Howard County General Hospital

Patient Tower Addition

Kelly Dooley
Structural Option
Advisor: Dr. Lepage

Howard County General Hospital

Patient Tower Addition

Columbia, MD

Project Team:

Owner – John Hopkins Medicine
Architect – Wilmot Sanz, Inc.
CM – Whiting-Turner
Structural – Rathgeber & Goss Assoc.
MEP – Leach Wallace Assoc., Inc.
Civil – Joyce Engineering Corp.

Building Statistics

Size – 114,261 SF addition
No. of Stories – Partially below grade basement, 4 stories, penthouse
Dates of Construction – April 07 to Present
Cost – Approx. $36.4 Million
Project Delivery – CM @ Risk

Architecture

- Façade composed of precast concrete, glass and metal panels
- Curtain wall on east side of addition to match existing hospital exterior
- Careful layout of patient rooms including stall-less showers and handrails

Structural

- Square and Rectangular spread footings under all columns
- 3 ¼" LW concrete on 2" metal deck at all floors and main roof/penthouse floor
- 1 ½" metal roof deck at penthouse roof
- Variety of W12 and W14 column sections
- 29 by 29 foot typical column bays
- 19 moment frames @ each floor
- Depressed slab at upper floors for stall-less showers

Mechanical

- Typical patient room require 6 minimum total air changes per hour (2 of which must be outside air changes)
- 2 new 70,000 CFM AHUs to accommodate required 140,000 CFM airflow
- Four additional boilers and one chiller to accommodate new PPH loads
- Radiant ceiling panels in patient rooms

Lighting/Electrical

- Anticipated 860 kW normal power and 1074 kW for new mechanical equipment
- 480Y/277 Volt Distribution
- Served by Baltimore Gas & Electric via 13.2 kV underground conductors
- Minimum 75 footcandle illuminance required for patient rooms
- T-8 fluorescent lamps with electronic ballast and 3500K CCT for typical lighting

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http://www.engr.psu.edu/ae/thesis/portfolios/2008/kmd293/

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I. EXECUTIVE SUMMARY

This report evaluates Howard County General Hospital patient tower addition as a cast-in-place concrete structure in comparison to its original design as a composite steel structure. All efforts were made to maintain the floor layout from the original design for architectural purposes.

The floor system was determined to be a 10" slab with 6" drop panels at all column locations. This floor thickness satisfies the flexural requirements with reasonable reinforcing steel and also complied with all deflection requirements. Columns were designed as 24" by 24" sections with typical reinforcing of (8) #8 bars. Normal weight concrete was used throughout the building with a 28 day compressive strength of 5000 psi.

For the existing composite design, it was very important to allow for a great deal of floor plan flexibility as the hospital's needs are ever changing and a future renovation is possible. For this reason, steel moment frames were used. For the new concrete system, it was desirable to maintain this same flexibility, so concrete moment frames were used. Wind loads for the new design were very similar to those for the existing design, however seismic loads greatly increased due to the increased building weight. A lateral analysis proved that the inherent lateral capacity of the slab and column system is sufficient to resist the lateral loads and shear walls were not required.

Wind drift was an issue in the existing composite system. The concrete system provided additional stiffness and resolved this issue as the total wind drift and story drift were both limited to H/400 in the new design. Seismic drift was also within the code mandated limits, proving to be acceptable.

A construction management study was performed to compare the two systems in terms of schedule and cost. It was found that the concrete system saved approximately $500,000. Both systems resulted in very similar schedules, with construction of the structural systems lasting approximately 16 weeks.

Finally an acoustics study evaluated various acoustical issues. Reverberation time was calculated and found to be between 0.5 and 0.7 seconds, which is assumed to be acceptable for the hospital. I also compared sound transmission through the new floor system and the existing system, finding that the concrete system achieved an STC rating of over 50, while the composite system was slightly below 50. Transmission through the walls was also assessed, as patient privacy is extremely important to the hospital. It was found that the typical partition walls separating the patient rooms have an STC rating of 51, which is above the target STC of 50 and therefore adequate to prevent sound transmission between rooms.

With the design was complete and the performance and cost of the new concrete system compared to that of the existing steel system, the concrete system is recommended over the steel system. Results prove that it performs more efficiently structurally and provided monetary savings.

II. ACKNOWLEDGEMENTS

I would like to extend my most sincere gratitude to those who helped me in any way with my senior thesis project. Without the help of the Architectural Engineering faculty, practicing professionals, and many others this project would not have been possible. Special thanks are extended to:

- Rathgeber/Goss Associates who provided me with structural and architectural drawings and also aided me with issues and questions throughout the year. Special thanks to Mike Goss and Justin Domire who were especially helpful.

