(4.5) + 7(135)(12.5)}{9(39) + 7(135)} = 10.2''$

$I_b = 2\left[\frac{39(9^3)}{12} + 39(9)(5.85^2) + \frac{135(7^3)}{12} + 135(7)2.2^2\right]$
$= 2\left[2369 + 11830 + 3859 + 4574\right]$
$= 45464\ in^4$

$I_{mid_A} = I_{AVG} = 2\left(\frac{6'}{60}\right)45464 + \frac{18}{60}(7718)$
$= 18186 + 4631$
$= 22817\ in^4$

$I_b = \frac{10'(9/12)^3}{12} + 10(9/12)\left(\frac{4.71}{12}\right)^2 + \frac{24'(7/12)^3}{12} + 24(7/12)\left(\frac{2.8}{12}\right)^2$
$= 0.352 + 1.414 + 0.40 + 0.76$
$= 2.95\ ft^4$
$= 60720\ in^4$

$\bar{y} = \frac{10'(9/12)(4.5/12) + 24(7/12)(12.5/12)}{10(9/12) + 24(7/12)}$
$= 0.81'$
$= 9.71''$

$I_s = \frac{24(7/12)^3}{12} = 0.397\ ft^4 = 8232\ in^4$

$I_{frame_B} = I_{AVG} = 2\left(\frac{5}{18}\right)60720 + \frac{8}{18}(8232)$
$= 33733 + 3659$
$= 37392\ in^4$

$I_b = \frac{72(16^3)}{12} = 24756\ in^4$

$I_s = \frac{72(7^3)}{12} = 2058\ in^4$

$I_{col_B} = I_{AVG} = 2\left(\frac{5}{18}\right)24756 + \left(\frac{8}{18}\right)2058$
$= 13753 + 915$
$= 14668\ in^4$

$I_1 = \frac{108(7^3)}{12} \cdot 2$

$= 6174 \text{ in}^4$

$\bar{y} = \frac{9(24)(4.5) + 7(108)(12.5)}{9(24) + 7(108)} = 9.9"$

$I_D = 2\left[\frac{24(9^3)}{12} + 24(9)5.4^2 + \frac{108(7^3)}{12} + 108(7)2.6^2\right]$

$= 2[1458 + 6299 + 3087 + 5111]$

$= 31909 \text{ in}^4$

$I_{mid_D} = I_{avg} = 2(5/18)(31909) + 8/18(6174)$

$= 17727 + 2744$

$= 20471 \text{ in}^4$

	A	D
I_{frame}	52,282 in^4	37,392 in^4
I_{col}	19,040 in^4	19,468 in^4
I_{mid}	22,817 in^4	20,471 in^4

Dead Only

$\Delta_{EFF_A} = \frac{wL^4}{384 E_c I_{frame_A}} = \frac{88(30)(30 \times 12)^4}{384(3.07e^6)(52282)(12)} = \frac{4.48e^9}{1.64e^{11}} = 0.059"$

* Assume $0.75M_o$ goes to CS and $0.25M_o$ goes to MS

$\Delta_{col_A} = \Delta_{EF_A} \frac{M_{col}}{M_{frame}} \frac{E_c I_{frame_A}}{E_c I_{col_A}}$

$= 0.059(0.75)\frac{52,282}{19,040}$

$= 0.123"$

$\Delta_{mid_A} = \Delta_{EF_A} \frac{M_{mid}}{M_{frame}} \frac{E_c I_{frame_A}}{E_c I_{mid_A}}$

$= 0.059(0.25)\frac{52,282}{22,817}$

$= 0.034"$

$$\Delta_{corr_B} = \frac{wL^4}{384 E_c I_{corr_B}} = \frac{88(30/(40 \times 12)^4)}{384(4)(5.97e^6)(37,392)} = \frac{4.45e^{15}}{9.41e^{14}} = 0.084''$$

$$\Delta_{t_{col_B}} = \frac{0.084(0.75)\,37,597}{14,668} = 0.160''$$

$$\Delta_{t_{mid_B}} = \frac{0.084(0.25)(37,392)}{26,471} = 0.038''$$

Dead only

$$\Delta_{max} = \Delta_{t_{col_A}} + \Delta_{t_{mid_B}} = 0.123 + 0.038 = 0.161''$$

$$= \Delta_{t_{col_B}} + \Delta_{t_{mid_A}} = 0.160 + 0.034 = 0.194''$$

Concept → 0.18''

