Final Report

Roberts Pavilion
Camden, NJ

Andrew Voorhees | Structural

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Faculty Advisor: Dr. Linda Hanagan

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)

Andrew Voorhees | Structural Option   www.engr.psu.edu/ae/thesis/portfolios/2013/apv5017/

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:

Sarah Bednarcik

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.

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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.

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.

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.

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.
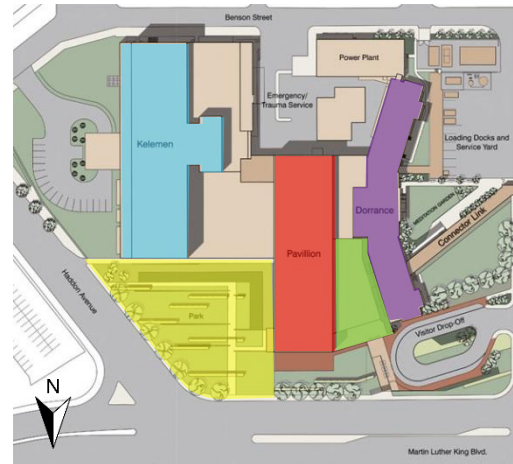
Figure 1: Site plan (courtesy of EwingCole)

Figure 2: Roberts Pavilion (courtesy of Halkin Photography, LLC)
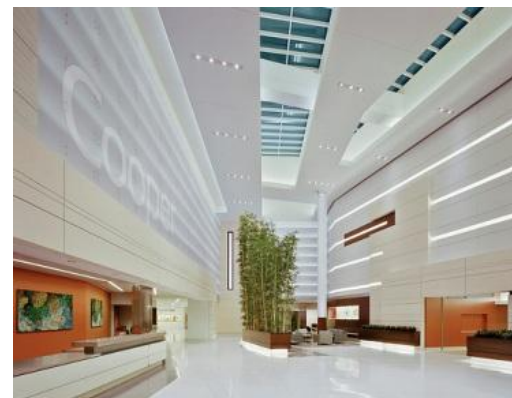
Figure 3: Lobby (courtesy of Eduard Hueber/Arch Photo, Inc.)

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.
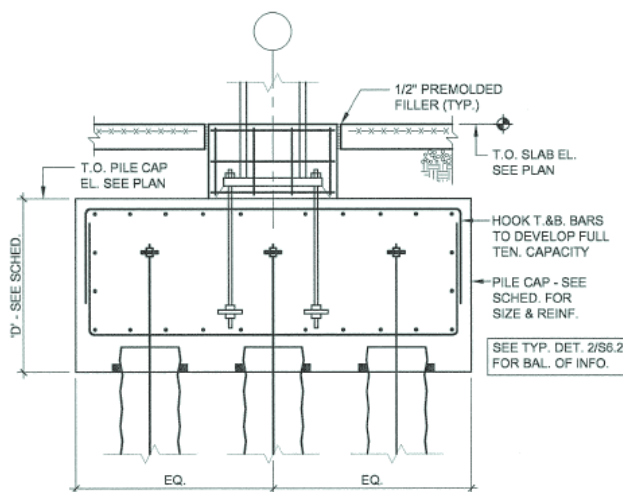
Figure 4: Typical pile cap

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.
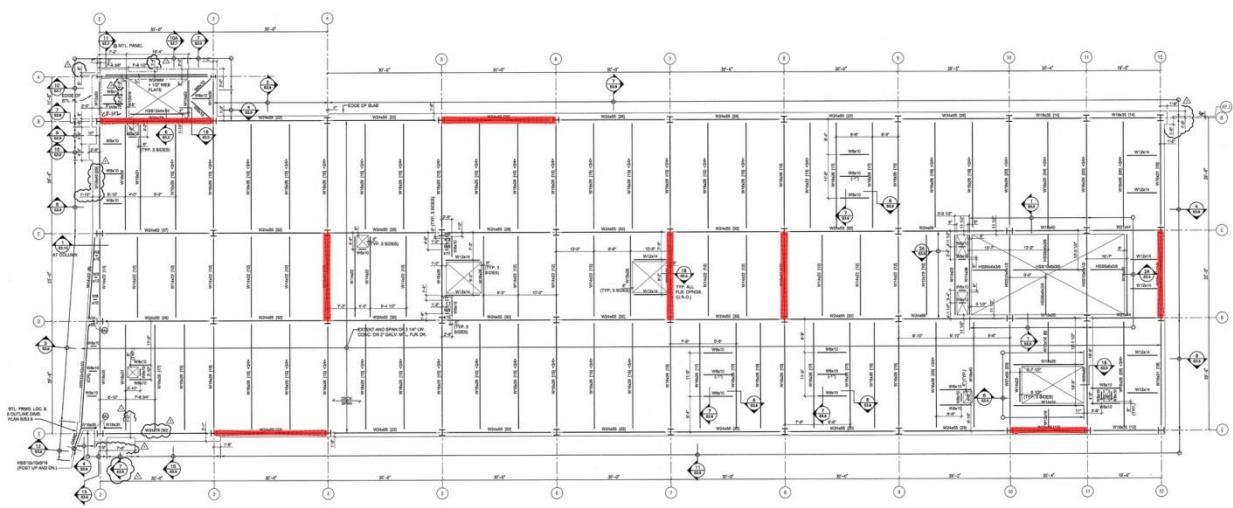
Figure 6: Braced frame locations

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.
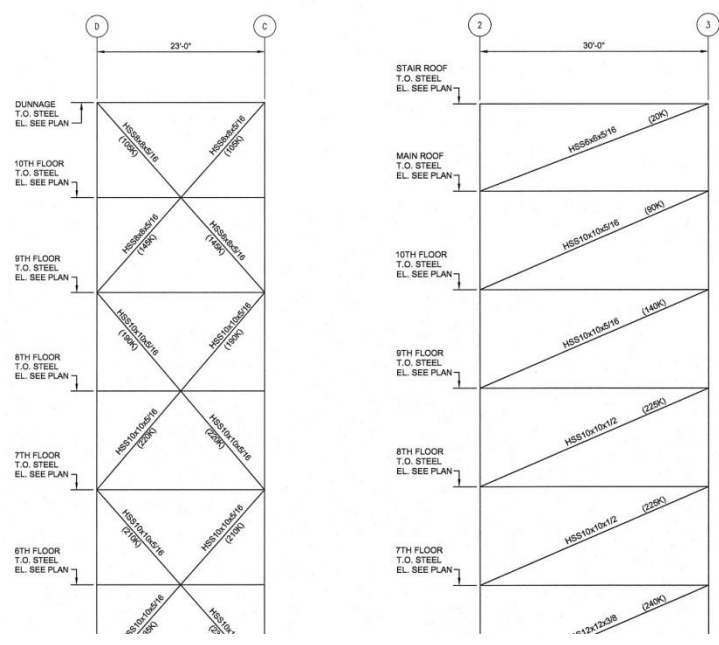
Figure 7: Typical braced frames

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

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

*Assumed Value

*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

295.39k
235.27k
206.41k
178.04k
150.22k
123.36k
99.71k
92.12k
56.43k
24.72k

V = 1461.68k

M = 138,177.23 k-ft
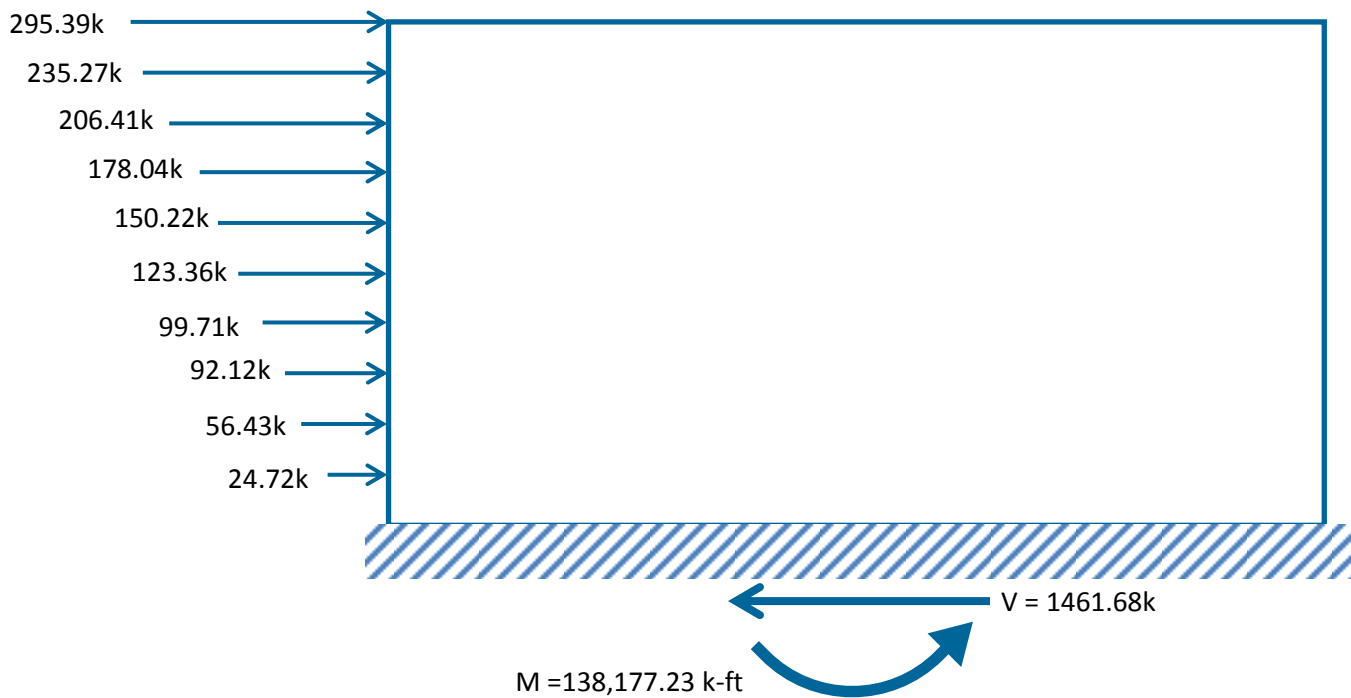
Figure 8: Seismic story forces

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		
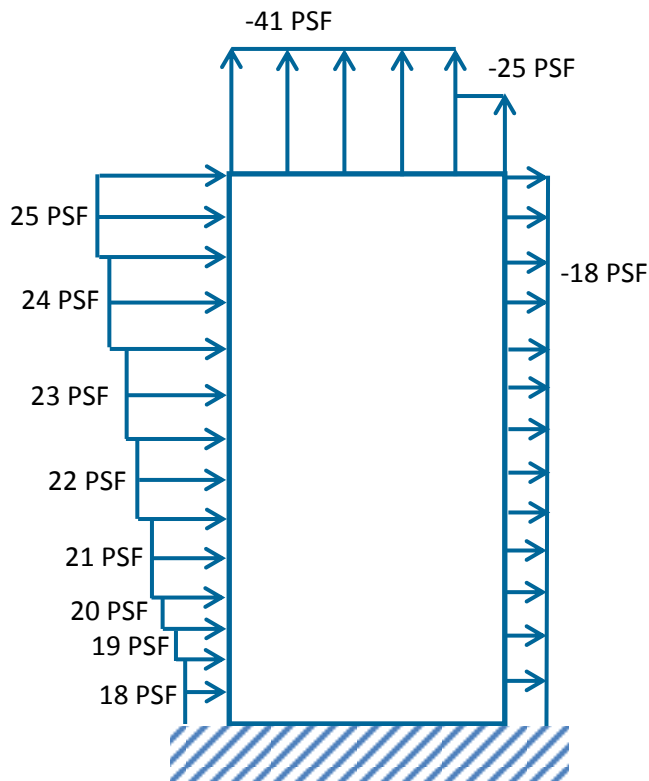
Figure 9: Wind loads E-W direction

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'.
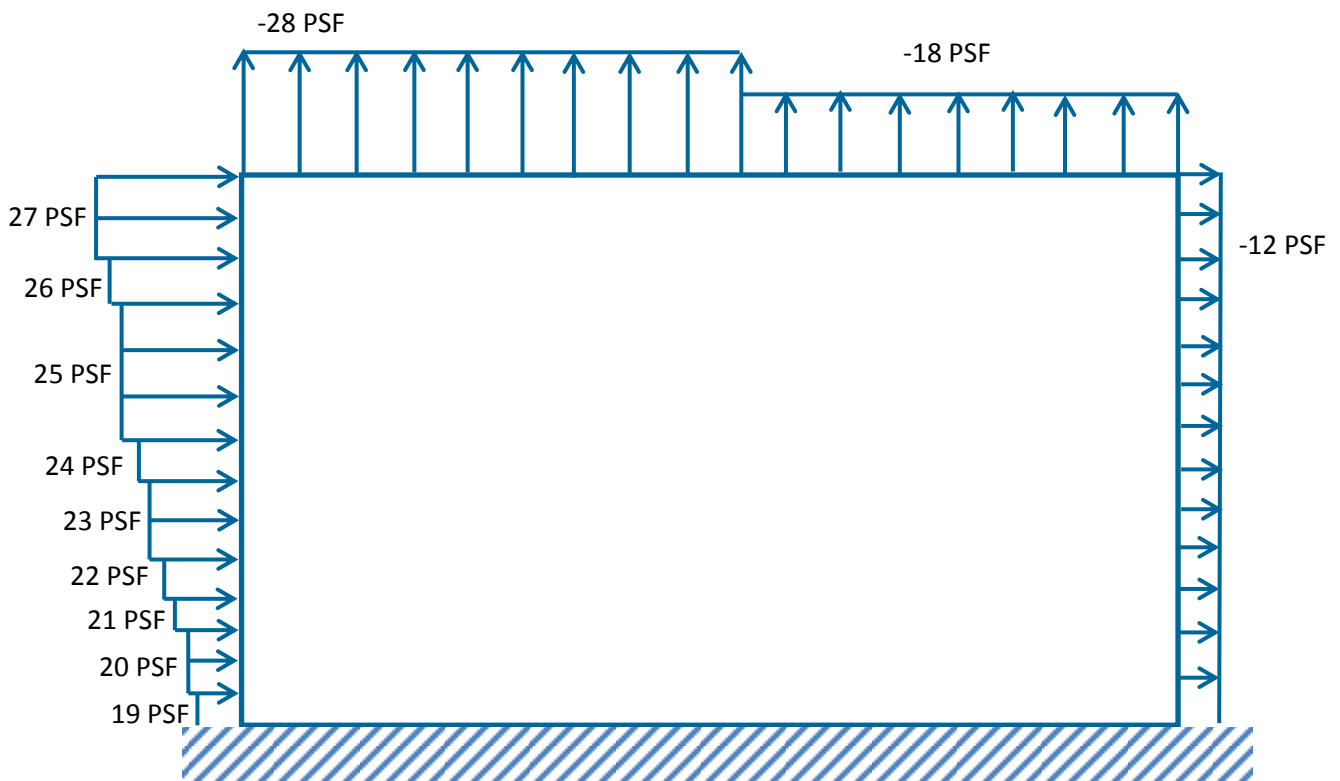
Figure 10: Wind loads N-S direction

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.
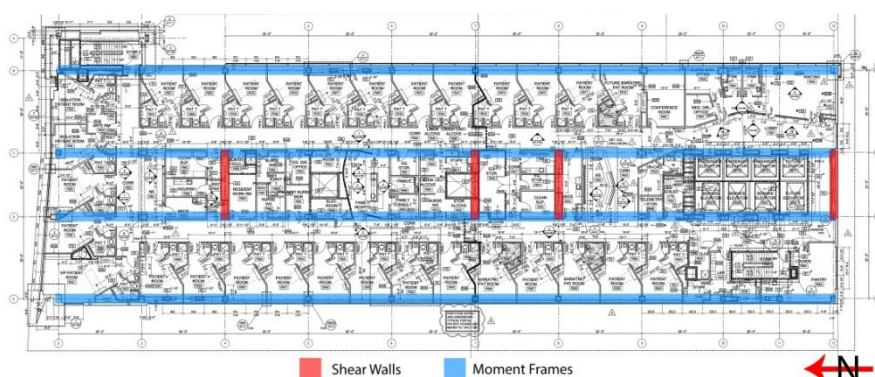
Figure 11: Proposed lateral system layout

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.
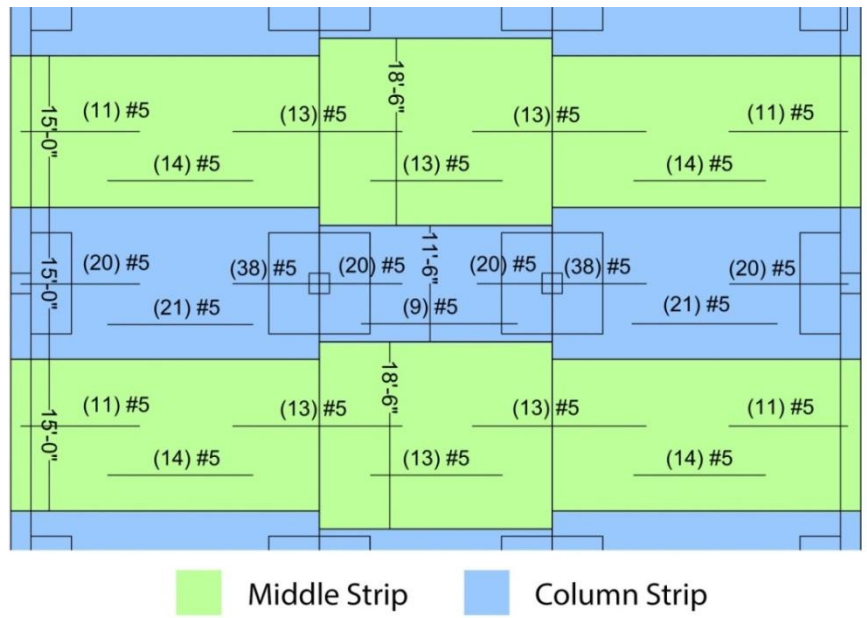
Figure 12: Required reinforcing E-W spans
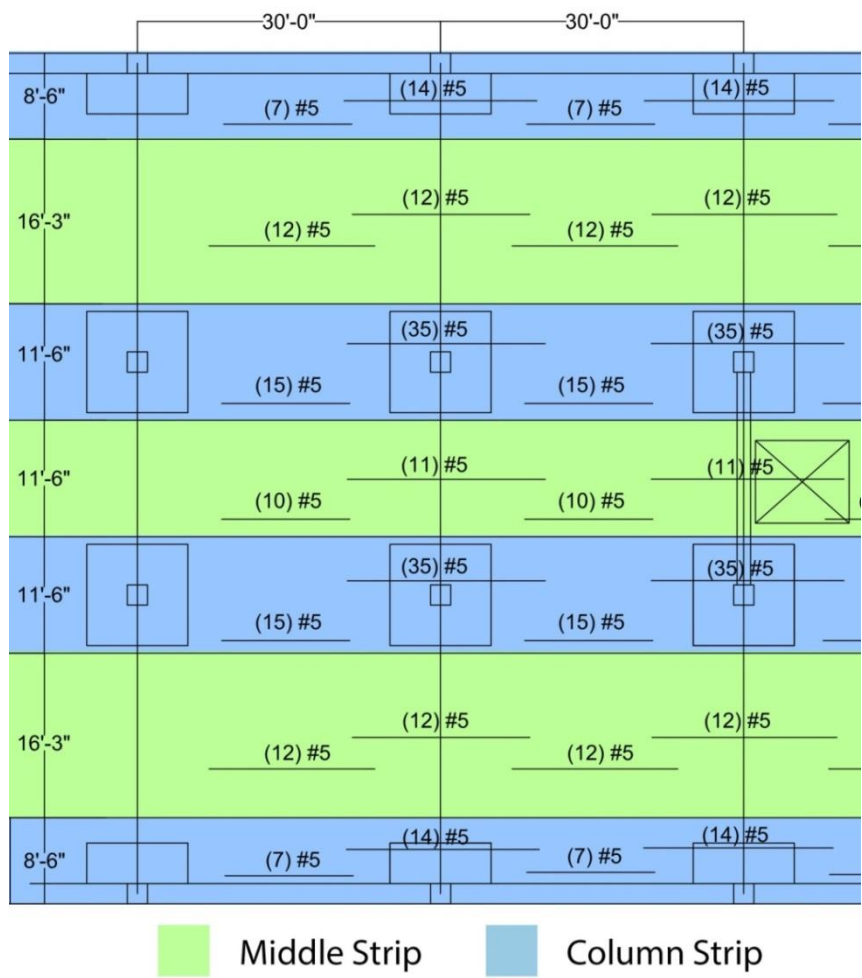
Figure 13: Required reinforcing N-S spans

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.
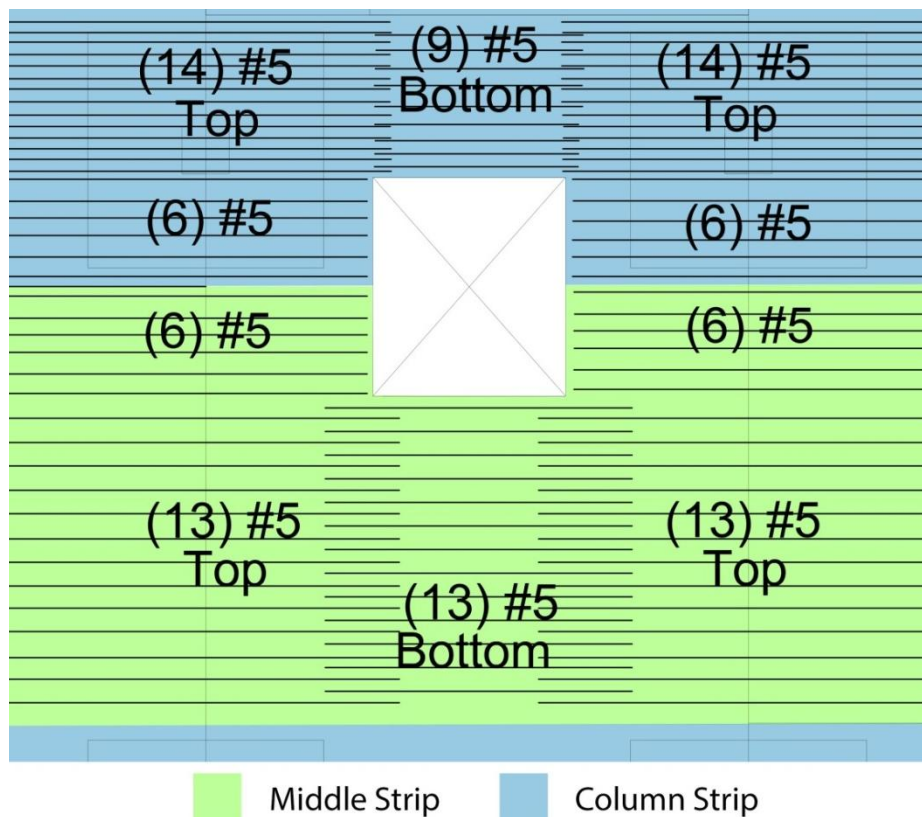
Figure 14: Reinforcement detailing around opening

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.
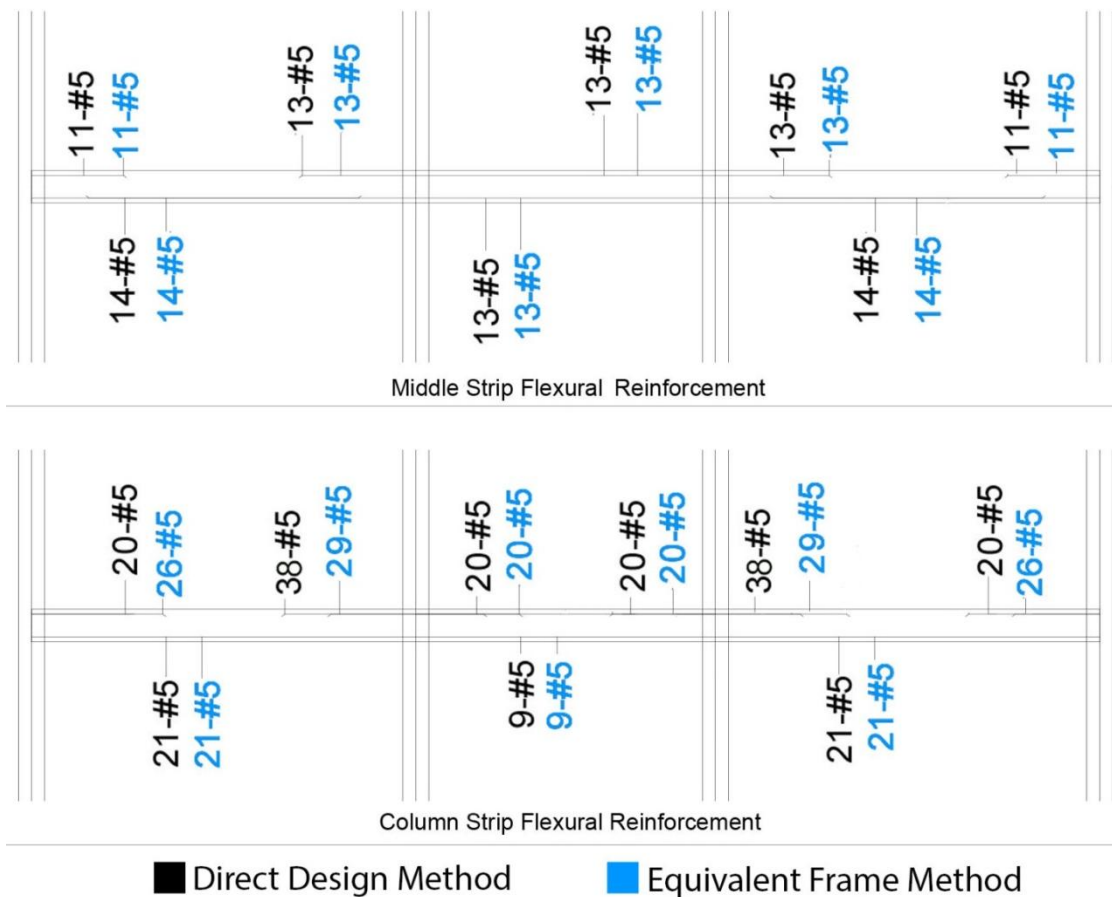
Figure 15: Reinforcement comparison - DDM vs. EFM

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.
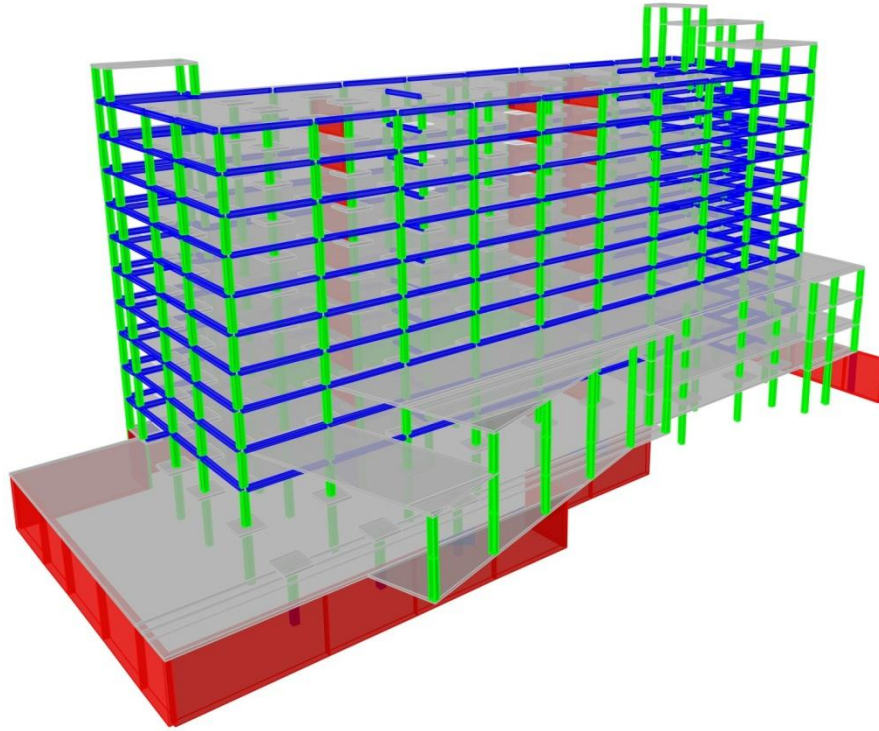
Figure 16: ETABS model
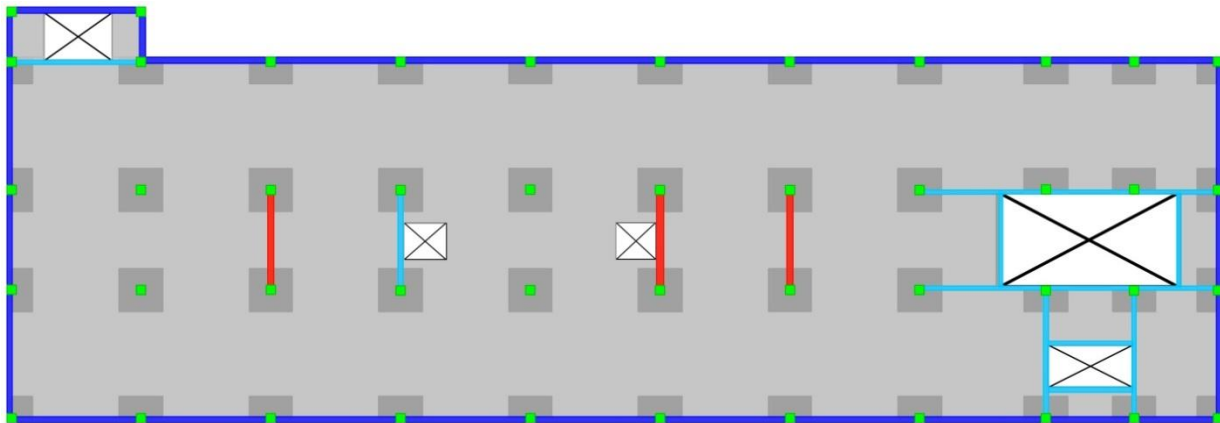
Figure 17: ETABS floor plan (Columns – green, Shear walls – red, Edge beams – dark blue, Interior Beams – light blue, Drop panels – dark gray)