- The Pennsylvania State University Architectural Engineering department and staff who have provided me with the foundation and education to pursue all of my career aspirations. Thank you especially to Professor Parfitt for teaching the senior thesis class and to Dr. Lepage, my thesis advisor, for his comments and assistance throughout the year.

- Beth Plavner of Howard County General Hospital who granted me permission to study the patient tower addition for my senior thesis project.

- Mike Lamdin of Leach Wallace Associates, Inc. who provided me with information on the mechanical and electrical systems.

- Dave Scovill of Whiting-Turner who supplied me with scheduling and cost data for the existing composite steel design.

- All of the mentors from the discussion board. Everyone was more than willing to help with all of our questions and provided detailed and prompt responses.

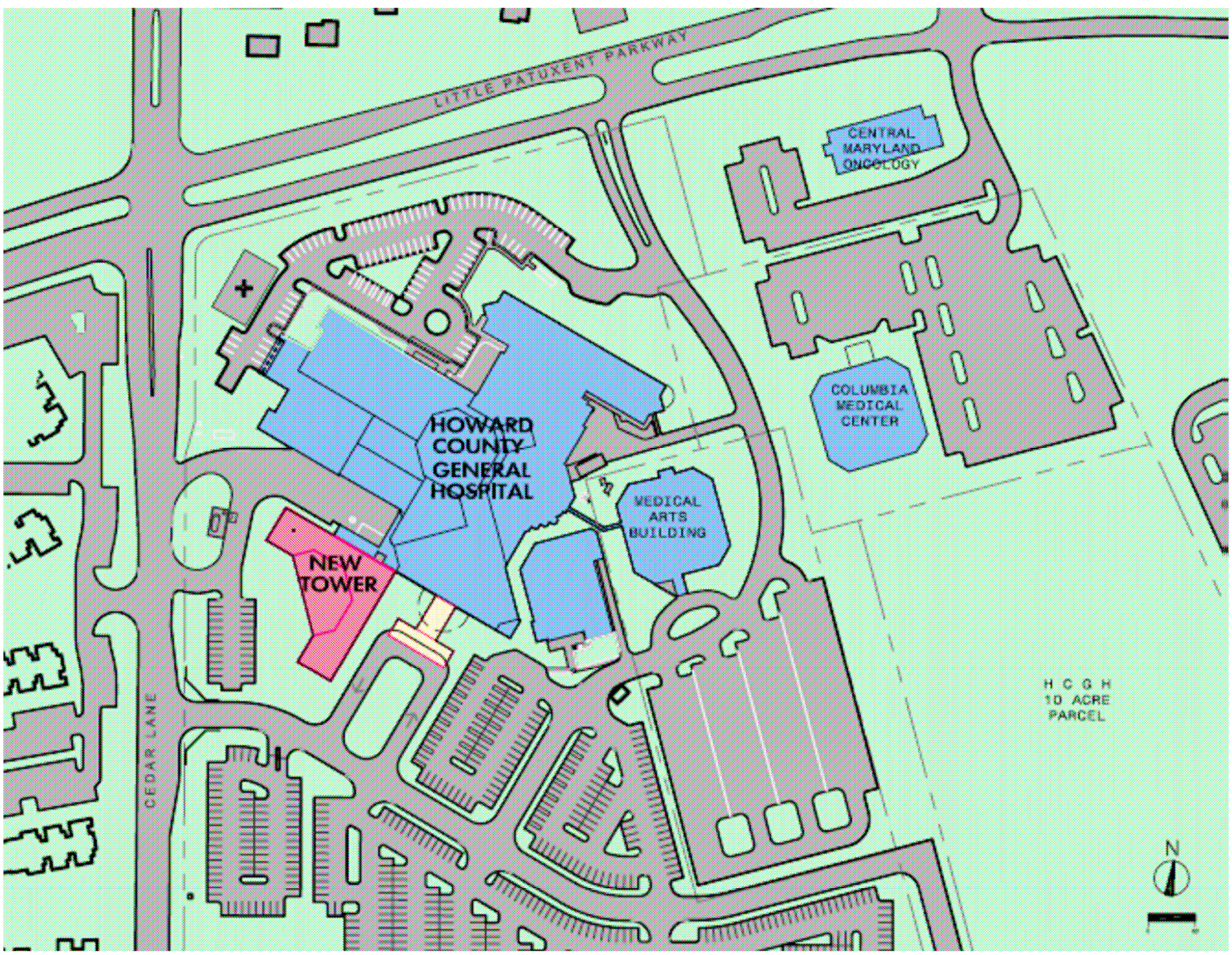
- My friends and family. You've provided me with encouragement, support, and laughter throughout all of my academic endeavors over the past five years.