Dead + Live

$$\Delta_{col_A} = \frac{168(30)(40 \times 12)^4}{384(4)(3.07e^6)\,72282} = 0.114''$$

$$\Delta_{t_{col_A}} = \frac{0.114(0.75)(72282)}{19,040} = 0.324''$$

$$\Delta_{t_{mid_A}} = \frac{0.114(0.25)(72282)}{22817} = 0.065''$$

$$\Delta_{col_B} = \frac{168(30)(40 \times 12)^4}{384(4)(3.07e^6)(37,392)} = 0.159''$$

$$\Delta_{t_{col_B}} = \frac{0.159(0.75)(37,392)}{14,668} = 0.304''$$

$$\Delta_{t_{mid_B}} = \frac{0.159(0.25)(37,392)}{22817} = 0.065''$$

Dead + Live

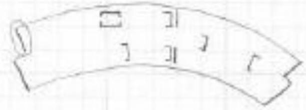
$$\Delta_{max} = \Delta_{t_{col_A}} + \Delta_{t_{mid_B}} = 0.234 + 0.065 = 0.30''$$

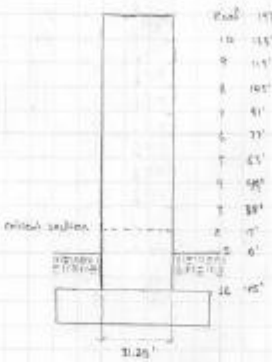
$$= \Delta_{t_{col_B}} + \Delta_{t_{mid_A}} = 0.304 + 0.065 = 0.37''$$

Concept → 0.35''

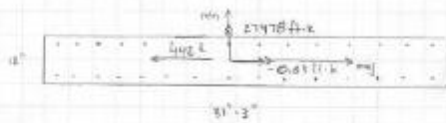
Shear Wall Design Check | Stephen Perkins

* Design assumptions
  - Rigid diaphragm
  - Neglect out-of-plane stiffness
  - Walls considered to be slender $h_w/\ell_w > 3$
  - Walls considered cracked $\therefore I = 0.35 I_g$





wall thickness = 12.0"



2 curtains of reinforcement V: #5 @ 10" o.c. H: #4 @ 12" o.c.
Normal weight concrete
$f'_c = 8000$ psi
Grade 60 steel ($f_y = 60$ ksi)

P	V_{maj}	V_{minor}	M_{maj}	M_{minor}
644	-18	0	27478	-0.03

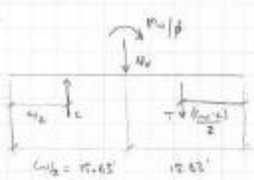
Minimum vertical Reinforcement (ACI 11.9.9)

$\rho_L = 0.0025 A_g$
$A_g = 1.0(31.25)(12) = 4500 \text{ in}^2$
$\rho_L = 0.0025(4500)$
$= 11.25 \text{ in}^2$
(78)#5 ⇒ $A_s = 24.2 \text{ in}^2$, $\rho_L = 0.0054$ ✓ ok

controlling load combo: 0.9D + 1.6W

Spacing of vertical reinforcement

$\min \begin{cases} \ell_w/3 = 25' \\ 3h = 3.0' \\ 18 = 18" \end{cases}$ ← controls > 10" ✓ ok



$C = T + N_u$ where $C = C_s + C_c$

$T = A_s f_y \left((\ell_w - c)/\ell_w \right)$

$C_s = A_s f_y \left(c/\ell_w \right)$

$C_c = 0.85 f'_c b a$ where $a = \beta c$, $\beta = 0.65$ for 8000 psi concrete

* Assume: Steel in tension zone yields in tension
  Steel in compression zone yields in compression
  Tension force acts at mid depth of tension zone
  Compression force acts at mid depth of compression zone

$c = \left(\dfrac{\alpha + \omega}{0.85\beta + 2\omega} \right) \ell_w$
where $\omega = \rho_L f_y / f'_c$
$\alpha = N_u / b \ell_w f'_c$

$\omega = 0.0059\,(^{60}/_{8})$
$= 0.0405$

$\alpha = 644/(12)(375")(8)$
$= 0.018$

$c = \left[(0.018 + 0.0405)/0.85(0.05) + 2(0.0405)\right] \times 375"$
$= 34.6"$

* Assume effective flexural depth $= 0.8\ell_w = 300$ in.
$c < 0.375\,d$ ∴ tension-controlled $\phi = 0.9$