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.
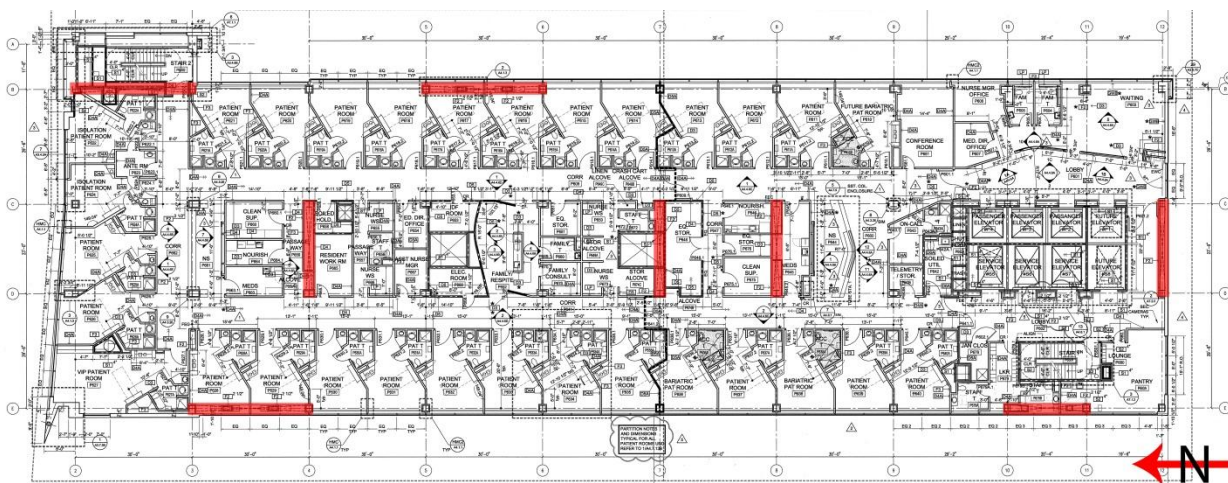
Figure 18: Braced frame layout

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.
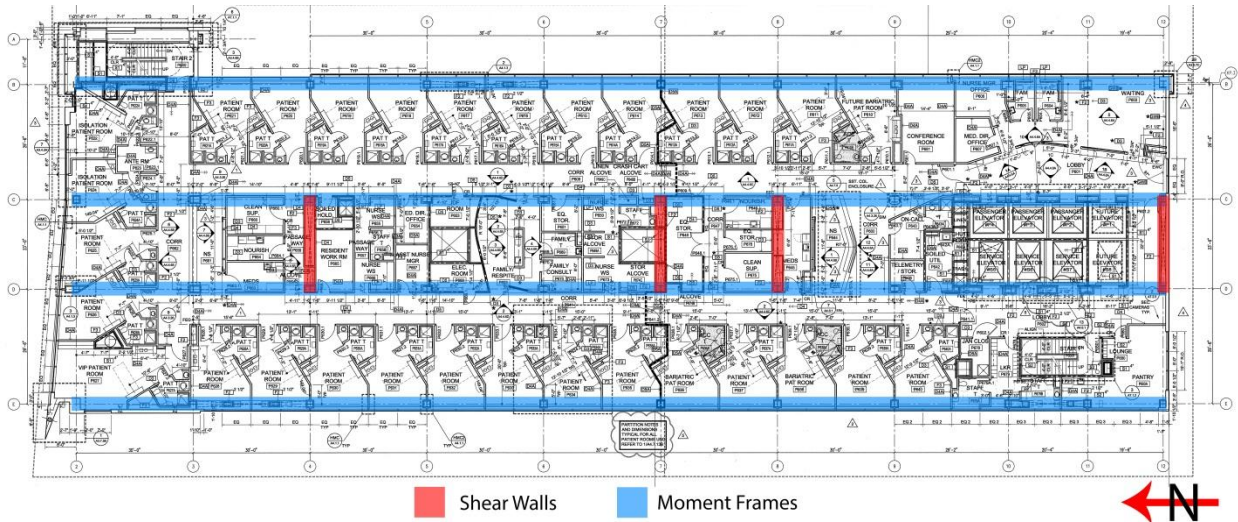
Figure 19: Proposed lateral system layout

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.
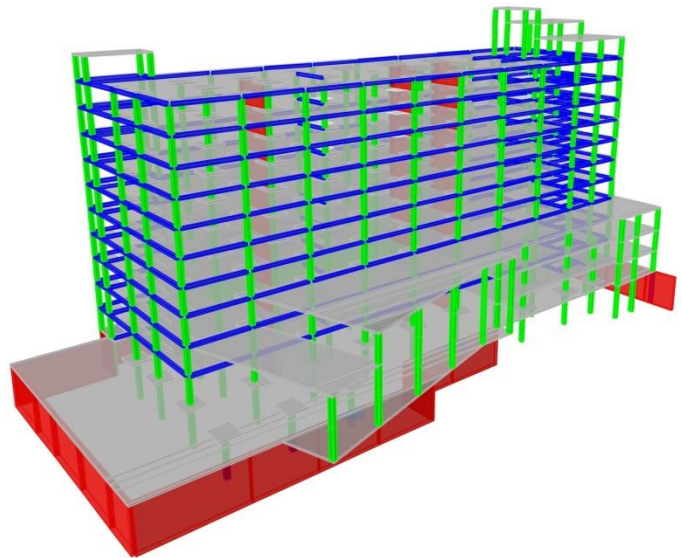
Figure 20: ETABS model

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.
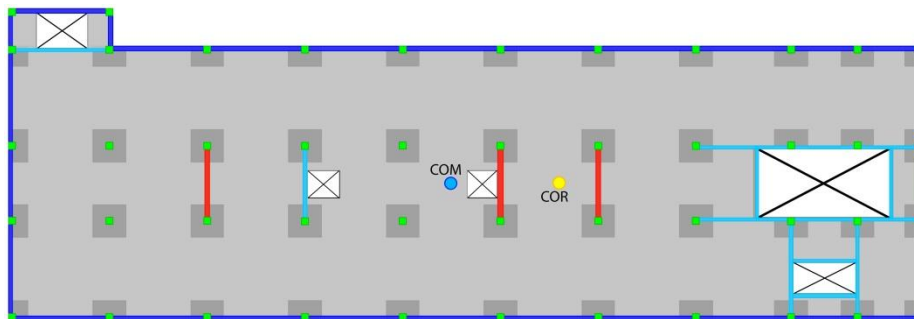
Figure 22: Center of rigidity for 4 wall layout
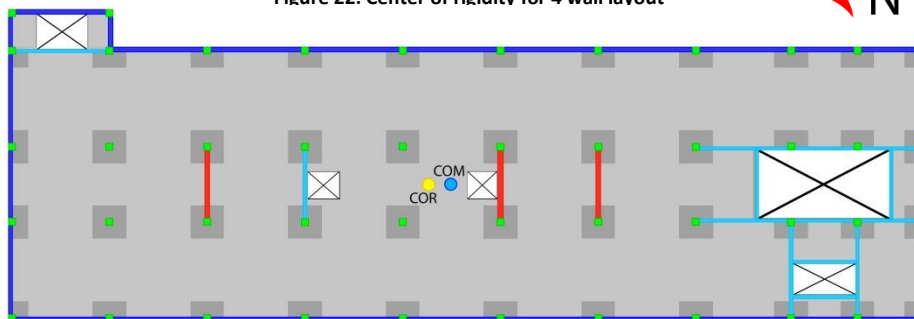
Figure 22: Center of rigidity for 3 wall layout

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.
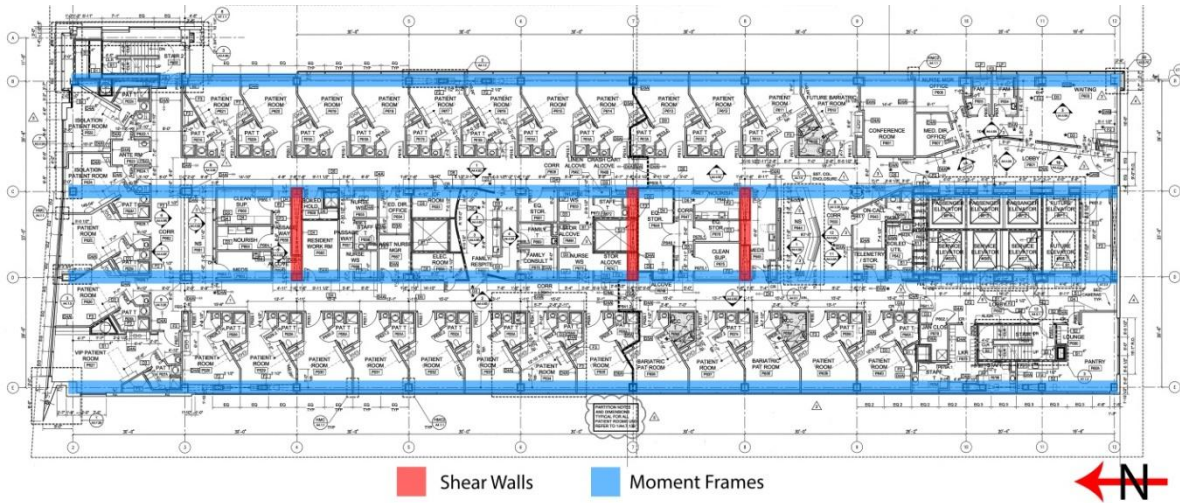
Figure 23: Final lateral system layout
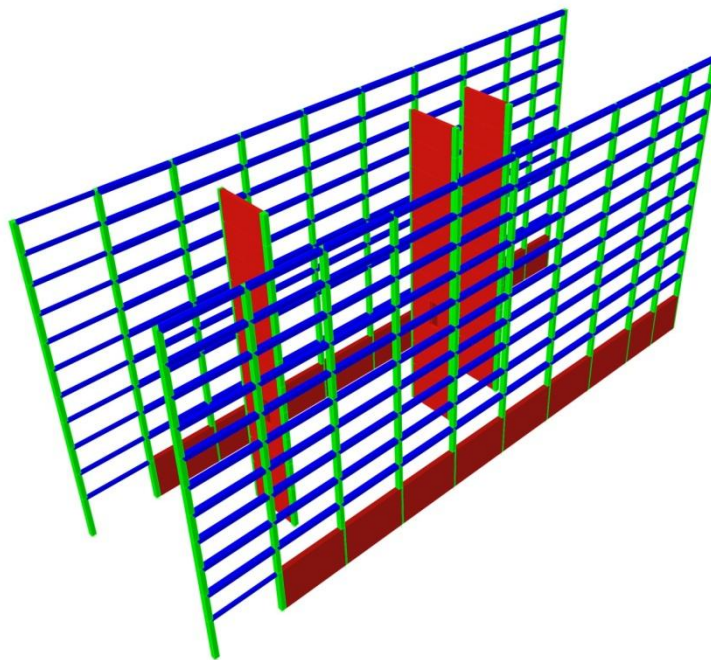
Figure 24: 3D view of ETABS lateral system

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.
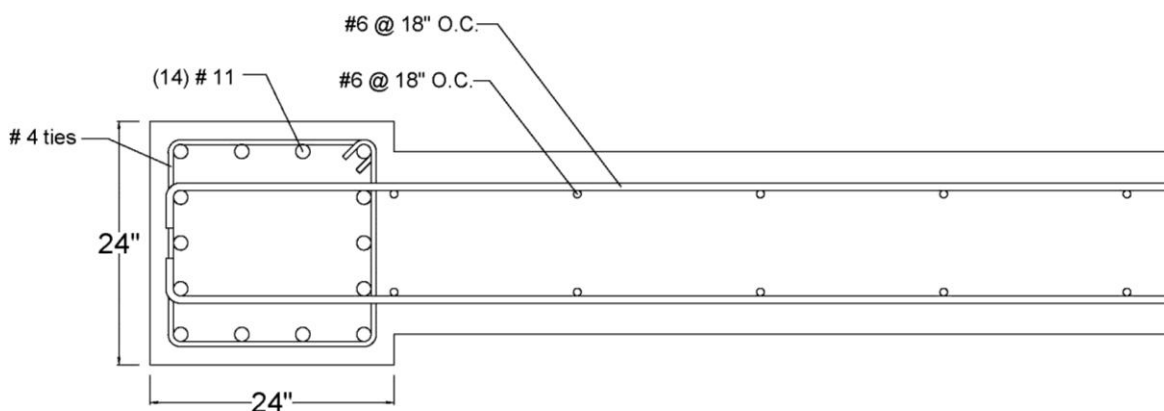
Figure 25: Shear wall detailing

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.
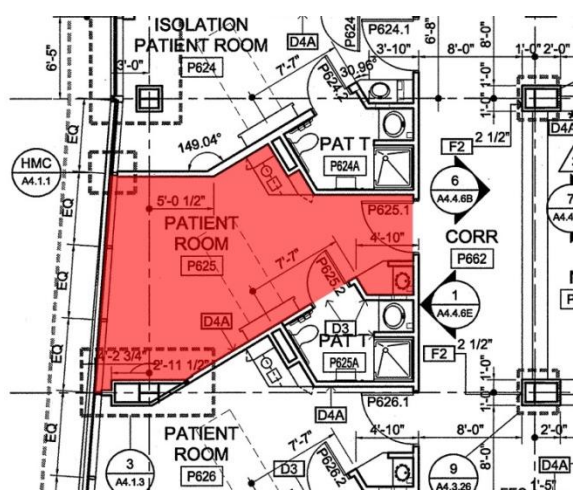
Figure 26: Patient room on 10th floor

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 (ft²)	Area (m²)	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.

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
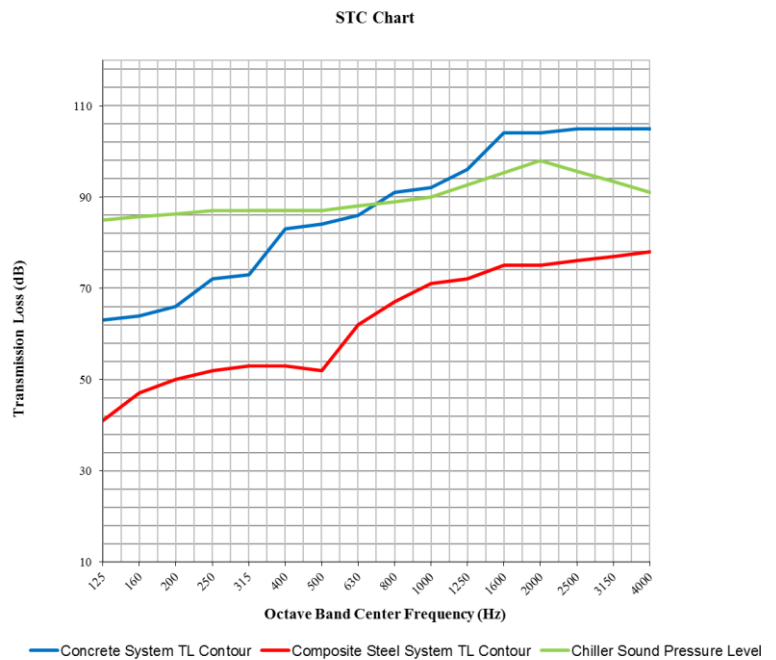
Figure 27: STC chart

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.

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.

Figure 28: Noise Criteria graph

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.
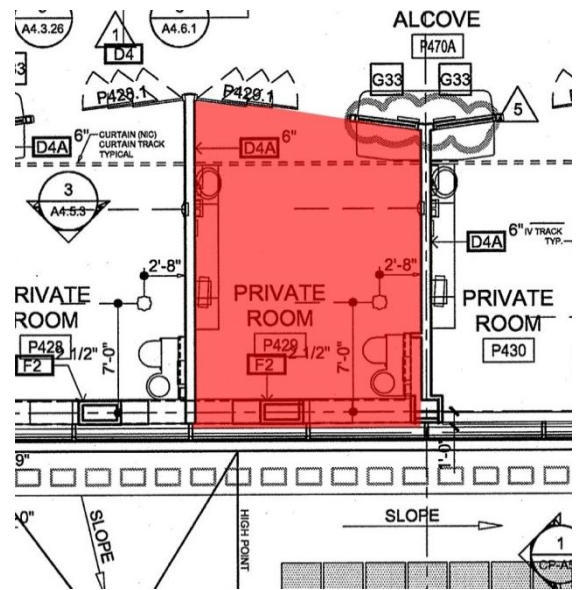
Figure 29: 4th floor patient room

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:

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

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

Noise Criteria (NC)
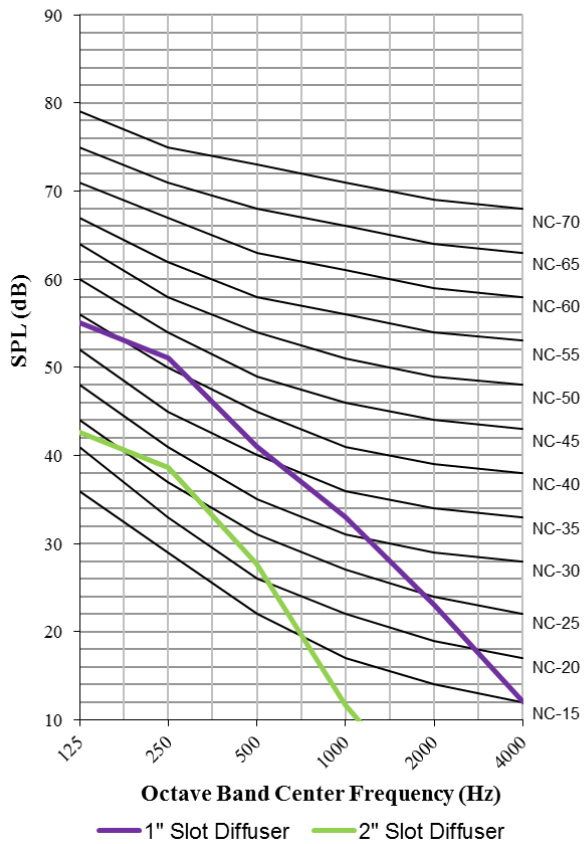
Figure 30: Noise Criteria Graph

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.

Room Criteria (RC)
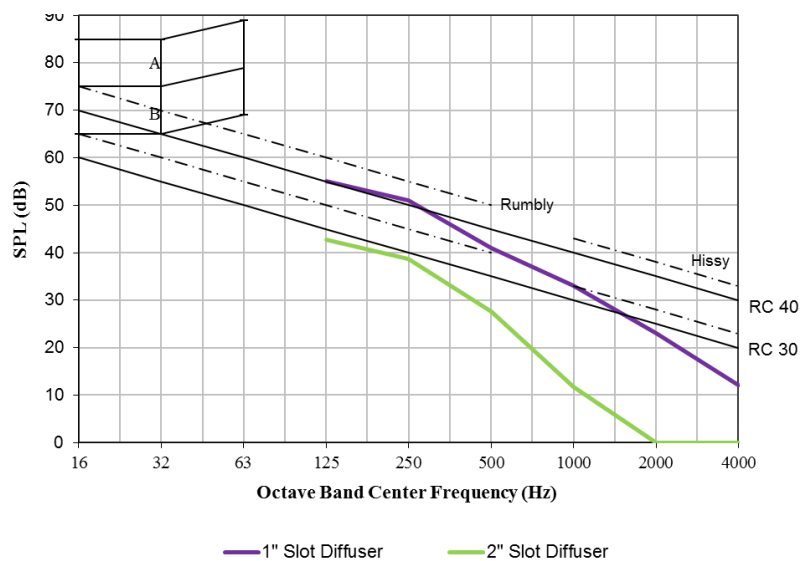
Figure 31: Room Criteria Chart

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

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**

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.

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.
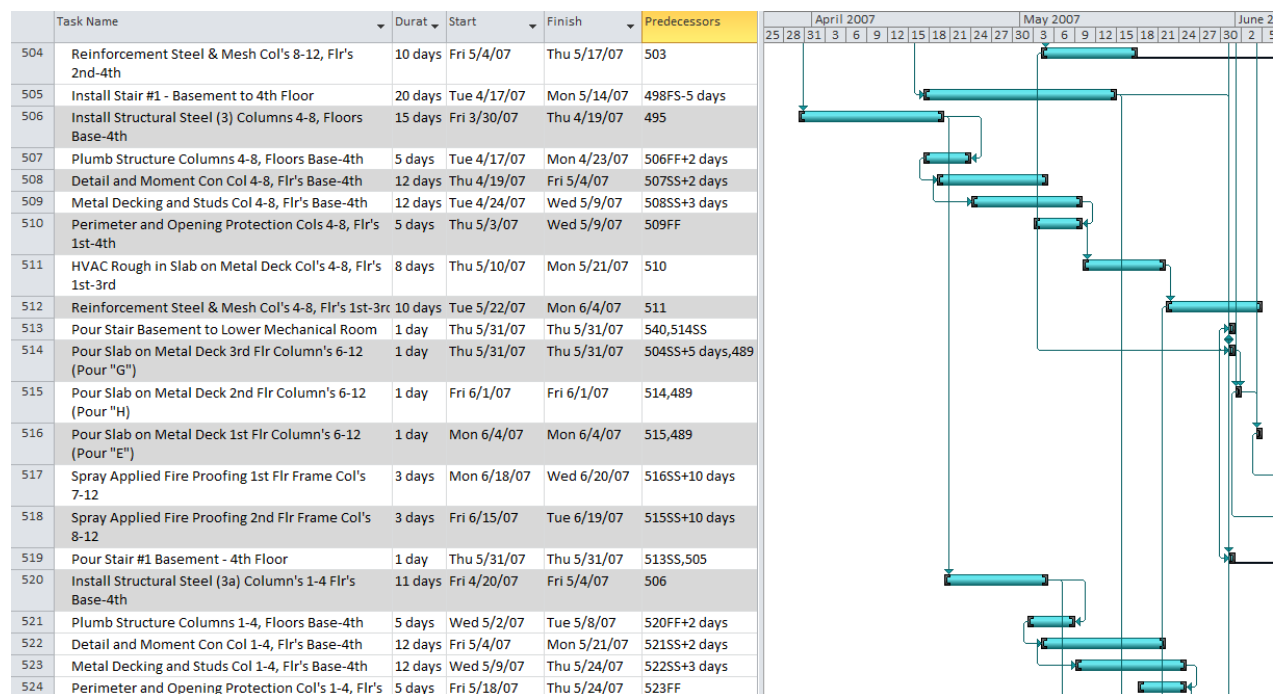
Figure 32: Steel schedule

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.

