Final Report

Roberts Pavilion
Camden, NJ

Andrew Voorhees | Structural

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Roberts Pavilion

Cooper University Hospital
Camden, NJ


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General Information

Function : Patient care center

Height : 10 Stories

New Construction : 320,000 GSF

Renovations : 51,000 GSF

Cost : $220 Million

Completed : December 2008

Delivery : Design-Bid-Build

Project Team

Owner :	Cooper University Hospital
Architect :	EwingCole
Structural :	EwingCole
MEP :	EwingCole
CM :	Turner / HSC
Civil :	Land Dimensions Engineering
Landscape Architecture:	Cairone & Kaupp, Inc.
Medical Equipment Planning :	Medequip International
Central Sterile Planning :	CT + Associates **LLC**
Elevators :	Zipf Associates, Inc.

Building Functions

- Intensive care units
- Clinical cardiology
- Operation suites
- Medical nursing units
- Clinical laboratories


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Structural Systems

Foundation : Slab on grade with pile caps and 16" diameter reinforced piles

Framing system : Steel frame using wide flange members with lightweight composite deck flooring

Lateral system : 8 ordinary concentrically braced frames, 4 in each direction


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Architecture

Designed to be a "healing garden," the interior spaces reflect a peaceful and relaxing atmosphere by incorporating an abundance of natural light, warm colors, and natural building materials such as stone, wood, and bamboo into the design. These themes are present in the lobby (shown left) and throughout the building.

The facade is composed of aluminum and glass panels. Renovations during construction updated the adjacent building facades to create a uniform appearance. The addition of the pavilion also serves as a link between the adjacent buildings by way of the lower floors.

Mechanical / Electrical

Mechanical : VAV system with 9 AHU's, 20,000 - 120,000 CFM
Three 750 ton Chillers

Electrical : 480/277 V 3-phase system
38 kV class switchgear
Two 2250 kW 13,200 V diesel generators (emergency power)

EXECUTIVE SUMMARY

The Roberts Pavilion is a patient care center located in Camden, NJ. It is part of the Cooper University Hospital and serves a large range of patient needs. Standing 10 stories above grade, it is a noticeable landmark when entering Camden. The pavilion was built between two existing hospital buildings and now serves to connect them. During construction, renovations updated the façades on the adjacent buildings to give a sense of uniformity to the complex. Aluminum and glass panels make up the main façade and provide patients with excellent views to the outside. Structurally, the building is framed in steel, with composite deck flooring. Lateral loads are resisted by four ordinary steel concentrically braced frames in each direction of the building.

Purpose and Scope

The following pages contain a detailed report on the Roberts Pavilion. An overview of the existing building is provided as part of this report. The second major portion is composed of a redesign of the building and the studies that were involved in that process. Originally the structure of the building was built out of steel. A choice was made to redesign the building with a reinforced concrete structure. This consisted of designing the gravity system as well as the lateral system.

The redesign was broken into two main portions, gravity and lateral systems. The gravity system was redesigned using a two way slab with drop panels. The lateral system was also adjusted. Braced frames were changed to shear walls and moment frames. To assist with lateral calculations, a computer model was created in ETABS. Both systems were also designed using hand calculations.

In addition to the main structural redesign, breadths in acoustics and construction were done. Acoustics were studied to find the impact of a concrete structure on building acoustics, as well as to study the noise levels in a typical patient room. The construction breadth was split into a cost analysis and a schedule analysis. Cost of the concrete structure was calculated and compared to the steel structure. To analyze the effect of a concrete structure on the project length, a schedule was created and compared to that of the steel structure.

It was determined in the end that a concrete system is feasible. However, it was shown that neither structure held a particular advantage over the other. A concrete structural system was able to be placed in roughly the same space as the steel structure, meaning very minimal changes to the architecture of the building, which was the primary concern. The first breath found that acoustically, the concrete structure performed better than the steel. The second breadth showed that the cost of the concrete system was found to be less than the steel. This was expected, but the cost was not as low as was previously thought. Finally, project length was increased, as would be expected with a concrete structure. Balancing the advantages with the disadvantages, it was decided that while a feasible alternative, a concrete structure offered no significant advantage over the existing steel structure.

Acknowledgements

This year would not have been possible without the following people:

I would like to thank EwingCole and the Cooper University Hospital for providing me with drawings and photos and allowing me to work on this project. In particular I would like to thank Brent Ellmann who was my contact and helped to answer my questions as well as worked to get permissions on my behalf.

Special thanks to the AE faculty. I am glad to have had the chance to learn from each of you. In particular, to my advisor Dr. Hanagan; thank you for all your help and encouragement.

Most of all I would like to thank my fellow classmates and friends:

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Dan Bodde

Sean Felton

Jon Fisher

Jon Gallis

Cheuk Tsang

It has been a blast to work with you guys. You all have kept me sane and made the late nights worth it. I will miss all of the conversations and the laughs that we have had. I could not have done this without you.

TABLE OF CONTENTS

BUILDING INTRODUCTION

The Roberts Pavilion, as shown in red in Figure 1, is a recently constructed patient care center at the Cooper University Hospital in Camden, New Jersey. Completed in December 2008, the project cost about $220 million. The pavilion is approximately 320,000 GSF and occupies 10 stories above grade as well as one basement level. Additionally, during construction, the adjacent Kelemen and Dorrance Buildings, shown in Figure 1 in blue and purple respectively, underwent 51,000 GSF of renovations.

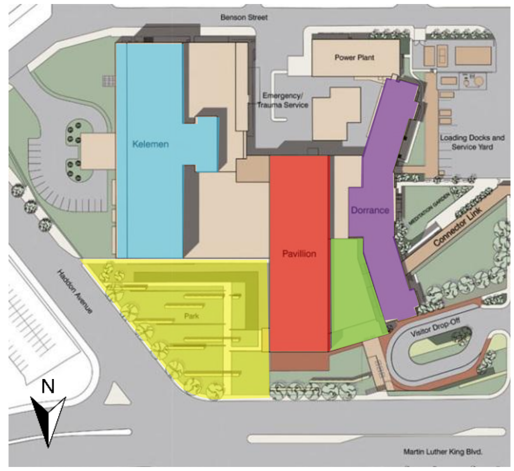

Figure 1: Site plan (courtesy of EwingCole)

Cooper has been a leading medical institution in southern New Jersey for many years. The Roberts Pavilion establishes Cooper's presence in Camden and upon entering the city, it is easily visible. Architecture and engineering systems were designed by EwingCole. They designed the façade, as shown in Figure 2, to be composed mostly of glass and aluminum panels. During renovations, façades of the adjacent buildings were updated to give the complex a sense of uniformity. The master plan also called for the demolition of the parking garage on the corner of Haddon Avenue and Martin Luther King Boulevard, as shown in yellow in Figure 1, and for the space to be turned into a park to improve the surrounding landscape.

Figure 2: Roberts Pavilion (courtesy of Halkin Photography, LLC)

The lobby, shown in green in Figures 1 and 3, is a grand, open space with an abundance of natural light and warm colors. It also acts as a link between the new pavilion and the existing Dorrance Building which is shown in purple in Figure 1. Bamboo plantings and natural materials give the space a garden-like feel. Cooper wanted the pavilion to feel like a "healing garden" where patients experience a calm and peaceful atmosphere seemingly distant from the city outside. This idea is evident in the design from the lobby to the upper floors.

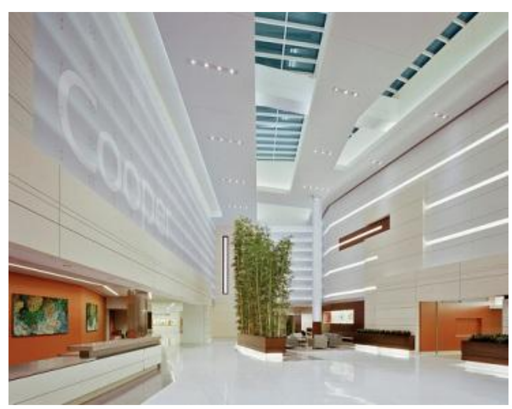

Figure 3: Lobby (courtesy of Eduard Hueber/Arch Photo, Inc.)

Each floor maintains a different function. The second floor houses clinical cardiology, while the third floor houses surgical suites, and the fourth and fifth floors hold the intensive care units. Typical patient rooms are located on floors six through ten.

STRUCTURAL OVERVIEW

Foundation

URS Corporation investigated the Roberts Pavilion site conditions by performing nine test borings. The top layer of soil in most of the drillings consisted of silty sand with some gravel and fragments of brick and concrete. This fill layer was classified as poorly to well-graded sand (SP-SW). Soil under the fill layer was classified as loose to dense silty sand with layers of clay becoming more firm with depth. 16" diameter reinforced piles were cast with a depth of -68' below the basement slab to reach firm soil. A minimum compressive strength of 4000 PSI concrete was used along with ASTM A615 Grade 60 reinforcement. Pile caps required concrete with minimum compressive strength of 5000 PSI and range in thickness from 3'-6" to 6'-0". The stratum layer under the footings was compacted to reach a bearing capacity of 4000 PSF.

The main basement will have an elevation of +8' above sea level (being about 5' above the water table), but elevator pits and mechanical space will be about +2' (1' below the water table). This means that the lower slab and walls will require waterproofing. Additionally these areas should be designed for hydrostatic uplift pressures. A permanent pump-operated subsurface drainage system was added to control the water level.

The main basement level is a 5" concrete slab, with a 16" slab poured in the north end under the mechanical room. Structural fill was placed for support under the foundations and used as backfill for the walls and footings. Soil pressures will need to be calculated when designing foundation walls.

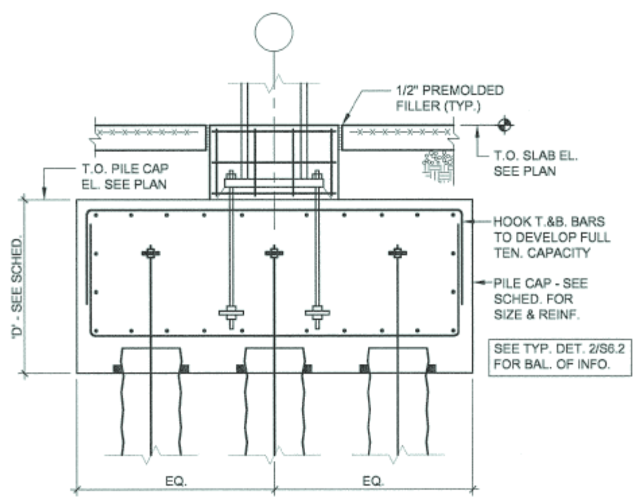

Figure 4: Typical pile cap

Floor System

Typical floor framing in the pavilion consists of a composite system. It incorporates a 2", 18-gauge steel deck with a 3¼" lightweight concrete topping reinforced with WWF (welded-wire-fabric). The Decking runs perpendicular to the beams and shear studs transfer the load to the beam to allow for composite behavior.

Framing System

All steel wide flange members in the building are A992 grade 50. Columns are typically spaced 30' on center in the North-South direction. In the East-West direction there are typically three bays; the interior span being 23', and the two exterior spans being 29'-6". Column spacing is shown in Figure 5. Column weights vary; with the heaviest being a W14x426. However, all columns are specified as W14s.

Beams on floors 4 - 10 are typically wide flange members W16x26 and W14x22 spaced at 10' (See Figure 5). Floors 1 (ground) - 3 have larger beams, being that they are supporting heavier equipment. The 3rd floor holds the operating suites and part of the trauma unit thus it supports larger dead and live loads than most of the floors. It uses mostly W21x44 beams spaced at 7'-6".

Figure 5: Typical column layout

Roof System

The roof of the pavilion supports mechanical equipment; specifically three cooling towers, an air cooled chiller, and three air handling units. It has two different levels, where the center level rises 3' above the main level to support the AHU's. Composite steel decking is also used on the roof, with the exception of the elevator core roof which is a poured slab. Wide flange members in the raised level are spaced at 6'-6" maximum to support the load from the mechanical units. In the south-west corner of the roof there is a small mechanical room with the roofing material being 1½", 20 gauge roof galvanized metal roof decking. All the mechanical systems on the roof are hidden by a 19' parapet.

Figure 6: Braced frame locations

Lateral System

The lateral resisting system in the pavilion consists of ordinary steel concentrically braced frames (OSCBF). There are four frames in each direction of the building as shown in Figure 6. Each frame extends through one full bay and through the full height of the building. Two typical frames are shown below in Figure 7. They consist of a variety of square HSS members with the most common being HSS10x10x1/2.

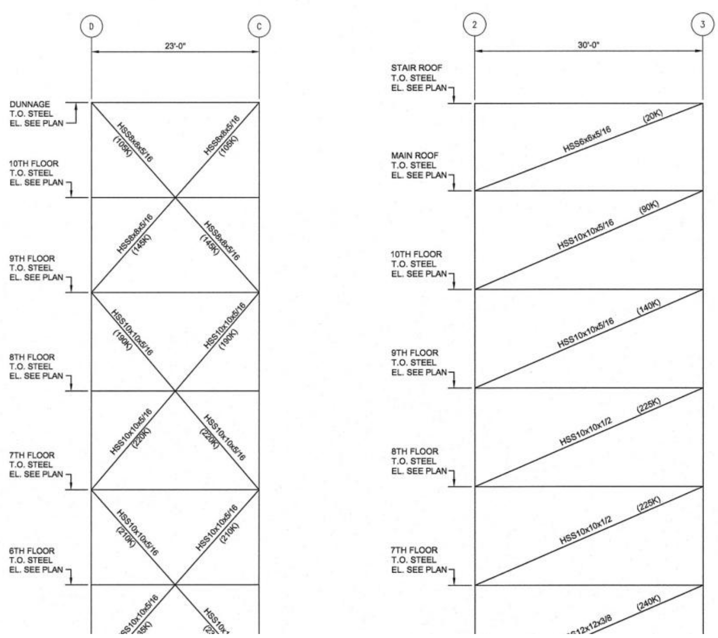

Figure 7: Typical braced frames

Design Codes

Below is a list of the codes and standards applicable to the design of the Roberts Pavilion as used by the design team. Codes that were utilized in this report for analysis are listed separately.

Codes Used In Original Design:

- IBC 2000 (New Jersey Edition)
- ASCE 7-02 (Minimum Design Load for Buildings and Other Structures)
- ACI 318-02 (Building Code Requirements for Structural Concrete)
- PCI (Manual for Structural Design of Architectural Precast Concrete)
- AISC 12th Edition (Manual of Steel Construction)
- AWS D1.1 (Structural Welding Code for Steel
- ASTM (American Society for Testing and Materials)

Codes Used In Analysis and Redesign:

- ASCE 7-05 (Minimum Design Load for Buildings and Other Structures)
- AISC 14th Edition (Manual of Steel Construction)
- ACI 318-11 (Building Code Requirements for Structural Concrete)

GRAVITY LOADS

Dead and Live Loads

Live load values were given on the structural drawings. These were similar to the values in ASCE 7-05 with the exception of several that aren't specified in the code. These values are denoted on the tables below with the value that was assumed. For spaces such as the operating rooms, that have a large difference between the code value and the value used for design, these calculations have used the value given in the drawings. This is because the live load may have been estimated larger because of specialized equipment, and it would be more conservative to use the larger value.