III. BUILDING BACKGROUND AND PROJECT INFO

Architecture and General Info:

Howard County General Hospital has been serving the Howard County community for over 30 years and joined John Hopkins Medicine in 1998. This new tower addition is part of an overall expansion known as the "Campus Development Plan" proposed to address an expanding population in Howard County, and hence an increase in the number of hospital visitors. This plan is attending to the community's needs not only with the new tower, but also a new 550 space parking garage and over 120,000 square feet of renovations to the existing hospital.

The new tower is located adjacent to the southwest side of the existing south building, close to Cedar Lane. The existing hospital is shown below in blue, with the new addition shown in red.



Consisting primarily of new private inpatient rooms, the tower is intended to help better accommodate the patients. Each typical floor, second through fourth, is made up of thirty patient rooms. Much thought was put into the layout of these rooms including the installation of stall-less showers and handrails running from bed to bath as many patients require such accommodations. In addition to the private patient rooms, each floor also includes several nurse stations, a lobby, and many other facilities for the hospital employees, patients, and visitors.

Differing from the upper levels, the first floor does not include patient rooms but houses a new outpatient center including rehabilitation and cardiopulmonary facilities.

The existing main entrance of the hospital is made up of a curtain wall system at the first floor and stripes of aluminum and glass paneling at the upper floors. Since the east façade of the building is adjacent to this existing entrance and clearly visible to all patients and visitors entering the hospital, the new curtain wall at the first floor mimics the existing curtain wall. This avoids a harsh contrast of materials and maintains the visual appeal of this primary façade. Rather than solely glass and aluminum, the upper floors on the east façade are made up of 12' precast panels in addition to glass and aluminum spandrel panels (similar to those on the existing façade). There is no curtain wall on the other sides of the new tower, the full height façade is composed of precast, aluminum, and glass panels.

Zoning:

Howard County General Hospital falls within the POR (planned office research) zoning district. The POR District was established to permit and encourage diverse institutional, commercial, office research and cultural facilities within the community. The hospital does not fall within one of Howard County's historical preservation districts, so that was not an issue during design or permit.

Structural System:

The Howard County Hospital tower addition utilizes composite steel and lightweight concrete for the floor system and moment frames for the lateral bracing system. A typical floor is composed of 3 ¼" lightweight concrete on 2" 18 gage galvanized metal deck for a total floor thickness of 5 ¼". This is present at all floors and the main roof/penthouse floor. The penthouse roof is 1 ½" deep 20 gage roof deck.

Infill floor beams are typically one of three sizes – W12x19, W14x22, or W16x26. The W12x19s are generally 19 feet long and spaced at 7’-3”. Both the W14x22 and W16x26 are generally 29 feet long, but the W14s are spaced at 7’-3” while the W16s are spaced at 9’-8”. Girder sizes range between W16s and W30s. Many of the beams and girders require ¾” or 1” camber to meet deflection requirements.

There is a wide range of column sizes used in the tower addition ranging from W12x40s to W14x193s. Typically, the column bays are 29 by 29 feet. Most columns and are spliced at the 2nd and 4th floors.

The tower’s main lateral force resisting system is composed of 19 typical moment frames at each floor. Of the 19 moment frames, 8 are located along the exterior of the building and 11 are located throughout the interior. The moment frames are double angle connections bolted to the columns and welded to the beam webs with top and bottom full penetration welds to connect the beam flanges. Moment connection required capacities range from 20 kips to 80 kips.

For the foundation, standard square spread footings were used at all interior and most exterior footings. A portion of the North wall abuts an existing retaining wall, so rectangular footings are required at those columns. At the basement level, the building is surrounded by a 16” concrete foundation wall with a 2’-4” continuous footing.

Mechanical System:

The mechanical systems required for the Howard County Hospital Patient Tower Addition include heating, ventilating, and air conditioning systems. Being that this is a hospital, there were also needs for special medical gas and vacuum systems.

The ventilation requirements for different rooms varied from two air changes per hour for corridors to fifteen air changes per hour for isolation patient rooms and operating rooms according to ASHRAE and AIA DHHS guidelines. Typical patient rooms, which occupy most of the space in the new tower, require six minimum total air changes per hour, 2 of which being outside air changes per hour. Based on these requirements, an approximate airflow 140,000 CFM was calculated. Two new 70,000 CFM aluminum factory constructed air handling units, each capable of providing a minimum of 25% outdoor air, are being added to the penthouse of the new tower. Medium pressure supply and return ducts will be routed into the ceilings of each floor then to VAV supply air terminal units in each room.