$T = 24.2(60)\left(^{(375-34.6)}/_{375}\right)$
$= 1318^k$

$\phi M_n = M_u$

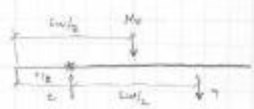
$M_n = {}^{M_u}/_{\phi}$

$C_s = 24.2(60)\left(^{34.6}/_{375}\right)$
$= 134^k$

$C_c = 0.85(8)(12)(0.85)(34.6)$
$= 1835^k$

$N_u = 644^k$

Flexural Capacity, M_n



Sum moments about c

$M_n = T\left(^{\ell_w}/_2\right) + N_u\left(\frac{\ell_w - c}{2}\right)$
$= 1318\left(^{375}/_2\right) + 644\left(^{(375-34.6)}/_2\right)$
$= 247125$ k-in $+ 109609$ k-in
$= 29728$ k-ft > 27478 k-ft ✓ ok

note: RAM output already applies ϕ so no need to apply strength reduction on M_n

check shear (ACI 11.9.6)

${}^{M_u}/_{V_u} - {}^{\ell_w}/_2$ will be negative ∴ use eqn 11-27

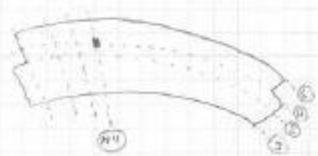
$V_c = 3.3\lambda\sqrt{f'_c}\,bd + {}^{N_u d}/_{4\ell_w}$ $\lambda = 1.0$ for NWC
$= 3.3(1.0)\sqrt{8000}\,(12)(300) + {}^{644(300)}/_{4(375)}$
$= 1063^k > 442^k$ ✓ ok

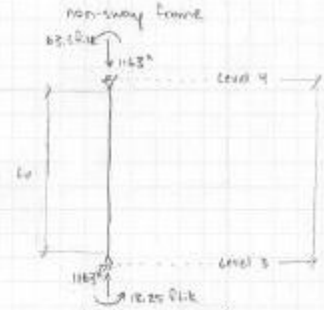
check horizontal reinforcement

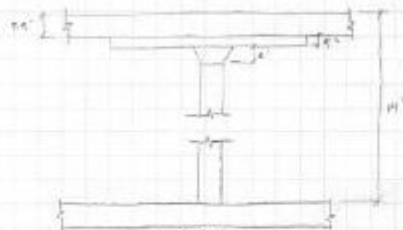
$\rho_t = {}^{A_v}/_{bs}$
$= \frac{2(0.20)}{12(12)}$
$= 0.0027 > 0.0025$ ✓ ok (ACI 11.9.9.2)

Column Design Check | Stephen Perkins

Design Assumptions
- column is braced against sidesway in both directions
- For simplicity of calculation, K will be taken as 1.0. This is conservative for a non-sway frame







$r = 0.3h$ (ACI 10.10.1.2)
$= 0.3(20)$
$= 6.0"$

$L_u = 14' - (9.5/12) - 2 = 11.13'$

$KL_u/r = 1.0(11.13)(12)/6 = 22.3 < 34 - 12(-11.85/63.58) = 36.5$ ∴ column is not slender

check non-sway assumption

ACI 10.10.1 → compression members are braced when bracing elements within a particular story are 12x as stiff as sum of column stiffness within story

* Assume column strip of slab behaves as wide, shallow beam connecting columns to form a frame.

$I_{cs_1} = 270894\ in^4$ at drop panel locations
$I_{cs_2} = 1248\ in^4$ at middle of span

$L = 5.0' = 60"$
$L_1 = 20' = 240"$
$L_2 = 8' = 96"$

$E = 4074$ ksi for slab + col

$I_{col_1} = 20(20^3)/12 = 13{,}333\ in^4$ $\quad L_1 = 14'(12) = 168"$
$I_{col_2} = 24(24^3)/12 = 27{,}648\ in^4$ $\quad L_2 = 18'(12) = 216"$

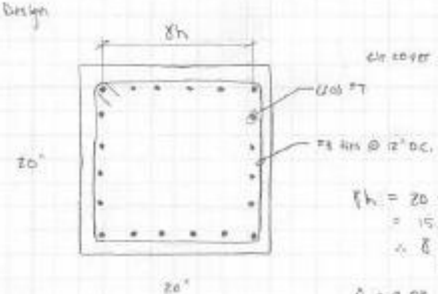
$\Sigma I_b/L_b = (270894/60)6 + (1248/240)2 + (1248/96) = 27113$

$\Sigma I_c/L_c = (13333/168)4 + (27648/216)4 = 830$

$\frac{27113}{830} = 32.6 >> 12$

∴ Braced assumption is valid

RAM Design



$\gamma h = 20 - 2(1.5) - 0.375(2) - 0.875$
$= 15.4"$
$\therefore \gamma = 15.4/20$
$= 0.77$