Dead loads are also shown below. An average value of 6.5 PSF for framing was calculated by summing the weight of framing on a given floor and dividing by the floor area. However, some floors are framed with larger members than the average floor, thus 10 PSF was estimated as the maximum value. Although the value is larger than average, it provides a more conservative analysis.

Live Loads (PSF)		
Occupancy or Use	As Designed	ASCE 7-05
Lobby/Public Areas	100	100
1st Floor Corridor	100	100
Corridors above 1st Floor	80	80
Patient Rooms + Partitions	40+20	40+20
O.R.	100	60
O.R. Core	125	*60
Medical Equipment Rooms	100	*100
Stairways	100	100
Mechanical Rooms	150	*150
Conference Rooms	100	*100
Kitchen	125	*125
Roof	30	20

*Assumed Value

Dead Loads (PSF)	
System	As Designed
Framing	*10
Superimposed	*10
MEP	*5
Composite Floor	42

*Assumed Value

Snow Loads

Snow loads were calculated using ASCE 7-05. The ground snow load was given in the code as 25 PSF. Calculations show that the maximum design value for snow drift is approximately 93 PSF (94 PSF given in the drawings). Values used to calculate the flat roof snow load are shown to the right.

Flat Roof Snow Load	
Variable	Value
P_g (PSF)	25
C_e	1
C_t	1
I	1.2
P_f (PSF)	24

LATERAL LOADS

Seismic Loads

Seismic loads were calculated based on ASCE 7-05 provisions. A major difference in the design of the building and the analysis is that the building was designed under ASCE 7-02. This difference was very evident in the response modification coefficient of the building, as well as ground acceleration factors. Shown below are different factors that are relevant to the seismic analysis calculations in this report.

Values for S_S, S_1, S_{DS}, and S_{D1} were determined via the USGS geo-hazards website. The values were then checked for accuracy by using the contour maps in ASCE 7-05 chapter 22.

After calculating the approximate fundamental period of the building, Cs was able to be determined. Then floor weights were totaled using an excel spreadsheet. Finally the base shear was able to be calculated. Forces were then distributed to each story level to find story forces and story shears. For simplicity, both roof level's masses were lumped together at the main roof level (h=133').

The base shear determined in this report's analysis was 1462 k while the base shear the building was designed for was 1300 k. This is approximately a 12% difference and was caused by the changes in code. Changes in the ground motion response maps affecting S_{D1} directly affected the value of C_S and by association, the base shear.

A computer model was not created for this stage of analysis. However, analyzing the building in a computer model would give different values of the fundamental frequency of the building. Since the code allows the use of the approximate period, the building's response to seismic activity is considered the same in all directions.

Seismic Design Values		
Factor/Parameter	Design	Analysis
R	5	3.25
C_d	4.5	3.25
Ω	2	2
I	1.5	1.5
Use Group	III	III
Design Category	C	C
Site Class	D	D
S_s	0.321	0.267
S_1	0.08	0.059
S_{DS}	0.3296	0.282
S_{D1}	0.128	0.095
Base Shear, V	1300	1462

Seismic Forces							
Level	Story Height, h_x (ft)	Story Weight, w_x (k)	$w_x h_x^k$	C_{vx}	Story Force, F_x (k)	Story Shear (k)	Overturning Moment (k-ft)
Ground	0	3237	0	0.00	0.00	1461.68	0.00
2nd	14	2563	52133	0.02	24.72	1461.68	346.14
3rd	28	2652	118994	0.04	56.43	1436.96	1580.12
4th	42	2725	194242	0.06	92.12	1380.52	3869.02
5th	55	2168	210239	0.07	99.71	1288.40	5483.84
6th	68	2106	260116	0.08	123.36	1188.70	8388.50
7th	81	2100	316751	0.10	150.22	1065.34	12167.77
8th	94	2100	375412	0.12	178.04	915.12	16735.69
9th	107	2100	435235	0.14	206.41	737.08	22085.93
10th	120	2100	496098	0.16	235.27	530.67	28233.00
Roof	133	2344	622862	0.20	295.39	295.39	39287.23
Sum		**26195**	**3082083**	**1.00**	**1461.68**		**138177.23**

*Table shows seismic force distribution per story height as well as overturning moment per level

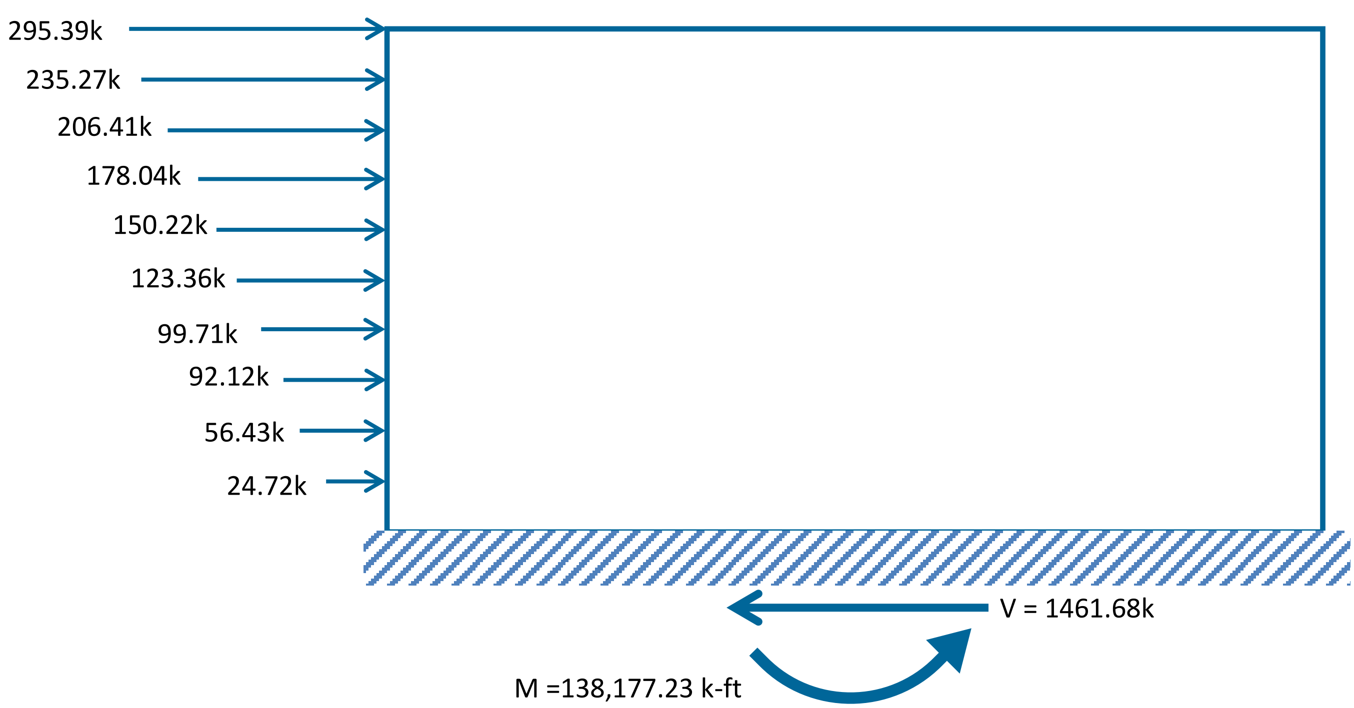

Figure 8: Seismic story forces

Wind Loads

Wind loads on the Main Wind-Force Resisting System (MWFRS) were calculated in accordance with ASCE 7-05. The code provisions call for the fundamental frequency to be calculated in order to determine if the building is flexible or not. From there, the gust factor can be determined. In order to determine the fundamental frequency, the code provides the approximation 75/H. This is more conservative than using the approximate frequency determined from $1/T_a$.

Calculations determined that the building was flexible; therefore the gust factor was determined by the procedure outlined in the code for a flexible building. Diagrams depicting the wind pressures on the building are shown on the next two pages. Also shown are the pressures for the roof. The values calculated were checked with those on the drawings and found to match.

Wind Pressure (PSF)						
			North-South		East-West	
Bldg Height	K_z	q_z	Windward	Leeward	Windward	Leeward
0-15	0.85	17.23	18.54	-11.34	17.36	-17.54
20	0.9	18.24	19.34	-11.34	18.08	-17.54
25	0.94	19.05	19.97	-11.34	18.66	-17.54
30	0.98	19.86	20.61	-11.34	19.24	-17.54
40	1.04	21.08	21.56	-11.34	20.11	-17.54
50	1.09	22.09	22.36	-11.34	20.84	-17.54
60	1.13	22.90	22.99	-11.34	21.42	-17.54
70	1.17	23.72	23.63	-11.34	22.00	-17.54
80	1.21	24.53	24.26	-11.34	22.58	-17.54
90	1.24	25.13	24.74	-11.34	23.01	-17.54
100	1.26	25.54	25.06	-11.34	23.30	-17.54
120	1.31	26.55	25.85	-11.34	24.03	-17.54
140	1.36	27.57	26.65	-11.34	24.75	-17.54
152	1.38	27.97	26.97	-11.34	25.04	-17.54

Roof			
East-West		North South	
Distance from edge	Suction	Distance from edge	Suction
0-76	-27.54	0-76	-40.67
76-152	-27.54	76-86	-24.23
152-285	-17.54		

Since the pavilion is not a perfect rectangular box on the first 4 floors, it was approximated as a rectangle with the dimensions 86'x285' which are the dimensions of the upper floors. Figure 10 shows the wind pressures in the North-South direction and Figure 9 shows the East-West direction.

It should be noted that for the wind analysis, the height of the building was taken as 152' which is the dimension to the top of the parapet. This is different from the seismic analysis which took the lumped roof mass at a height of 133'.

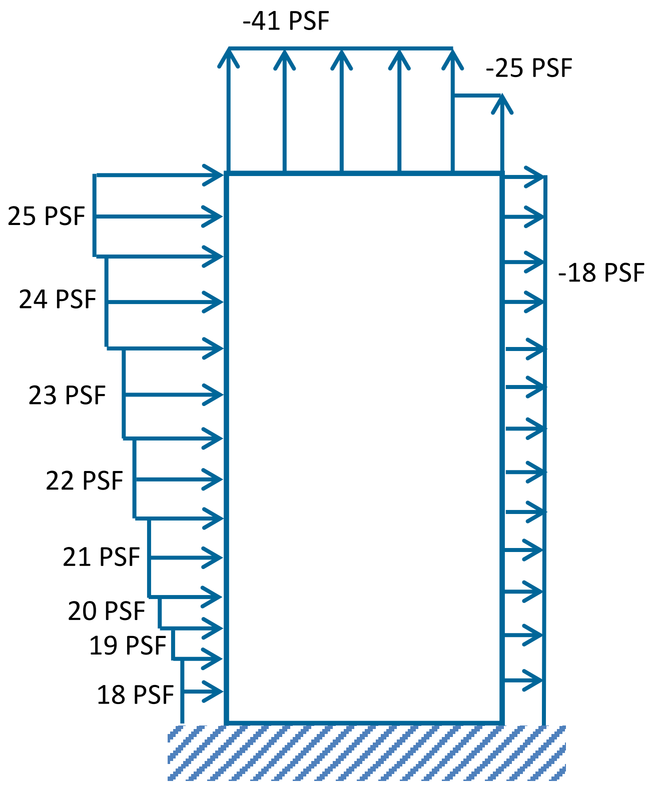

Figure 9: Wind loads E-W direction

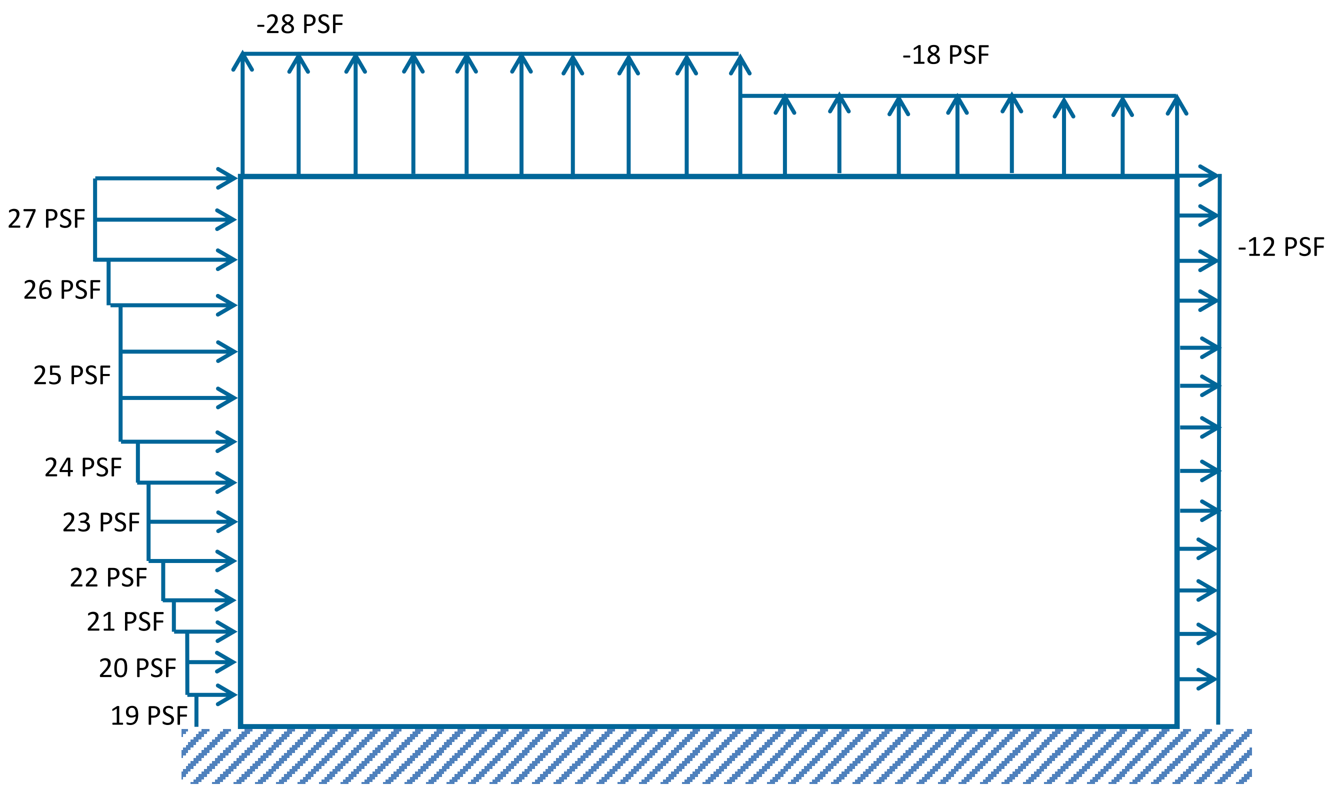

Figure 10: Wind loads N-S direction

PROBLEM STATEMENT

As previously discussed, the Robert's Pavilion is a steel framed building with composite deck flooring. This is a good system being lightweight and capable of supporting large spans. However, as far as cost is concerned, it was shown in Technical Report II that a concrete system may be more economical. The cost of materials, formwork, and labor for a concrete building may possibly be cheaper than a steel building. Another advantage of concrete construction is decreased floor-to-floor heights. This would make a large impact in the cost of piping and ductwork for the building if every floor was decreased slightly. Additionally, a concrete slab is also good at damping vibrations and controlling noise transmittance; two issues that are critical in a hospital.