Appendices

Appendix A: Typical Plans

Legend:
- Patient Rooms (green)
- Family Areas (orange)
- Nursing Stations (red)
- Staff/Back of House (blue)
- Vertical Circulation (yellow)
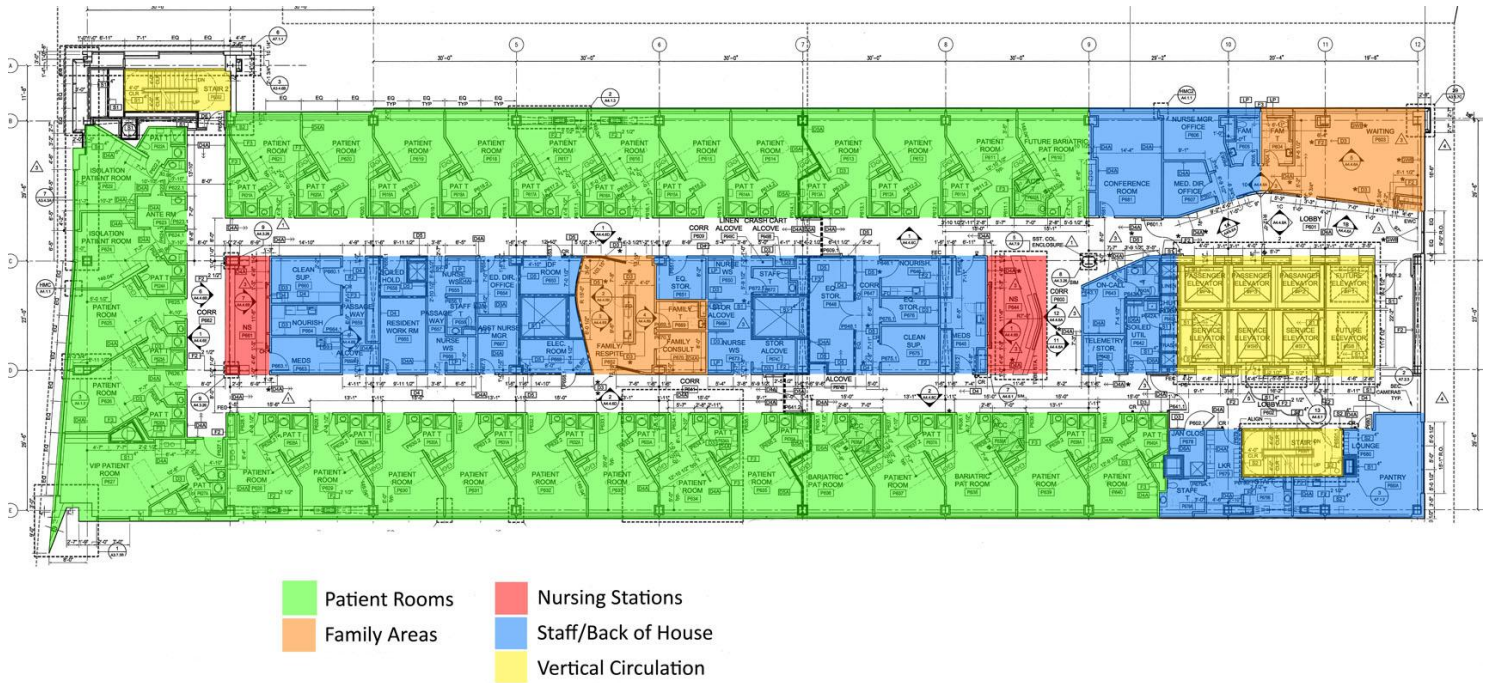
Figure 34: Typical patient floor
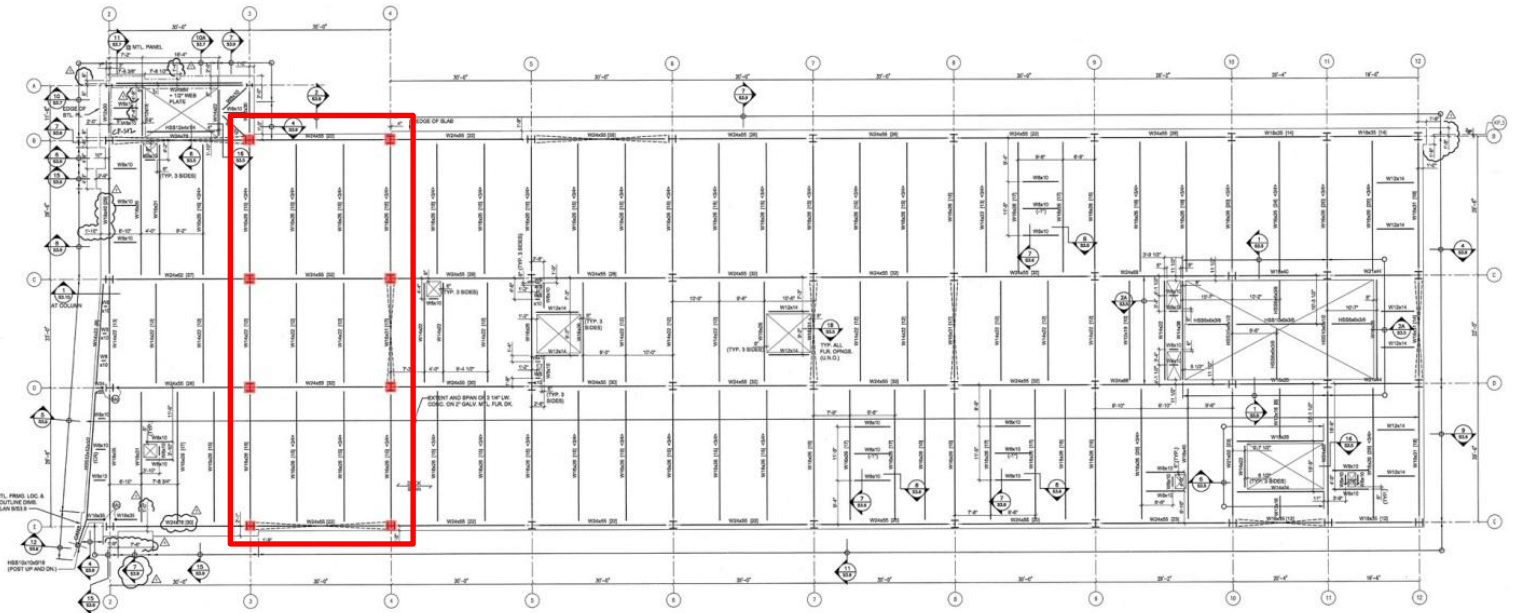
Figure 35: Typical bay - steel frame

Figure 37: Lobby layout
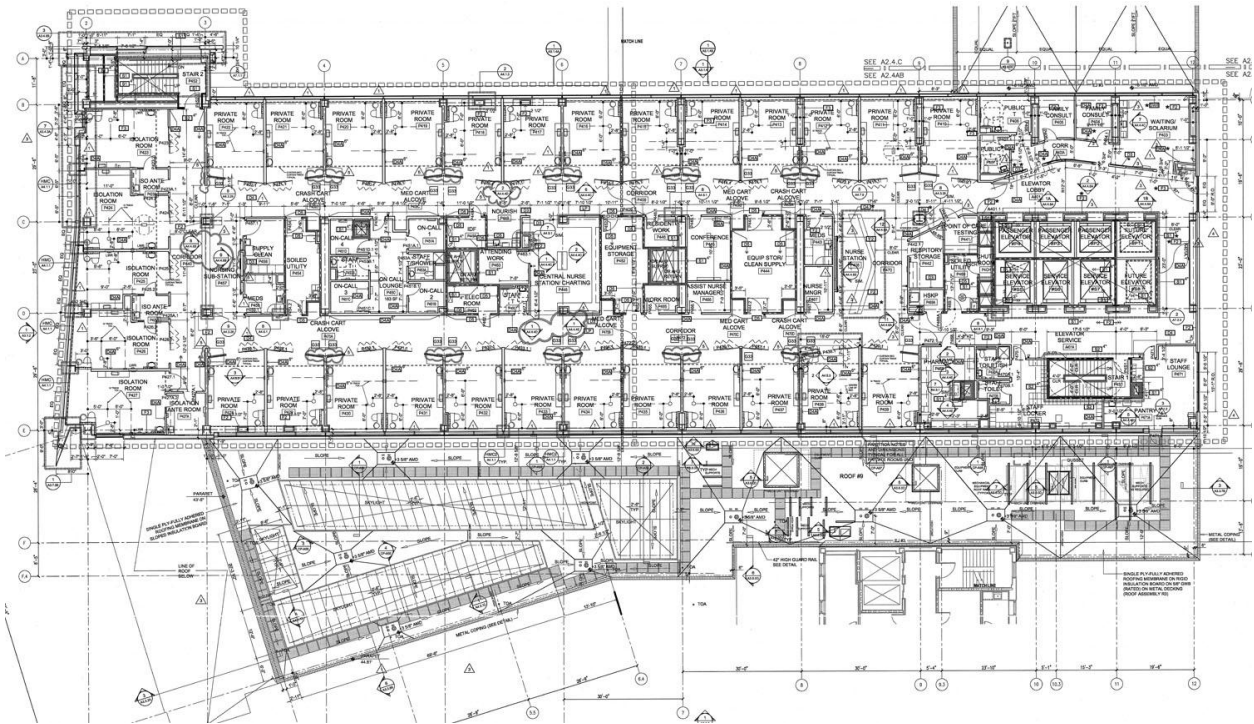
Figure 36: 4th Floor/Lobby roof plan
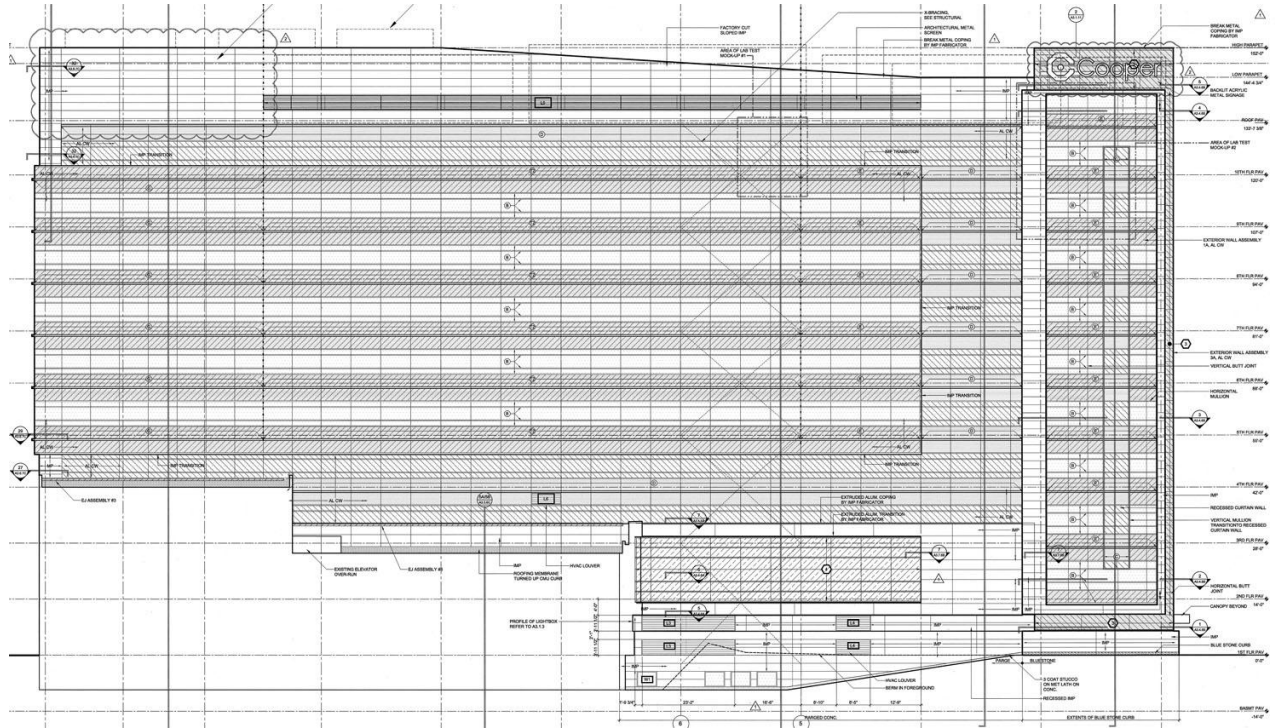
Figure 39: East elevation
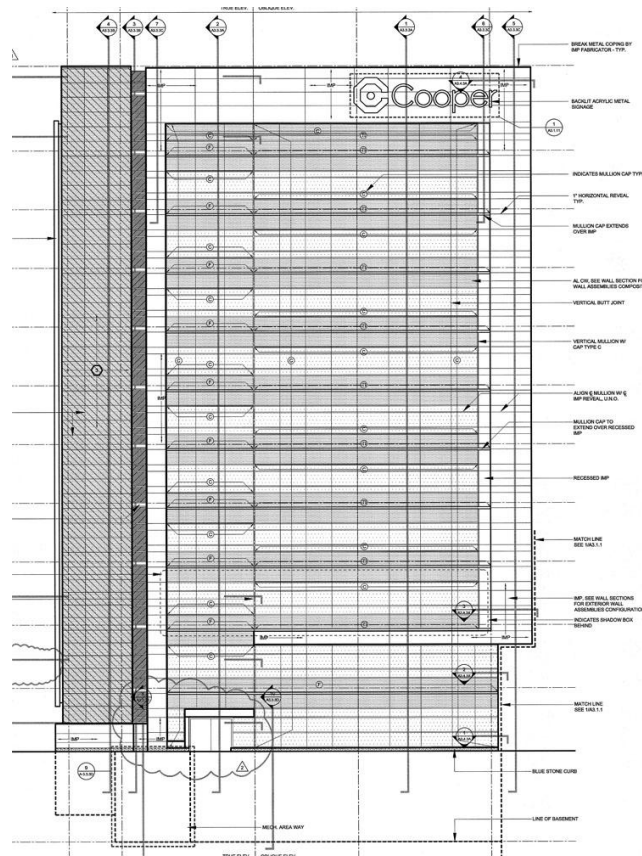
Figure 38: North elevation

Appendix B: Wind Load Calculations

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Wind Load Parameters • ASCE 7-05

$V = 90$ mph
$I = 1.15$
Exposure C
$K_d = 0.85$
$K_{zt} = 1.0$

$G =$ see below
$GC_{pi} = \pm 0.18$

$K_z = 1.38 @ 152'$

Location: Camden, NJ (Fig 6-1)
Occupancy Category IV (Table 6-1)(Table 1-1)
(§6.5.6.2)
Building MWRFS (Table 6-4)
Homogeneous Topography

Enclosed Bldg (Fig. 6-5)

$$\frac{152 - 140}{160 - 140} = \frac{K_z - 1.36}{1.39 - 1.36}$$ (Table 6-3)

Velocity Pressures

Building Height	K_z	q_z (psf)
0-15	0.85	17.23
20	0.90	18.24
25	0.94	19.05
30	0.98	19.86
40	1.04	21.08
50	1.09	22.09
60	1.13	22.90
70	1.17	23.72
80	1.21	24.53
90	1.24	25.13
100	1.26	25.54
120	1.31	26.55
140	1.36	27.57
152	1.38	27.97

$$q_z = 0.00256 \, K_z \, K_{zt} \, K_d \, V^2 \, I$$

@ 152': $q_z = 0.00256(1.38)(1.0)(0.85)(90)^2(1.15) = 27.97$ psf

Calculating G

fundamental frequency → $n_1 = 100/H = 100/152 = 0.658$
$n_1 = 75/H = 75/152 = \boxed{0.493}$

$n_1 = 0.493 < 1$ * More conservative to use lower bound

∴ Consider Building as Flexible (§6.5.8.2)

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$$g_Q = g_v = 3.4$$

$$g_R = \sqrt{2\ln(3600n_1)} + \frac{0.577}{\sqrt{2\ln(3600n_1)}}$$

$$= \sqrt{2\ln(3600 \times 0.493)} + \frac{0.577}{\sqrt{2\ln(3600 \times 0.493)}} = \boxed{4.0174}$$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6}$$

$c = 0.2$ (Table 6-2)

$\bar{z} = 0.6h = 0.6(152) = 91.2$

$$= 0.2\left(\frac{33}{91.2}\right)^{1/6} = \boxed{0.1688}$$

Determine Q:

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$$

$\bar{\epsilon} = 1/5$ (Table 6-2)
$\ell = 500$ (Table 6-2)

$$= 500\left(\frac{91.2}{33}\right)^{1/5} = 612.73$$

Q: N-S

$$B = 86'$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1 + 0.63\left(\frac{86+152}{612.73}\right)^{0.63}}} = \boxed{0.8616}$$

Q: E-W

$$B = 285'$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{285+152}{612.73}\right)^{0.63}}} = \boxed{0.8140}$$

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$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right)$$

$\bar{\alpha} = 1/6.5$ (Table 6-2)
$\bar{b} = 0.65$ (Table 6-2)
$V = 90\ mph$

$$= 0.65\left(\frac{91.2}{33}\right)^{1/6.5} 90\left(\frac{88}{60}\right)$$

$$\boxed{= 100.32\ ft/s}$$

$$N_1 = \frac{n_1\ L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{0.493 \times 612.73}{100.32} \boxed{= 3.01}$$

$$R_n = \frac{7.47\ N_1}{(1 + 10.3\ N_1)^{5/3}} = \frac{7.47\,(3.01)}{[1 + 10.3\,(3.01)]^{5/3}} \boxed{= 0.0697}$$

$$\eta_h = \frac{4.6\,n_1\,h}{\bar{V}_{\bar{z}}} = \frac{4.6\,(0.493)(152)}{100.32} = 3.436$$

$$R_h = \frac{1}{\eta_h} - \frac{1}{2\eta_h^2}(1 - e^{-2\eta_h}) = \frac{1}{3.436} - \frac{1}{2(3.436^2)}\left(1 - e^{-2(3.436)}\right)$$

$$\boxed{= 0.2487}$$

$$\eta_B = \frac{4.6\,n_1\,B}{\bar{V}_{\bar{z}}} \qquad B = 86'\ N\text{-}S \Rightarrow \frac{4.6\,(0.493)(86')}{100.32} = 1.944\quad N\text{-}S$$

$$B = 285'\ E\text{-}W \Rightarrow \frac{4.6\,(0.493)(285)}{100.32} = 6.443\quad E\text{-}W$$

$$R_B = \frac{1}{\eta_B} - \frac{1}{2\eta_B^2}(1 - e^{-2\eta_B}) = \frac{1}{1.944} - \frac{1}{2(1.944^2)}\left(1 - e^{-2(1.944)}\right) \boxed{= 0.3848\ N\text{-}S}$$

$$= \frac{1}{6.443} - \frac{1}{2(6.443^2)}\left(1 - e^{-2(6.443)}\right) \boxed{= 0.1432\ E\text{-}W}$$

$$\eta_L = \frac{15.4\,n_1\,L}{\bar{V}_{\bar{z}}} \qquad L = 285'\ N\text{-}S \Rightarrow \frac{15.4\,(0.493)(285)}{100.32} = 21.569\quad N\text{-}S$$

$$L = 86'\ E\text{-}W \Rightarrow \frac{15.4\,(0.493)(86)}{100.32} = 6.508\quad E\text{-}W$$

$$R_L = \frac{1}{\eta_L} - \frac{1}{2\eta_L^2}(1 - e^{-2\eta_L}) = \frac{1}{21.569} - \frac{1}{2(21.569^2)}\left(1 - e^{-2(21.569)}\right) \boxed{= 0.0453\ N\text{-}S}$$

$$= \frac{1}{6.508} - \frac{1}{2(6.508^2)}\left(1 - e^{-2(6.508)}\right) \boxed{= 0.1419\ E\text{-}W}$$

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Assume $\beta = 0.01$ (§ C6.5.8)

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

N-S: $R = \sqrt{\frac{1}{0.01} (0.0697)(0.2487)(0.3848)(0.53 + 0.47 \times 0.0453)}$

$$\boxed{= 0.6064 \quad N\text{-}S}$$

E-W: $R = \sqrt{\frac{1}{0.01} (0.0697)(0.2487)(0.1432)(0.53 + 0.47 \times 0.1419)}$

$$\boxed{= 0.3849 \quad E\text{-}W}$$

Determine G_f:

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

N-S: $G_f = 0.925 \left[\dfrac{1 + 1.7(0.1688)\sqrt{3.4^2(0.8616^2) + 4.0174^2(0.6064^2)}}{1 + 1.7(3.4)(0.1688)} \right]$

$$\boxed{G_f = 0.98 \quad N\text{-}S}$$

E-W: $G_f = 0.925 \left[\dfrac{1 + 1.7(0.1688)\sqrt{3.4^2(0.8140^2) + 4.0174^2(0.3849^2)}}{1 + 1.7(3.4)(0.1688)} \right]$

$$\boxed{G_f = 0.89 \quad E\text{-}W}$$

Wind Loads   Tech 1 Report   Andrew Voorhees   5

Design Wind Pressures for MWRFS

Simplified Building shape

285'

86'

↑N
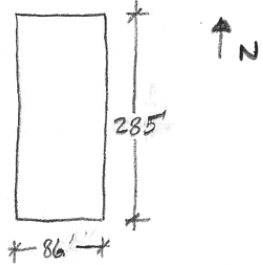
C_p: Walls

__N-S__

Windward Wall pressure coeff. = 0.8
Leeward Wall pressure coeff. :

$L/B = 279/82 = 3.4$

$\dfrac{3.4 - 2}{4 - 2} = \dfrac{C_p - (-0.3)}{-0.2 - (-0.3)}$ ⇒ $C_p = -0.23$

__E-W__

Windward Wall pressure coeff. = 0.8
Leeward Wall pressure coeff. :

$L/B = 82/279 = 0.29$

$0 < 0.29 < 1$ ⇒ $C_p = -0.5$
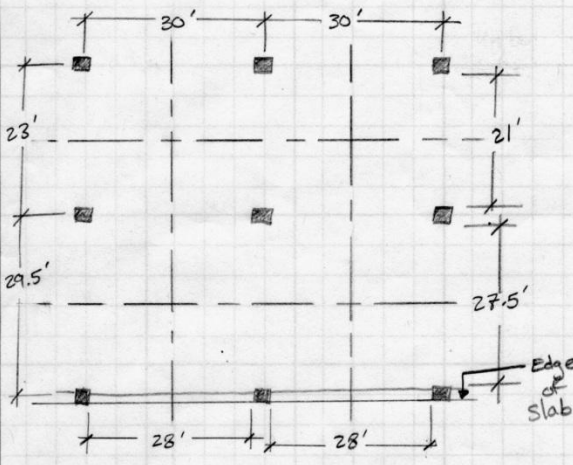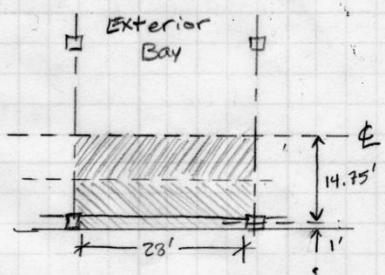
C_p: Roof

$0 = \theta < 10°$

$h/L = 152/279 = 0.54$ ⇒ 0 - h/2 $C_p = -0.9$
          $152/82 = 1.85$ h/2 - h $C_p = -0.9$
                                       h - 2h $C_p = -0.5$
                                       > 2h $C_p = -0.3$

⇓

0 - h/2 $C_p = -1.3$
> h/2 $C_p = -0.7$

See Excel tables for calculations

Appendix C: Slab Design

Slab Design (Flrs 6-10) | Final Report | Andrew Voorhees | 1

Using the Direct Design Method

* Assume Column Dimensions of 24" × 24"

Using Drop panels and Edge Beams

Loads

Live = 80 psf
Dead
 - MEP = 5 psf
 - SDL = 10 psf

ACI 318 - 11

From Table 9.5 (c) : Minimum slab thickness — $h = l_n/36$ for Ext. + Int. panels
Two-way slab with drop panels ($f_y = 60,000$)

$$\frac{l_n}{36} = \frac{28' \times 12''}{36} = 9.33''$$

use h = 10"

Self weight of slab

$$= \left(\frac{10''}{12''}\right) \times 150 \, pcf = 125 \, psf$$

$$q_u = 1.6(80) + 1.2(15+125) = 296 \, psf = 0.296 \, ksf$$

Exterior Bay

14.75'

23'

1'

* considered an "interior" span

$$M_o = \frac{q_u \, l_2 \, l_n^2}{8}$$

$$= \frac{0.296 \, ksf \, (15.75')(28')^2}{8}$$

$$\boxed{M_o = 457^{ft-k}}$$

Moment Distribution Coefficients

$$M_u^- = 0.65 M_o \qquad M_u^+ = 0.35 M_o$$

Slab Design (Flrs 6-10) | Final Report | Andrew Voorhees | 2

$M_u^- = 0.65 (457) = 297^{ft\cdot k}$ $M_u^+ = 0.35 (457) = 160^{ft\cdot k}$

Design Edge Beams

$\alpha_f \geq 0.8$

Try 24" x 18"

$\text{Centroid} = \dfrac{24(18)(9) + (8)(10)(13)}{24(18) + (8)(10)} = 9.625"$ from bottom

$I_b = \dfrac{24(18^3)}{12} + 24(18)(0.625)^2 + \dfrac{8(10)^3}{12} + 8(10)(3.375)^2$

$I_b = 13410.67 \text{ in}^4$

$I_s = (15.75 \times 12") \times \dfrac{10^3}{12} = 15750 \text{ in}^4$

$\alpha_f = \dfrac{I_b}{I_s} = \dfrac{13410.67}{15,750} = 0.85 \geq 0.80 \quad \checkmark \text{ ok}$

$\alpha_f \, \ell_2/\ell_1 = 0.85 (15.75/30) = 0.446 \leq 1.0$

$\ell_2/\ell_1 = \dfrac{15.75}{30} = 0.525$

Neg. Moment

	0.5	0.525	1.0
0	75	75	75
0.446		81.36	
1.0	90	89.25	75

To Col. Strip

$M_u^- = \dfrac{81.36}{100} (297) = \boxed{241.64^{ft\cdot k}}$

To Mid. Strip

$M_u^- = \boxed{55.36^{ft\cdot k}}$

Pos Moment

	0.5	0.525	1.0
0	60	60	60
0.446		73.05	
1.0	90	89.25	75

To Col. Strip

$M_u^+ = 0.7305 (160) = \boxed{116.88^{ft\cdot k}}$

To Mid. Strip

$M_u^+ = \boxed{43.12^{ft\cdot k}}$

Col. Strip Distributes to Beam + Col. Strip as well

ACI 318-11 § 13.6.5.2 → Beam strip portion interpolated

$x_f, l_2/l_1 = 0.446 \leq 1.0$

∴ $\dfrac{85 - 0}{1.0 - 0} = \dfrac{x - 0}{0.446 - 0}$ $x = 37.91\%$

To Beam   To Slab

$M_u^- = 0.3791(241.64) = \boxed{91.6^{ftk}}$ $M_u^- = \boxed{150.04^{ftk}}$

$M_u^+ = 0.3791(116.88) = \boxed{44.31^{ftk}}$ $M_u^+ = \boxed{72.57^{ftk}}$

Interior Bay

$q_u = 0.296\ ksf$

$M_o = \dfrac{q_u\, l_2\, l_n^2}{8}$

5.75'

$= \dfrac{0.296(26.25')(28')^2}{8}$

7.375'

29.5'

$\boxed{M_o = 761.5^{ftk}}$

30'
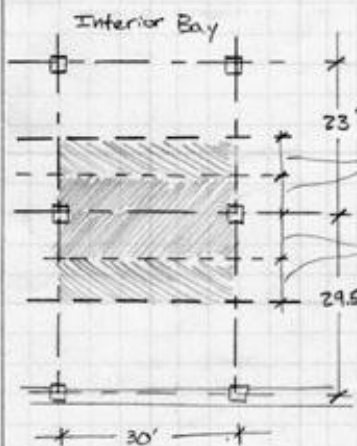
$M_u^- = 0.65\ M_o$ $M_u^+ = 0.35\ M_o$

$M_u^- = 0.65(761.5)$ $= 0.35(761.5)$

$M_u^- = 495^{ftk}$ $M_u^+ = 266.5^{ftk}$

Distribute to Col + Mid Strips

Col. Strip = 5.75 + 7.375 = 13.125 ft

No interior Beams ⇒ $x_f = 0$

M_u^- Col. strip = 0.75(495) = $\boxed{371.25^{ftk}}$

M_u^- Mid strip = 0.25(495) = $\boxed{123.75^{ftk}}$ → Half to each strip

M_u^+ Col. strip = 0.6(266.5) = $\boxed{160^{ftk}}$

M_u^+ Mid strip = 0.4(266.5) = $\boxed{106.6^{ftk}}$ → Half to each strip

Slab Design (Flrs 6-10) | Final Report | Andrew Voorhees | 4.