$\rho_g = 0.03$

$P_u = 1163.2$ k
$M_{ux} = -2.7$ ft-k
$M_{uy} = 63.0$ ft-k

For M_{ux}, Find max P_n

$\phi M_{nx}/bh^2 = 2.7(12)/20(20^2) = 0.004$ Using Fig A-10b $f'_c = 5$ ksi, $f_y = 60$ ksi, $\gamma = 0.75$

$\phi P_n/bh = 3.08 \rightarrow \phi P_n = 3.08(20)(20)$
$\phi P_{nx0} = 1232$ k

Using Fig A-10c $f'_c = 5$ ksi, $f_y = 60$ ksi, $\gamma = 0.90$

$\phi P_n/bh = 3.50 \rightarrow \phi P_n = 3.5(20)(20)$
$\phi P_{nx0} = 1400$ k

interpolate $\phi P_{nx0} = \frac{(77-75)(1400-1232)}{(90-75)} + 1232 = 1245.4$ k

$\phi P_{ny0} = 1245.4$ k

$1/\phi P_n = 1/\phi P_{nx0} + 1/\phi P_{ny0} - 1/\phi P_0$

$\phi P_0 = [0.85 f'_c (A_g - A_s) + f_y A_s] 0.65$
$= [0.85(4)(400-12) + 60(2)] 0.65$
$= 1540$ k

$1/\phi P_n = 1/1245 + 1/1245 - 1/1540$
$= 1/9.57e^{-4}$
$\phi P_n = 1045$ k

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$f_n = \pi/2 \left[g E_s I_T / wL^4 \right]^{1/2}$

$\Downarrow$

$0.18\sqrt{g/\Delta}$

where

$\Delta = 5wL^4/384 E_s I_T$

$g = 386$ in/s²
$E_s = 29000$ ksi
w = weight per unit length (unfactored)
L = span length

combined (joist and girder)

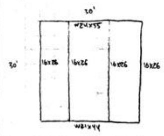
$f_n = 0.18\sqrt{g/(\Delta_j + \Delta_g)}$

$\Delta_p = \Delta_{jp} + \Delta_{gp}/2$

$\Delta_{jp} = \dfrac{\Delta_{oj}}{n_{eff}}$

Δ_{oj}, Δ_{gp} estimated by $\Delta = P L^3/96 E I_T$

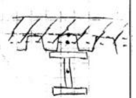
Beam mode



w_L = 15 psf (Actual, not design)
w_D = 33 psf (superimposed)

slab: 3.25 + 3.0 = 6.25"
120 pcf (4.75"/12") = 47.5 psf
deck = 3 psf
total = 51 psf

Slab width = 10'(12") = 10' <
0.4(30') = 12'
∴ entire slab width is used



$A_b = 7.68$ in² $\quad I_b = 301$ in⁴
$A_g = 16.20$ in² $\quad I_g = 1830$ in⁴
$d_b = 15.7$ in
$d_g = 23.6$ in

$n = E_s/1.35E_c$
$E_c = 57000\sqrt{f'c}$
$= 4031$ ksi
$n = 29000/1.35(4031)$
$= 5.33$

$\bar{y} = \dfrac{7.68(1.5 + 15.7/2) - [120''(4.75'')(4.75''/2)/(5.33)]}{7.68 + (120(4.75)/5.33)} = \dfrac{71.81 - 253.9}{114.62} = -1.59''$ (1.59" above middle of deck)

$I_b = 301 + 7.68(15.7/2 + 1.5 + 1.59)^2 + 120(4.75^3)/12(5.33) + 120(4.75)(4.75/2 - 1.59)^2/(5.33)$

$= 301 + 919. + 201 + 65.9$
$= 1487$ in⁴

$w_b = (15 + 33 + 51 + 26/(10))10 = 1016$ plf

$\Delta_b = 5(1016)(30^4)(1728)/384(29000)(1487)(1000) = 0.429''$

$f_b = 0.18\sqrt{g/\Delta} = 0.18\sqrt{386/0.429} = 5.40$ Hz

$d_e = 3.25 + 1.5 = 4.75''$

$D_s = 12 d_e^3/(12n) = 12(4.75^3)/12(5.33) = 20.11$ in⁴/ft

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$D_b = I_b/s = 1487$ in⁴$/(10') = 148.7$ in⁴/ft $\qquad s = 10.0'$

Effective Beam Panel width $\quad C_b = 2.0$

$B_b = C_b(D_s/D_b)^{1/4} L_b \qquad L_b$ = span length = 30'