Technical Report III addressed an additional issue with the steel structure in relation to the lateral system. The Robert's Pavilion was designed under the 2002 ASCE code. However, the loads determined via ASCE 7-05 in Technical Report III were larger. This difference was due to code changes and in no way suggests that the structure is unsafe; simply that the lateral forces changed. In fact, Technical report III determined that even with the increased forces, the lateral members still provided sufficient strength. However, a serviceability issue surfaced and it was shown that there were new issues with drift. Therefore it is proposed that with a new concrete gravity system, a new concrete lateral system also be designed.

PROPOSED SOLUTION

Structural Depth

With the intention of designing the most cost-efficient building, different concrete systems were studied in Technical Report II. In order to find the most economical design, the systems discussed in that report will be considered in relation to feasibility and cost. The most practical design will be determined from among a one-way slab with beams and a two-way flat slab with shear caps or drop panels as necessary. The most efficient systems will be chosen to use in the new structural design. The slabs in the building will then be designed and detailed for the given loads. Columns will also be sized and designed to be placed on existing column lines in order to avoid changing the architecture in any major way.

The lateral system will be redesigned to incorporate shear walls and concrete moment frames. Placement of the walls will coincide with the location of the current braced frames acting in the East-West direction. Current braced frames in the North-South direction are located at the exterior of the building, and placing a shear wall in the same location would result in the loss of windows in patient rooms or a major façade redesign. Therefore, the lateral system in the North-South direction will be redesigned to incorporate concrete moment frames. Beams will be added at the edges of the slab and will be considered along the two interior spans as well. In the event that these beams are not sufficient to resist lateral loads, return walls will be added in the core of the building to resist loads in the North-South direction. Shear walls in the center of the building may conflict with the architecture in ways such as wall thickness and placement of doors. These issues will be addressed as necessary and shear walls will be designed to include openings where required.

Floor systems, columns, and the lateral system will be designed by hand. Then a detailed model will be created in ETABS using the final design. Through the program, members will be checked for their required gravity loads and an analysis of the lateral systems will be completed as well. The figure below shows the proposed layout for the new lateral system. Shown in red are shear walls. These will be input at the same location of the current braced frames. It is possible that not all four shear walls will be required to resist lateral loads, in which case two or three will be used instead. Shown in blue are the proposed locations for beams that will create moment frames in the North-South direction. This design will be modified as more in depth calculations are completed.

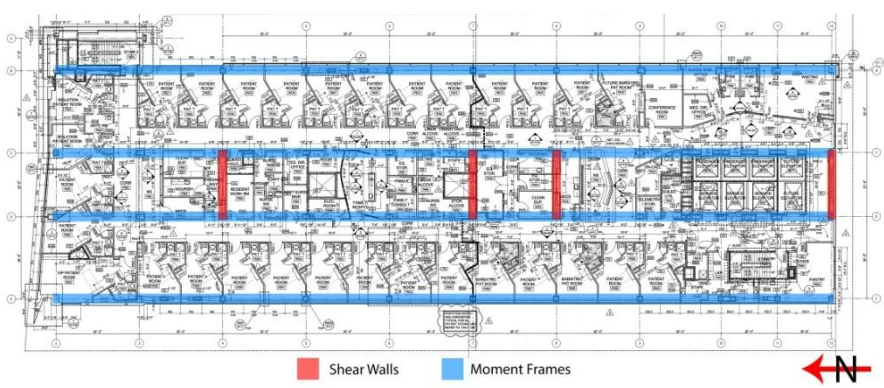

Figure 11: Proposed lateral system layout

Breadth 1: Acoustics

Changing the structure of the building from steel to concrete should result in better acoustical performance. As concrete is more massive, it blocks noise better. Therefore, a study of the sound transmission of the concrete structure will be compared to that of the steel structure. Particular attention will be given to the roof level. The roof holds mechanical equipment and it is very important that the slab is able to block the noise to the patient rooms below. Another patient room on a lower floor will also be studied. HVAC noise will be modeled for the space to study the acoustics. It is very important, especially in a hospital, that noise levels are controlled.

Breadth 2: Cost and Schedule

It is reasonable assumption that changing from steel to concrete will result in a less expensive construction cost. For this reason, a cost estimate of the concrete system will be completed. Using that estimate, the cost of the concrete system will be compared to that of the existing steel system to determine the feasibility of each. In addition, the impact on the construction schedule will be studied. Changing the structure to concrete will most likely impact the critical path and length of construction. These effects will be studied and compared to the steel structure. It can then be determined which system is more economical.

MAE Requirements

Graduate level work will be incorporated into this design work particularly from AE 530: Advanced Computer Modeling. The ETABS model will be very important for determining the building's reaction to both gravity and lateral loads. Additional work from AE 538: Earthquake Engineering, will be considered as well. This will be of particular use when designing the shear walls and moment frames for seismic loads.

STRUCTURAL DEPTH: GRAVITY SYSTEM DESIGN

Slab Design

The first step of designing the gravity system was to determine the type of floor system that would be used. A one-way slab with beams was considered along with a two-way slab with drop panels. A major plus of using a two-way slab was a smaller floor depth. Reducing this would cause a reduction in total building height providing a large cost savings. Additional cost could be saved by using a two-way slab because of formwork. The forming of beams would be more time consuming and expensive than forming a flab slab. Thus it was thought that the most cost effective concrete solution was a flat slab.

Floors 6 through 10 have a similar layout and experience the same loading. Therefore the sixth floor slab was chosen to design first. The floor is mostly composed of private patient rooms and requires a design live load of 80 psf. Based on ACI 318-11 table 9.5(c), a slab with drop panels and a 28’ clear span required a slab depth equal to L_n/36. Therefore a trial slab depth of 10” was chosen. Using this table also meant that deflection criteria were met. To meet the minimum requirement of L/6, the drop panels were sized as 10’x10’ squares. Required depth of the drop panels was 1.25h minimum and therefore, a 2.5” drop panel would be used to start.

Shear calculations were done next to determine if the drop panel thickness was adequate. Column dimensions were assumed at 24”x24", see following pages for column discussion. Calculations proved that the 2.5” depth was adequate for resisting two-way punching shear around the columns on upper floors where live loads were 80 psf. On the lower floors where live load is increased to 100 psf or greater, a drop panel depth of 5” or greater was used to control punching shear. See Appendix C for complete calculations.

Reinforcement for the slab was designed next. It was determined that the floor system met all the requirements to use the direct design method and therefore the slabs would be designed using this method. The statical moment M_o was found for each span based on bay spacing and floor loading. This moment was then divided into positive moment and negative moment and then distributed to column and middle strips based on DDM coefficients. Edge beams were assumed to be 18”x24” and were included in this analysis in order to assist in taking some of the moment. To save time, an excel spreadsheet was used to calculate moments based on input of span length, location (edge or interior span), and loading. This spreadsheet can be seen in Appendix F.

Once the moment was distributed, the required reinforcing in the slab was able to be determined. A spreadsheet in excel was programmed to determine top and bottom steel based on the moments input. This made it easier to achieve an output quickly when variables such as the span or loads were changed. The spreadsheet was cross checked with hand calculations for accuracy. Two typical spans are shown on the next page detailed with the required number of bars. In each strip the upper lines represent top steel and the lower lines represent bottom steel.

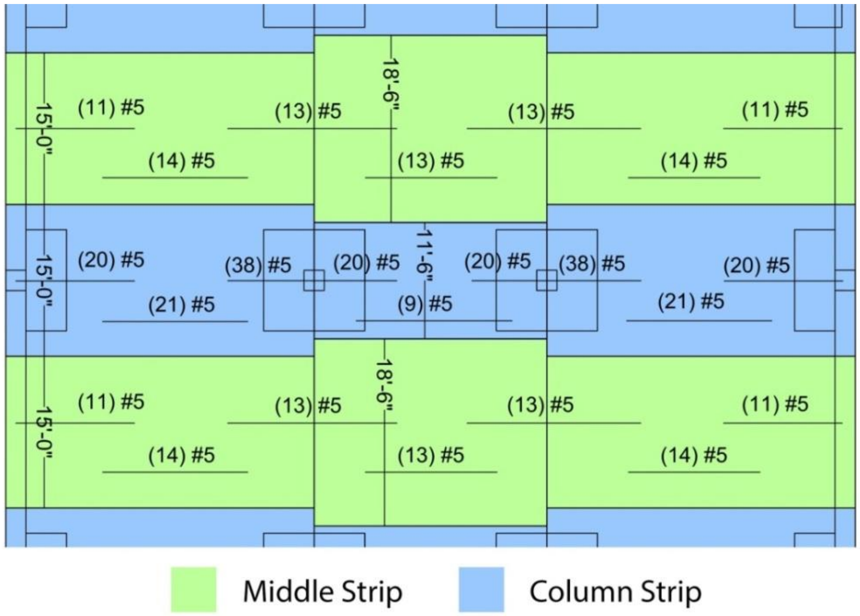

Figure 12: Required reinforcing E-W spans

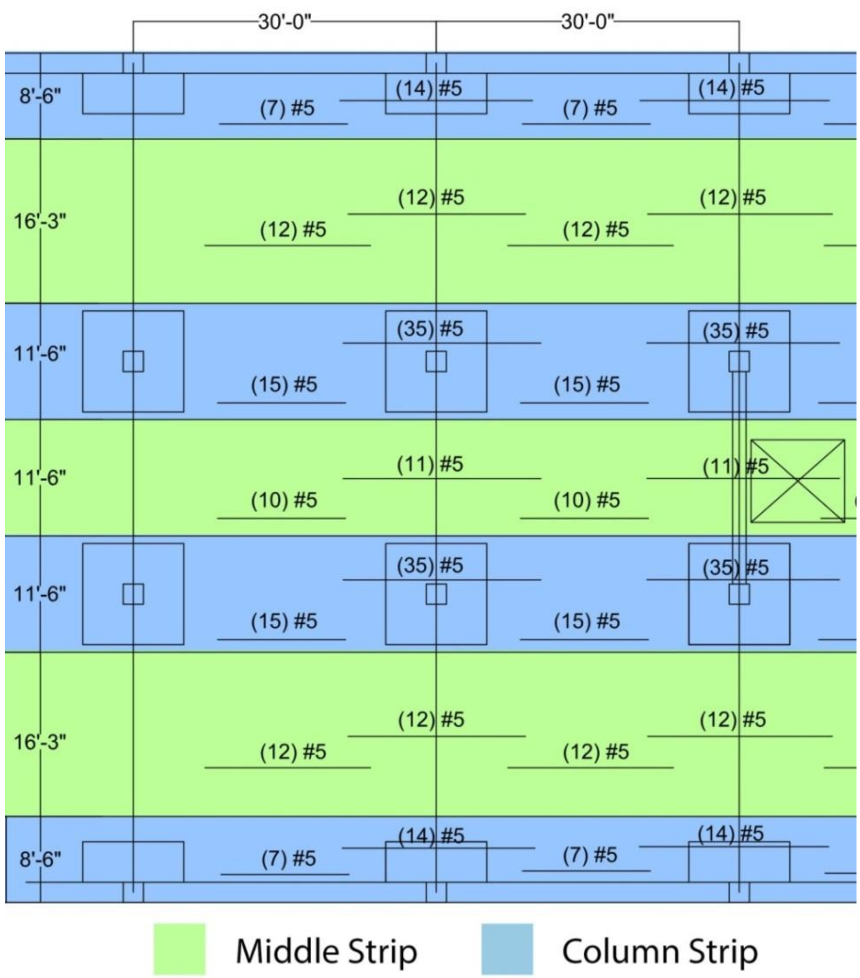

Figure 13: Required reinforcing N-S spans

Slab Openings

Hospitals often have a significant amount of floor penetrations due to HVAC systems. This is one reason a steel structure may be preferred to concrete; because in a steel building it is easier to create holes in the floor. Not including the stair wells and elevator core, the Roberts Pavilion is no exception to the amount of floor openings. Most of these are small, being about 5'x5' or smaller. However, on the upper floors there are two large openings for mechanical chases with dimensions about 11'x9'. These openings were too large to be ignored.

Typically, reinforcement around an opening is pulled to the sides and continued around the hole. As there is no live load or dead load being applied over the area of the opening, the existing reinforcement should be adequate to support the remaining load on the slab. One of the large openings on the sixth floor is shown below. The opening occurs between the column strip and the middle strip and therefore interrupts reinforcing in both of these. Negative reinforcement at the column strip required (20) #5 bars, and because part of the strip was interrupted, 14 of these bars were pulled to the side and continue past the hole. The remaining 6 bars will be continued to the edge of the opening. Positive reinforcement in the column strip was also pulled closer together. If this reinforcing were to become too congested, a beam would be recommended to be placed between the columns to hold the steel. On the middle strip side of the opening the reinforcement was also condensed into the remaining width. More details for the reinforcing around the opening can be seen in Appendix E.