Corner Bay

Int. Support $M_u^- = 0.70$

Ext. Support $M_u^- = 0.30$

$M_u^+ = 0.50$

29.5'

30'

18" 10" 24"

Calculate C:

18" 10" 8" 24"

32" 10" 8" 24"

$$C = \Sigma \left(1 - 0.63 \tfrac{x}{y}\right) \tfrac{x^3 y}{3}$$

$$= \left[1 - 0.63\left(\tfrac{18}{24}\right)\right]\tfrac{18^3(24)}{3} + \left[1 - 0.63\left(\tfrac{8}{10}\right)\right]\tfrac{8^3(10)}{3}$$

$$= 25,457.55 \quad \text{controls}$$

$$C = \left[1 - 0.63\left(\tfrac{10}{32}\right)\right]\tfrac{10^3(32)}{3} + \left[1 - 0.63\left(\tfrac{8}{24}\right)\right]\tfrac{8^3(24)}{3}$$

$$= 11802.51$$

$$\beta_t = \frac{C}{2 I_s} \qquad I_s = 15,750 \, in^4$$

$$\beta_t = \frac{25,457.55}{2(15,750)} = 0.808$$
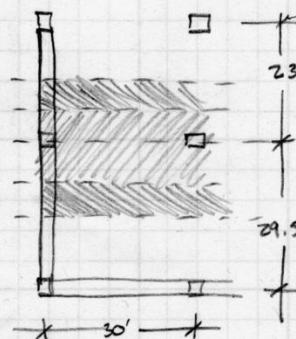
Interpolate l_2/l_1

β_t =	0.5	0.525	1.0	
0	100	100	100	$\alpha_{f_1} l_2/l_1 = 0$
0.808		91.92		
2.5	75	75	75	

β_t =	0.5	0.525	1.0	
0	100	100	100	$\alpha_{f_1} l_2/l_1 \geq 1.0$
0.808		96.53		
2.5	90	89.25	75	

$\alpha_{f_1} l_2/l_1 = 0.446$

0	91.92	
0.446	x	$\dfrac{x - 91.92}{0.446} = \dfrac{96.53 - 91.92}{1.0}$
1.0	96.53	$x = 93.98\%$

Slab Design (Flrs 6-10) | Final Report | Andrew Voorhees | 5

Int. Support $M_u^- = 0.70(457) = 319.9$

$\qquad$ 81.36% to Col. Strip $\Rightarrow$ $\boxed{260.27 \text{ ft·k}}$

$\qquad$ to Mid Strip $\Rightarrow$ $\boxed{59.63 \text{ ft·k}}$

Ext. Support $M_u^- = 0.3(457) = 137.1 \text{ ft·k}$

$\qquad$ 93.96% to Col. Strip $\Rightarrow$ $\boxed{128.85 \text{ ft·k}}$

$\qquad$ to Mid strip = $\boxed{8.25 \text{ ft·k}}$

Pos. Moment $M_u^+ = 0.5(457) = 228.5$

$\qquad$ 73.05% to Col strip $\Rightarrow$ $\boxed{166.92 \text{ ft·k}}$

$\qquad$ to Mid strip $\Rightarrow$ $\boxed{61.58 \text{ ft·k}}$

*Distribute Col. Strip Moments to Col. + Beam strips
   — See excel calculations

Exterior Span

$M_0 = 761.5 \text{ ft·k}$ $\alpha_{f1} = 0$, No Beams

$I_s = \dfrac{12 \times 26.25' \times 10^3}{12} = 26250$

$\beta_t = \dfrac{25457.55}{2(26,250)} = 0.485$

β_t =	0		100	100	100
	0.485			95.15	
	2.5		75	75	75

Int. Support
$M_u^- = 0.70(761.5) = 533.05$

$\qquad$ 75% to Col. Strip $\Rightarrow$ $\boxed{399.79}$ to Mid Strip = $\boxed{133.26 \text{ ft·k}}$

Ext. Support
$M_u^- = 0.3(761.5) = 228.45 \text{ ft·k}$

$\qquad$ 95.15% to Col. Strip = $\boxed{217.37 \text{ ft·k}}$ to Mid Strip = $\boxed{11.08 \text{ ft·k}}$

Pos Moment $M_u^+ = 0.5(761.5) = 380.75 \text{ ft·k}$

$\qquad$ 0.60(380.75) to Col strip = $\boxed{228.45 \text{ ft·k}}$ to Midstrip = $\boxed{152.3 \text{ ft·k}}$

Slab Design (Flrs 6-10)   Final Report   Andrew Voorhees   6

-217.4 -399.8 -371.25   -371.25   Column Strip
   +228.45   +160

-11.08 -133.26 -123.75 +66.6 -123.75   Middle Strip
   +152.3

-217.4 -399.8 -371.25   -371.75   Column Strip
   +228.45   +160

-5.54 +76.15 -66.63 -61.87 +53.3 -61.87
-8.25 +61.58 -59.63 -55.36 +43.12 -55.36   Middle Strip
-13.79 +137.73 -126.26 -47.24 +96.42 -117.24

-74.87   -150.04   -150.04   Column Strip
   +103.5   -161.35   +72.57

-48.94 +63.43 -48.88 -91.6 +44.31 -91.6
   Beam Strip   Beam Strip

②   ③   ④

Ⓔ   Ⓓ   Ⓒ   Ⓑ   Ⓐ

② -48.80 +63.20 -98.38 -67.56 +33.54 -67.56
   -77.49 +100.35 -156.22 +68.75 -68.75
                        +34.14

   -8.02 +60.30 -58.79 -33.39 -33.39
   -5.345 +83.95 -73.45 -39.78 +23.69 -39.78
   -13.37 +144.25 -132.24 -73.17 +23.69 -73.17
                        +57.46

③ -241.15   -440.7 -238.64   -238.64
      +251.83   +102.86

   -10.69   -146.9 -79.55   -79.55
      +167.89   +68.53

④

Shear Checks   Final Report   Andrew Voorhees   1

Check one-way shear   Assuming #5 bars $d_b = 0.625''$

$l_2 = 26.25'$ for long direction $d = 10'' - \frac{3}{4}'' - \frac{d_b}{2}$

13.25' $d = 8.94''$

          for short direction $d = 10'' - \frac{3}{4}'' \frac{3}{2} d_b$

$b \times l_2 = 26.25' \times 13.25'$ $d = 8.31''$
$= 347.81$

critical section
at $d \leq 8.94'$

          $d = 8.31''$

13.06   14.75'

15'

$q_u = 0.296 \text{ ksf}$ $b \times l_2 = 30' \times 13.06 = 391.8$ controls

$V_u = 0.296 \times 391.8 = 115.97^k$

$V_c = 2\lambda\sqrt{f'_c} \, bd = 2\sqrt{4000} \, (13.06 \times 12)(8.31) = 164.73^k$

$\phi V_c = 0.75 (164.73) = 123.5 > 115.97 = V_u$

$\checkmark$ ok

Check Two-Way Shear

          Min drop panel thickness
          $1.25h = 2.5''$ below slab

          10''

          2.5''

          width $\geq l/6 = 30/6 = 5'$ in
          each direction

          24''

60'' $\frac{5'}{=60''}$

b_o around column $= 2(24'' + 24'' + 2(8.31 + 2.5)) = 139.24'' = 34.81'' \times 34.81''$

b_o around drop panel $= 2(120 + 120 + 2(8.31)) = 513.24 = 128.31'' \times 128.31''$

q_{du} drop panel $= 1.2(2.5/12)(150) = 37.5 \text{ psf}$

$V_u = 0.296^{ksf}\left[(26.25 \times 30) - \left(\frac{34.81}{12}\right)^2\right] + 0.0375\left[10^2 - \left(\frac{34.81}{12}\right)^2\right]$

$= 234.04^k$

Shear   Checks | Final Report | Andrew Voorhees | 2

$$V_c \leq \begin{cases} 2 + 4/\beta & = 6 \\ \dfrac{\alpha_s d}{b_o} + 2 & = \dfrac{40(10.81)}{139.24} + 2 = 5.1 \\ 4 & = 4 \leftarrow \text{controls} \end{cases}$$

$$\phi V_c = 4(0.75)(1)\sqrt{4000}\,(139.24)(10.81) = 285.6^k$$

$$\phi V_c > V_u \qquad 285.6 > 234.04^k$$

$$\checkmark \text{ ok}$$

Check shear around drop panel

$$V_u = 0.296 \left[(26.25 \times 30) - \left(\dfrac{128.31}{12} \right)^2 \right] = 199.26^k$$

$$V_c \leq \begin{cases} 2 + 4/\beta & = 6 \\ \dfrac{\alpha_s d}{b_o} + 2 & = \dfrac{40(8.31)}{513.24} + 2 = 2.65 \leftarrow \text{controls} \\ 4 & = 4 \end{cases}$$

$$\phi V_c = 2.65(0.75)\sqrt{4000}\,(513.24)(8.31)$$

$$= 536.12^k > V_u = 199.26^k$$

$$\checkmark \text{ ok}$$

Appendix D: Slab Reinforcement
N-S Gridlines
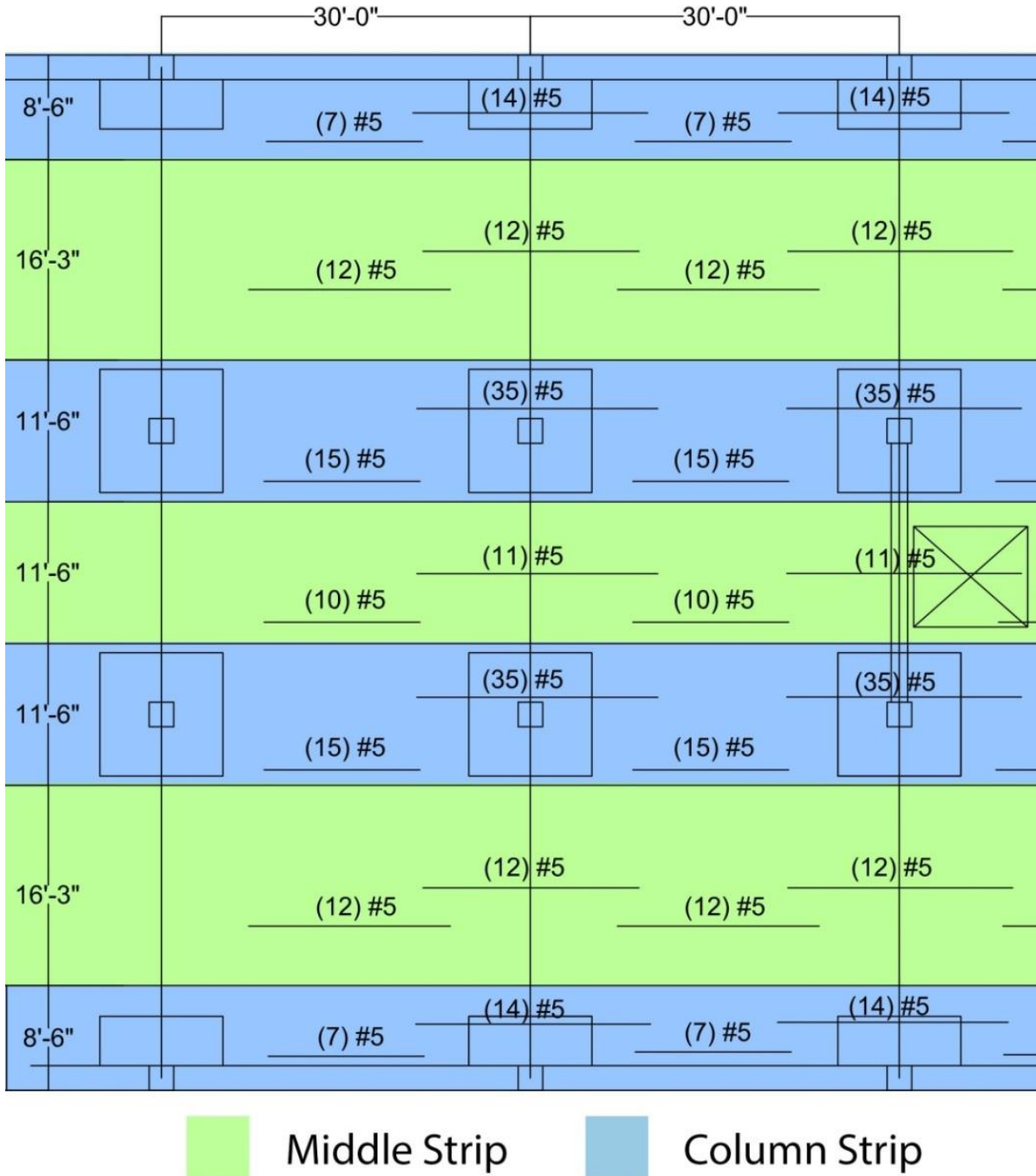
E-W Gridlines
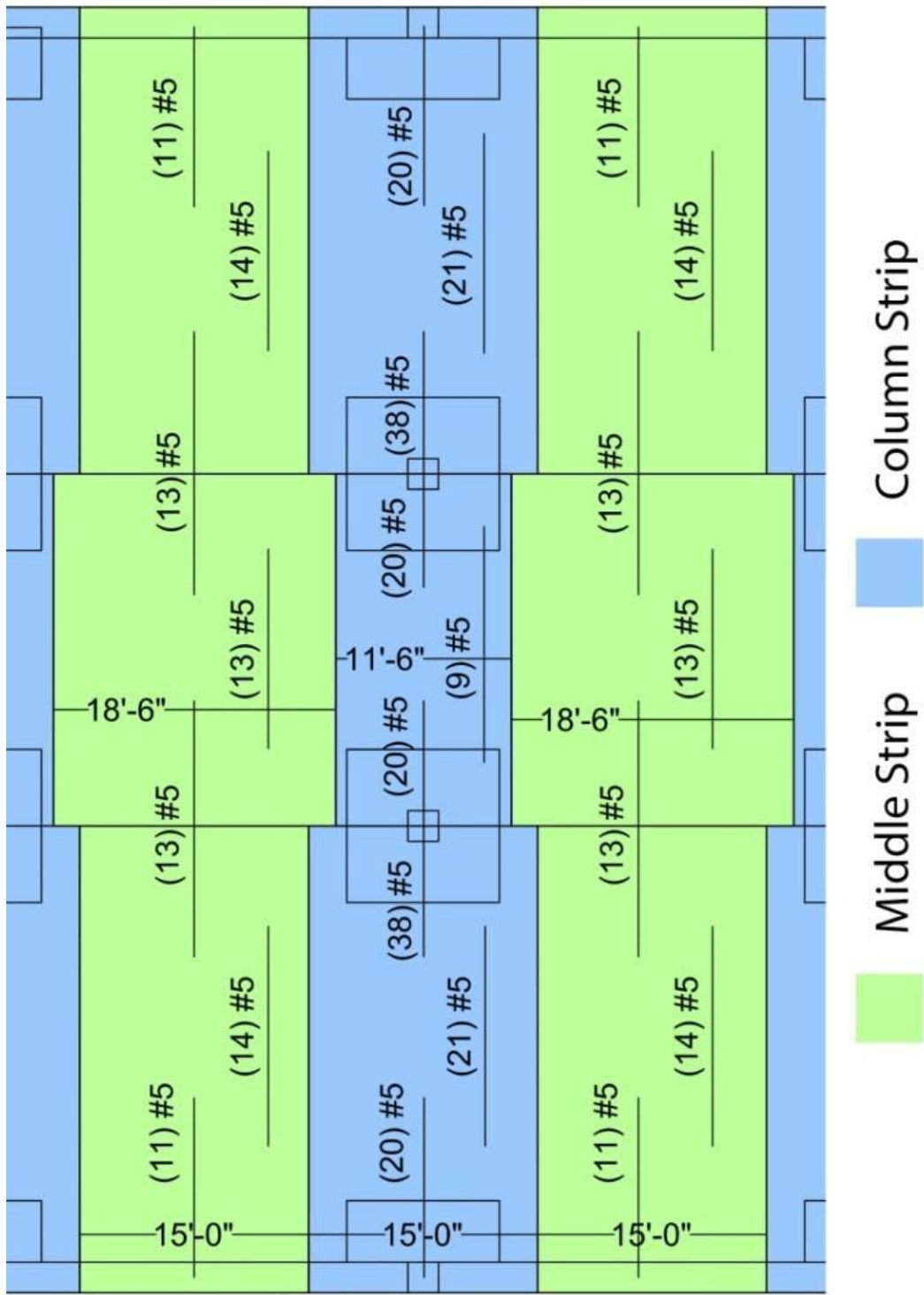
DDM vs EFM - Reinforcing

Figure 41: spSlab input

Column Strip Flexural Reinforcement

■ Direct Design Method

Middle Strip Flexural Reinforcement

■ Equivalent Frame Method
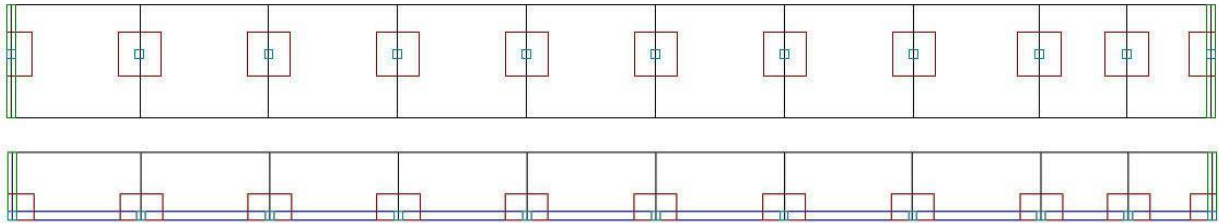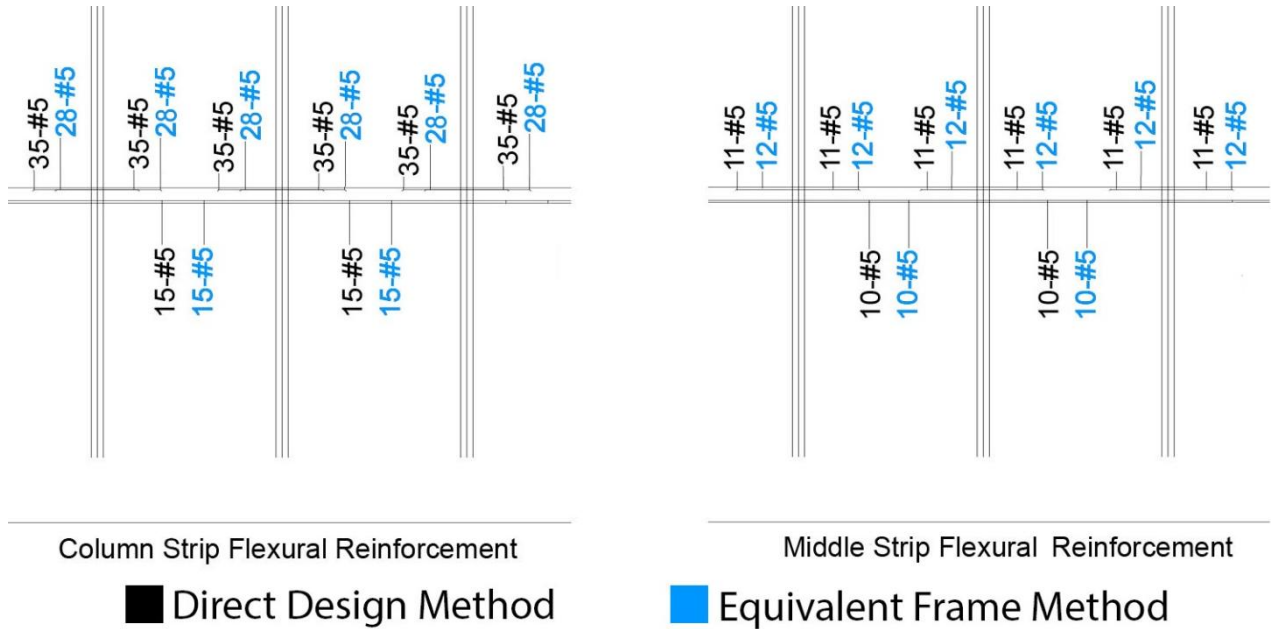
Figure 40: Interior frame

Column Strip Flexural Reinforcement

■ Direct Design Method

Middle Strip Flexural Reinforcement

■ Equivalent Frame Method
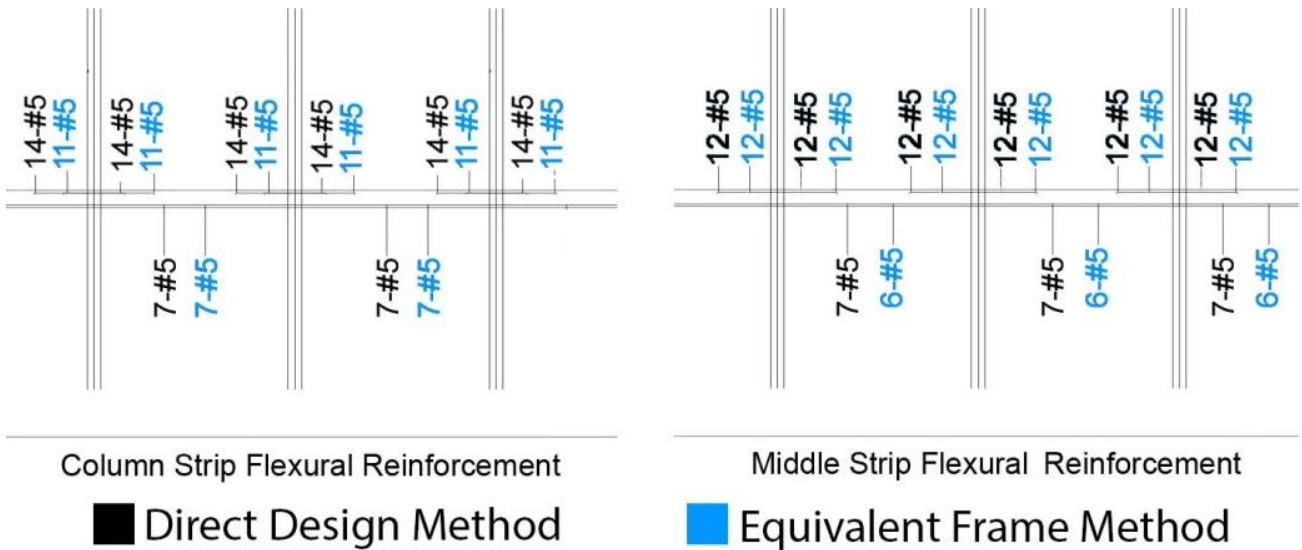
Figure 42: Exterior frame

Appendix E: Slab Opening Details
N-S Details
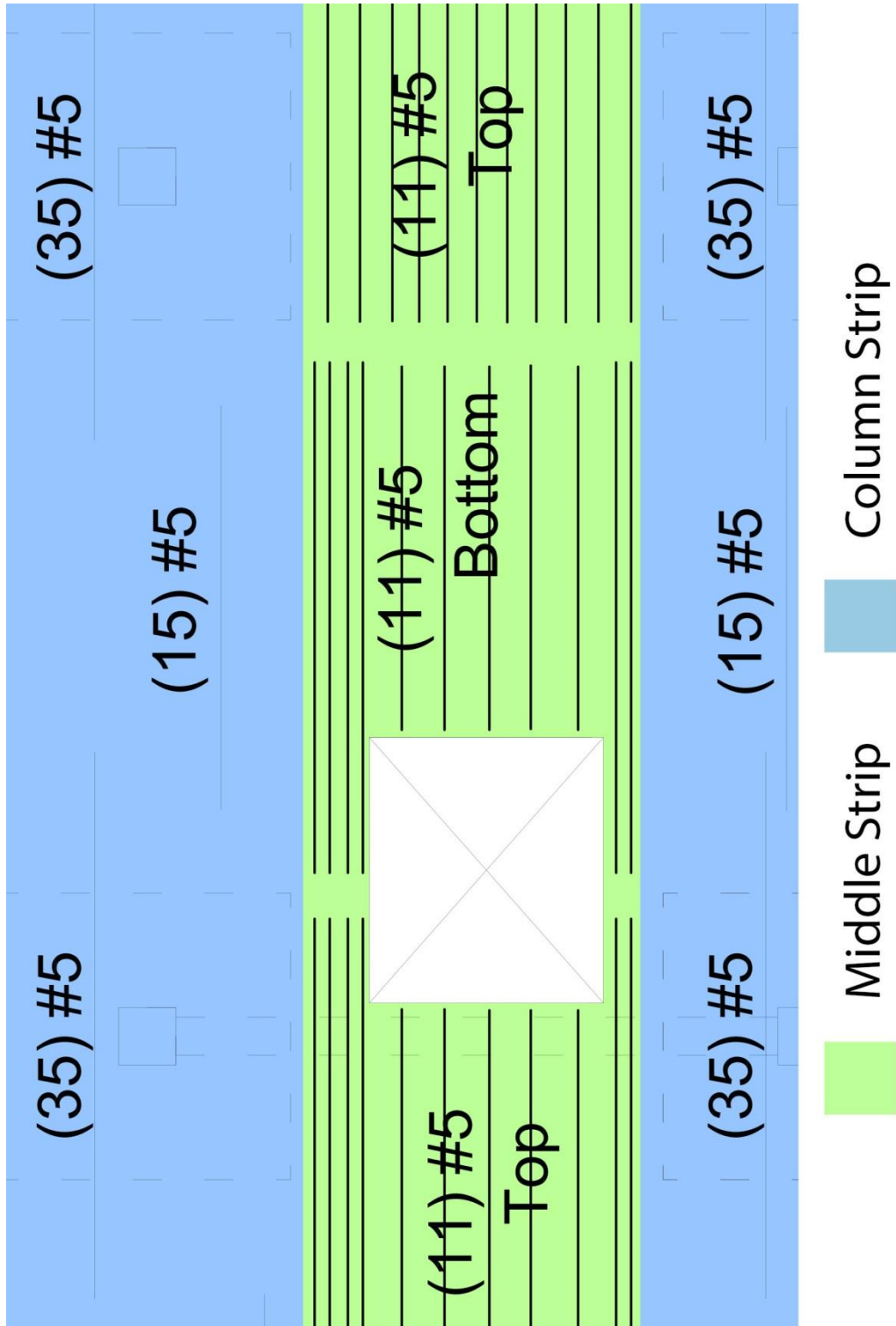
E-W Details
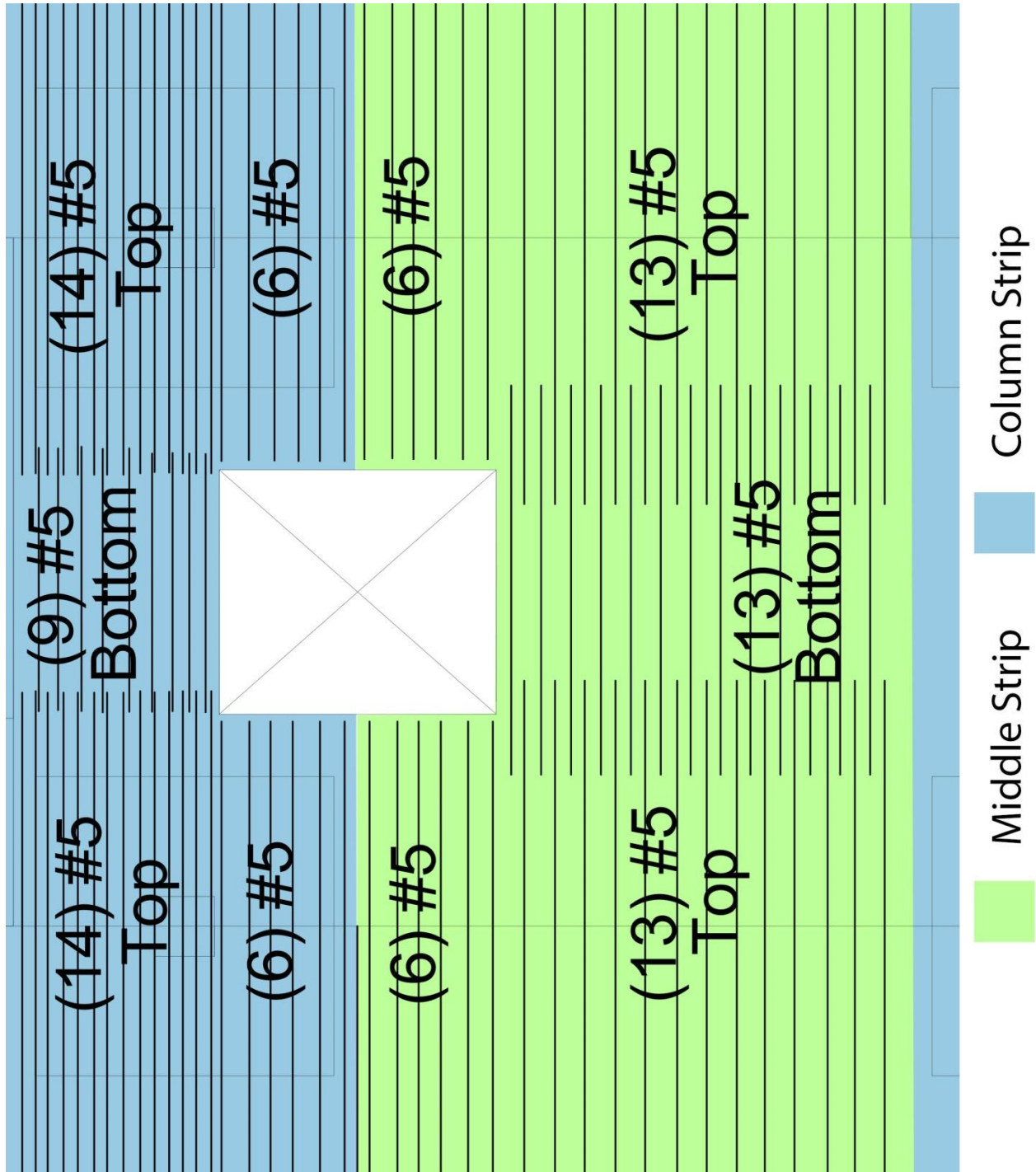
Appendix F: Sample Excel Spreadsheet – Slab Calculations