$= 2.0(20.11/148.7)^{1/4}(30')$
$= 36.39'$

Weight of beam panel

$W_b = (w/s) B_b L_b = (1016\#/ft)/(10 ft)(36.4')(30')/1000$
$= 111^k$

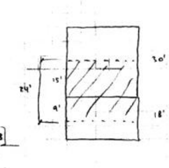
Girder Mode

Effective slab width

$0.4L_g = 0.4(24)(12)$
$= 115.''$ * controls

$30'(12) = 360''$

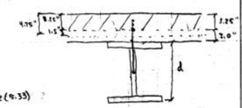
Depth concrete = $3.25 + (3.0/2)$
$= 4.75''$



$\bar{y} = \dfrac{16.20(23.6/2 + 3.0) - [115(3.25)(3.25/2)/5.33]}{16.20 + [115(3.25)/(5.33)]}$

$= \dfrac{239.8 - 114}{86.32}$

$= 1.46''$ below T/deck



$I_g = 1830 + 16.2(23.6/2 + 3 - 1.46)^2 + 115(3.25)^3/12(5.33)$
$+ 115(3.25)(3.25/2 + 1.46)^2/5.33$

$= 1830 + 2882.9 + 62 + 667$
$= 5442$ in⁴

$w_g = (1016/10)30 + 55 = 3103$ plf.

$\Delta_g = 5(3103)(30^4)(1728)/384(29000)(5442)(1000) = 0.358''$

$f_g = 0.18\sqrt{g/\Delta} = 0.18\sqrt{386/0.358} = 5.91$ Hz

$B_g = C_g(D_b/D_g)^{1/4} L_b$
$= 1.8(149/227)^{1/4}(30)$
$= 48.6'$

$D_b = 148.7$
$D_g = 5442/24 = 227$

$W_g = (w/s) B_b L_b = (3103/24)\,48.6\,(30)/1000 = 189^k$

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$\Delta_g' = L_g/B_b\,(\Delta_g)$

$= (30/36.39)(0.358'')$ → effectively stiffened (can reduce)
* girder span (30') is less than beam panel width (36')

$= 0.295''$

Total:

$f_n = 0.18\sqrt{g/(\Delta_b + \Delta_g')} = 0.18\sqrt{386/(0.429 + 0.295)} = 4.16$ Hz

↳ within range of human perception
∴ must check for walking excitation

$W = \dfrac{\Delta_b}{(\Delta_b + \Delta_g')} W_b + \dfrac{\Delta_g'}{(\Delta_b + \Delta_g')} W_g$

$= 0.429/(0.429 + 0.295) \times 111^k + 0.295/(0.429 + 0.295) \times 189^k$

$= 65.8^k + 77.0^k$
$= 143^k$

$\beta W = 0.05(143^k)$
$= 7.14^k$

$\dfrac{a_p}{g} = \dfrac{P_o e^{-0.35 f_n}}{\beta W} = \dfrac{65(e^{-0.35(4.16)})}{7140} = 0.0023 = 0.23\%g < 0.25\%g$ for operating room

* Concluded that existing floor system is satisfactory for walking excitation. Standard for comparison is an operating room. If floors meet this criteria, then they will meet all criteria for other hospital spaces.

→ Now must consider sensitive equipment (quasi static vibration)

$V = U_v \Delta_p / f_n \qquad$ where $U_v = \pi F_m f_o^2$

Design Assumption: weight of person walking = 185 lb
walking rate = 100 steps/min

$F_m/W = 1.7$ (Fig 6.4 AISC Design Guide 11)
$F_m = 315$ lb

$f_o = 1/t_o = 5.0$ (Fig 6.4 AISC Design Guide 11)

$U_v = \pi(315)(5^2) = 24740$ lb/s²

$\Delta_{ob} = L^3/46EI = 30^3(1728)/46(29000)(1487)(1000) = 1.13e^{-5}$ in/lb

$\Delta_{gp} = 30^3(1728)/46(29000)(5442)(1000) = 3.08e^{-6}$ in/lb

$\Delta_p = \dfrac{\Delta_{ob}}{N_{eff}} + \dfrac{\Delta_{gp}}{2} = \dfrac{1.13e^{-5}}{2} + \dfrac{3.08e^{-6}}{2} = 7.19e^{-6}$ in/lb

$V = U_v \Delta_p / f_n = 24740(7.19e^{-6}$ in/lb$)/4.16 = 0.04$ in/s
* 43000 μin/s × Does not meet sensitive criteria for operating room

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