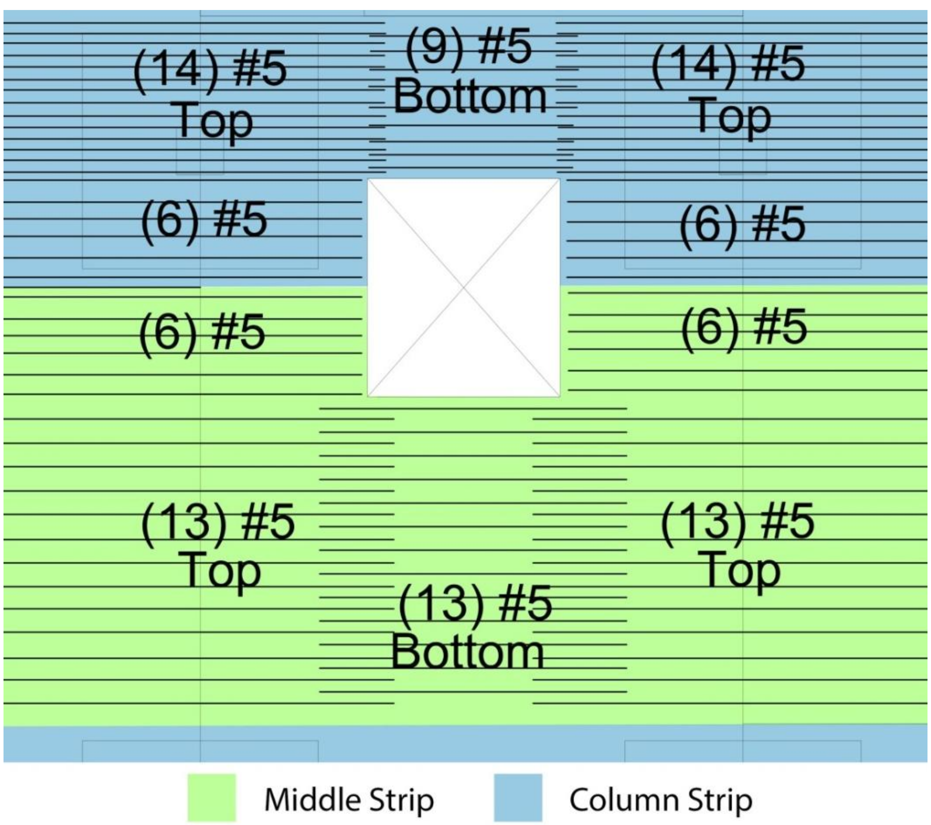

Figure 14: Reinforcement detailing around opening

Direct Design vs. Equivalent Frame

Although hand calculations were done using the direct design method, an analysis was also run in spSlab which uses the equivalent frame method. This was done to gain a more thorough understanding of the difference between the two methods and to verify that results were close. It was known that each method distributes moments differently, however the statical moment between the two methods should be the same. As a check for accuracy, spreadsheet calculations and output from spSlab were compared and verified to have the same statical moment.

As previously stated, the methods distribute moments differently. It was noted that the equivalent frame method distributed more moment to the exterior supports than the direct design method. This can be seen in the difference between required reinforcement. A comparison of the reinforcement for each method is shown below. It was found that required steel for each method was very close across the whole span, but that it was distributed differently to supports and edge beams. As the majority of calculations were done using the DDM, the final design used reinforcement determined from the DDM. However, this was a good exercise in discovering the difference between methods.

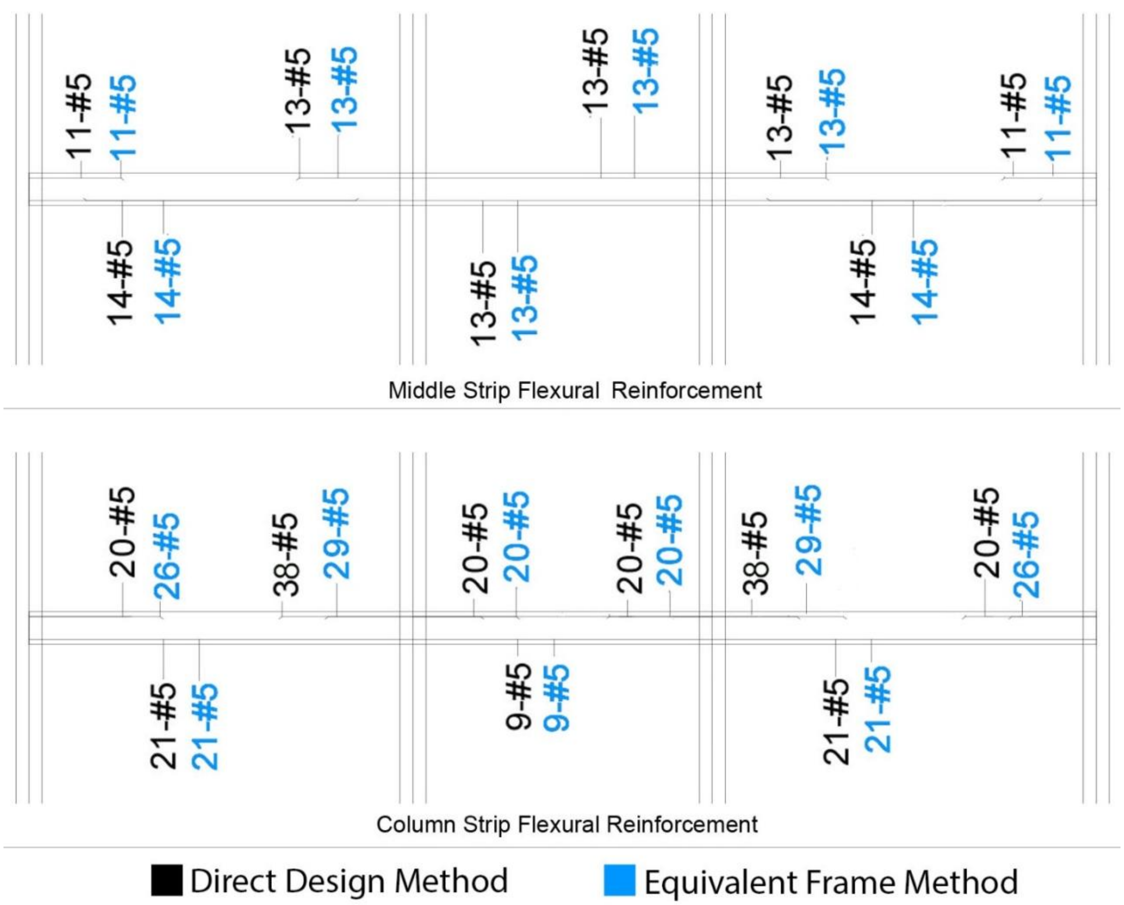

Figure 15: Reinforcement comparison - DDM vs. EFM

Columns

Columns were designed after slabs were finished. One of the benefits of using a concrete floor system is that the overall height between floors was decreased. The steel floor system required girders with a depth of 29". However, with the two-way slab, the depth was able to be decreased to 12.5". This meant a savings of approximately 16". This allowed for each story of the building to be decreased. To be conservative, because of unknown MEP requirements, each story height was reduced by 12", resulting in a total building decrease of 10'. Floor heights on the upper levels were reduced to 12' and the lower floor were reduced to 13'. This reduction will result in a large material's savings as well as cost savings in piping and duct lengths.

The first step in designing the concrete columns for the building was determining a base size. Steel columns in the building, after they are fireproofed and boxed out, occupy approximately 4 square feet. Therefore, in order to least disturb the existing layout of the building, a column size of 24" x 24" was chosen to work with. It was also decided to keep the column size the same throughout the building to allow for easier setting of formwork. Steel reinforcing will vary with story level as loading decreases.

Column loads were tabulated in excel. Live load reduction was taken into account where appropriate. It was found that interior columns' supporting the ground level experience axial loads of up to 2,220 kips. Using a compressive strength of 4000 psi, the reinforcing required in these columns would be approximately (10) #18's, with a reinforcing ratio of 6.94%. In order to decrease the amount of steel required, the cross section of the column could have been increased. However, the basement level of the pavilion has been fitted out to house laboratories and other functional spaces. Changing the cross section to a larger size would impact the architecture and was determined to be a last resort. An increase in the compressive strength of the concrete was therefore determined to be the best solution. Concrete with a compressive strength of 6000 psi was utilized and resulted in a lower steel ratio of 4.33% and the ability to use (16) #11 bars. Based on ETABS analyses, it was determined best to use 6000 psi concrete for columns on the basement level, ground, and second floor, so as to save in steel reinforcing.

Moments on the columns were found by using the unbalanced moment from the slab. Using these moments, the axial load, and design aids from the Wight & MacGregor concrete textbook, the required reinforcment ratio was estimated and in turn the number of bars and their sizes were chosen. Hand calculations found that reinforcing for the interior columns on the basement level was on the order of (16) # 11's, while edge columns could pass with (8) # 8's. Hand calculations for columns can be seen in Appendix G.

To verify design results, columns as deisgned were input into SpColumn and checked for adequacy. An additional check was done by comparing an ETABS analysis of the gravity loads which found axial loads to be within 5% of those calculated. Shear walls were included in the model and assumed to be load bearing. Results from sp Column and ETABS can be seen in Appendices H & J respectively. A 3D view and a floor plan of the ETABS model is shown on the following page.

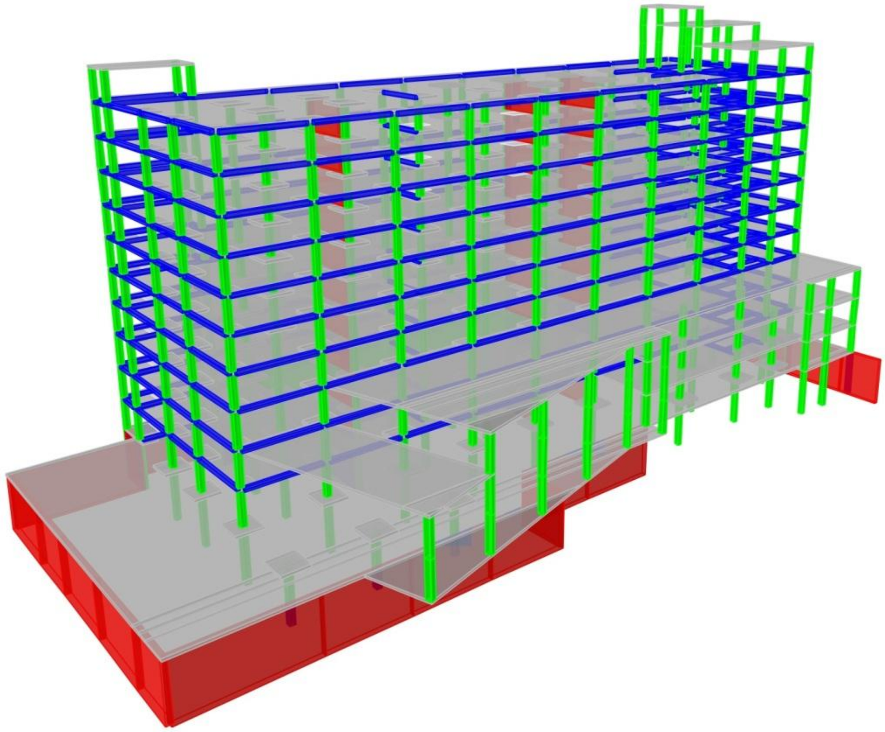

Figure 16: ETABS model

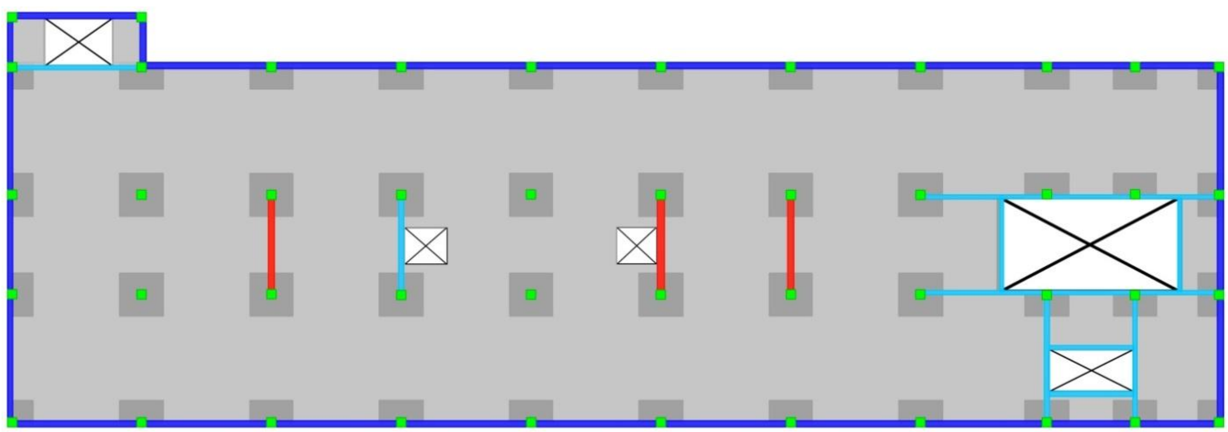

Figure 17: ETABS floor plan (Columns – green, Shear walls – red, Edge beams – dark blue, Interior Beams – light blue, Drop panels – dark gray)

Foundations

Increased building weight will increase the demand on foundations. The orininal geotechnical report calls for the maximum column loads of 2000 kips. Loads on the interior columns for the concrete building can reach about 2200 kips. Therefore, foundations would need to be addressed. They consist of 16" piles and pile caps. Drawings give pile capacity at 120 tons. The pile cap under column C-6 was chosen to check. It was determined that for the given load of 2200 kips, 9 piles would be required. This would mean a pile cap of 12'x12' to satisfy spacing dimensions. Original size was 8'x12'. Thickness was unchanged at 4'-6". Shear and flexure were checked and bars were increased from #9's to #10's. Calculations are shown in Appendix U.

LATERAL SYSTEM DESIGN

Wall and Frame Layout

Originally the lateral system in the building had been braced frames, shown in red in the figure below. As part of the structural depth, it was determined to change the lateral system to a concrete alternative. One of the largest concerns when designing the lateral system was the impact on the architecture. The East and West facades of the building are composed of a glass curtain wall the entire height of the building. Patient rooms run the entire length of the building on each side and impeding the room's view was simply not an option. Original braced frames were easily able to be boxed in and pass in front of the windows of several rooms.

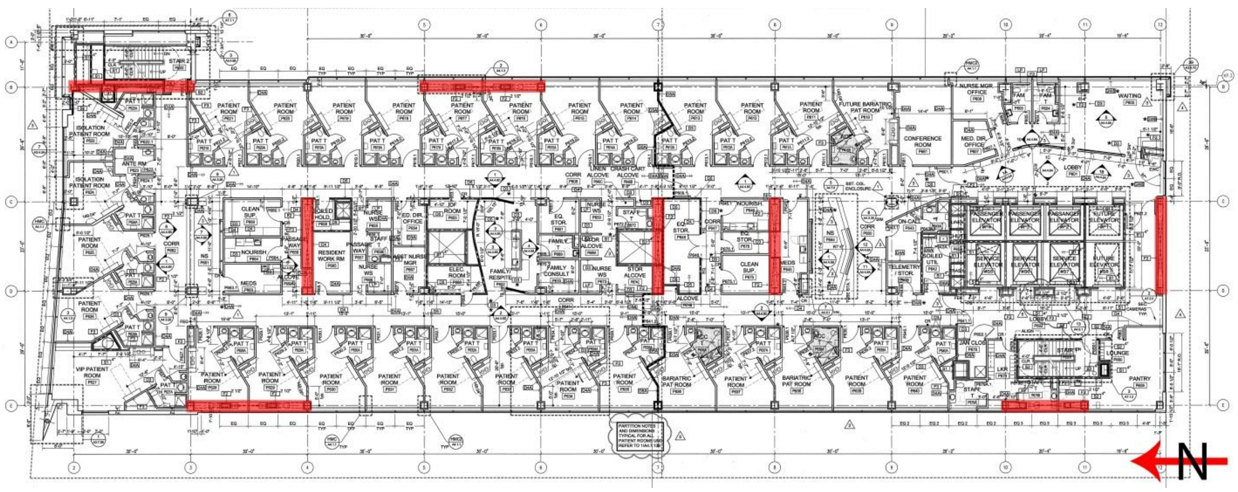

Figure 18: Braced frame layout

The first option considered for the redesign was concrete shear walls. These could be placed in the same locations as the braced frames in the E-W direction of the building. However there was no way to place shear walls in the braced frame locations in the N-S direction as they would be blocking views of the outside. Several options were considered for alternative placement, mainly placing the walls in the core of the building. Upon further observation, it proved to be difficult to find a suitable location. This was due to several factors. Floor layouts on lower levels did not allow for certain placement as the wall would be obstructing open spaces such as the lobby. The option of placing walls along the elevator or stairwell core was also considered. However, openings in the elevator core made it difficult to place a wall there, and it was difficult to transfer forces into any wall around the stair core. An additional disadvantage of placing walls along the southern stair core was that there were no columns to tie them into.