								No Beams			With Beams		
								Coeff	M			Coeff	M
	Column Strip Interior Neg. Moments						Column Strip	0.814	241.64	→	Column Strip	0.620	149.83
	l_2/l_1	0.5	0.525	1	0.525	2				→	Beam	0.380	91.82
	$\alpha_f L_2/L_1 = 0$	75	75	75	75	75	Middle Strip	0.186	55.33				
	0.4470	x	81.37	x	81.37	x							
	$\alpha_f L_2/L_1 > 1$	90	89.25	75	89.25	45							
	Column Strip Exterior Neg. Moments						Column Strip	0.919	272.97	→	Column Strip	0.620	169.25
$\alpha_f L_2/L_1 = 0$	l_2/l_1	0.5	0.525	1	0.525	2				→	Beam	0.380	103.72
	$\beta_t = 0$	100	100	100	100	100	Middle Strip	0.081	24.00				
	0.8082	x	91.92	x	91.92	x							
	$\beta_t = 2.5$	75	75	75	75	75							
	Column Strip Exterior Neg. Moments						Column Strip	0.940	279.08	→	Column Strip	0.620	173.04
$0 < \alpha_f L_2/L_1 < 1$	l_2/l_1	0.5	0.525	1	0.525	2				→	Beam	0.380	106.04
	$\alpha_f L_2/L_1 = 0$	x	91.92	x	91.92	x	Middle Strip	0.060	17.89				
	0.4470	x	93.98	x	93.98	x							
	$\alpha_f L_2/L_1 = 1$	x	96.52	x	96.52	x							
	Column Strip Exterior Neg. Moments						Column Strip	0.965	286.65	→	Column Strip	0.620	177.73
$\alpha_f L_2/L_1 > 1$	l_2/l_1	0.5	0.525	1	0.525	2				→	Beam	0.380	108.92
	$\beta_t = 0$	100	100	100	100	100	Middle Strip	0.035	10.32				
	0.8082	x	96.52	x	96.52	x							
	$\beta_t = 2.5$	90	89.25	75	89.25	45							
	Column Strip Positive Moments						Column Strip	0.731	116.85	→	Column Strip	0.620	72.45
	l_2/l_1	0.5	0.525	1	0.525	2				→	Beam	0.380	44.40
	$\alpha_f L_2/L_1 = 0$	60	60	60	60	60	Middle Strip	0.269	43.05				
	0.4470	x	73.08	x	73.08	x							
	$\alpha_f L_2/L_1 > 1$	90	89.25	75	89.25	45							
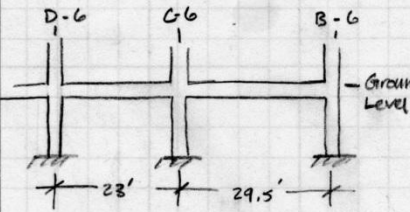
L_1	30	ft
L_n	28	ft
L_2	15.75	ft
h	10	in
L	80	psf
D	15	psf
Self Wt	125	psf
S	94	psf
L_R	30	psf
q_u	0.296	ksf
M_o	456.88	k-ft
Span Type	1	
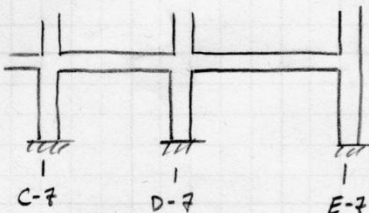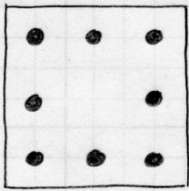
I_b (calc)	13410.67	in^3
I_s	15750	in^4
α_f	0.8515	
$\alpha_f L_2/L_1$	0.4470	
C	25457.55	
β_t	0.808176	

		Int M_u^-	M_u^+	Int M_u^-
1	Interior	0.65	0.35	0.65
		Int M_u^-	M_u^+	Ext M_u^-
2	End No Edge Beam	0.7	0.52	0.26
3	End w/ Edge Beam	0.7	0.5	0.3
4	Ext Edge Fully Restrained	0.65	0.35	0.65
		M_u^-	M_u^+	M_u^-
	Coeff (from above)	0.65	0.35	0.65
	M =	297	160	297

Figure 43: Input data

Appendix G: Column Design Hand Calculations

Column Design | Final Report | Andrew Voorhees | 1

C-6 Interior Column (Ground Floor)

D-6   G-6   B-6

— Ground Level

├─ 23' ─┼─ 29.5' ─┤

Unbalanced Moment
$M_u = 453.47 - 264.43 = \boxed{189.04}$ ᶠᵗᵏ

$P_u = \boxed{2219.81}$ ᵏ from Excel Tabulations

$e = \dfrac{M_u}{P_u} = \dfrac{189.04 \times 12}{2219.81} = 1.02''$

*Assume $d' = 2.5''$

$\dfrac{(h - 2(d'))}{h} = \dfrac{24'' - 2(2.5'')}{24} = 0.79 = \gamma$ $e/h = 1.02/24'' = 0.0425$

$\dfrac{\phi P_n}{bh} = \dfrac{2219.81}{24(24)} = 3.85$

$\dfrac{\phi M_n}{bh^2} = \dfrac{189.04 \times 12}{24(24^2)} = 0.164$

Using MacGregor + Wight Design Aids

$f'_c = 6000$ psi $f_y = 60$ ksi

$\gamma = 0.75$ $\gamma = 0.90$
$\rho = 4.3\%$ $\rho = 4.3\%$

$\therefore \gamma = 0.79$
$\rho = 4.3\% < 5\%$ ✓ok

$A_s = \dfrac{4.3}{100}(24 \times 24) = 24.77 \text{ in}^2 \Longrightarrow$ #10's — 20 bars
                                                        #11's — 16 bars

Use (16) #11's , #4 ties   use ↑

$S_{ties} \leq \begin{cases} 16(1.410) = 22.56 \leftarrow \text{controls} \\ 48(0.5) = 24 \\ 24 \end{cases}$ use ties @ 18"

check b_{min}
$= 1.5''(2) + 2(0.5'') + 5(1.410) + 4(1.5 \times 1.410) = 19.51 < 24''$ ✓ok

$\boxed{\therefore \text{ use (16) #11's on 4 faces with #4 ties @ 18''}}$

Column Design | Final Report   Andrew Voorhees   2

B-7 Edge Column (Ground Floor)

Unbalanced Moment

$M_u = 242$ #k

$P_u = 1429$ k   Tabulated in Excel

C-7   D-7   E-7

$e = \dfrac{M_u}{P_u} = \dfrac{242 \times 12}{1429} = 2.03''$

assume $d' = 2.5''$

$\dfrac{h - 2(d')}{h} = \dfrac{24 - 2(2.5)}{24} = 0.79 = \gamma$ $e/h = 2.03/24 = 0.085$

$\dfrac{\phi P_n}{bh} = \dfrac{1429}{24(24)} = 2.48$

$\dfrac{\phi M_n}{bh} = \dfrac{242 \times 12}{24(24)^2} = 0.21$

using Wight & MacGregor Design Aids

$f'_c = 6000$ psi $f_y = 60$ ksi

$\gamma = 0.75$ $\gamma = 0.90$

$\rho = 1.0\%$ $\rho = 1.0\%$

∴ @ $\gamma = 0.79 \rightarrow \rho = 1.0\%$

$A_s = \rho bh = 0.01(24 \times 24) = 5.76$ in²   → $\rho = 1.10\%$

use (8) #8; ,#4 ties   → (8) #8 = 6.32 in² ← use #8;
        @ 18"   (6) #9 = 6.0 in²
                       (5) #10 = 6.35

bar spacing
$24 - [\, 2(1.5'') + 2(0.5) + 2(0.5)\,] = 19''$

$19/2 = 9.5 - 2(0.5) = 8.5''$ clr. spacing ✓ok

use (8) #8 with #4 ties @ 18"

Appendix H: sp Column Output
Column C-6

24 x 24 in

Code: ACI 318-11

Units: English

Run axis: About X-axis

Run option: Design

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 01/29/13

Time: 15:14:09

P (kip)
3000
(Pmax)   (Pmax)
1
fs=0   fs=0
fs=0.5fy   fs=0.5fy
-1200   1200
Mx (k-ft)
(Pmin)   (Pmin)
-1500

spColumn v4.81. Licensed to: Penn State University. License ID: 59919-1033951-4-22545-2CF68

File: F:\Spring\Columns\C - 6 (2).col

Project: Cooper Hospital

Column: C - 6   Engineer: Andrew

f'c = 6 ksi	fy = 60 ksi	Ag = 576 in^2	16 #11 bars
Ec = 4415 ksi	Es = 29000 ksi	As = 24.96 in^2	rho = 4.33%
fc = 5.1 ksi		Xo = 0.00 in	Ix = 27648 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 27648 in^4
Beta1 = 0.75		Min clear spacing = 3.24 in	Clear cover = 2.00 in
Confinement: Tied			
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65			

Column B-7

24 x 24 in

Code: ACI 318-11

Units: English

Run axis: About X-axis

Run option: Design

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 02/05/13

Time: 14:10:47

P (kip)
2500

(Pmax)   (Pmax)

fs=0   fs=0

+1

fs=0.5fy   fs=0.5fy

-700   700

Mx (k-ft)

(Pmin)   (Pmin)

-500

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File: F:\Spring\Columns\B-7.col

Project: Cooper Hospital

Column: C - 6   Engineer: Andrew

f'c = 6 ksi	fy = 60 ksi	Ag = 576 in^2	8 #8 bars
Ec = 4415 ksi	Es = 29000 ksi	As = 6.32 in^2	rho = 1.10%
fc = 5.1 ksi		Xo = 0.00 in	Ix = 27648 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 27648 in^4
Beta1 = 0.75		Min clear spacing = 8.50 in	Clear cover = 2.00 in
Confinement: Tied			
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65			

Column C-6 Upper Floor

24 x 24 in

Code: ACI 318-11

Units: English

Run axis: About X-axis

Run option: Design

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 02/05/13

Time: 15:05:13

P (kip)
1800

(Pmax)   (Pmax)

fs=0   fs=0

1

fs=0.5fy   fs=0.5fy

-700   700

Mx (k-ft)

(Pmin)   (Pmin)
-600

spColumn v4.81. Licensed to: Penn State University. License ID: 59919-1033951-4-22545-2CF68

File: F:\Spring\Columns\C-6 (6th floor).col

Project:

Column: C-6 6th Flr   Engineer:

f'c = 4 ksi	fy = 60 ksi	Ag = 576 in^2	12 #8 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 9.48 in^2	rho = 1.65%
fc = 3.4 ksi		Xo = 0.00 in	Ix = 27648 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 27648 in^4
Beta1 = 0.85		Min clear spacing = 5.33 in	Clear cover = 2.00 in
Confinement: Tied			
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65			

Appendix I: Column Load Calculations

C - 6 Interior Column													
Level	Trib Area	Live Load	Reduced Live Load	Dead	Super	Snow	1.6 L	1.2 D	1.6 S	1.6 L x A_T	1.2 D x A_T	1.6 S x A_T	1.2 x Swt.	Total Wt.
Penthouse Roof	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Mech Platform	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Roof	787.5	30	15.52	125	30	94	-	186	150	-	146.475	118.44	-	264.92
10	787.5	80	41.38	125	15	-	66	168	-	52.14	132.3	-	8.64	457.99
9	787.5	80	41.38	125	15	-	66	168	-	52.14	132.3	-	8.64	651.07
8	787.5	80	41.38	125	15	-	66	168	-	52.14	132.3	-	8.64	844.15
7	787.5	80	41.38	125	15	-	66	168	-	52.14	132.3	-	8.64	1037.23
6	787.5	80	41.38	125	15	-	66	168	-	52.14	132.3	-	8.64	1230.31
5	615	80	44.19	125	15	-	71	168	-	43.49	103.32	-	-	
	172.5	100	80	-	-	-	128	-	-	22.08	-	-	8.64	1407.84
4	615	80	44.19	125	15	-	71	168	-	43.49	103.32	-	-	
	172.5	100	80	-	-	-	128	-	-	22.08	-	-	8.64	1585.37
3	442.5	100	80	125	15	-	128	168	-	56.64	74.34	-	-	
	345	125	100	-	-	-	160	-	-	55.20	-	-	9.36	1780.91
2	787.5	100	80	125	15	-	128	168	-	100.80	132.3	-	9.36	2023.37
Ground	345	100	80	125	15	-	128	168	-	44.16	57.96	-	-	
	442.5	150	120	-	-	-	192	-	-	84.96	-	-	9.36	2219.81
										Sum (No Col DL)	2131.25		Sum	**2219.81**

B - 2 Corner Column													
Level	Trib Area	Live Load	Reduced Live Load	Dead	Super	Snow	1.6 L	1.2 D	1.6 S	1.6 L x A_T	1.2 D x A_T	1.6 S x A_T	1.2 x Swt.	Total Wt.
Penthouse Roof	87.5	30	31.55	125	15	94	-	168	150	-	14.7	13.16	-	27.86
Mech Platform	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Roof	221.25	30	22.63	125	30	94	-	186	150	-	41.1525	33.276	-	
	87.5	100	80				80	-	-	7.00	-	-	8.64	117.9285
10	221.25	80	60.34	125	15	-	97	168	-	21.36	37.17	-	-	
	87.5	100	80				128	-	-	11.20	-	-	8.64	196.298
9	221.25	80	60.34	125	15	-	97	168	-	21.36	37.17	-	-	
	87.5	100	80				128	-	-	11.20	-	-	8.64	274.6675
8	221.25	80	60.34	125	15	-	97	168	-	21.36	37.17	-	-	
	87.5	100	80				128	0	-	11.20	-	-	8.64	353.037
7	221.25	80	60.34	125	15	-	97	168	-	21.36	37.17	-	-	
	87.5	100	80				128	0	-	11.20	-	-	8.64	431.4065
6	221.25	80	60.34	125	15	-	97	168	-	21.36	37.17	-	-	
	87.5	100	80				128	-	-	11.20	-	-	8.64	509.776
5	221.25	80	60.34	125	15	-	97	168	-	21.36	37.17	-	-	
	87.5	100	80				128	-	-	11.20	-	-	8.64	588.1455
4	221.25	80	60.34	125	15	-	97	168	-	21.36	37.17	-	-	
	87.5	100	80				128	-	-	11.20	-	-	8.64	667.235
3	308.75	100	80	125	15	-	128	168	-	39.52	51.87	-	9.36	767.985
2	308.75	100	80	125	15	-	128	168	-	39.52	51.87	-	9.36	868.735
Ground	640.625	100	80	125	15	-	128	168	-	82.00	107.63	-	9.36	1067.72
										Sum (No Col DL)	969.80		Sum	**1067.72**

B-7 Edge Column													
Level	Trib Area	Live Load	Reduced Live Load	Dead	Super	Snow	1.6 L	1.2 D	1.6 S	1.6 L x A_T	1.2 D x A_T	1.6 S x A_T	1.2 x S.wt	Total Wt.
Penthouse Roof	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Mech Platform	-	-	-	-	-	-	-	-	-	-	-	-	-	-
Roof	442.5	30	18.20	125	30	94	-	186	150	-	82.305	66.552	8.64	157.50
10	442.5	80	48.52	125	15	-	78	168	-	34.35	74.34	-	8.64	274.83
9	442.5	80	48.52	125	15	-	78	168	-	34.35	74.34	-	8.64	392.17
8	442.5	80	48.52	125	15	-	78	168	-	34.35	74.34	-	8.64	509.50
7	442.5	80	48.52	125	15	-	78	168	-	34.35	74.34	-	8.64	626.83
6	442.5	80	48.52	125	15	-	78	168	-	34.35	74.34	-	8.64	744.17
5	442.5	80	48.52	125	15	-	78	168	-	34.35	74.34	-	8.64	861.50
4	442.5	80	48.52	125	15	-	78	168	-	34.35	74.34	-	9.36	979.56
3	442.5	100	80	125	15	-	128	168	-	56.64	74.34	-	9.36	1119.90
2	442.5	100	80	125	15	-	128	168	-	56.64	74.34	-	9.36	1260.24
Ground	442.5	150	120	125	15	-	192	168	-	84.96	74.34	-	9.36	1428.90
										Sum (No Col DL)	1330.98		Sum	**1428.90**

Appendix J: Etabs Column Output

```
ACI 318-08/IBC 2009 COLUMN SECTION DESIGN  Type: Non Sway  Units: Kip-ft  (Summary)

Level        :  1ST FLOOR              L=15.000
Element      :  C40                    B=2.000       D=2.000        dc=0.208
Section ID   :  C-6                    E=576000.000  fc=864.000     Lt.Wt. Fac.=1.000
Combo ID     :  COMB1                  fy=8640.000   fys=8640.000
Station Loc  :  15.000                 RLLF=0.800

Phi(Compression-Spiral): 0.750
Phi(Compression-Tied):   0.650
Phi(Tension Controlled): 0.900
Phi(Shear):              0.750
Phi(Seismic Shear):      0.600
Phi(Joint Shear):        0.850


AXIAL FORCE & BIAXIAL MOMENT CHECK FOR PU, M2, M3
          Capacity      Design       Design       Design      Minimum     Minimum
          Ratio         Pu           M2           M3          M2          M3
          0.808         2214.408     0.000        -287.316    198.995     198.995

AXIAL FORCE & BIAXIAL MOMENT FACTORS
                        Cn           Delta_ns     Delta_s      K           L
                        Factor       Factor       Factor       Factor      Length
     Major Bending(M3)  1.000        1.444        1.000        1.000       15.000
     Minor Bending(M2)  1.000        1.444        1.000        1.000       15.000

SHEAR DESIGN FOR V2,V3
                        Rebar        Shear        Shear        Shear       Shear
                        Av/s         Vu           phi*Vc       phi*Vs      Vp
     Major Shear(V2)    0.000        0.000        154.102      0.000       0.000
     Minor Shear(V3)    0.000        0.000        154.102      0.000       0.000

JOINT SHEAR DESIGN
                        Joint Shear  Shear        Shear        Shear       Joint
                        Ratio        VuTop        VuTot        phi*Vc      Area
     Major Shear(V2)    N/A          N/A          N/A          N/A         N/A
     Minor Shear(V3)    N/A          N/A          N/A          N/A         N/A

(6/5) BEAM/COLUMN CAPACITY RATIOS
                        Major        Minor
                        Ratio        Ratio
                        N/A          N/A


Notes:
   N/A: Not Applicable
   N/C: Not Calculated
   N/N: Not Needed
```

Appendix K: Wind Loads

Level	Story Height, hx (ft)	Trib Area (SF)	Story Force, F_x (k)	Overturning Moment (k-ft)	Trib Area (SF)	Story Force, F_y (k)	Overturning Moment (k-ft)
			Wind Forces				
			N-S			**E-W**	
Ground	0	559	16.35	0.00	1852.5	68.56	0.00
2	13	1118	33.17	431.18	3705	107.78	1401.12
3	26	1118	34.52	897.43	3705	141.26	3672.81
4	39	1075	35.35	1378.51	3562.5	142.39	5553.40
5	51	1032	36.63	1868.12	3420	146.33	7462.98
6	63	1032	37.77	2379.53	3420	149.83	9439.51
7	75	1032	38.97	2922.91	3420	153.52	11514.12
8	87	1032	39.85	3466.78	3420	156.21	13590.22
9	99	1032	40.72	4031.67	3420	158.90	15730.83
10	111	1032	41.44	4599.49	3420	161.09	17880.45
Roof	123	2150	88.65	10903.80	7125	342.72	42154.56
Sum			**443.41**	**32,879.42**		**1728.60**	**128,400.00**

Appendix L: Wind Drift

	Level	P_x				P_y				Δ_{allow}
		δ_x	δ_y	Δ_x	Δ_y	δ_x	δ_y	Δ_x	Δ_y	
Case 1	Roof	1.0924	-0.0591	0.0483	-0.0049	-0.0262	0.4322	-0.0024	0.0435	0.24
	10	1.0441	-0.0542	0.0660	-0.0052	-0.0238	0.3887	-0.0025	0.0450	0.24
	9	0.9781	-0.0490	0.0807	-0.0055	-0.0213	0.3437	-0.0026	0.0456	0.24
	8	0.8974	-0.0435	0.0944	-0.0058	-0.0187	0.2981	-0.0027	0.0462	0.24
	7	0.8030	-0.0377	0.1081	-0.0061	-0.0160	0.2519	-0.0028	0.0461	0.24
	6	0.6949	-0.0316	0.1213	-0.0064	-0.0132	0.2058	-0.0027	0.0452	0.24
	5	0.5736	-0.0252	0.1302	-0.0068	-0.0105	0.1606	-0.0027	0.0428	0.24
	4	0.4434	-0.0184	0.1521	-0.0085	-0.0078	0.1178	-0.0023	0.0416	0.26
	3	0.2913	-0.0099	0.1601	-0.0065	-0.0055	0.0762	-0.0027	0.0368	0.26
	2	0.1312	-0.0034	0.1300	-0.0039	-0.0028	0.0394	-0.0028	0.0394	0.26
	Ground	0	0	0	0	0	0	0	0	0

	Level	0.75 P_x (+e_x)				0.75 P_x (-e_x)				Δ_{allow}
		δ_x	δ_y	Δ_x	Δ_y	δ_x	δ_y	Δ_x	Δ_y	
Case 2	Roof	0.8224	0.0647	0.0368	0.0058	0.8512	-0.1534	0.0388	-0.0131	0.24
	10	0.7856	0.0589	0.0500	0.0061	0.8124	-0.1403	0.0522	-0.0139	0.24
	9	0.7356	0.0528	0.0611	0.0063	0.7602	-0.1264	0.0633	-0.0146	0.24
	8	0.6745	0.0465	0.0715	0.0065	0.6969	-0.1118	0.0738	-0.0153	0.24
	7	0.6030	0.0400	0.0815	0.0067	0.6231	-0.0965	0.0841	-0.0158	0.24
	6	0.5215	0.0333	0.0914	0.0067	0.5390	-0.0807	0.0942	-0.0163	0.24
	5	0.4301	0.0266	0.0977	0.0062	0.4448	-0.0644	0.1010	-0.0165	0.24
	4	0.3324	0.0204	0.1131	0.0060	0.3438	-0.0479	0.1183	-0.0186	0.26
	3	0.2193	0.0144	0.1197	0.0061	0.2255	-0.0293	0.1238	-0.0159	0.26
	2	0.0996	0.0083	0.0996	0.0083	0.1017	-0.0134	0.1017	-0.0134	0.26
	Ground	0	0	0	0	0	0	0	0	0
	Level	0.75 P_y (+e_y)				0.75 P_y (-e_y)				Δ_{allow}
		δ_x	δ_y	Δ_x	Δ_y	δ_x	δ_y	Δ_x	Δ_y	
	Roof	-0.0515	0.4332	-0.0043	0.0421	0.0123	0.2873	0.0008	0.0283	0.24
	10	-0.0472	0.3911	-0.0046	0.0437	0.0115	0.2590	0.0008	0.0289	0.24
	9	-0.0426	0.3474	-0.0048	0.0447	0.0107	0.2301	0.0009	0.0294	0.24
	8	-0.0378	0.3027	-0.0049	0.0455	0.0098	0.2007	0.0009	0.0298	0.24
	7	-0.0329	0.2572	-0.0052	0.0459	0.0089	0.1709	0.0010	0.0298	0.24
	6	-0.0277	0.2113	-0.0052	0.0453	0.0079	0.1411	0.0011	0.0294	0.24
	5	-0.0225	0.1660	-0.0055	0.0435	0.0068	0.1117	0.0014	0.0285	0.24
	4	-0.0170	0.1225	-0.0058	0.0435	0.0054	0.0832	0.0025	0.0321	0.26
	3	-0.0112	0.0790	-0.0058	0.0386	0.0029	0.0511	0.0017	0.0253	0.26
	2	-0.0054	0.0404	-0.0054	0.0404	0.0012	0.0258	0.0012	0.0258	0.26
	Ground	0	0	0	0	0	0	0	0	0

	Level	0.75 Px + 0.75 Py				0.75 Px - 0.75 Py				Δallow
		δx	δy	Δx	Δy	δx	δy	Δx	Δy	
Case 3	Roof	2.2995	0.3217	0.1047	0.0303	0.8750	-0.9655	0.0414	-0.0979	0.24
	10	2.1948	0.2914	0.1433	0.0312	0.8336	-0.8676	0.0549	-0.1010	0.24
	9	2.0515	0.2602	0.1754	0.0318	0.7787	-0.7666	0.0662	-0.1027	0.24
	8	1.8761	0.2284	0.2058	0.0324	0.7125	-0.6639	0.0767	-0.1040	0.24
	7	1.6703	0.1960	0.2352	0.0329	0.6358	-0.5599	0.0869	-0.1037	0.24
	6	1.4351	0.1631	**0.2636**	0.0328	0.5489	-0.4562	0.0969	-0.1017	0.24
	5	1.1715	0.1303	**0.2797**	0.0333	0.4520	-0.3545	0.1035	-0.0965	0.24
	4	0.8918	0.0970	**0.3153**	0.0404	0.3485	-0.2580	0.1189	-0.0939	0.26
	3	0.5765	0.0566	**0.3194**	0.0307	0.2296	-0.1641	0.1256	-0.0812	0.26
	2	0.2571	0.0259	0.2571	0.0259	0.1040	-0.0829	0.1040	-0.0829	0.26
	Ground	0	0	0	0	0	0	0	0	0

	Level	0.563 Px (-ex) + 0.563 Py (-ey)				0.563 Px (-ex) + 0.563 Py (+ey)				Δallow
		δx	δy	Δx	Δy	δx	δy	Δx	Δy	
Case 4	Roof	1.9387	0.8757	0.0963	0.0796	1.8834	0.8656	0.0949	0.0820	0.24
	10	1.8424	0.7961	0.1265	0.0836	1.7885	0.7836	0.1248	0.0861	0.24
	9	1.7159	0.7125	0.1514	0.0867	1.6637	0.6975	0.1496	0.0890	0.24
	8	1.5645	0.6258	0.1750	0.0898	1.5141	0.6085	0.1729	0.0916	0.24
	7	1.3895	0.5360	0.1977	0.0919	1.3412	0.5169	0.1953	0.0930	0.24
	6	1.1918	0.4441	0.2196	0.0925	1.1459	0.4239	0.2160	0.0924	0.24
	5	0.9722	0.3516	0.2328	0.0915	0.9299	0.3315	0.2253	0.0882	0.24
	4	0.7394	0.2601	**0.2636**	0.1018	0.7046	0.2433	0.2476	0.0876	0.26
	3	0.4758	0.1583	**0.2630**	0.0829	0.4570	0.1557	0.2499	0.0767	0.26
	2	0.2128	0.0754	0.2128	0.0754	0.2071	0.0790	0.2071	0.0790	0.26
	Ground	0	0	0	0	0	0	0	0	0
	Level	0.563 Px (+ex) + 0.563 Py (-ey)				0.563 Px (+ex) + 0.563 Py (+ey)				Δallow
		δx	δy	Δx	Δy	δx	δy	Δx	Δy	
	Roof	0.8694	-1.4541	0.0488	-0.1393	0.7437	-1.0141	0.0398	-0.0977	0.24
	10	0.8206	-1.3148	0.0602	-0.1457	0.7039	-0.9164	0.0504	-0.1014	0.24
	9	0.7604	-1.1691	0.0693	-0.1499	0.6535	-0.8150	0.0592	-0.1040	0.24
	8	0.6911	-1.0192	0.0782	-0.1537	0.5943	-0.7110	0.0676	-0.1062	0.24
	7	0.6129	-0.8655	0.0864	-0.1556	0.5267	-0.6048	0.0755	-0.1073	0.24
	6	0.5265	-0.7099	0.0944	-0.1545	0.4512	-0.4975	0.0829	-0.1062	0.24
	5	0.4321	-0.5554	0.1005	-0.1489	0.3683	-0.3913	0.0862	-0.1027	0.24
	4	0.3316	-0.4065	0.1162	-0.1500	0.2821	-0.2886	0.0977	-0.1113	0.26
	3	0.2154	-0.2565	0.1175	-0.1294	0.1844	-0.1773	0.0995	-0.0893	0.26
	2	0.0979	-0.1271	0.0979	-0.1271	0.0849	-0.0880	0.0849	-0.0880	0.26
	Ground	0	0	0	0	0	0	0	0	0

Appendix M: Seismic Parameters

Seismic Parameters | Final Report   Andrew Voorhees   1

Ordinary Reinforced Concrete Moment Frames (N-S)

$R = 3$ $C_d = 2.5$ $\Omega_0 = 3$

Ordinary Reinforced Concrete Shear Walls (E-W)

$R = 4$ $C_d = 4$ $\Omega_0 = 2.5$

$I = 1.5$
use group III
Design Category C
Site Class D

$S_s = 0.267$
$S_1 = 0.059$
$S_{DS} = 0.282$
$S_{D1} = 0.095$
$T_L = 6$ sec

Approximate Fundamental Period for N-S direction

$T_a = C_t h_n{}^x$ for conc Moment-resiting Frames (Table 12.8-2)

$T_a = 0.016 (123)^{0.9}$

$C_t = 0.016$
$x = 0.9$
$h_n = 123^{ft}$

$= 1.22$ sec

Fundamental Period for N-S direction

$T = T_a C_u$ (from Table 12.8-1)

$C_u = 1.7$

$T = 1.22 (1.7) = 2.074$ sec → shall not exceed

Seismic Response Coefficient (N-S)

$C_s = \dfrac{S_{DS}}{(R/I)} = \dfrac{0.282}{3(1.5)} = 0.141$

$T < T_L \therefore C_s \leq \dfrac{S_{D1}}{T(R/I)} = \dfrac{0.095}{1.22(3/1.5)} = 0.0389 < 0.141 = C_s$

controls

$C_s = 0.0389$ N-S direction

Seismic Parameters   Final Report   Andrew Voorhees   2

Approximate Fundamental Period for E-W direction

$$T_a = C_t h_n^x$$

for concrete shear walls (other systems)
$$C_t = 0.02$$
$$x = 0.75$$

$$= 0.02 (123)^{0.75}$$

$$= 0.739 \text{ sec}$$

Fundamental Period for E-W direction

$$T = T_a C_u = 1.7 (0.739) = \boxed{1.256 \text{ sec}} \leftarrow \text{shall not exceed}$$

Seismic Response Coefficient

$$C_s = \frac{S_{DS}}{(R/I)} = 0.141$$

$$T < T_L \therefore C_s \leq \frac{S_{D1}}{T(R/I)} = \frac{0.095}{0.739(3/1.5)} = 0.0643 < 0.141 = C_s$$

controls

$$\boxed{C_s = 0.0643} \quad \text{E-W direction}$$

Base Shears

$$W = 48,794.30^k$$

North-South
Conc. Mom.-Resisting Frames

$$C_s = 0.0389$$

$$V_b = C_s W$$

$$= 0.0389 (48,794.3)$$

$$\boxed{V_b = 1898^k}$$

East-West
Conc. shear walls

$$C_s = 0.0643$$

$$V_b = C_s W$$

$$= 0.0643 (48,794.3)$$

$$\boxed{V_b = 3138^k}$$

These values are much larger than those used for the steel structure's foundation. If time permits the foundations will be evaluated to account for this larger base shear.