Based on these difficulties, a moment frame system was considered. Moment frames in the long direction of the building would work well because of the longer length. They are also a good decision because they don't impact the architectural layout of the building. Although moment frames may be more expensive, it was also thought that this would be a good learning opportunity to contrast the moment frame system with the shear walls. The preliminary proposed layout is shown on the next page.

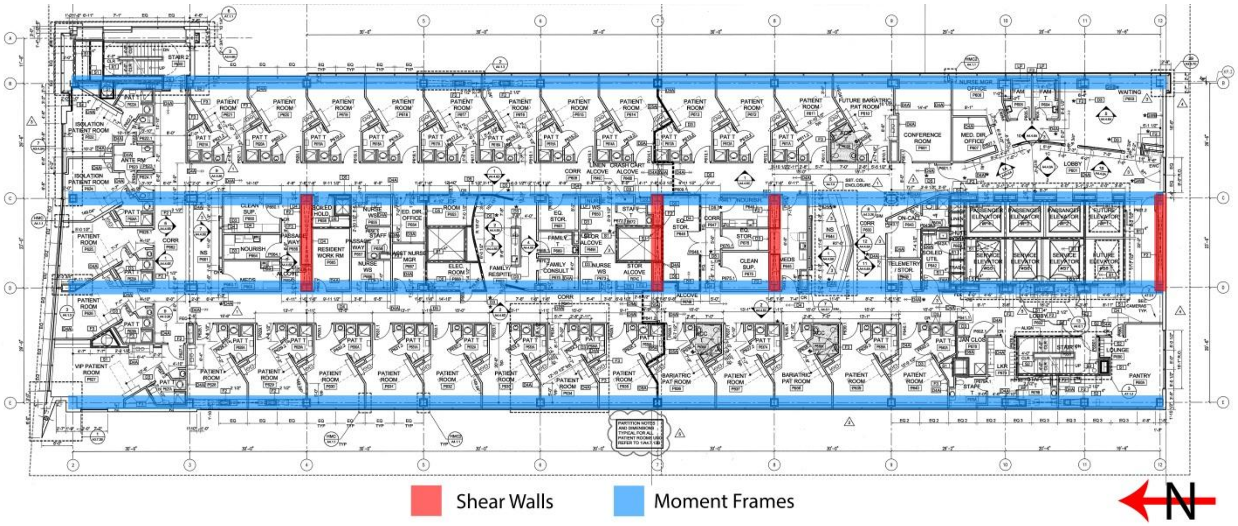

Figure 19: Proposed lateral system layout

ETABS Model

An ETABS model of the lateral system was created using the existing gravity model and making necessary changes. Upon changing the building to concrete, it was found that seismic loads now controlled the lateral system design due to increased building weight. Therefore, loads were input with the adjusted seismic values. Seismic loads were applied in each direction of the building, as well as with a 5% code required offset from the center of mass. Based on time constraints it was decided that wind loads would be calculated by ETABS. Wind parameters were input and all 12 load cases were checked for accuracy. For applied seismic and wind loads see Appendices K & N.

Modeling Considerations:

Based on ACI 318-11, property modifiers were assigned to members in the lateral model. Beam and slab stiffnesses were modified by 0.35 and 0.25 respectively. Columns were given a factor of 0.70 to the moment of inertia in both bending directions. Shear walls were assumed to be uncracked with a modifier of 0.70 to neglect out of plane stiffness. Bases of columns were assumed to be fixed. Rigid end offsets were used on beams with a rigid zone factor of 0.5 as is appropriate for concrete. Floors were modeled as shell elements with a rigid diaphragm constraint. Shear walls were meshed at 4'. Foundation walls were included on the basement level but were not meshed. To correct for a shear reversal at the ground floor, a flexible diaphragm was used on the ground level and the slab was meshed.

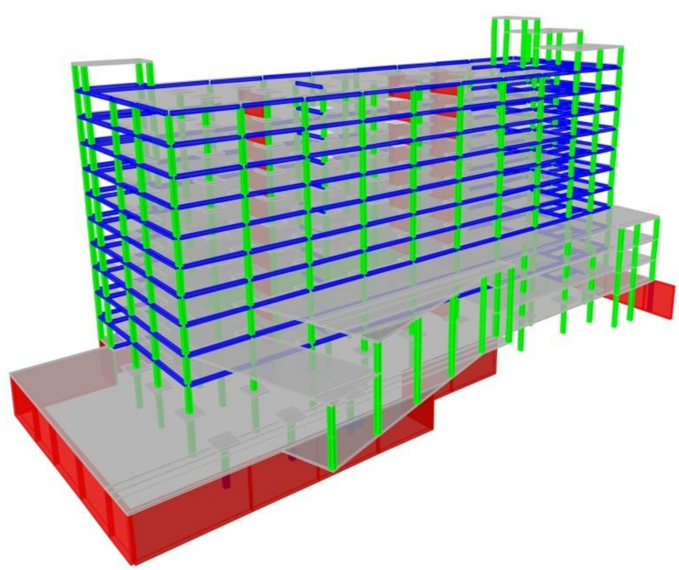

Figure 20: ETABS model

Walls were taken to be 18” thick and span between columns in the center bay being 23’ long. Columns were considered as boundary elements for the walls. Originally it was determined that 4 shear walls would be a good decision; dividing the base shear into approximately 780 kips per wall. However, after modeling the earthquake and wind loads in ETABS, it was determined that 3 walls would be better. Using 4 walls resulted with the center of rigidity towards the right side of the floor plan and created a large eccentricity from the center of mass, shown in Figures 21 & 22 below. When the earthquake load was applied at a negative 5% offset from the center of mass, it produced a large torsional shear in the wall at the north end of the building and the building was found to exhibit torsional irregularity.

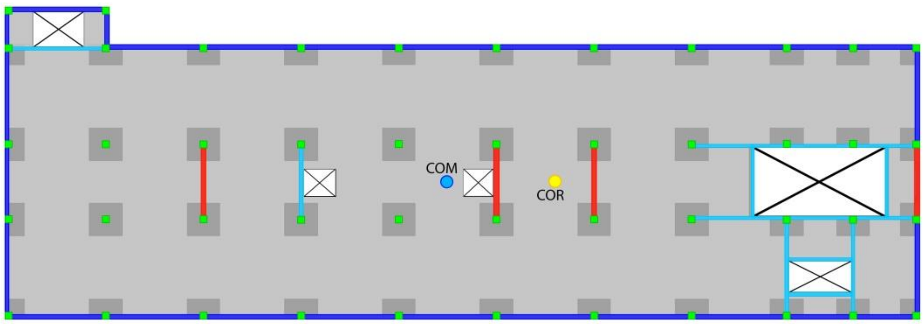

Figure 22: Center of rigidity for 4 wall layout

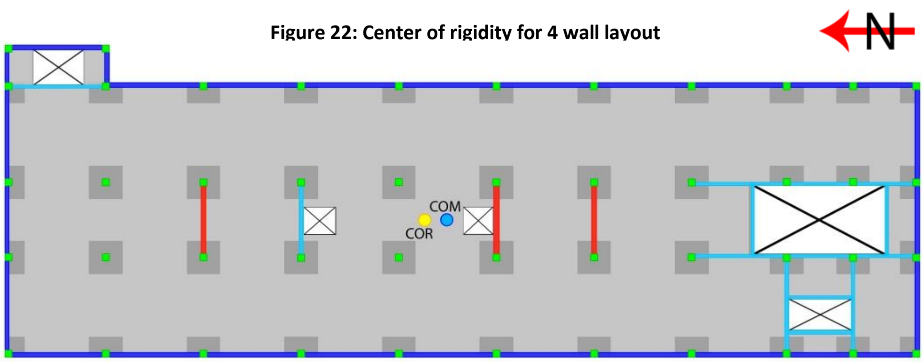

Figure 22: Center of rigidity for 3 wall layout

To lessen the torsional shear, the wall at the south end of the building was removed. This shifted the center of rigidity to the left and consequently, closer to the center of mass, shown in Figure 22 above. The eccentricity between the center of mass and the center of rigidity in the x-direction was now 4.7’. This greatly lessened the torsional effects on the building and it was found to no longer exhibit torsional irregularity.

The moment frames on the exterior of the building would consist of columns and edge beams, while the interior frames were composed of columns and the slab. Beams were sized as part of the gravity system with a trial size of 18” x 24”. The depth of these beams was picked to least impact the space between the drop ceiling and the slab. Width was chosen based on the column size being 24”. Forming the beams with the columns would be much easier and take less time if they are the same width. Preliminary sizes were input into the model and run to check deflections. It was found that seismic drifts were too large and thus it was decided that increasing the beam size would be the best option. Beams were then changed to 24”x24”. The model was run again, and drifts were found to be acceptable this time.

As was mentioned above, the model was used to check deflections based on preliminary member dimensions. It was found that displacements for the center of mass in the y-direction due to the earthquake loads were within the code accepted limits. An additional check of the stress in the concrete due to the given loads was found to be acceptable, therefore shear wall dimensions did not need to be changed. Seismic drifts in the x-direction were accepted after the change to 24” deep beams. This increased the stiffness of the frame and decreased the deflection. Wind displacements were checked at the edges of the building and compared with the limit of L/600. Every wind case passed the drift limits with the exception of a few lower floors due to case 3 and 4. The drift on these floors passed a limit on the order of L/490 or better. Drifts were therefore deemed acceptable. Deflection tables are shown in Appendices L & 0. The final design for the lateral system layout is shown in Figure 23 and 24 below.

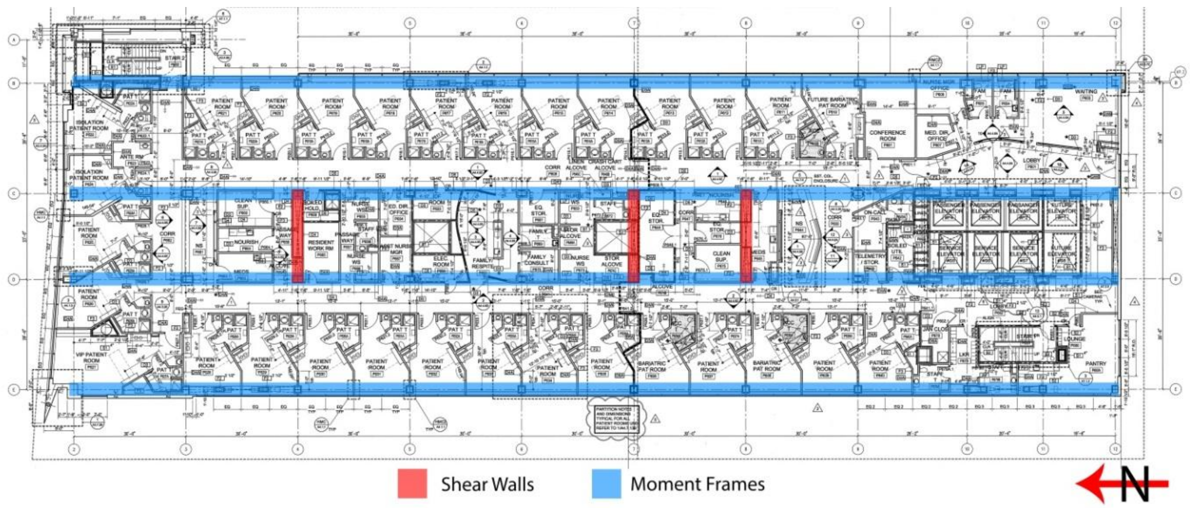

Figure 23: Final lateral system layout

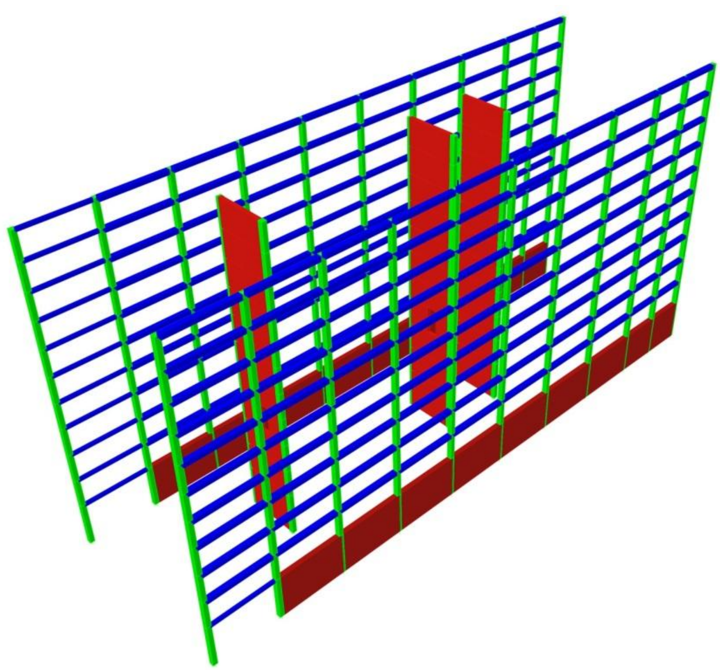

Figure 24: 3D view of ETABS lateral system

Shear Wall Design

After deflections were found to be acceptable, the beams and walls were detailed. This consisted of hand calculations to determine the required reinforcing. For the walls, shear was tabulated in ETABS and found to be reasonable in each wall. Shears in the walls at the ground floor were recorded for each load case and are shown below. The shear per story level in the walls does not add up to the total story shear calculated. This is because in reality, other frames in the same direction take some of the load. Wall 4 was chosen to be designed by hand as it experienced the worst case shear of 1038 kips. This was largely due to the fact that this wall was the farthest away from the center of rigidity and thus experienced the largest torsional shear.