Appendix N: Seismic Loads

Building Weight (k)						
Level	Floor (k)	Columns (k)	MEP (k)	SDL (k)	Walls (k)	Total (k)
Ground	4924.75	368.00	196.99	590.97	55.07	6135.78
2	3483.00	368.00	139.32	417.96	119.32	4527.60
3	3643.88	368.00	145.76	437.27	112.86	4707.75
4	3812.00	368.00	152.48	457.44	169.57	4959.49
5	2866.38	368.00	114.66	343.97	205.15	3898.15
6	2866.38	368.00	114.66	343.97	205.15	3898.15
7	2866.38	368.00	114.66	343.97	205.15	3898.15
8	2866.38	368.00	114.66	343.97	205.15	3898.15
9	2866.38	368.00	114.66	343.97	205.15	3898.15
10	2866.38	368.00	114.66	343.97	205.15	3898.15
Roof	3268.38	512.00	115.43	692.58	486.40	5074.79
				Total Building Weight		48,794.30

Seismic Forces N-S						
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	6136	0	0.00	0.00	1898.00	0.00
2nd	13	4528	148194.24	0.01	19.87	1898.00	258.26
3rd	26	4708	395528.29	0.03	53.02	1878.13	1378.59
4th	39	4959	723243.63	0.05	96.95	1825.11	3781.24
5th	51	3898	818754.42	0.06	109.76	1728.16	5597.69
6th	63	3898	1091343.25	0.08	146.30	1618.40	9216.95
7th	75	3898	1383380.14	0.10	185.45	1472.10	13908.75
8th	87	3898	1692795.14	0.12	226.93	1286.65	19742.81
9th	99	3898	2018004.28	0.14	270.52	1059.72	26781.97
10th	111	3898	2357748.46	0.17	316.07	789.19	35083.73
Roof	123	5075	3529301.02	0.25	473.12	473.12	58194.12
Sum		48794	14,158,292.87	1.00	1898.00		173,944.13

Seismic Forces E-W						
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	6136	0	0.00	0.00	3138.00	0.00
2nd	13	4528	80072.78	0.02	51.31	3138.00	666.99
3rd	26	4708	180960.57	0.04	115.95	3086.69	3014.73
4th	39	4959	300212.74	0.06	192.36	2970.74	7502.15
5th	51	3898	318666.80	0.07	204.19	2778.38	10413.55
6th	63	3898	403756.60	0.08	258.71	2574.19	16298.66
7th	75	3898	490825.18	0.10	314.50	2315.48	23587.39
8th	87	3898	579588.53	0.12	371.37	2000.98	32309.54
9th	99	3898	669837.78	0.14	429.20	1629.61	42490.96
10th	111	3898	761412.42	0.16	487.88	1200.41	54154.52
Roof	123	5075	1112017.12	0.23	712.53	712.53	87641.20
Sum		48794	4,897,350.51	1.00	3138.00		278,079.69

473 k →

316 k →

271 k →

227 k →

185 k →

146 k →

110 k →

97 k →

53 k →

20 k →

V = 1898 k

M = 173,944 k-ft
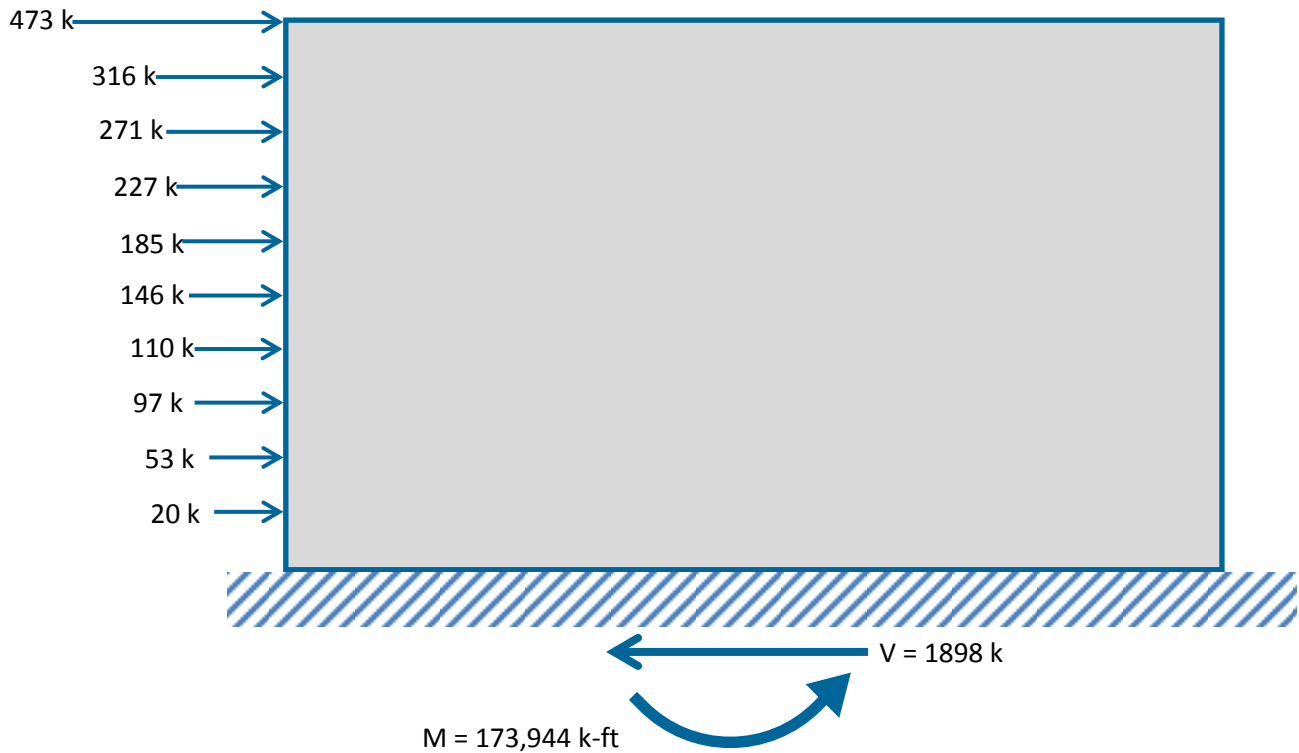
Figure 44: N-S Seismic loads

713 k →

488 k →

429 k →

371 k →

315 k →

259 k →

204 k →

192 k →

116 k →

51 k →

V = 3138 k

M = 278,080 k-ft
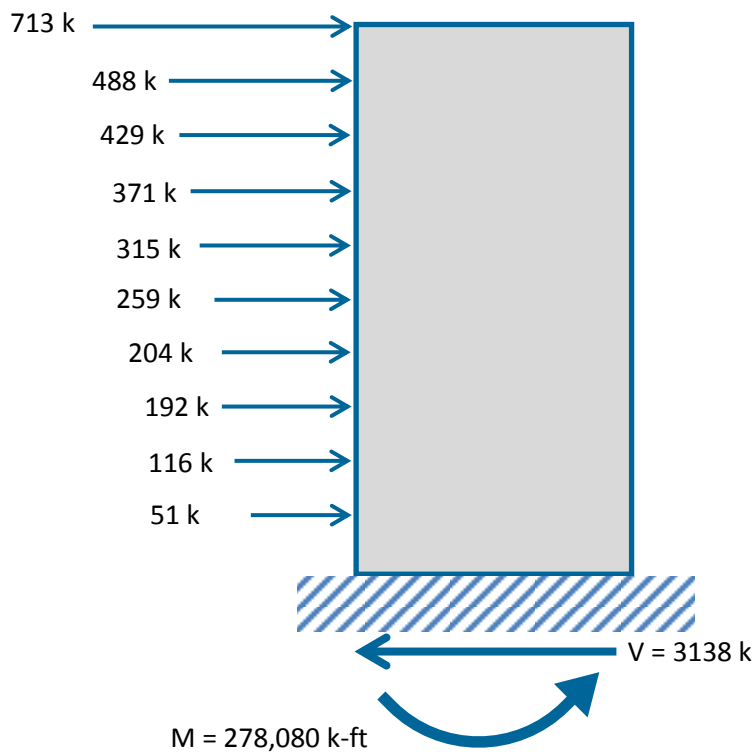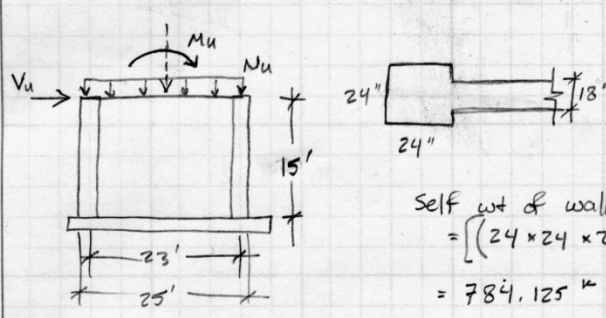
Figure 45: E-W Seismic loads
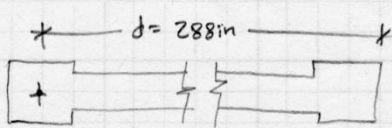
Appendix O: Seismic Drift

		Drift N-S (x - direction)									
	E_X			$E_X + E_{XT}$ (+e_y)			$E_X + E_{XT}$ (-e_y)			
Level	δ_{xe}	$C_d\delta_{xe}/I$	Δ_x	δ_{xe}	$C_d\delta_{xe}/I$	Δ_x	δ_{xe}	$C_d\delta_{xe}/I$	Δ_x	Δ_{allow}
Roof	6.108	10.179	0.434	6.122	10.203	0.435	6.093	10.155	0.432	1.44
10	5.847	9.745	0.595	5.861	9.768	0.597	5.834	9.723	0.593	1.44
9	5.490	9.150	0.775	5.503	9.171	0.777	5.478	9.130	0.773	1.44
8	5.025	8.375	0.934	5.036	8.394	0.936	5.014	8.357	0.932	1.44
7	4.465	7.442	1.066	4.475	7.458	1.069	4.455	7.425	1.064	1.44
6	3.825	6.375	1.171	3.834	6.389	1.173	3.817	6.361	1.168	1.44
5	3.123	5.205	1.248	3.130	5.216	1.251	3.116	5.193	1.246	1.44
4	2.374	3.957	1.458	2.380	3.966	1.466	2.369	3.948	1.451	1.56
3	1.499	2.499	1.391	1.500	2.500	1.392	1.498	2.497	1.390	1.56
2	0.665	1.108	1.108	0.665	1.108	1.108	0.665	1.108	1.108	1.56
Ground	0	0	0	0	0	0	0	0	0	0

		Drift E-W (y - direction)									
	E_Y			$E_Y + E_{YT}$ (+e_x)			$E_Y + E_{YT}$ (-e_x)			
Level	δ_{ye}	$C_d\delta_{ye}/I$	Δ_y	δ_{ye}	$C_d\delta_{ye}/I$	Δ_y	δ_{ye}	$C_d\delta_{ye}/I$	Δ_y	Δ_{allow}
Roof	3.706	9.883	1.034	3.761	10.029	0.970	3.652	9.738	1.098	1.44
10	3.319	8.849	1.098	3.397	9.059	1.123	3.240	8.640	1.074	1.44
9	2.907	7.751	1.105	2.976	7.935	1.131	2.837	7.566	1.080	1.44
8	2.492	6.646	1.101	2.552	6.805	1.126	2.433	6.487	1.075	1.44
7	2.079	5.545	1.078	2.129	5.678	1.104	2.030	5.412	1.052	1.44
6	1.675	4.467	1.033	1.715	4.574	1.059	1.635	4.360	1.008	1.44
5	1.288	3.434	0.961	1.318	3.516	0.985	1.257	3.352	0.937	1.44
4	0.927	2.473	0.926	0.949	2.531	0.894	0.906	2.415	0.958	1.56
3	0.580	1.547	0.775	0.614	1.637	0.821	0.546	1.456	0.729	1.56
2	0.290	0.772	0.772	0.306	0.817	0.817	0.273	0.727	0.727	1.56
Ground	0	0	0	0	0	0	0	0	0	0

Appendix P: Shear Wall Design

Shear Wall Design   Final Report   Andrew Voorhees   1

$V_u = 1076^k$
$M_u = 52,236.2^{k \cdot ft}$
$N_u = 2888.66^k$

Self wt of walls above
$= \left[(24 \times 24 \times 2) + (18 \times 23 \times 12) \right] \div 144 \times 150 \times 123'$
$= 784.125^k$

W_t of slabs + SDL
$= 11 \times \left[(29.5' + 23') \times 30' \right] \times (125 + 15)$
11 floors
$= 2425.5^k$

Total DL $= 3209.63^k$

$N_u = 0.9 (3209.63) = \boxed{2888.66^k}$

Shear Capacity

ACI 318-11 §11.9.6 Minimum of:

$V_c = 3.32 \sqrt{f'_c} \, hd + \dfrac{N_u d}{4 l_w}$ — $d = 0.8 \, l_w = 0.8 \times 25' \times 12''$
$= 240''$

$= 3.3 \sqrt{6000} \, (18)(240) + \dfrac{2888.66 \, (240)}{4 \, (25 \times 12)} \times 1000 = \boxed{1682.00}$

$V_c = \left[0.6 \lambda \sqrt{f'_c} + \dfrac{l_w \left(1.25 \lambda \sqrt{f'_c} + \dfrac{0.2 \, N_u}{l_w h} \right)}{\dfrac{M_u}{N_u} - l_w/2} \right] hd$

$= \left[0.6 \sqrt{6000} + \dfrac{(25 \times 12) \left(1.25 \sqrt{6000} + \dfrac{0.2 (2888.66) \times 1000}{(25 \times 12 \times 18)} \right)}{\dfrac{52,236.2 \times 12}{2888.66} - \dfrac{25 \times 12}{2}} \right] \times 240 \times 18$

$= 67.0$

$= \boxed{4143.17^k}$

$\therefore V_c = \boxed{1682^k}$ $V_u > 0.5 \, \phi \, V_c$ $\therefore$ Reinf shall be provided
by 11.9.9

Shear Wall Design | Final Report | Andrew Voorhees | 2

Horizontal Reinforcement

$$V_u \geq 0.5(0.75)(1682) = 630.75^k$$

but $V_u > 0.75(1682) = 1261.5^k$

∴ Reinf. provided based on $A_{v,min}$

$$\rho_t \geq 0.0025 = \frac{A_v}{sh}$$

$$s \leq \begin{cases} \ell_w/5 = 60" \\ 3h = 54" \\ 18" \leftarrow controls \end{cases}$$

$$0.0025 = \frac{A_v}{18"(18")}$$

$$A_v \geq 0.81 \div 2 = 0.405 \, in^2 \quad per \ curtain$$

$$\#6 \quad A_b = 0.44 \, in^2 \qquad \rho = \frac{0.88 \, in^2}{18(18)} = 0.0027$$
√ok

∴ use 2 curtains of #6 @ 18" o.c.

Vertical Reinforcement

$$\rho_\ell \geq \begin{cases} 0.0025 \\ 0.0025 + 0.5\left(2.5 - \frac{15}{25}\right)(0.0027 - 0.0025) = 0.00269 \end{cases}$$

∴ use 2 curtains of #6 @ 18" o.c. $\rho_\ell = 0.0027$

Flexural Design

$$T = A_s F_y$$

$$C = 0.85 f'_c \, ba$$

$$a = \frac{T + N_u}{0.85 f'_c \, b} = \frac{A_s F_y + N_u}{0.85 f'_c \, b}$$

$$\phi M_n = \phi\left[T\left(d - \frac{a}{2}\right) + N_u\left(\frac{\ell_w - a}{2}\right)\right]$$

substitute for a, solve for A_s

$$\phi M_n = \phi\left[A_s F_y\left(d - \left(\frac{A_s F_y + N_u}{0.85 f'_c \, b(2)}\right)\right) + \frac{N_u}{2}\left(\ell_w - \left(\frac{A_s F_y + N_u}{0.85 f'_c \, b}\right)\right)\right]$$

$$\phi M_n \geq M_u$$

d = 288in

Shear Wall Design | Final Report | Andrew Voorhees   3

$$\frac{M_u}{\phi} = A_s F_y \left(d - \frac{N_u}{0.85 f'_c b (2)} - \frac{A_s F_y}{0.85 f'_c b (2)} \right) + \frac{N_u}{2} \left(l_w - \frac{N_u}{0.85 f'_c b} - \frac{A_s F_y}{0.85 f'_c b} \right)$$

$$\frac{52,236.2 \times 12}{0.9} = 60 A_s \left(288 - \frac{2888.66}{0.85(6)(18)(2)} - \frac{60 A_s}{0.85(6)(18)(2)} \right) +$$

$$+ \frac{2888.66}{2} \left(25 \times 12 - \frac{2888.66}{0.85(6)(18)} - \frac{60 A_s}{0.85(6)(18)} \right)$$

$$696,482.67 = 16,335.99 A_s - 19.61 A_s^2 + 387,850.43 - 944.61 A_s$$

$$0 = -19.61 A_s^2 + 15391.98 A_s - 308,632.24$$

$$\boxed{A_s = 20.59 \, in^2}$$

\# 13's = 16.2 bars ≈ 18 bars
\# 11's = 13.2 bars ≈ 14 bars
↓

$$\boxed{\therefore \text{use } (14) \; \# 11\text{'s}}$$

$A_s = 14 (1.56) = 21.84 \, in^2$

$T = A_s F_y$
$= 14 (1.56)(60)$
$= 1310.4^k$

$$a = \frac{T + N_u}{0.85 f'_c b} = \frac{1310.4 + 2888.66}{0.85(6)(18)} = 45.74''$$

$$\phi M_n = 0.9 \left[1310 \left(288 - \frac{45.74}{2} \right) + 2888.66 \left(300 - \frac{45.74}{2} \right) \right]$$

$$= 643,100^{"k} = 53,591.67^{k ft}$$

$$\phi M_n \geq M_u \qquad \underline{\checkmark \, Ok}$$

Tension controlled ?
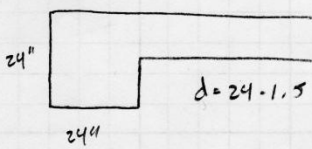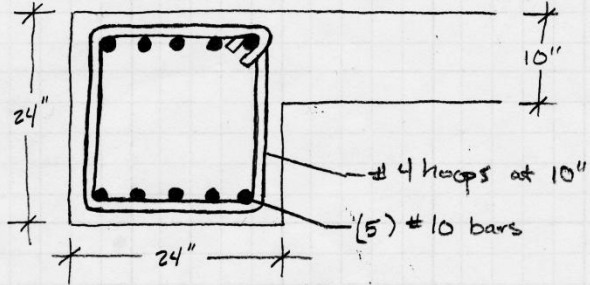
$$c = a / \beta_1 = 45.74 / 0.85 = 60.99'' < 0.375 d = 108''$$

$$\therefore \text{Tension controlled}, \quad \phi = 0.9 \quad \checkmark$$

(14) \#11's

\#6 @ 18"

\#6 @ 18"

18"

24"

Appendix Q: Moment Frame Design

Moment Frame Calcs   Final Report   Andrew Voorhees   1

35.7   71.4   71.4   71.4   71.4   71.4   71.4   71.4   71.4   71.4   35.7

7.6 →   2   3   4   5   6   7   8   9   10   11   12

36.1   72.2   72.2   72.2   72.2   72.2   72.2   72.2   72.2   72.2   36.1

30'   30'   30'   30'   30'   30'   30'   30'   30'   30'

Exterior Frame

Story shear @ second story = 1878^k

Frame takes 38% ⟹ 714^k

Story Force @ second story = 19.87^k

38% ⟹ 7.6^k

shear in ground columns = (714 + 7.6) ÷ 10 = 72.2^k

Compare to etabs output : shear = 60.51^k ✓ ok

Frame B ↗

Etabs distributed forces to other frames ie. lobby
More conservative to use hand calcs.

35.7
7.6 →   2   ↑

36.1

$\Sigma M_2 = 35.7 \times 7' + 36.1 \times 7' - F_{23} \times 15'$

$F_{23} = 33.5^k$

$M_{23} = 33.5 \times 15' = 502.6^{ft\cdot k}$

71.4
33.5   3   ↑ F_{34}

72.2

$\Sigma M_3 = 71.4 \times 7' + 72.2 \times 7' - 33.5 \times 15'$
                                          $- F_{34} \times 15'$

$F_{34} = 33.5^k$

$M = 502.7$ ✓ ok   should be the same

compare with etabs output

Frame E : $M = 277.0^{ft\cdot k}$ → Moment is lower b/c shear is lower

Moment Frame Calcs   Final Report   Andrew Voorhees   **2**

Moments from DDM:

Live = 80 psf   EQ. $M = 502.6^{ftk}$
MEP = 5 psf
SDL = 10 psf
S.wt = 125 psf

Load Combos:
  1.2D + 1.0E + L + 0.2S   ←
  0.9D + 1.0E + 1.6H

$q_u = 1.0(80 \text{ psf}) + 1.2(15 + 125) = 0.248 \text{ ksf}$

$M_o = \dfrac{q_u \ell_2 \ell_n^2}{8} = \dfrac{0.248(15.75')(28)^2}{8} = \boxed{M_o = 382.8 \text{ kft}}$

$M_u^+ = 0.35 M_o \qquad M_u^- = 0.65 M_o$

$\quad = 134^{kft} \qquad\qquad = 249^{kft}$

134

249   249

Distribute to Beam

$M_u^- = 38\% \quad = 94.62^{ftk}$

$M_u^+ = 38\% \sim 50.92^{ftk}$

502.6

$M_u^- = 502.6 + 50.92 = \boxed{553.52^{ftk}}$

Reversible ∴ M = 553.52 at Top and Bottom

Shear: $0.248 \times 15.75 = 3.91 \text{ klf}$

$V_{max} = \dfrac{3.91 \times 28}{2} = 54.7^k$

$V_{from \, EQ} = 33.5^k$

$V_{max} = 88.2^k$

Moment Frame Calcs | Final Report | Andrew Voorhees | 3

Design Edge Beams

$M = 553.52^{ftk}$

24"

$d = 24 - 1.5$

24"

$A_s = \dfrac{M_a}{4d} = \dfrac{553.52}{4(22.5)} = 6.15$

Try #10's $A_s = 1.27 \rightarrow$ Need 5 bars
$A_s = 6.35\ in^2$
Top & Bottom

$a = \dfrac{A_s F_y}{0.85 f'_c b} = \dfrac{6.35(60)}{0.85(4)(24)} = 4.67\ in$

$c = \dfrac{4.67}{0.85} = 5.5"$

$d = 22.5 - 0.635$
$= 21.865"$

$\varepsilon_s = \dfrac{0.003}{5.5}(22.5 - 5.5) = 0.0092 > 0.005 \quad \therefore \phi = 0.9$

$\phi M_n = 0.9(6.35)(21.865 - \dfrac{4.67}{2})(\dfrac{60}{12}) = 558.1^{ftk} > M_u$ \quad V OK

∴ use (5) #10 bars Top and Bottom

10"

24"

#4 hoops at 10"

(5) #10 bars

24"

Shear reinf.

$s = \dfrac{A_v f_y d}{V_s}$

try #4 hoops

$V_c = 2\lambda \sqrt{f'_c}\ b_w d$
$= 2\sqrt{4000}(24)(21.865) = 66.37$

$V_s = \dfrac{V_u}{\phi} - V_c = \dfrac{88.2}{0.75} - 66.37 = \underline{51.23^k}$

$s = \dfrac{(0.20 \times 2)60(21.865)}{51.23} = 10.24"$

∴ use #4 hoops at 10"

Moment Frame (Int.)   Final Report   Andrew Voorhees   4

Interior Frame

$11.3 \times 7' + 11.4 \times 7' - F_{23} \times 15' = 0$

$F_{23} = 10.59^k$

$M = 158.9^{ft \cdot k}$

Moments from DDM:

$q_u = 0.248 \text{ ksf}$ Load Combo $1.2D + 1.0E + L$

$M_o = \dfrac{0.248(28)^2(26.25)}{8} = 638.0^{ft \cdot k}$

$M_u^- = 0.65 M_o$
$M_u^+ = 0.35 M_o$

$M_u^- = 415 + 159 = 574^{ft \cdot k}$

$M_u^+ = 223^{ft \cdot k}$

Distribute to Col. and Middle strips

M_u^- to Col. $= 0.75 (574) = 430.5^{ft \cdot k}$

M_u^- to Mid $= 0.25 (574) = 143.5^{ft \cdot k} \rightarrow$ half to each

M_u^+ to Col $= 0.60 (223) = 133.8^{ft \cdot k}$

M_u^+ to Mid $= 0.40 (223) = 89.2^{ft \cdot k} \rightarrow$ half to each

Reinf designed with excel spreadsheet

Neg. Reinf @ Col. = 41 bars   Pos. Reinf. @ Col. strip. = 12 bars
         @ Mid = 13 bars   @ Mid Strip = 10 bars

The image shows handwritten structural engineering calculations.

Check Col shear due to Lateral Loads   Andrew Voorhees

Check Column B-7 (Part of Moment Frame)

ties → #4's @ 18" oc.

V_u from lateral Load = 72.2^k , N_u = 1428^k

$V_c = 2 \left(1 + \dfrac{N_u}{2000 A_g} \right) \lambda \sqrt{f'_c} \, b_w d$ (Eqn. 11-4)

$2 \left(1 + \dfrac{1428 \times 1000}{2000 (24 \times 24)} \right) (1) \sqrt{6000} \, (24)(21.5) = 179.03^k$

$V_u \leq 0.5 \, \phi \, V_c$?

$= 0.5 \, (0.75)(179.03) = 67.14^k$

$V_u \geq 67.14^k$ ∴ Must satisfy § 11.4.5.1

spacing ≤ $d/2$ = 10.75 ⟹ use 10"

§ 11.4.6.3 $A_{v\,min} = 0.75 \sqrt{f'_c} \, \dfrac{b_w \, S}{f_{yt}}$

$= 0.75 \sqrt{6000} \, \dfrac{24 \, (10)}{60000} = 0.23 \text{ in}^2$ ← controls

and $\geq \dfrac{50 \, b_w \, s}{f_{yt}} = \dfrac{50 \, (24)(10)}{60,000} = 0.2$

2 legs of a #4

$A_v = 0.20 \times 2 = 0.40 \text{ in}^2$ √ ok

∴ use #4 hoops @ 10" oc.

Appendix R: Krueger Diffuser Specifications

KRUEGER — Excellence in Air Distribution

LINEAR SLOT DIFFUSERS E1

DesignFlo® | Architectural Linear Slot Diffuser

DFL Performance Data: Vertical Throw

IP/METRIC DATA: DFL, CONTINUOUS SLOT (VT BLADES)

Slot Width	Slots	Air Flow CFM/ft	Press Ps "WG	Perpendicular Throw ft	NC	Air Flow L/s/m	Press Ps Pa	Perpendicular Throw m	2	3	4	5	6	7
1.0" Slot Width	1 Slot	20	0.003	2 - 3 - 6	-	31	0.8	0.5 - 0.9 - 1.8	-	11	-	-	-	-
		80	0.054	8 - 12 - 19	19	124	13.3	2.5 - 3.7 - 5.7	41	42	34	30	22	13
		110	0.101	11 - 16 - 22	27	171	25.2	3.4 - 4.7 - 6.7	49	49	42	38	31	23
		140	0.164	14 - 18 - 25	33	217	40.8	4.3 - 5.3 - 7.6	55	54	47	44	38	31
		200	0.335	17 - 21 - 30	42	310	83.3	5.2 - 6.4 - 9.0	65	62	56	52	49	42
	2 Slots	50	0.005	4 - 5 - 11	-	78	1.3	1.1 - 1.6 - 3.3	14	19	-	-	-	-
		150	0.047	11 - 16 - 26	20	233	11.7	3.3 - 4.9 - 7.8	42	43	36	31	23	14
		200	0.084	14 - 21 - 30	28	310	20.8	4.3 - 6.4 - 9.0	50	50	42	38	32	23
		250	0.131	18 - 23 - 33	34	388	32.5	5.4 - 7.1 - 10.1	56	54	48	44	38	30
		350	0.256	23 - 28 - 39	42	543	63.8	6.9 - 8.4 - 11.9	64	62	56	52	48	41
1.5" Slot Width	1 Slot	30	0.006	2 - 4 - 7	-	47	1.4	0.7 - 1.1 - 2.3	13	-	-	-	-	-
		100	0.063	8 - 12 - 21	14	155	15.7	2.5 - 3.8 - 6.4	42	37	31	25	17	-
		135	0.115	11 - 17 - 24	23	210	28.6	3.4 - 5.1 - 7.4	49	44	39	34	30	22
		170	0.182	14 - 19 - 27	30	264	45.3	4.3 - 5.9 - 8.3	55	50	45	42	40	33
		240	0.363	19 - 23 - 33	44	373	90.3	5.7 - 7.0 - 9.9	63	58	54	52	55	51
	2 Slots	40	0.003	1 - 3 - 7	-	62	0.6	0.4 - 0.9 - 2.1	-	-	-	-	-	-
		160	0.040	9 - 14 - 27	11	248	10.0	2.8 - 4.3 - 8.1	39	34	28	21	11	-
		220	0.076	13 - 19 - 31	20	342	19.0	3.9 - 5.8 - 9.5	47	42	37	31	24	14
		280	0.123	16 - 24 - 35	27	435	30.7	5.0 - 7.4 - 10.7	53	48	43	38	35	27
		400	0.252	23 - 30 - 42	39	621	62.7	7.1 - 9.0 - 12.8	62	57	52	50	50	45
2.0" Slot Width	1 Slot	20	0.002	1 - 1 - 4	-	31	0.4	0.2 - 0.4 - 1.3	30	25	-	-	-	-
		100	0.044	7 - 11 - 21	20	155	10.9	2.2 - 3.3 - 6.4	45	43	32	15	-	-
		140	0.086	10 - 15 - 25	24	217	21.3	3.0 - 4.6 - 7.6	48	47	39	25	17	-
		180	0.141	13 - 19 - 28	28	279	35.2	3.9 - 5.9 - 8.6	51	49	44	33	27	16
		260	0.295	19 - 24 - 34	37	404	73.5	5.6 - 7.3 - 10.3	54	54	52	44	43	32
	2 Slots	40	0.002	1 - 2 - 6	-	62	0.4	0.3 - 0.6 - 1.8	33	28	-	-	-	-
		180	0.035	9 - 14 - 27	22	279	8.8	2.8 - 4.1 - 8.3	47	45	33	15	-	-
		250	0.068	13 - 19 - 33	26	388	17.0	3.8 - 5.8 - 10.1	50	48	39	25	15	-
		320	0.112	16 - 24 - 38	30	497	27.8	4.9 - 7.4 - 11.4	53	51	44	32	25	13
		460	0.231	23 - 32 - 45	37	714	57.5	7.1 - 9.7 - 13.7	56	55	52	44	40	30
2.5" Slot Width	1 Slot	20	0.001	0 - 1 - 4	-	31	0.3	0.1 - 0.3 - 1.2	-	-	-	-	-	-
		120	0.049	8 - 11 - 23	11	186	12.2	2.3 - 3.5 - 7.0	38	34	28	13	-	-
		170	0.098	11 - 16 - 27	19	264	24.5	3.3 - 4.9 - 8.3	44	41	35	24	21	-
		220	0.165	14 - 21 - 31	25	342	41.0	4.3 - 6.4 - 9.5	49	46	40	32	30	19
		320	0.348	20 - 27 - 38	33	497	86.7	6.2 - 8.1 - 11.4	55	54	48	44	43	38
	2 Slots	40	0.001	1 - 1 - 5	-	62	0.3	0.2 - 0.4 - 1.6	-	-	-	-	-	-
		220	0.041	10 - 15 - 30	13	342	10.2	3.0 - 4.5 - 9.1	40	35	29	13	-	-
		310	0.082	14 - 21 - 37	20	481	20.3	4.3 - 6.4 - 11.2	46	42	36	24	21	-
		400	0.136	18 - 27 - 42	26	621	33.9	5.5 - 8.2 - 12.8	50	47	41	32	30	17
		580	0.286	26 - 36 - 51	34	900	71.2	8.0 - 10.9 - 15.4	56	55	49	44	43	36
3.0" Slot Width	1 Slot	20	0.001	0 - 1 - 3	-	31	0.3	0.1 - 0.2 - 0.9	-	-	-	-	-	-
		140	0.051	8 - 12 - 24	-	217	12.6	2.5 - 3.7 - 7.4	40	31	22	14	-	-
		200	0.104	12 - 17 - 30	16	310	25.8	3.5 - 5.3 - 9.0	47	39	31	24	21	-
		260	0.175	15 - 23 - 34	23	404	43.6	4.6 - 6.9 - 10.3	51	45	38	31	29	19
		380	0.374	22 - 29 - 41	33	590	93.1	6.7 - 8.8 - 12.4	59	54	47	41	41	34
	2 Slots	50	0.002	1 - 2 - 6	-	78	0.4	0.2 - 0.5 - 1.9	-	-	-	-	-	-
		250	0.040	10 - 15 - 31	-	388	10.1	3.1 - 4.7 - 9.4	41	32	23	14	-	-
		350	0.079	14 - 22 - 39	16	543	19.8	4.4 - 6.6 - 11.9	47	39	31	23	20	-
		450	0.131	19 - 28 - 45	23	699	32.7	5.6 - 8.5 - 13.5	52	45	37	30	28	16
		650	0.274	27 - 38 - 54	33	1009	68.1	8.1 - 11.5 - 16.3	59	54	46	40	39	31

NOTES: Throw values are given for terminal velocities of 150, 100, and 50 FPM (0.75, 0.50, and 0.25 m/s). NC values are based on octave band 2 - 7 sound power levels minus a room absorption of 10dB, re 10^{-12} Watts. Dash in space denotes a NC or dB value of less than 10. Data was obtained from tests conducted in accordance with ANSI/ASHRAE Standard 70, ISO Standard 5219, and ISO Standard 3741. 2-Way, 1-Slot throw is split throw. Refer to page E1-4 and E1-5 for directional airflow descriptions. Pressures are for diffuser section only. Plenums will add to sound level and pressure drop. Keep inlet velocities below 800 FPM to reduce plenum generated sound level and pressure drop. See correction factors on page E1-14. See selection software for performance data not shown, including octave band data.

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LINEAR SLOT DIFFUSERS

DESIGNFLO

Providing You With Air Distribution Solutions   **E1-15**

Appendix S: Steel Cost Estimate

Steel Deck - 05 31 13.50 (5400)						
Level	**SF**	**Material**	**Labor**	**Equipment**	**Total**	**Tot Incl O&P**
		2.34	**0.47**	**0.04**	**2.85**	**3.45**
Ground	39398	92191.32	18517.06	1575.92	112284.3	135923.1
2nd	27864	65201.76	13096.08	1114.56	79412.4	96130.8
3rd	29151	68213.34	13700.97	1166.04	83080.35	100570.95
4th	30496	71360.64	14333.12	1219.84	86913.6	105211.2
5th	22931	53658.54	10777.57	917.24	65353.35	79111.95
6th	22931	53658.54	10777.57	917.24	65353.35	79111.95
7th	22931	53658.54	10777.57	917.24	65353.35	79111.95
8th	22931	53658.54	10777.57	917.24	65353.35	79111.95
9th	22931	53658.54	10777.57	917.24	65353.35	79111.95
10th	22931	53658.54	10777.57	917.24	65353.35	79111.95
Roof	23086	54021.24	10850.42	923.44	65795.1	79646.7
Total	**287581**	**$672,939.54**	**$135,163.07**	**$11,503.24**	**$819,605.85**	**$ 992,154.45**

Applied Fireproofing - Steel Deck - 07 81 16.10 (0500)						
Level	**SF**	**Material**	**Labor**	**Equipment**	**Total**	**Tot Incl O&P**
		0.79	**0.71**	**0.1**	**1.6**	**2.05**
Ground	39398	31124.42	27972.58	3939.8	63036.8	80765.9
2nd	27864	22012.56	19783.44	2786.4	44582.4	57121.2
3rd	29151	23029.29	20697.21	2915.1	46641.6	59759.55
4th	30496	24091.84	21652.16	3049.6	48793.6	62516.8
5th	22931	18115.49	16281.01	2293.1	36689.6	47008.55
6th	22931	18115.49	16281.01	2293.1	36689.6	47008.55
7th	22931	18115.49	16281.01	2293.1	36689.6	47008.55
8th	22931	18115.49	16281.01	2293.1	36689.6	47008.55
9th	22931	18115.49	16281.01	2293.1	36689.6	47008.55
10th	22931	18115.49	16281.01	2293.1	36689.6	47008.55
Roof	23086	18237.94	16391.06	2308.6	36937.6	47326.3
Total	**287581**	**$227,188.99**	**$204,182.51**	**$28,758.10**	**$460,129.60**	**$ 589,541.05**

Placing Concrete - 03 31 05.70 (1400)					
Level	C.Y.	Material	Labor	Equipment	Total	Tot Incl O&P
		0	17.25	5.5	22.75	32.5
Ground	395.20	0	6817.13	2173.58	8990.71	12843.87
2nd	279.50	0	4821.38	1537.25	6358.63	9083.75
3rd	292.41	0	5044.07	1608.25	6652.32	9503.32
4th	305.90	0	5276.80	1682.46	6959.25	9941.79
5th	230.02	0	3967.81	1265.10	5232.90	7475.58
6th	230.02	0	3967.81	1265.10	5232.90	7475.58
7th	230.02	0	3967.81	1265.10	5232.90	7475.58
8th	230.02	0	3967.81	1265.10	5232.90	7475.58
9th	230.02	0	3967.81	1265.10	5232.90	7475.58
10th	230.02	0	3967.81	1265.10	5232.90	7475.58
Roof	231.57	0	3994.63	1273.65	5268.28	7526.11
Total	2884.69	$ -	$ 49,760.83	$15,865.77	$ 65,626.61	$ 93,752.29

Finishing Concrete - 03 35 29.30 (0250)					
Level	SF	Material	Labor	Equipment	Total	Tot Incl O&P
		0	0.56	0.03	0.49	0.86
Ground	39398	0	22062.88	1181.94	19305.02	33882.28
2nd	27864	0	15603.84	835.92	13653.36	23963.04
3rd	29151	0	16324.56	874.53	14283.99	25069.86
4th	30496	0	17077.76	914.88	14943.04	26226.56
5th	22931	0	12841.36	687.93	11236.19	19720.66
6th	22931	0	12841.36	687.93	11236.19	19720.66
7th	22931	0	12841.36	687.93	11236.19	19720.66
8th	22931	0	12841.36	687.93	11236.19	19720.66
9th	22931	0	12841.36	687.93	11236.19	19720.66
10th	22931	0	12841.36	687.93	11236.19	19720.66
Roof	23086	0	12928.16	692.58	11312.14	19853.96
Total	2884.685957	$ -	$161,045.36	$ 8,627.43	$140,914.69	$ 247,319.66

		Concrete Topping - 03 30 53.40 (3300)				
Level	**SF**	**Material**	**Labor**	**Equipment**	**Total**	**Tot Incl O&P**
		1.16	**0.95**	**0.31**	**2.42**	**3.06**
Ground	39398	45701.68	37428.1	12213.38	95343.16	120557.88
2nd	27864	32322.24	26470.8	8637.84	67430.88	85263.84
3rd	29151	33815.16	27693.45	9036.81	70545.42	89202.06
4th	30496	35375.36	28971.2	9453.76	73800.32	93317.76
5th	22931	26599.96	21784.45	7108.61	55493.02	70168.86
6th	22931	26599.96	21784.45	7108.61	55493.02	70168.86
7th	22931	26599.96	21784.45	7108.61	55493.02	70168.86
8th	22931	26599.96	21784.45	7108.61	55493.02	70168.86
9th	22931	26599.96	21784.45	7108.61	55493.02	70168.86
10th	22931	26599.96	21784.45	7108.61	55493.02	70168.86
Roof	23086	26779.76	21931.7	7156.66	55868.12	70643.16
Total	**287581**	**$ 333,593.96**	**$ 273,201.95**	**$ 89,150.11**	**$ 695,946.02**	**$ 879,997.86**

							Structural Steel - Beams & Girders (6th Floor) 05 12 23.77 (0900)				
Member			**#**	**Length**	**L.F.**	**Tons**	**Material**	**Labor**	**Equipment**	**Total**	**Tot Incl O&P**
							2600	**440**	**126**	**3166**	**3750**
W	12	x	14	10	10.0	100.0	0.70	1820.00	308.00	88.20	2216.20	2625.00
W	12	x	16	1	19.5	19.5	0.16	405.60	68.64	19.66	493.90	585.00
W	12	x	19	1	23.0	23.0	0.22	568.10	96.14	27.53	691.77	819.38
W	12	x	19	1	11.7	11.7	0.11	288.25	48.78	13.97	351.00	415.74
W	12	x	30	2	11.7	23.3	0.35	910.26	154.04	44.11	1108.42	1312.88
W	14	x	22	1	29.5	29.5	0.32	843.70	142.78	40.89	1027.37	1216.88
W	14	x	22	2	11.7	23.3	0.26	667.52	112.97	32.35	812.84	962.78
W	14	x	22	19	23.0	437.0	4.81	12498.20	2115.08	605.68	15218.96	18026.25
W	14	x	34	1	20.3	20.3	0.35	898.59	152.07	43.55	1094.20	1296.04
W	14	x	38	1	23.0	23.0	0.44	1136.20	192.28	55.06	1383.54	1638.75
W	16	x	26	4	23.0	92.0	1.20	3109.60	526.24	150.70	3786.54	4485.00
W	16	x	26	52	29.5	1534.0	19.94	51849.20	8774.48	2512.69	63136.37	74782.50
W	16	x	26	1	20.3	20.3	0.26	687.15	116.29	33.30	836.74	991.09
W	16	x	31	4	29.5	118.0	1.83	4755.40	804.76	230.45	5790.61	6858.75
W	16	x	31	4	23.0	92.0	1.43	3707.60	627.44	179.68	4514.72	5347.50
W	18	x	35	1	29.5	29.5	0.52	1342.25	227.15	65.05	1634.45	1935.94
W	18	x	35	3	20.3	61.0	1.07	2775.05	469.62	134.48	3379.15	4002.47
W	18	x	35	2	19.5	39.0	0.68	1774.50	300.30	86.00	2160.80	2559.38
W	18	x	40	2	29.5	59.0	1.18	3068.00	519.20	148.68	3735.88	4425.00
W	18	x	40	1	20.3	20.3	0.41	1057.16	178.90	51.23	1287.30	1524.75
W	21	x	44	2	19.5	39.0	0.86	2230.80	377.52	108.11	2716.43	3217.50
W	21	x	50	1	29.5	29.5	0.74	1917.50	324.50	92.93	2334.93	2765.63
W	24	x	62	1	29.5	29.5	0.91	2377.70	402.38	115.23	2895.31	3429.38
W	24	x	62	1	30.0	30.0	0.93	2418.00	409.20	117.18	2944.38	3487.50
W	24	x	84	1	30.0	30.0	1.26	3276.00	554.40	158.76	3989.16	4725.00
W	24	x	76	2	30.0	60.0	2.28	5928.00	1003.20	287.28	7218.48	8550.00
W	24	x	55	27	30.0	810.0	22.28	57915.00	9801.00	2806.65	70522.65	83531.25
W	24	x	68	2	29.2	58.3	1.98	5156.73	872.68	249.90	6279.31	7437.59
HSS	10x6x1/2		2	23.0	46.0	1.12	2921.23	494.36	141.57	3557.16	4213.31
HSS	10x6x1/2		1	20.3	20.3	0.50	1291.06	218.49	62.57	1572.11	1862.10
HSS	6x6x3/8		4	11.5	46.0	0.63	1643.30	278.10	79.64	2001.04	2370.15
HSS	12x12x1/2		1	30.0	30.0	1.14	2966.73	502.06	143.77	3612.56	4278.94
Total						**70.85**	**$ 184,204.37**	**$ 31,173.05**	**$ 8,926.83**	**$ 224,304.25**	**$ 265,679.39**

Columns 05 12 23.77 (0900)										
Member			#	Length	LF	Tons	Material	Labor	Equipment	Total	Tot Incl O&P
							2600	440	126	3166	3750
W	14	x	43	2	58	116	2.49	6484.40	1097.36	314.24	7896.00	9352.50
W	14	x	43	1	50	50	1.08	2795.00	473.00	135.45	3403.45	4031.25
W	14	x	61	4	58	232	7.08	18397.60	3113.44	891.58	22402.62	26535.00
W	14	x	61	4	52	208	6.34	16494.40	2791.36	799.34	20085.10	23790.00
W	14	x	61	4	50	200	6.10	15860.00	2684.00	768.60	19312.60	22875.00
W	14	x	68	2	52	104	3.54	9193.60	1555.84	445.54	11194.98	13260.00
W	14	x	74	5	52	260	9.62	25012.00	4232.80	1212.12	30456.92	36075.00
W	14	x	82	1	42	42	1.72	4477.20	757.68	216.97	5451.85	6457.50
W	14	x	82	1	52	52	2.13	5543.20	938.08	268.63	6749.91	7995.00
W	14	x	90	6	28	168	7.56	19656.00	3326.40	952.56	23934.96	28350.00
W	14	x	90	6	39	234	10.53	27378.00	4633.20	1326.78	33337.98	39487.50
W	14	x	90	2	42	84	3.78	9828.00	1663.20	476.28	11967.48	14175.00
W	14	x	90	4	50	200	9.00	23400.00	3960.00	1134.00	28494.00	33750.00
W	14	x	90	11	52	572	25.74	66924.00	11325.60	3243.24	81492.84	96525.00
W	14	x	90	12	58	696	31.32	81432.00	13780.80	3946.32	99159.12	117450.00
W	14	x	99	3	58	174	8.61	22393.80	3789.72	1085.24	27268.76	32298.75
W	14	x	99	5	52	260	12.87	33462.00	5662.80	1621.62	40746.42	48262.50
W	14	x	109	1	52	52	2.83	7368.40	1246.96	357.08	8972.44	10627.50
W	14	x	109	1	42	42	2.29	5951.40	1007.16	288.41	7246.97	8583.75
W	14	x	120	6	52	312	18.72	48672.00	8236.80	2358.72	59267.52	70200.00
W	14	x	132	1	55	55	3.63	9438.00	1597.20	457.38	11492.58	13612.50
W	14	x	132	4	56	224	14.78	38438.40	6504.96	1862.78	46806.14	55440.00
W	14	x	145	2	56	112	8.12	21112.00	3572.80	1023.12	25707.92	30450.00
W	14	x	145	2	52	104	7.54	19604.00	3317.60	950.04	23871.64	28275.00
W	14	x	159	1	62	62	4.93	12815.40	2168.76	621.05	15605.21	18483.75
W	14	x	159	7	52	364	28.94	75238.80	12732.72	3646.19	91617.71	108517.50
W	14	x	176	2	52	104	9.15	23795.20	4026.88	1153.15	28975.23	34320.00
W	14	x	176	1	75	75	6.60	17160.00	2904.00	831.60	20895.60	24750.00
W	14	x	176	4	56	224	19.71	51251.20	8673.28	2483.71	62408.19	73920.00
W	14	x	176	1	62	62	5.46	14185.60	2400.64	687.46	17273.70	20460.00
W	14	x	193	2	56	112	10.81	28100.80	4755.52	1361.81	34218.13	40530.00
W	14	x	193	2	62	124	11.97	31111.60	5265.04	1507.72	37884.36	44872.50
W	14	x	193	2	52	104	10.04	26093.60	4415.84	1264.54	31773.98	37635.00
W	14	x	211	1	52	52	5.49	14263.60	2413.84	691.24	17368.68	20572.50
W	14	x	211	3	56	168	17.72	46082.40	7798.56	2233.22	56114.18	66465.00
W	14	x	211	1	62	62	6.54	17006.60	2878.04	824.17	20708.81	24528.75
W	12	x	230	6	13	78	8.97	23322.00	3946.80	1130.22	28399.02	33637.50
W	14	x	233	4	52	208	24.23	63003.20	10662.08	3053.23	76718.51	90870.00
W	14	x	233	1	62	62	7.22	18779.80	3178.12	910.10	22868.02	27086.25
W	14	x	233	2	42	84	9.79	25443.60	4305.84	1233.04	30982.48	36697.50
W	14	x	233	4	69	276	32.15	83600.40	14147.76	4051.40	101799.56	120577.50
W	14	x	257	2	56	112	14.39	37419.20	6332.48	1813.39	45565.07	53970.00
W	14	x	257	2	42	84	10.79	28064.40	4749.36	1360.04	34173.80	40477.50
W	14	x	257	2	52	104	13.36	34746.40	5880.16	1683.86	42310.42	50115.00
W	14	x	342	5	56	280	47.88	124488.00	21067.20	6032.88	151588.08	179550.00
W	14	x	370	1	56	56	10.36	26936.00	4558.40	1305.36	32799.76	38850.00
W	14	x	426	2	56	112	23.86	62025.60	10496.64	3005.86	75528.10	89460.00
Total						547.79	$1,424,248.80	$241,026.72	$69,021.29	$1,734,296.81	$2,054,205.00

Member		#	L.F.	lb/ft	Tons	Material	Labor	Equipment	Total	Tot Incl O&P
						2600	440	126	3166	3750
	Sturcural Steel - Braced Frames - 05 12 23.77 (0900)									

Member		#	L.F.	lb/ft	Tons	Material 2600	Labor 440	Equipment 126	Total 3166	Tot Incl O&P 3750
Column Line E	HSS 10x10x1/2	2	22.4	62.46	1.40	3637.67	615.61	176.29	4429.56	5246.64
	HSS 10x10x3/8	2	17.5	47.9	0.84	2179.45	368.83	105.62	2653.90	3143.44
	HSS 10x10x5/16	4	17.5	40.35	1.41	3671.85	621.39	177.94	4471.18	5295.94
	HSS 10x10x5/16	4	16.5	40.35	1.33	3462.03	585.88	167.78	4215.69	4993.31
	HSS 8x8x5/16	6	16.5	31.84	1.58	4097.81	693.48	198.59	4989.87	5910.30
	HSS 6x6x5/16	6	16.5	23.34	1.16	3003.86	508.35	145.57	3657.77	4332.49
	HSS 10x10x5/16	1	36	40.35	0.73	1888.38	319.57	91.51	2299.47	2723.63
	HSS 12x12x3/8	2	36	58.1	2.09	5438.16	920.30	263.54	6622.01	7843.50
	HSS 12x12x1/2	2	20.5	76.07	1.56	4054.53	686.15	196.49	4937.17	5847.88
	HSS 10x10x1/2	4	20.5	62.46	2.56	6658.24	1126.78	322.67	8107.68	9603.23
	HSS 10x10x1/2	4	19.8	62.46	2.47	6430.88	1088.30	311.65	7830.84	9275.31
	HSS 10x10x3/8	2	19.8	47.9	0.95	2465.89	417.30	119.50	3002.70	3556.58
	HSS 10x10x5/16	2	19.8	40.35	0.80	2077.22	351.53	100.67	2529.41	2995.99
	HSS 8x8x5/16	6	19.8	31.84	1.89	4917.37	832.17	238.30	5987.84	7092.36
Column Line B	HSS 12x12x1/2	2	20.5	76.07	1.56	4054.53	686.15	196.49	4937.17	5847.88
	HSS 12x12x3/8	4	20.5	58.1	2.38	6193.46	1048.12	300.14	7541.73	8932.88
	HSS 10x10x1/2	2	19.8	62.46	1.24	3215.44	544.15	155.83	3915.42	4637.66
	HSS 10x10x3/8	6	19.8	47.9	2.85	7397.68	1251.91	358.50	9008.09	10669.73
	HSS 8x8x5/16	4	19.8	31.84	1.26	3278.25	554.78	158.87	3991.90	4728.24
	HSS 6x6x5/16	2	19.8	23.34	0.46	1201.54	203.34	58.23	1463.11	1733.00
	HSS 10x10x5/16	1	22.8	40.35	0.46	1195.97	202.40	57.96	1456.33	1724.96
	HSS 10x10x5/16	1	27.6	40.35	0.56	1447.76	245.01	70.16	1762.92	2088.11
	HSS 12x12x3/8	3	33.1	58.1	2.88	7500.13	1269.25	363.47	9132.85	10817.49
	HSS 12x12x3/8	3	32.7	58.1	2.85	7409.49	1253.91	359.08	9022.48	10686.77
	HSS 10x10x1/2	2	32.7	62.46	2.04	5310.35	898.67	257.35	6466.37	7659.16
	HSS 10x10x5/16	2	32.7	40.35	1.32	3430.56	580.56	166.25	4177.36	4947.92
	HSS 6x6x5/16	1	32.7	23.34	0.38	992.18	167.91	48.08	1208.17	1431.03
Column Line 12	HSS 10x10x1/2	4	18.1	62.46	2.26	5878.74	994.86	284.89	7158.49	8478.95
	HSS 10x10x5/16	2	18.1	40.35	0.73	1898.87	321.35	92.02	2312.24	2738.76
	HSS 10x10x5/16	10	17.3	40.35	3.49	9074.72	1535.72	439.77	11050.21	13088.53
	HSS 8x8x5/16	4	17.3	31.84	1.10	2864.33	484.73	138.81	3487.87	4131.24
	HSS 6x6x5/16	2	29.4	23.34	0.69	1784.11	301.93	86.46	2172.50	2573.24
Column Line 8	HSS 10x10x3/8	2	23.1	47.9	1.11	2876.87	486.86	139.42	3503.15	4149.34
	HSS 10x10x1/2	4	18.1	62.46	2.26	5878.74	994.86	284.89	7158.49	8478.95
	HSS 10x10x3/8	2	18.1	47.9	0.87	2254.17	381.48	109.24	2744.89	3251.21
	HSS 10x10x3/8	2	17.3	47.9	0.83	2154.54	364.61	104.41	2623.57	3107.51
	HSS 10x10x5/16	8	17.3	40.35	2.79	7259.77	1228.58	351.82	8840.17	10470.83
	HSS 8x8x5/16	4	17.3	31.84	1.10	2864.33	484.73	138.81	3487.87	4131.24
Column Line 7	HSS 10x10x1/2	2	23.1	62.46	1.44	3751.35	634.84	181.80	4567.99	5410.60
	HSS 10x10x1/2	6	18.1	62.46	3.39	8818.10	1492.29	427.34	10737.74	12718.42
	HSS 10x10x1/2	2	17.3	62.46	1.08	2809.45	475.45	136.15	3421.05	4052.09
	HSS 10x10x3/8	4	17.3	47.9	1.66	4309.08	729.23	208.82	5247.14	6215.03
	HSS 10x10x5/16	4	17.3	40.35	1.40	3629.89	614.29	175.91	4420.08	5235.41
	HSS 8x8x5/16	4	17.3	31.84	1.10	2864.33	484.73	138.81	3487.87	4131.24
Column Line 4	HSS 12x12x5/8	2	23.1	93.34	2.16	5606.00	948.71	271.68	6826.38	8085.58
	HSS 12x12x1/2	2	18.1	76.07	1.38	3579.85	605.82	173.49	4359.16	5163.25
	HSS 12x12x3/8	2	18.1	58.1	1.05	2734.19	462.71	132.50	3329.40	3943.54
	HSS 12x12x1/2	2	18.1	76.07	1.38	3579.85	605.82	173.49	4359.16	5163.25
	HSS 12x12x1/2	2	17.3	76.07	1.32	3421.63	579.04	165.82	4166.49	4935.04
	HSS 12x12x3/8	4	17.3	58.1	2.01	5226.68	884.51	253.29	6364.48	7538.48
	HSS 10x10x3/8	2	17.3	47.9	0.83	2154.54	364.61	104.41	2623.57	3107.51
	HSS 10x10x5/16	2	17.3	40.35	0.70	1814.94	307.14	87.95	2210.04	2617.71
	HSS 8x8x5/16	4	17.3	31.84	1.10	2864.33	484.73	138.81	3487.87	4131.24
Total					80.22	$ 208,564.06	$ 35,295.46	$10,107.34	$ 253,966.86	$ 300,813.56