Load Case	Shear (k)		
	Wall 4	Wall 7	Wall 8
EX	-5.4	-2.47	13.74
EY	-796.68	-754.69	-748.72
EX+EXT	-49.67	5.42	43.69
EX-EXT	38.94	-10.38	-16.26
EY+EYT	-555.72	**-797.97**	**-911.83**
EY-EYT	**-1037.63**	-711.4	-585.61
W	-7.57	1.59	7.71
W-2	-86.11	-90.28	-93.67
W-3	17.05	-2.09	-9.27
W-4	-28.41	4.48	20.83
W-5	-41.85	-70.99	-85.3
W-6	-87.32	-64.43	-55.2
W-7	183.15	201.33	213.82
W-8	-84.81	-64.27	-51.51
W-9	291.62	125.75	57.16
W-10	-16.66	176.52	263.85
W-11	90.48	-73.64	-142.01
W-12	-217.8	-22.86	64.68

The wall was designed for the basement level where it experiences the largest overturning moment in addition to the shear. Axial load was included from the floors above and helped to resist the overturning. As was previously stated, seismic loads controlled the lateral system. Reducing the dead load would cause less resistance to the overturning and thus the controlling load combination for the shear wall was $0.9D + 1.0E + 1.6H$.

It was found that to resist the tension and compression from the overturning moment, (14) #11 bars could be used. This is less than the required steel found from the gravity calculations for the interior ground floor column, and thus was thought reasonable. All the tension and compression steel will be placed in the columns acting as a boundary element. A detail of the wall reinforcing is shown below. Design calculations can be seen in Appendix P.

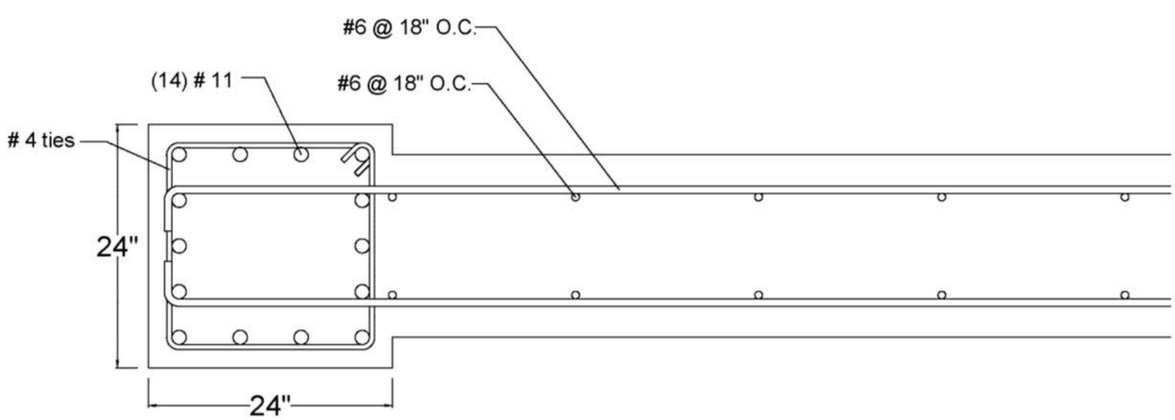

Figure 25: Shear wall detailing

Moment Frame Design

To design the moment frames, first the computer model was used to find the shear that each moment frame takes. It was speculated that the exterior frames would have a larger stiffness because of the edge beams. Interior frames have the slab area; however, the stiffness modifier of 0.25 as prescribed by code largely reduced this resistance. The ETABS model gave the shear in the exterior frame to be approximately 660 kips, while the interior bay was approximately 200 kips. The exterior frames each take about 38% of the load while the interior each take 12%. As this distribution was reasonable, the output was accepted to be used to design the beams. Shears in the frames at the ground floor are shown below.

Load Case	Shear (k)			
	Frame B	Frame C	Frame D	Frame E
EX	654.93	195.93	204.68	666.77
EY	-24.38	4.02	-4.66	13.48
EX+EXT	**663.06**	**196.94**	352.64	660.73
EX-EXT	646.79	194.91	**353.62**	**672.81**
EY+EYT	-68.26	-1.45	-1.51	46.06
EY-EYT	19.5	9.48	-6.76	-19.1
W	162.86	51.23	53.42	166.1
W-2	-5.34	0.31	-0.6	3.33
W-3	116.95	37.76	40.19	128.44
W-4	127.34	39.08	39.94	120.71
W-5	-9.19	-0.43	-0.32	6.36
W-6	1.19	0.89	-0.57	-1.37
W-7	132.99	37.78	41.29	117.82
W-8	316.88	99.6	102.96	326.63
W-9	68.63	24.43	31.72	111.64
W-10	131.02	32.29	30.27	65.25
W-11	206.68	70.83	78.01	268.38
W-12	269.07	78.69	76.57	221.99

A simple portal frame analysis was conducted by hand and found to yield results fairly close to the design moments from the program. These results were then used to design the reinforcement. Moments in the exterior frame were assumed to be directly resisted by the edge beams. Reinforcing would be placed in the beam, but continuous bars were necessary at the top and bottom of the beam because the moments are reversible. Moments in the interior frames were resisted by the slab and were distributed to the column and middle strips where they were added to the moments from the gravity analysis to find the combined required steel. When the lateral load was added, the load combination for slab moments was changed to *1.2D + 1.0E + L + 0.2S.*

Moments were distributed by the direct design method. After the required reinforcing was determined, it was shown that 3 extra bars were needed in the column strip in addition to those determined by gravity loads alone. Middle strip reinforcing remained unchanged. For detailed calculations see Appendix Q.

After lateral loads were applied, columns also needed to be adjusted. Adding the lateral load increased the moment and shear in the columns and they needed to be checked to verify that they were still adequate. Column B-7 was checked in spColumn for adequacy. It was found that with the additional moment, the column was not reinforced enough. The original reinforcing required by the unbalanced moment from the slab was (8) # 8's. Accounting for the additional lateral load added a moment in the perpendicular direction, loading the column bi-axially. For this loading state, the reinforcing was bumped up to require (12) # 9's. This was an increase in A_s of 5.68 in^2; a considerable change. Columns in the interior frames also required more steel but the moments were lower there.

BREADTH 1: ACOUSTICS STUDY

Tenth Floor Patient Room

One of the major differences between a concrete structure and a steel structure is the way it handles noise and vibrations. A concrete structure, because of its larger mass, is better as controlling vibrations as well as transmitting less noise than a steel system. As the Roberts Pavilion is a hospital, noise control is paramount and thus the purpose of this breadth was to study the noise transmission of each system. Two particular patient rooms were chosen to study. One room is on the tenth floor with mechanical equipment positioned on the roof above, and the second is an intensive care unit located on the fourth floor. These rooms were chosen because of the importance of sound isolation in each.

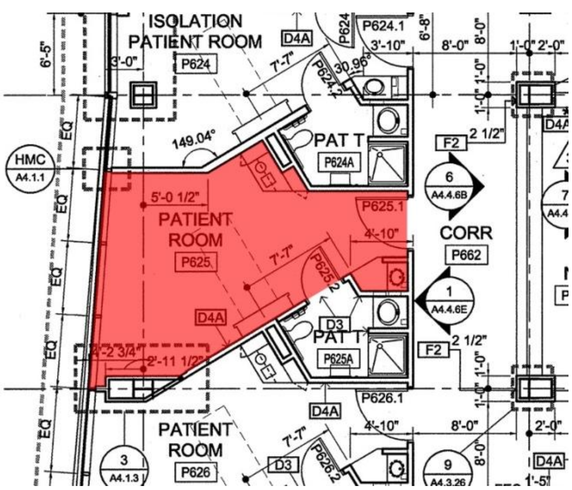

Figure 26: Patient room on 10th floor

The first space that was studied was the patient room on the tenth floor, shown highlighted in the figure to the left. Its location was critical because a process chiller is located directly above on the roof. Air handling units on the roof are located on a raised platform that allows for ducts to run from underneath the equipment, and therefore are less of a concern for noise transmission. Additionally, cooling towers on the roof are placed three feet above the roof, most likely to prevent vibrations in the roof and to allow for maintenance.

It was determined to find the amount of noise that was transmitted through the roof. First, the absorption of the room was found. This was done by adding up the absorption of all the materials in the room at each octave band frequency. The absorption coefficients of all the materials in the room are shown in the table below. These coefficients were then multiplied by the given material's surface area, and summed to give the room's absorption of sound energy as a whole, which was determined to be 211 sabins.

Patient Room 10th Floor								
			Absorption Coefficient (sabins)					
Material	Area (ft^2)	Area (m^2)	125	250	500	1000	2000	4000
Gyp WB	558	51.84	0.55	0.14	0.08	0.04	0.12	0.11
Glass	83.75	7.78	0.18	0.06	0.04	0.03	0.02	0.02
Doors	49	4.55	0.10	0.07	0.05	0.04	0.04	0.04
Sheet Vinyl Flooring	294	27.31	0.02	0.03	0.03	0.03	0.03	0.02
Ceiling Tile	294	27.31	0.76	0.93	0.83	0.99	0.99	0.94
Gyp WB	558	51.84	28.51	7.26	4.15	2.07	6.22	5.70
Glass	83.75	7.78	1.40	0.47	0.31	0.23	0.16	0.16
Doors	49	4.55	0.46	0.32	0.23	0.18	0.18	0.18
Sheet Vinyl Flooring	294	27.31	0.55	0.82	0.82	0.82	0.82	0.55
Ceiling Tile	294	27.31	20.76	25.40	22.67	27.04	27.04	25.67
					Total Absorption		211.14	

Patient Room 10th Floor																
Frequency (Hz)	**125**	**160**	**200**	**250**	**315**	**400**	**500**	**630**	**800**	**1000**	**1250**	**1600**	**2000**	**2500**	**3150**	**4000**
Chiller, L_1	85	-	-	87	-	-	87	-	-	90	-	-	98	-	-	91
Concrete TL	63	64	66	72	73	83	84	86	91	92	96	104	104	105	105	105
NR	62	63	65	71	72	82	83	85	90	91	95	103	103	104	104	104
L_2	23	-	-	16	-	-	4	-	-	0	-	-	0	-	-	0
Steel TL	41	47	50	52	53	53	**52**	62	67	71	72	75	75	76	77	78
NR	40	46	49	51	52	52	**51**	61	66	70	71	74	74	75	76	77
L_2	45	-	-	36	-	-	36	-	-	20	-	-	24	-	-	14
RC-30	45	-	-	40	-	-	35	-	-	30	-	-	25	-	-	20
NR Req	40	-	-	47	-	-	52	-	-	60	-	-	73	-	-	71
TL Req	**41**	**-**	**-**	**48**	**-**	**-**	**53**	**-**	**-**	**61**	**-**	**-**	**74**	**-**	**-**	**72**

Noise levels from the process chiller were taken from the textbook “Architectural Acoustics” by Egan. The levels varied between 85 and 98 dB over each frequency as shown in the table above. In order to compare the ability of the floor system to block the noise from the chiller in the patient room below, each system was matched with its closest equivalent floor system from the book “Architectural Acoustics” by Marshall Long. The steel floor system was modeled as a 5” thick concrete slab on metal decking (42 psf), a 16” airspace and acoustic ceiling tile (STC 60). This approximation seemed the most accurate because the actual floor’s weight was 42 psf and the airspace was about 16”. The concrete floor was modeled as a 6” slab, a layer of R-11 insulation and acoustical ceiling tile (STC 84). This was the closest approximation to the actual concrete system and as a 6” slab will show to provide adequate sound isolation, a 10” slab will be even better. Transmission loss values (in dB) for each system are shown in the table above.

The Ceilings & Interior Systems Construction Association reported that “the low-frequency noise often created by mechanical systems in hospitals can potentially be a source of annoyance and result in higher blood pressure and sleep disruption in patients.” This meant that it was imperative to control low-frequency transmittance. As seen in the table above, a steel system just meets the required NR value at the lower frequencies meaning a concrete floor is a “safer bet” when it comes to blocking low frequencies.

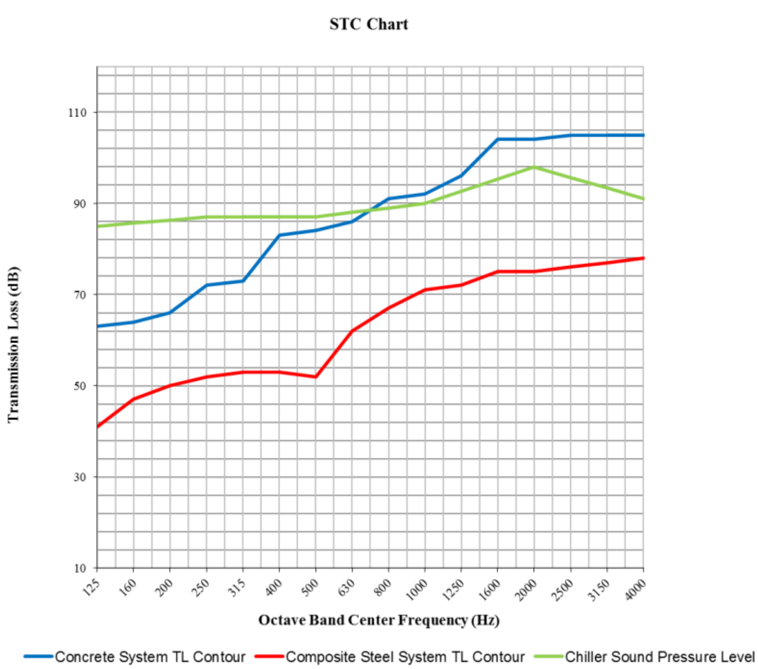

Figure 27: STC chart

Transmission loss values were plotted on a graph across the different octave bands. This graphical representation of sound transmission is shown in the figure to the right. The closer the line is to the chiller pressure level, shown in green, the less noise that is transmitted. Where the floor system line crosses the chiller pressure line, the floor system is able to completely block the noise. This is seen where the blue line passes above the green line between the 630 and 800 Hz octave bands. The STC graph also shows at a glance that the concrete system is a better sound isolator than the steel, as it is higher on the chart.

Noise levels in a hospital are recommended to meet NC 25-35 criteria. Therefore the target level was set at NC 30. In order to keep background noise levels below 30 dB the floor system needed to be able to reduce the noise level at each frequency to below the required level for NC 30 at each frequency. The required transmission loss values were determined and are shown in the table on the previous page. Both systems met the requirement with the exception of the steel at the 500 Hz frequency.

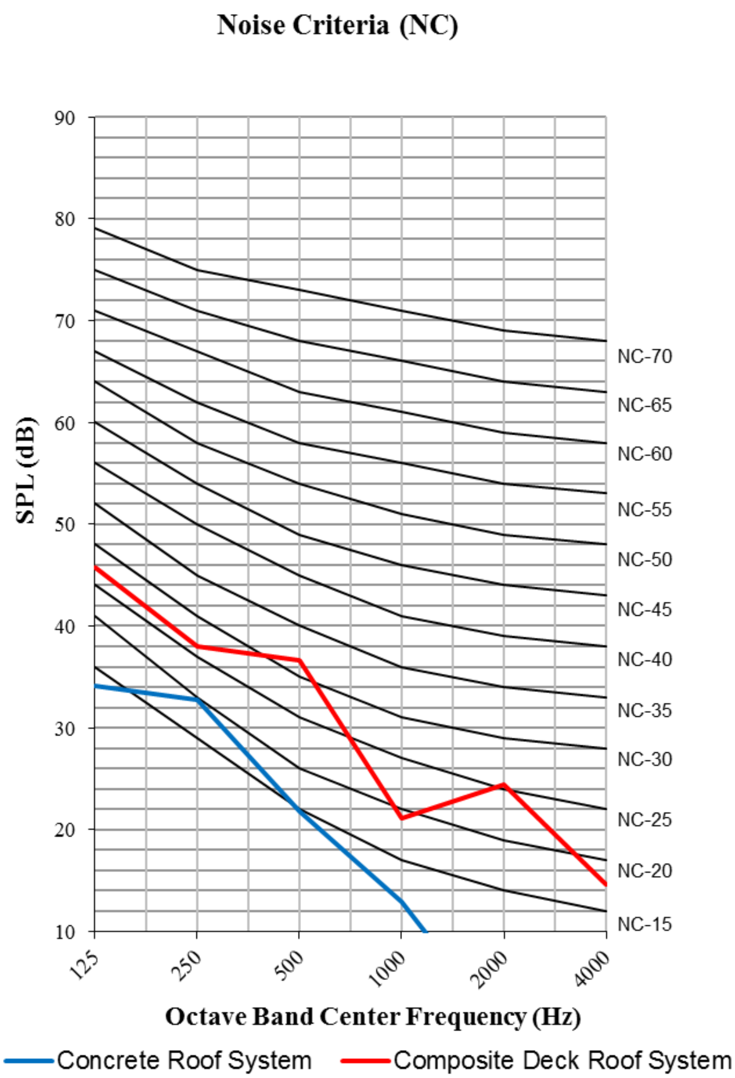

Figure 28: Noise Criteria graph

A noise criteria graph is shown to the left. The background noise considered was only composed of the noise transmitted through the floor from the chiller. This did not take into account any transmission from HVAC systems in the ceiling. See the next section of this breadth for calculations on diffuser noise. As shown in the graph the steel system just misses the NC 30 rating. The concrete system passes with a NC rating of 20. This is quite low. In a room with this rating it may be uncomfortable because of how quiet it would seem. However, these ratings should in reality be higher than noted because the background noise would also include any HVAC noise in the ceiling as well as noise from outside of the room. Without proper data these would be hard to predict, however the ratings noted by this simplified method were reasonable and thus were accepted as usable.

Fourth Floor Patient Room

The second part of this acoustics breath was studying noise effects on a lower floor. For this part, an intensive care unit was chosen to study on the 5th floor of the hospital. The room is shown in the figure to the right. This room was smaller than the room on the 10th floor and had a different layout. Absorption values were recorded and the total room absorption was found to be 192 sabins, as shown in the table below. Originally the floor system was going to be studied to find the transmittance of sound from HVAC systems to the floor above. However, as was shown in the previous room study, the concrete floor does a great job at blocking sound. Therefore, a study of the background noise in the room was completed.

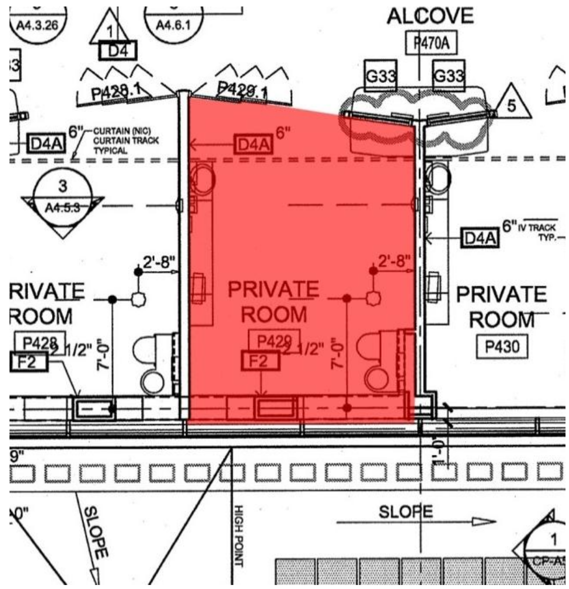

Figure 29: 4th floor patient room

Patient Room 4th Floor								
			Absorption Coefficient (sabins)					
Material	Area (ft^2)	Area (m^2)	125	250	500	1000	2000	4000
Gyp WB	433	40.23	0.55	0.14	0.08	0.04	0.12	0.11
Glass	157	14.59	0.18	0.06	0.04	0.03	0.02	0.02
Cabinets	48	4.46	0.10	0.07	0.05	0.04	0.04	0.04
Sheet Vinyl Flooring	276	25.64	0.02	0.03	0.03	0.03	0.03	0.02
Ceiling Tile	276	25.64	0.76	0.93	0.83	0.99	0.99	0.94
Gyp WB	433	40.23	22.12	5.63	3.22	1.61	4.83	4.42
Glass	157	14.59	2.63	0.88	0.58	0.44	0.29	0.29
Cabinets	48	4.46	0.45	0.31	0.22	0.18	0.18	0.18
Sheet Vinyl Flooring	276	25.64	0.51	0.77	0.77	0.77	0.77	0.51
Ceiling Tile	276	25.64	19.49	23.85	21.28	25.38	25.38	24.10
					Total Absorption		192.05	

Background noise in a hospital room can come from many different sources, such as HVAC systems and noise from an adjacent room or hallway. For this portion of the breadth study, the effect of air diffusers was studied. Diffuser locations and specifications were taken from mechanical drawings. In the fourth floor room, two slotted diffusers were located in the ceiling in front of the window. A list of manufacturers was taken from the specifications and a supplier, Kreuger, was chosen based on the availability of sound pressure level data for their diffusers. A complete specification for the chosen product can be seen in Appendix R.

Two diffusers were chosen based on the required air flow output of 350 cfm. One chosen had (2) 1” slots and the other had (2) 2” slots. The 1” slot diffuser was specified at NC 42, while the 2” slot was specified at NC 30. Ceiling transmission loss values were found next. As the air would be diffusing in a half-cylinder shape, the ceiling would absorb some of the sound. Losses due to air were also calculated using the equation:

$$L_r = L_s - \Delta L_{TL} + 10log(4cos\theta) + 10log\left[\frac{S_w Q}{16\pi\left[z + \sqrt{\frac{S_w Q}{4\pi}}\right]^2} + \frac{S_w}{R_r}\right]$$

Where ΔL_{TL} is the loss to the ceiling, L_s is the sound pressure level at the diffuser, S_w is the surface area of the diffuser, Q is the directivity coefficient of the sound, 2 for a cylinder, R_r is the room constant. Z is the distance from the source to the receiver which was taken as 10’ to a patient sitting in the bed. R_r was found to be 6114 sabins. Using these parameters the equation was solved for each octave band and is shown in the tables below. The NC rating was taken at the 500 Hz giving the 1” slot NC 41, and the 2” slot NC 28. Compare this to the manufacturer’s specifications and you get NC 42 and NC 30 respectively. This was close and thus calculations were considered correct.

Noise Transmission from Ceiling Diffusers (1" Slot Diffusers)						
Frequency (Hz)	**125**	**250**	**500**	**1000**	**2000**	**4000**
Diffuser SPL	64	62	56	52	48	41
Ceiling TL	7	9	13	17	23	27
Air TL	2	2	2	2	2	2
L at Reciever (dB)	55	51	41	33	23	12
					NC	**41**

Noise Transmission from Ceiling Diffusers (2" Slot Diffusers)						
Frequency (Hz)	**125**	**250**	**500**	**1000**	**2000**	**4000**
Diffuser SPL	53	51	44	32	25	13
Ceiling TL	7	9	13	17	23	27
Air TL	3	3	3	3	3	3
L at Reciever (dB)	43	39	28	12	0	0
					NC	**28**

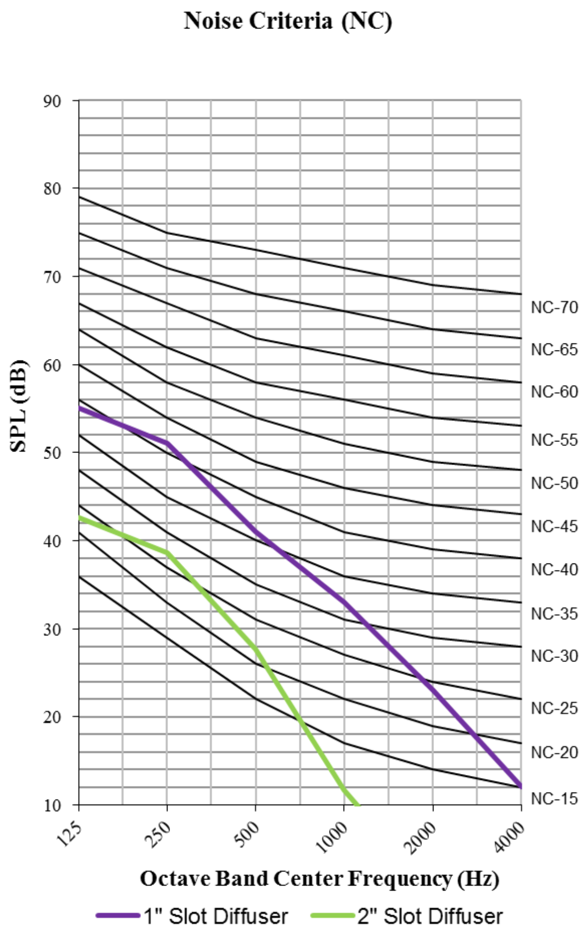

Figure 30: Noise Criteria Graph

A graphical representation of these values is shown to the left. From these outputs it was determined that to maintain a lower background noise level the 2" diffuser would be a better choice. It should be noted that this sound level is only taking into consideration the diffusers. Including other sources of background noise would raise these levels and possibly change the NC rating.

The room criteria was also calculated for each diffuser. Shown in the figure below, the 2" diffuser satisfies an RC value of 30. It is necessary to keep the diffuser noise from being too rumbly or hissy as is shown on the graph. With either diffuser, the RC value was chosen in order to keep it below the rumble or hiss line. Keeping these values low will maintain the desired level for a hospital room, and therefore a 2" diffuser would be a good choice.

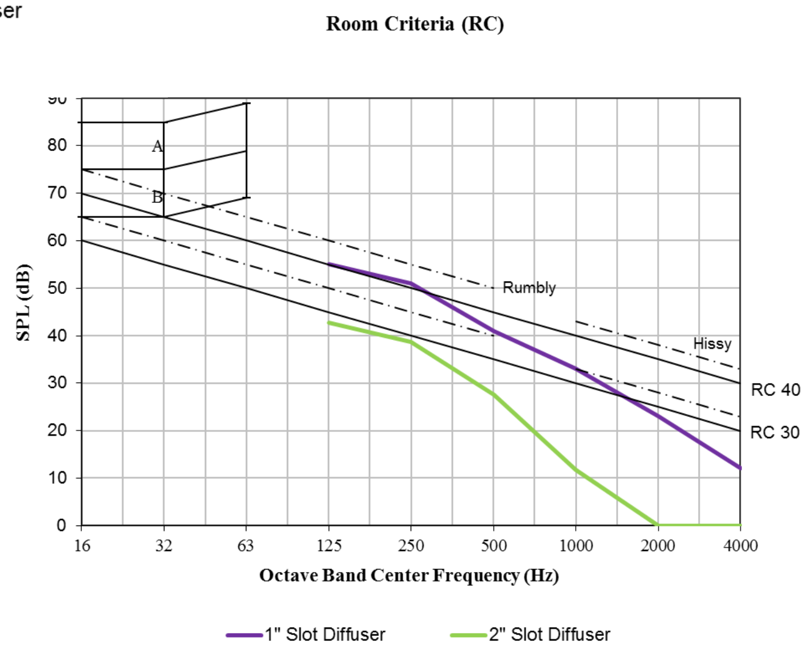

Figure 31: Room Criteria Chart

BREADTH 2: COST AND SCHEDULE ANALYSIS

Steel Estimate

It was determined in Technical Report II that a concrete structural system would possibly be cheaper than a steel structure. Therefore, this second breadth took an in depth look at the impact on the cost and schedule of a concrete structure.

The first step in completing this task was to create an estimate for the steel structure. A detailed estimate of the individual elements of the structure was done using RS Means 2012. This included steel deck, concrete topping, applied fireproofing, structural steel members, and concrete placing and finishing. The total cost of a steel structure was found to be $9 million. This is about 4% of the total project cost of $220 million. However, this is not uncommon for a hospital. It is probable that the total project cost was driven by specialized hospital MEP systems. Cost for steel members was found by tonnage and came out to approximately 67% of the cost of the structure. A cost breakdown is shown to the right. For detailed tables containing the full estimating process, see Appendix S.

Beams	$ 3,669,944.99
Columns	$ 2,054,205.00
Braces	$ 300,813.56
Fireproofing	$ 791,217.15
Steel Decking	$ 992,154.45
Conc Topping	$ 879,997.86
Placing Conc	$ 93,752.29
Finishing Conc	$ 247,319.66
Total	**$ 9,029,404.96**

Concrete Estimate

Construction of the concrete structure was broken down into five components: formwork, steel reinforcing, concrete mix, placing, and finishing. Formwork accounts for the largest percentage of the cost at approximately 56%. This was determined to be reasonable. The second most expensive component was the reinforcing steel at about 20% of the cost. Outputs from spSlab were used to find the approximate length of top and bottom reinforcing bars. Then reinforcement in the slab was totaled by the amount of steel in a typical bay multiplied by the number of bays on that floor. Column reinforcing was found using ETABS output. Required area of steel per column was averaged on each floor to find total reinforcing required. Wall reinforcing was based on the shear wall designed by hand. An additional 10% to account for waste was added into the total by a recommendation by RS Means. Detailed estimate calculations are shown in Appendix T.

Formwork	$ 4,684,331.89
Conc Vol	$ 1,254,047.49
Placing	$ 438,709.10
Finishing	$ 376,472.45
Reinf Steel	$ 1,653,306.02
Total	**$ 8,406,866.95**

Some considerations that were taken into account for the concrete estimate should be noted. First of all, placing costs were divided between the lower floors and the upper floors. It was estimated that floors seven and below would be able to take advantage of a concrete pump, however the upper floors were assumed to use crane and bucket. This was decided based on the assumption of a concrete boom pump with a vertical reach of about 100 ft. Also, columns between the basement and 3rd floor were assumed to be using 6000 psi concrete. See the gravity design section of the report for more information on column specifics.