Appendix T: Concrete Cost Estimate

					Formwork - Columns - 03 11 13.25 (6500)				
Level	Height	Dim.	#	S.F.C.A.	Material	Labor	Equipment	Total	Tot Incl O&P
					2.57	7.05	0	9.62	13.65
Basement	15	24"x24"	38	4560	11719.2	32148	0	43867.2	62244
Ground	13	24"x24"	44	4576	11760.32	32260.8	0	44021.12	62462.4
2nd	13	24"x24"	44	4576	11760.32	32260.8	0	44021.12	62462.4
3rd	13	24"x24"	44	4576	11760.32	32260.8	0	44021.12	62462.4
4th	12	24"x24"	44	4224	10855.68	29779.2	0	40634.88	57657.6
5th	12	24"x24"	44	4224	10855.68	29779.2	0	40634.88	57657.6
6th	12	24"x24"	44	4224	10855.68	29779.2	0	40634.88	57657.6
7th	12	24"x24"	44	4224	10855.68	29779.2	0	40634.88	57657.6
8th	12	24"x24"	44	4224	10855.68	29779.2	0	40634.88	57657.6
9th	12	24"x24"	44	4224	10855.68	29779.2	0	40634.88	57657.6
10th	12	24"x24"	44	4224	10855.68	29779.2	0	40634.88	57657.6
Total				47856	122989.92	$337,384.80	$ -	$460,374.72	$ 653,234.40

				Formwork - Exterior Beams - 03 11 13.20 (1500)				
Level	Dim.	L.F.	S.F.C.A.	Material	Labor	Equipment	Total	Tot Incl O&P
				2	7.8	0	9.8	14.15
Ground	24"x18"	0	0	0	0	0	0	0
2nd	24"x18"	722	3610	7220	28158	0	35378	51081.5
3rd	24"x18"	722	3610	7220	28158	0	35378	51081.5
4th	24"x18"	722	3610	7220	28158	0	35378	51081.5
5th	24"x18"	722	3610	7220	28158	0	35378	51081.5
6th	24"x18"	722	3610	7220	28158	0	35378	51081.5
7th	24"x18"	722	3610	7220	28158	0	35378	51081.5
8th	24"x18"	722	3610	7220	28158	0	35378	51081.5
9th	24"x18"	722	3610	7220	28158	0	35378	51081.5
10th	24"x18"	722	3610	7220	28158	0	35378	51081.5
Roof	24"x18"	722	3610	7220	28158	0	35378	51081.5
Total		7220	36100	$ 72,200.00	$281,580.00	$ -	$353,780.00	$ 510,815.00

Formwork - Interior Beams - 03 11 13.20 (2000)								
Level	Dim.	L.F.	S.F.C.A.	Material	Labor	Equipment	Total	Tot Incl O&P
				3.03	6.85		9.88	13.9
Ground	12"x18"	289	676.26	2049.0678	4632.381	0	6681.4488	9400.014
2nd	12"x18"	289	676.26	2049.0678	4632.381	0	6681.4488	9400.014
3rd	12"x18"	289	676.26	2049.0678	4632.381	0	6681.4488	9400.014
4th	12"x18"	331	774.54	2346.8562	5305.599	0	7652.4552	10766.106
5th	12"x18"	331	774.54	2346.8562	5305.599	0	7652.4552	10766.106
6th	12"x18"	331	774.54	2346.8562	5305.599	0	7652.4552	10766.106
7th	12"x18"	331	774.54	2346.8562	5305.599	0	7652.4552	10766.106
8th	12"x18"	331	774.54	2346.8562	5305.599	0	7652.4552	10766.106
9th	12"x18"	331	774.54	2346.8562	5305.599	0	7652.4552	10766.106
10th	12"x18"	331	774.54	2346.8562	5305.599	0	7652.4552	10766.106
Roof	12"x18"	331	774.54	2346.8562	5305.599	0	7652.4552	10766.106
Total		3515	8225.1	$ 24,922.05	$ 56,341.94	$ -	$ 81,263.99	$ 114,328.89

Formwork - Shear Walls - 03 11 13.85 (2400)								
Level	Height	Wall Length	S.F.C.A.	Material	Labor	Equipment	Total	Tot Incl O&P
				2.4	7.35	0	9.75	13.95
Ground	15	21	1890	4536	13891.5	0	18427.5	26365.5
2nd	13	21	1638	3931.2	12039.3	0	15970.5	22850.1
3rd	13	21	1638	3931.2	12039.3	0	15970.5	22850.1
4th	13	21	1638	3931.2	12039.3	0	15970.5	22850.1
5th	12	21	1512	3628.8	11113.2	0	14742	21092.4
6th	12	21	1512	3628.8	11113.2	0	14742	21092.4
7th	12	21	1512	3628.8	11113.2	0	14742	21092.4
8th	12	21	1512	3628.8	11113.2	0	14742	21092.4
9th	12	21	1512	3628.8	11113.2	0	14742	21092.4
10th	12	21	1512	3628.8	11113.2	0	14742	21092.4
Roof	12	21	1512	3628.8	11113.2	0	14742	21092.4
Total			17388	$ 41,731.20	$127,801.80	$ -	$169,533.00	$242,562.60

							Structural Concrete - Columns - 03 31 05.35 (0411)			
Level	Height	Dim.	#	f'c	C.Y.	Material	Labor	Equipment	Total	Tot Incl O&P
						124	0	0	124	136
Basement	15	24"x24"	38	6000	84.44	10471.11	0	0	10471.11	11484.44
Ground	13	24"x24"	44	6000	84.74	10507.85	0	0	10507.85	11524.74
2nd	13	24"x24"	44	6000	84.74	10507.85	0	0	10507.85	11524.74
						Structural Concrete - Columns - 03 31 05.35 (0300)				
Level	Height	Dim.	#	f'c	C.Y.	Material	Labor	Equipment	Total	Tot Incl O&P
						103	0	0	103	113
3rd	13	24"x24"	44	4000	84.74	8728.30	0	0	8728.30	9575.70
4th	12	24"x24"	44	4000	78.22	8056.89	0	0	8056.89	8839.11
5th	12	24"x24"	44	4000	78.22	8056.89	0	0	8056.89	8839.11
6th	12	24"x24"	44	4000	78.22	8056.89	0	0	8056.89	8839.11
7th	12	24"x24"	44	4000	78.22	8056.89	0	0	8056.89	8839.11
8th	12	24"x24"	44	4000	78.22	8056.89	0	0	8056.89	8839.11
9th	12	24"x24"	44	4000	78.22	8056.89	0	0	8056.89	8839.11
10th	12	24"x24"	44	4000	78.22	8056.89	0	0	8056.89	8839.11
		Total			886.22	$ 96,613.33	$ -	$ -	$ 96,613.33	$ 105,983.41

							Structural Concrete - Slabs and Drop Panels - 03 31 05.35 (0300)			
Level	S.F.	Slab Thickness	Drop Thickness	Drop Dimensions	# of Drops	C.Y.	Material	Labor	Equipment	Total	Tot Incl O&P
							103	0	0	103	113
Ground	39398	10	2.5	10'x10'	39	1246.08	128346.27	0	0	128346.27	140807.07
2nd	27864	10	2.5	10'x10'	27	880.83	90725.83	0	0	90725.83	99534.17
3rd	29151	10	5	10'x10'	27	941.39	96963.06	0	0	96963.06	106376.94
4th	30496	10	2.5	10'x10'	27	962.07	99092.99	0	0	99092.99	108713.67
5th	22931	10	2.5	10'x10'	27	728.58	75043.77	0	0	75043.77	82329.57
6th	22931	10	2.5	10'x10'	27	728.58	75043.77	0	0	75043.77	82329.57
7th	22931	10	2.5	10'x10'	27	728.58	75043.77	0	0	75043.77	82329.57
8th	22931	10	2.5	10'x10'	27	728.58	75043.77	0	0	75043.77	82329.57
9th	22931	10	2.5	10'x10'	27	728.58	75043.77	0	0	75043.77	82329.57
10th	22931	10	2.5	10'x10'	27	728.58	75043.77	0	0	75043.77	82329.57
Roof	23086	10	2.5	10'x10'	27	733.36	75536.51	0	0	75536.51	82870.15
			Total			9135.22	$ 940,927.25	$ -	$ -	$ 940,927.25	$ 1,032,279.41

							Structural Concrete - Beams - 03 31 05.35 (0300)			
Level	L.F. Int Beams	Dim Int Beams	L.F. Ext Beams	Dim Ext Beams	C.Y.	Material	Labor	Equipment	Total	Tot Incl O&P
						103	0	0	103	113
Ground	289	12"x18"	0	24"x18"	7.14	734.99	0	0	734.99	806.35
2nd	289	12"x18"	722	24"x18"	42.79	4407.38	0	0	4407.38	4835.28
3rd	289	12"x18"	722	24"x18"	42.79	4407.38	0	0	4407.38	4835.28
4th	331	12"x18"	722	24"x18"	43.83	4514.20	0	0	4514.20	4952.47
5th	331	12"x18"	722	24"x18"	43.83	4514.20	0	0	4514.20	4952.47
6th	331	12"x18"	722	24"x18"	43.83	4514.20	0	0	4514.20	4952.47
7th	331	12"x18"	722	24"x18"	43.83	4514.20	0	0	4514.20	4952.47
8th	331	12"x18"	722	24"x18"	43.83	4514.20	0	0	4514.20	4952.47
9th	331	12"x18"	722	24"x18"	43.83	4514.20	0	0	4514.20	4952.47
10th	331	12"x18"	722	24"x18"	43.83	4514.20	0	0	4514.20	4952.47
Roof	331	12"x18"	722	24"x18"	43.83	4514.20	0	0	4514.20	4952.47
		Total			443.33	$ 45,663.33	$ -	$ -	$ 45,663.33	$ 50,096.67

				Structural Concrete - Shear Walls - 03 31 05.35 (0411)					
Level	Height	Wall Length	Wall Width	C.Y. (3 Walls)	Material 124	Labor 0	Equipment 0	Total 124	Tot Incl O&P 136
Basement	15	21	1.5	52.50	6510	0	0	6510.00	7140.00
Ground	13	21	1.5	45.50	5642	0	0	5642.00	6188.00
2nd	13	21	1.5	45.50	5642	0	0	5642.00	6188.00
3rd	13	21	1.5	45.50	5642	0	0	5642.00	6188.00
4th	12	21	1.5	42.00	5208	0	0	5208.00	5712.00
5th	12	21	1.5	42.00	5208	0	0	5208.00	5712.00
6th	12	21	1.5	42.00	5208	0	0	5208.00	5712.00
7th	12	21	1.5	42.00	5208	0	0	5208.00	5712.00
8th	12	21	1.5	42.00	5208	0	0	5208.00	5712.00
9th	12	21	1.5	42.00	5208	0	0	5208.00	5712.00
10th	12	21	1.5	42.00	5208	0	0	5208.00	5712.00
Total				483.00	$ 59,892.00	$ -	$ -	$ 59,892.00	$ 65,688.00

				Placing Concrete - Slabs and Drop Panels - 03 31 05.70 (1500)						
Level	S.F.	Slab Thickness	Drop Thickness	Drop Dimensions	# of Drops	C.Y.	Material 0	Labor 15.1	Equipment 4.82	Total 19.92	Tot Incl O&P 28.5
Ground	39398	10	2.5	10'x10'	39	1246.08	0	18815.81	6006.11	24821.92	35513.29
2nd	27864	10	2.5	10'x10'	27	880.83	0	13300.58	4245.62	17546.20	25103.75
3rd	29151	10	5	10'x10'	27	941.39	0	14214.97	4537.49	18752.47	26829.58
4th	30496	10	2.5	10'x10'	27	962.07	0	14527.23	4637.17	19164.39	27418.94
5th	22931	10	2.5	10'x10'	27	728.58	0	11001.56	3511.76	14513.32	20764.54
6th	22931	10	2.5	10'x10'	27	728.58	0	11001.56	3511.76	14513.32	20764.54
7th	22931	10	2.5	10'x10'	27	728.58	0	11001.56	3511.76	14513.32	20764.54
				Placing Concrete - Slabs and Drop Panels - 03 31 05.70 (1550)						
Level	S.F.	Slab Thickness	Drop Thickness	Drop Dimensions	# of Drops	C.Y.	Material 0	Labor 25	Equipment 10.7	Total 35.7	Tot Incl O&P 50
8th	22931	10	2.5	10'x10'	27	728.58	0	18214.51	7795.81	26010.31	36429.01
9th	22931	10	2.5	10'x10'	27	728.58	0	18214.51	7795.81	26010.31	36429.01
10th	22931	10	2.5	10'x10'	27	728.58	0	18214.51	7795.81	26010.31	36429.01
Roof	23086	10	2.5	10'x10'	27	733.36	0	18334.10	7847.00	26181.10	36668.21
Total						9135.22	$ -	$ 166,840.90	$61,196.08	$ 228,036.98	$ 323,114.41

				Placing Concrete - Beams - 03 31 05.70 (0050)					
Level	L.F. Int Beams	Dim Int Beams	L.F. Ext Beams	Dim Ext Beams	C.Y.	Material 0	Labor 40	Equipment 12.85	Total 52.85	Tot Incl O&P 75.5
Ground	289	12"x18"	0	24"x18"	7.14	0	285.43	91.70	377.13	538.75
2nd	289	12"x18"	722	24"x18"	42.79	0	1711.60	549.85	2261.46	3230.65
3rd	289	12"x18"	722	24"x18"	42.79	0	1711.60	549.85	2261.46	3230.65
4th	331	12"x18"	722	24"x18"	43.83	0	1753.09	563.18	2316.27	3308.95
5th	331	12"x18"	722	24"x18"	43.83	0	1753.09	563.18	2316.27	3308.95
6th	331	12"x18"	722	24"x18"	43.83	0	1753.09	563.18	2316.27	3308.95
7th	331	12"x18"	722	24"x18"	43.83	0	1753.09	563.18	2316.27	3308.95
				Placing Concrete - Beams - 03 31 05.70 (0100)					
Level	L.F. Int Beams	Dim Int Beams	L.F. Ext Beams	Dim Ext Beams	C.Y.	Material 0	Labor 61	Equipment 26	Total 87	Tot Incl O&P 122
8th	331	12"x18"	722	24"x18"	43.83	0	2673.46	1139.51	3812.96	5346.91
9th	331	12"x18"	722	24"x18"	43.83	0	2673.46	1139.51	3812.96	5346.91
10th	331	12"x18"	722	24"x18"	43.83	0	2673.46	1139.51	3812.96	5346.91
Roof	331	12"x18"	722	24"x18"	43.83	0	2673.46	1139.51	3812.96	5346.91
Total					443.33	0	$ 21,414.81	$ 8,002.14	$ 29,416.96	$ 41,623.52

Placing Concrete - Shear Walls - 03 31 05.70 (5350)									
Level	Height	Wall Length	Wall Width	C.Y. (3 Walls)	Material 0	Labor 20	Equipment 6.4	Total 26.4	Tot Incl O&P 37.5
Basement	15	21	1.5	52.50	0	1050.00	336.00	1386.00	1968.75
Ground	13	21	1.5	45.50	0	910.00	291.20	1201.20	1706.25
2nd	13	21	1.5	45.50	0	910.00	291.20	1201.20	1706.25
3rd	13	21	1.5	45.50	0	910.00	291.20	1201.20	1706.25
4th	12	21	1.5	42.00	0	840.00	268.80	1108.80	1575.00
5th	12	21	1.5	42.00	0	840.00	268.80	1108.80	1575.00
6th	12	21	1.5	42.00	0	840.00	268.80	1108.80	1575.00
Placing Concrete - Shear Walls - 03 31 05.70 (5400)									
Level	Height	Wall Length	Wall Width	C.Y. (3 Walls)	Material 0	Labor 29	Equipment 12.4	Total 41.4	Tot Incl O&P 57.5
7th	12	21	1.5	42.00	0	1218.00	520.80	1738.80	2415.00
8th	12	21	1.5	42.00	0	1218.00	520.80	1738.80	2415.00
9th	12	21	1.5	42.00	0	1218.00	520.80	1738.80	2415.00
10th	12	21	1.5	42.00	0	1218.00	520.80	1738.80	2415.00
Total				483.00	$ -	$ 11,172.00	$ 4,099.20	$ 15,271.20	$ 21,472.50

Placing Concrete - Columns - 03 31 05.70 (0800)									
Level	Height	Dim.	#	C.Y.	Material 0	Labor 26	Equipment 8.4	Total 34.4	Tot Incl O&P 49
Basement	15	24"x24"	38	84.44	0	2195.56	709.33	2904.89	4137.78
Ground	13	24"x24"	44	84.74	0	2203.26	711.82	2915.08	4152.30
2nd	13	24"x24"	44	84.74	0	2203.26	711.82	2915.08	4152.30
3rd	13	24"x24"	44	84.74	0	2203.26	711.82	2915.08	4152.30
4th	12	24"x24"	44	78.22	0	2033.78	657.07	2690.84	3832.89
5th	12	24"x24"	44	78.22	0	2033.78	657.07	2690.84	3832.89
6th	12	24"x24"	44	78.22	0	2033.78	657.07	2690.84	3832.89
Placing Concrete - Columns - 03 31 05.70 (0850)									
Level	Height	Dim.	#	C.Y.	Material 0	Labor 39	Equipment 16.8	Total 55.8	Tot Incl O&P 78
7th	12	24"x24"	44	78.22	0	3050.67	1314.13	4364.80	6101.33
8th	12	24"x24"	44	78.22	0	3050.67	1314.13	4364.80	6101.33
9th	12	24"x24"	44	78.22	0	3050.67	1314.13	4364.80	6101.33
10th	12	24"x24"	44	78.22	0	3050.67	1314.13	4364.80	6101.33
Total				886.22	$ -	$ 27,109.33	$ 10,072.53	$ 37,181.87	$ 52,498.67

Concrete Finishing - Slabs - 03 35 29.30 (0200)						
Level	S.F.	Material	Labor	Equipment	Total	Tot Incl O&P
		0	0.76	0	0.76	1.13
Ground	39398	0	29942.48	0	29942.48	44519.74
2nd	27864	0	21176.64	0	21176.64	31486.32
3rd	29151	0	22154.76	0	22154.76	32940.63
4th	30496	0	23176.96	0	23176.96	34460.48
5th	22931	0	17427.56	0	17427.56	25912.03
6th	22931	0	17427.56	0	17427.56	25912.03
7th	22931	0	17427.56	0	17427.56	25912.03
8th	22931	0	17427.56	0	17427.56	25912.03
9th	22931	0	17427.56	0	17427.56	25912.03
10th	22931	0	17427.56	0	17427.56	25912.03
Roof	23086	0	17545.36	0	17545.36	26087.18
Total		$ -	$218,561.56	$ -	$218,561.56	$324,966.53
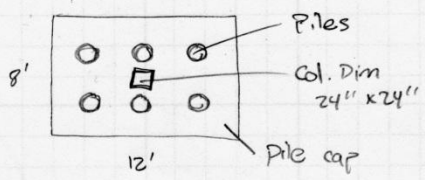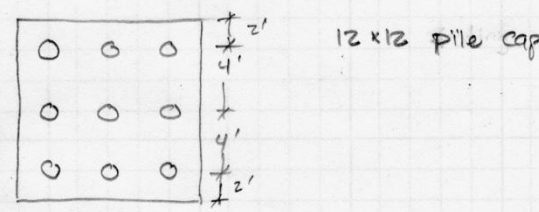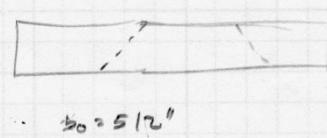
Concrete Finishing - Walls & Columns - 03 35 29.60 (0020)										
Level	Height	Wall Length	S.F. (2 Sides)	SFCA (Cols)	SF Total	Material	Labor	Equipment	Total	Tot Incl O&P
						0.03	0.63	0	0.66	0.96
Basement	15	21	630	4560	5190	156	3269.70	0	3425.40	4982.40
Ground	13	21	546	4576	5122	154	3226.86	0	3380.52	4917.12
2nd	13	21	546	4576	5122	154	3226.86	0	3380.52	4917.12
3rd	13	21	546	4576	5122	154	3226.86	0	3380.52	4917.12
4th	12	21	504	4224	4728	142	2978.64	0	3120.48	4538.88
5th	12	21	504	4224	4728	142	2978.64	0	3120.48	4538.88
6th	12	21	504	4224	4728	142	2978.64	0	3120.48	4538.88
7th	12	21	504	4224	4728	142	2978.64	0	3120.48	4538.88
8th	12	21	504	4224	4728	142	2978.64	0	3120.48	4538.88
9th	12	21	504	4224	4728	142	2978.64	0	3120.48	4538.88
10th	12	21	504	4224	4728	142	2978.64	0	3120.48	4538.88
Total					53652	$ 1,609.56	$ 33,800.76	$ -	$ 35,410.32	$ 51,505.92

Reinforcement Bars - Slabs - 03 21 10.60 (0400)					
Level	Tons	Material	Labor	Equipment	Total	Tot Incl O&P
		1050	540	0	1590	2025
Ground	101.02	106074	54552.37	0	160626.43	204571.40
2nd	71.66	75239	38694.13	0	113932.70	145102.97
3rd	77.17	81026	41670.60	0	122696.76	156264.74
4th	45.18	47439	24397.30	0	71836.51	91489.89
5th	45.18	47439	24397.30	0	71836.51	91489.89
6th	45.18	47439	24397.30	0	71836.51	91489.89
7th	45.18	47439	24397.30	0	71836.51	91489.89
8th	45.18	47439	24397.30	0	71836.51	91489.89
9th	45.18	47439	24397.30	0	71836.51	91489.89
10th	45.18	47439	24397.30	0	71836.51	91489.89
Roof	45.18	47439	24397.30	0	71836.51	91489.89
Total (+10%)	672.42	$ 706,037.67	$ 363,105.09	$ -	$ 1,069,142.76	$ 1,361,644.08

Reinforcement Bars - Columns - 03 21 10.60 (0250)							
Level	Height	#	Tons	Material	Labor	Equipment	Total	Tot Incl O&P
				980	685	0	1665	2175
Basement	15	38	18.17	17807	12446.77	0	30253.82	39520.75
Ground	13	44	12.31	12060	8430.01	0	20490.46	26766.81
2nd	13	44	7.78	7624	5328.75	0	12952.37	16919.76
3rd	13	44	14.77	14473	10116.01	0	24588.55	32120.17
4th	12	44	7.18	7037	4918.85	0	11956.03	15618.24
5th	12	44	7.18	7037	4918.85	0	11956.03	15618.24
6th	12	44	5.64	5526	3862.74	0	9389.00	12264.91
7th	12	44	5.64	5526	3862.74	0	9389.00	12264.91
8th	12	44	5.64	5526	3862.74	0	9389.00	12264.91
9th	12	44	5.64	5526	3862.74	0	9389.00	12264.91
10th	12	44	5.64	5526	3862.74	0	9389.00	12264.91
Total (+10%)			105.14	$103,036.25	$ 72,020.24	$ -	$175,056.48	$228,677.39

Reinforcement Bars - Walls - 03 21 10.60 (0750)							
Level	Height	Length	Tons (3 Walls)	Material	Labor	Equipment	Total	Tot Incl O&P
				930	525	0	1455	1850
Basement	15	21	3.36	3129	1766.21	0	4894.91	6223.77
Ground	13	21	2.92	2712	1530.71	0	4242.26	5393.93
2nd	13	21	2.92	2712	1530.71	0	4242.26	5393.93
3rd	13	21	2.92	2712	1530.71	0	4242.26	5393.93
4th	12	21	2.69	2503	1412.96	0	3915.93	4979.02
5th	12	21	2.69	2503	1412.96	0	3915.93	4979.02
6th	12	21	2.69	2503	1412.96	0	3915.93	4979.02
7th	12	21	2.69	2503	1412.96	0	3915.93	4979.02
8th	12	21	2.69	2503	1412.96	0	3915.93	4979.02
9th	12	21	2.69	2503	1412.96	0	3915.93	4979.02
10th	12	21	2.69	2503	1412.96	0	3915.93	4979.02
Total (+10%)			34.05	$ 31,662.50	$ 17,873.99	$ -	$ 49,536.50	$ 62,984.55

Appendix U: Foundation Checks

Foundation Check   Andrew Voorhees   1

Check Existing Footing   Column C-6

$P_u = 2200^k$

Pile Cap Dimensions $8' \times 12' \times 4'-6''$

Long Dir : Top = (19) #8
           Bot = (21) #8
Short Dir : Top = (19) #8
            Bot (19) #8

Piles $16'' \phi$, Capacity = 120 Tons
                              = 240 kips

# of Piles = $\dfrac{2200}{240}$ = 9.16 → use 9

12 x 12 pile cap

$b_o = 512''$

9 piles = $\dfrac{2160^k}{144 \text{ s.f.}}$ = 15,000 psf = q⁻

Check punching shear

$$d^2 (4v_c + q) + d(2v_c + q)(b+c) = q (BL - cb)$$

$$v_c \leq \begin{cases} \dfrac{4}{\beta_c} + 2 = \dfrac{4}{1} + 2 = 6 \\ \dfrac{\alpha_s d}{b_o} = \dfrac{40(52)}{512} = 4.06 \\ 4 \quad = 4 \leftarrow \text{controls} \end{cases}$$

d = 52''

$V_c = 4(\sqrt{4000}) \times 512 \times 52 = 6735.4^{kips}$

$$P = \dfrac{1}{144 \times 144} \left[2(52'')(6735.4^k)(24'' + 52'') + 2(52)(6735.4)(24 + 52) + 15(24 + 52)(24 + 52) \right]$$

= 5138.8^k to punch > 2200^k ∴ punching <u>OK</u>

2

Check Flexure
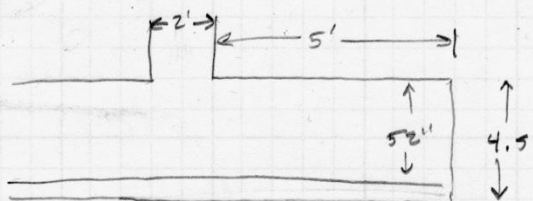
$q_u = 15 \text{ ksf}$

$M_u = \dfrac{15 \, (1')(5')^2}{2} = 187.5 \,'^k$

$a = \dfrac{A_s f_y}{0.85 f'_c b} = \dfrac{60 \, (A_s)}{0.85 \, (4)(12'')} = 1.47 \, A_s$

$M_u \leq \phi M_n = \phi A_s F_y \left(d - a/2\right)$

$187.5 \,'^k \, (12) = 0.9 \, A_s \, (60) \left(52'' - \dfrac{1.47 \, A_s}{2}\right)$

$2250 \,^{k \cdot in} = 2808 \, A_s - 39.69 \, A_s^2$

$\therefore \; A_s \geq 0.81 \text{ in}^2/\text{ft}$

Try (1) #9 /ft = $A_s = 1 \text{ in}^2/\text{ft}$

$\rho_{min} = \dfrac{1 \text{ in}^2/\text{ft}}{54'' \, (12'')} = 0.0015 < 0.0018$
⤷ use this

$0.0018 \, (54)(12) = A_s = 1.16 \quad \therefore \; \text{use} \; \#10 @ 12'' \quad A_s = 1.27 \text{ in}^2/\text{ft}$

$\varepsilon = \dfrac{0.003}{c} = \dfrac{\varepsilon_s}{(d-c)} \;\Rightarrow\; \dfrac{0.003}{2.2} \, (52 - 2.2)$

$= 0.07 > 0.005 \quad \therefore \; \phi = 0.9$

$a = 1.47 \, (1.27) = 1.87$

$c = \dfrac{1.87}{0.85} = 2.2$

$\boxed{\therefore \; \text{use} \; (12) \; \#10\text{'s in each direction}}$

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