Cost Comparison

After this breadth study, it was proven to be true that a concrete structure is cheaper than a steel structure. However, the difference is not as large as it was expected to be. The total cost of the steel structure was $9 million, while the concrete cost was $8.4 million. This represents about a 7% savings in cost. However, foundations were not included in these estimates. Adding the foundations to the estimates would increase the cost of each system, but more so for the concrete. The increased building weight of a concrete structure requires larger foundations. Thus the price difference between the two systems would be decreased.

Square footage costs not including foundations are approximately $28.22 – steel, and $26.27 – concrete. This was thought reasonable as the concrete cost should probably be in the $25-$27 range. Based on material availability in the Northeast region, construction is normally controlled by steel. For this reason, the cost of either system would be fairly close.

Steel Schedule

A schedule was obtained from EwingCole and was then input into Microsoft Project. The steel erection portion of the project took approximately 188 work days, and lasted from February 4th until October 24th. The schedule was mostly comprised of installing structural steel, decking, applying fireproofing, MEP rough in, and pouring slabs. Activities such as MEP rough in were found to have no impact on the length of the schedule as they were not predecessors for any activities. Part of the steel schedule is shown below. The main installation of structural steel was the determining factor in the length of the project. Each phase could only be started after the previous section was completed.

	Task Name	Durat	Start	Finish	Predecessors
504	Reinforcement Steel & Mesh Col's 8-12, Flr's 2nd-4th	10 days	Fri 5/4/07	Thu 5/17/07	503
505	Install Stair #1 - Basement to 4th Floor	20 days	Tue 4/17/07	Mon 5/14/07	498FS-5 days
506	Install Structural Steel (3) Columns 4-8, Floors Base-4th	15 days	Fri 3/30/07	Thu 4/19/07	495
507	Plumb Structure Columns 4-8, Floors Base-4th	5 days	Tue 4/17/07	Mon 4/23/07	506FF+2 days
508	Detail and Moment Con Col 4-8, Flr's Base-4th	12 days	Thu 4/19/07	Fri 5/4/07	507SS+2 days
509	Metal Decking and Studs Col 4-8, Flr's Base-4th	12 days	Tue 4/24/07	Wed 5/9/07	508SS+3 days
510	Perimeter and Opening Protection Cols 4-8, Flr's 1st-4th	5 days	Thu 5/3/07	Wed 5/9/07	509FF
511	HVAC Rough in Slab on Metal Deck Col's 4-8, Flr's 1st-3rd	8 days	Thu 5/10/07	Mon 5/21/07	510
512	Reinforcement Steel & Mesh Col's 4-8, Flr's 1st-3rc	10 days	Tue 5/22/07	Mon 6/4/07	511
513	Pour Stair Basement to Lower Mechanical Room	1 day	Thu 5/31/07	Thu 5/31/07	540,514SS
514	Pour Slab on Metal Deck 3rd Flr Column's 6-12 (Pour "G")	1 day	Thu 5/31/07	Thu 5/31/07	504SS+5 days,489
515	Pour Slab on Metal Deck 2nd Flr Column's 6-12 (Pour "H)	1 day	Fri 6/1/07	Fri 6/1/07	514,489
516	Pour Slab on Metal Deck 1st Flr Column's 6-12 (Pour "E")	1 day	Mon 6/4/07	Mon 6/4/07	515,489
517	Spray Applied Fire Proofing 1st Flr Frame Col's 7-12	3 days	Mon 6/18/07	Wed 6/20/07	516SS+10 days
518	Spray Applied Fire Proofing 2nd Flr Frame Col's 8-12	3 days	Fri 6/15/07	Tue 6/19/07	515SS+10 days
519	Pour Stair #1 Basement - 4th Floor	1 day	Thu 5/31/07	Thu 5/31/07	513SS,505
520	Install Structural Steel (3a) Column's 1-4 Flr's Base-4th	11 days	Fri 4/20/07	Fri 5/4/07	506
521	Plumb Structure Columns 1-4, Floors Base-4th	5 days	Wed 5/2/07	Tue 5/8/07	520FF+2 days
522	Detail and Moment Con Col 1-4, Flr's Base-4th	12 days	Fri 5/4/07	Mon 5/21/07	521SS+2 days
523	Metal Decking and Studs Col 1-4, Flr's Base-4th	12 days	Wed 5/9/07	Thu 5/24/07	522SS+3 days
524	Perimeter and Opening Protection Col's 1-4, Flr's	5 days	Fri 5/18/07	Thu 5/24/07	523FF

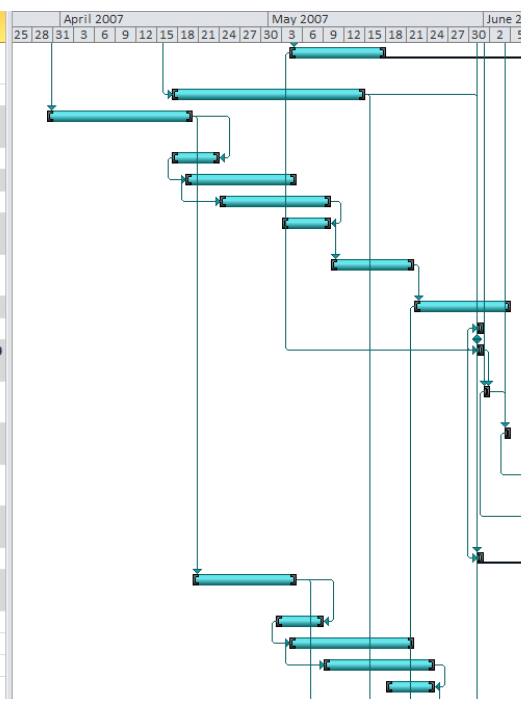

Figure 32: Steel schedule

Concrete Schedule

The concrete schedule created follows a similar path for each level of the building. Steel reinforcing for columns would be placed before forming the columns. After forms were set, the columns were poured and forming the slab above would commence. Next, slab reinforcing was placed and the slab was poured. Finally, perimeter and opening protection would be set up and finishing would occur.

The amount of time in days for each activity was found using the daily output values from RS Means. Some activities were shortened by allowing for multiple crews on the project at once. Rebar setting and formwork could be done with multiple teams at once. Length of concrete placing was determined using one crew based on the assumption of one pump for the lower levels and one crane for the upper levels.

Setting column and wall steel would take between 2 and 3 days, while column forms would take between 5 and 6. It was decided that forms should be started a day after placing the steel in order to avoid congestion on the site. Pouring columns would take about 2 days and was determined to start at such a time in order to finish a day after forms were placed. Forming the slabs would take an average of 9 days, and therefore was started a day after the columns started to be poured. Steel was to be set to finish a day after slab forms were finished being placed. Pouring the slab would start a day after steel began to be set. Then perimeter and opening protection could start to be set the day after the slab was poured. Finishing could not occur until reshores were removed, which was estimated at two weeks. These two weeks also impacted MEP rough in. However, this would cause no major delay in the project because there was a large enough time delay between rough in and any successive events. The whole procedure would start over again after the perimeter protection was in place, and the columns on the next level would be set. Part of the concrete schedule is shown below.

	Task Name	Dura	Start	Finish	Predecessors
498	Pour Slab, 2nd Floor	7 days	Wed 4/18/07	Thu 4/26/07	497SS+1 day
499	Perimeter and Opening Protection, Slab, 2nd Floor	2 days	Fri 4/27/07	Mon 4/30/07	498
500	Finish Columns and Walls, Ground Floor	5 days	Fri 5/11/07	Thu 5/17/07	498FS+10 days
501	Finish Slab, Ground Floor	5 days	Fri 5/11/07	Thu 5/17/07	498FS+10 days
502	Set Steel Reinforcing, Columns and Walls, 2nd Floor	4 days	Mon 4/30/07	Thu 5/3/07	499SS+1 day
503	Form Columns and Walls, 2nd Floor	6 days	Wed 5/2/07	Wed 5/9/07	502SS+2 days
504	Pour Columns and Walls, 2nd Floor	2 days	Wed 5/9/07	Thu 5/10/07	503FF+1 day
505	Form Slab, 3rd Floor	9 days	Thu 5/10/07	Tue 5/22/07	504SS+1 day
506	Set Steel Reinforcing, Slab, 3rd Floor	5 days	Thu 5/17/07	Wed 5/23/07	505FF+1 day
507	Pour Slab, 3rd Floor	7 days	Fri 5/18/07	Mon 5/28/07	506SS+1 day
508	Perimeter and Opening Protection, Slab, 3rd Floor	2 days	Tue 5/29/07	Wed 5/30/07	507
509	Finish Columns and Walls, 2nd Floor	5 days	Tue 6/12/07	Mon 6/18/07	507FS+10 days
510	Finish Slab, 2nd Floor	4 days	Tue 6/12/07	Fri 6/15/07	507FS+10 days
511	Set Steel Reinforcing, Columns and Walls, 3rd Floor	4 days	Wed 5/30/07	Mon 6/4/07	508SS+1 day
512	Form Columns and Walls, 3rd Floor	6 days	Fri 6/1/07	Fri 6/8/07	511SS+2 days

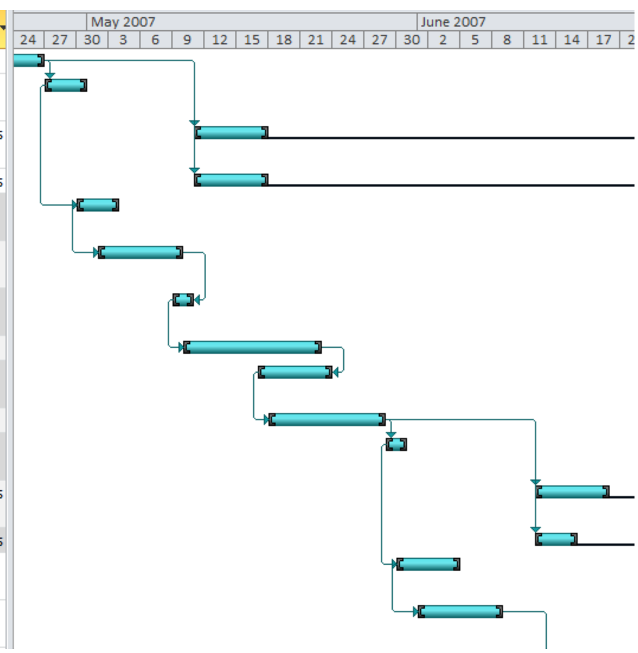

Figure 33: Concrete schedule

Schedule Comparison

Total construction length of the concrete structure was found to be 260 work days, or about 14.4 weeks longer than the steel construction length, which was about 188 days. This makes sense because a concrete structure normally takes longer to construct. With the concrete structure, fireproofing was made unnecessary, which removed about 80 days of work from the schedule. However, with the increased length of forming and placing steel, this savings was inconsequential.

An issue that may arise from this increase in schedule length is placing concrete in the winter. The structure would be started in February of 2007 and finished in February of 2008. Precautions must be taken to ensure that the concrete cures correctly. Admixtures may be considered to help with the temperature. Tarps and heaters may be necessary to keep the concrete from freezing. Additional lighting may be necessary as well because of the shorter days in winter and costs for electricity could add up. Snow must also be kept off of the formwork and slabs. These issues all present a supportive position for why steel is a better choice because of its “quick” construction time.

If the concrete system were much cheaper than the steel, an increased schedule length may be worthwhile. However, with such a competitive steel cost, the schedule increase would be a downside. This may outweigh the cost savings. As with either system there are positives and negatives.

COMPARISON – STEEL VS. CONCRETE

A concrete structure has benefits as well as drawbacks. Among one of its advantages, is the cost. The concrete structure was found to be cheaper than the steel, although not by as much as had previously been thought. The cost difference was not as large as would be hoped if changing to concrete for cost savings. Also considering that the location of this project is Camden, NJ, may change the price. The Northeast is a primarily steel controlled region. This means that in reality because of availability of materials, a steel building may be more economical.

Considering why the original building was composed of steel could have had something to do with the location factor, although another possibility is that hospitals have a lot of floor penetrations. Creating an opening in a steel floor system is much easier than in a concrete system. This may have led to a one way slab with beams being a good alternative. However for simplicity, a two way slab is better because it requires less labor having no beams to form.

Acoustically, it was shown that a concrete floor is better than a steel floor. The mass of a concrete floor system blocks noise more efficiently than a steel floor. This is a big issue in hospitals. Noise levels affect how quickly patients recover and the comfort level during their stay. Concrete also provides superior vibration control. For these reasons, a concrete system is recommended over a steel system.

Finally, the schedule impact of each system was studied. It was found that the length of construction of the concrete was much longer than that of the steel. If schedule was of no consequence, then a concrete structure wouldn’t be an issue. However, a schedule increase will most likely increase the cost of the project.

Based on all these considerations, it was thought that although a concrete structure is perfectly feasible, it may not be the best choice based on this project type and location. The benefits of acoustic performance are outweighed by the schedule increase. As cost was so close to that of the steel, the schedule impact would probably be the determining factor. Thus a steel structure is probably the most efficient and cost effective choice for the Roberts Pavilion.

CONCLUSION

This report consisted of an analysis of the Roberts Pavilion. An analysis of the existing steel structure was done in the fall semester. Having knowledge of the gravity and lateral systems, a judgment was made to redesign the structure out of reinforced concrete. The gravity system was redesigned using a two-way slab with drop panels. Slabs were designed by the direct design method, although a comparison was made between this method and the equivalent frame procedure. Moments were calculated and reinforcement was determined. After slabs were designed, columns were designed. Loads were summed and columns on the ground floor were designed and detailed. An analysis in spColumn was used to verify results. An additional check of the foundations was completed as part of the gravity loads section.

The next major part of this report contained the lateral system redesign. Shear walls and moment frames were used to resist lateral loads. After determining the location of these elements, hand calculations determined the required reinforcing. A computer model was created in ETABS to assist with modeling the building's behavior in wind and seismic loading. Drifts were checked with code acceptable values and found to pass for strength and serviceability.

A breadth in acoustics was done to assess the capability of the concrete structure to block noise. It was found that the concrete system did a much better job at blocking mechanical noise to the patient room than the steel. An additional room was modeled for background noise from the mechanical equipment in the ceiling. Levels were found to be acceptable and within the recommended requirements for a hospital.

The second breadth dealt with schedule length and cost of the structure. Estimates found that the concrete building would be slightly cheaper than the steel building. This was due to cheaper material costs. However, the difference was less than was expected at about 7% less. A schedule analysis was also completed and it was found to take 14 weeks longer to construct the concrete building. This was a large increase.

The benefits of a concrete system include good acoustic performance, better drift control, and lower cost. However, because the cost is so close to that of the steel, it is likely that the schedule increase would likely lead to the steel building to be a better choice. Concrete is of course a feasible alternative if the schedule is not an issue. Either way, both systems have their strengths and should be considered as equal options.