For Heating and Cooling, design parameters of 0 degrees winter and 95 degrees dry bulb and 79 degrees wet bulb were used. Four additional boilers and a new chiller are being added to accommodate the new PPH loads. The existing Johnson Controls system will be extended to the new tower allowing a set point adjustment between 68 and 75 degrees within the spaces. In addition, radiant ceiling panels are being provided in each patient room by the windows for supplemental heating.

Electrical/Lighting Systems:

Currently, Howard County Hospital is served by Baltimore Gas & Electric via 13.2 kV underground conductors terminating at a 15 kV switchgear with 7 feeders serving various parts of the hospital.

The National Electric Code requires certain amounts of branch circuits, emergency branch circuits, outlets and receptacles based on whether rooms are critical patient care areas, general patient care areas, or non-patient care areas. The new demand load of the whole hospital with the new patient tower was calculated to be 4634 kW including a 25% growth factor. The new patient tower alone is anticipated to require 860 kW of normal power plus 1074 kW for the new mechanical equipment.

For the new normal power loads, at 480Y/277 volts, the existing substation has the capacity and space within the switchgear to serve the estimated 1150 amp load. A new 3P-1600 amp circuit breaker will be installed within the switchgear and a 1600 amp feeder will terminate in the new electrical room at a new 480Y/277 volt rated switchboard.

For the mechanical equipment, a new 800 amp feeder will be routed up from the switchgear (which has available space) in the existing substation to the new penthouse to carry the loads of the new AHUs. The new chiller will be served via a new 600 amp circuit breaker directly from the existing substation.

In terms of lighting, illuminance was the most important criteria since it is critical for proper diagnosis and treatment of patients. A minimum of 75 footcandles is required for patient rooms, which comprise the majority of the tower. Leach Wallace decided on T-8 fluorescent lamps with electronic ballasts and a CCT of 3500 K for general lighting. There will also be special fixtures such as examination lights and under cabinet fixtures for task lighting.

Construction:

Whiting-Turner is the Construction Manager for the Patient Tower addition. Excavation for the basement began in September of 2007, requiring a significant amount of preparation as the addition is adjacent to the existing hospital.

The existing footings were underpinned and soldier piles driven into the ground to take the horizontal earth pressure. Between the soldier piles, wood lagging (shown in the adjacent photo) was installed in five-foot lifts to one foot below each of the temporary tie back elevations. The lagging must be in firm contact with the soil to avoid soil movement. Seven wire post tension cables were used for the tie backs. This lagging process continues as excavation proceeds to subgrade elevation.

Following the excavation, Whiting-Turner prepared formwork for the new footings and slab on grade. The concrete foundation walls and slab on grade were then formed and poured. This concrete work was finished by the end of 2007, in preparation for the arrival of steel in January of 2008.

Steel fabrication began in June of 2007 and erection began in January of 2008. Erection of the columns, then beams, will occur for all levels creating the building's structural frame. This process began in January and finished in March. Once the building frame is complete, the steel deck will be placed followed by the concrete fill poured, forming the floor slabs. After the floors are completed, interior work can begin.

Currently, the projected completion date for the Patient Tower addition is in the summer of 2009.

A GMP bid of almost $40 million was provided for the tower construction alone. The entire Campus Development Plan was originally estimated at around $73 million, but is now projected to be closer to $100 million.

Fire Protection:

The existing hospital currently has a sprinkler system installed for fire protection, and the new patient tower will include one as well. A six-inch fire service will enter through the mechanical room on the ground floor. In addition, the existing fire pump does not have enough capacity for the new tower, so a new fire pump with a 750 GPM capacity is required.

Multiple fire alarm devices will also be installed, including manual pull devices at all exits and at each nurse's station. Audible and visual devices will be utilized in accordance with NFPA and with the ADA.

The majority of the building is rated for 2 hours of fire protection. This is achieved with the composite floor system described in the structural summary.

Telephone and Communication Systems:

A new 2-way communications system will be installed throughout the new tower with stations in all patient rooms, staff stations, and nursing areas. New amplifiers will be added in the electrical closets to extend the paging system to the new patient tower.

Cable television outlets will be located in all patient rooms, waiting rooms, and dialysis stations.

For security and access control, magnetic door locks, card readers, remote release pushbuttons, and local intercom systems will be provided to restrict access where necessary. A four-inch square outlet box will be provided at each security device location.

Building Envelope:

The building envelope of the tower creates a horizontal striping effect that mimics the existing façade of the adjacent hospital. It consists mainly of 12’ high precast concrete panels for durability, glass spandrel panels to allow for lots of natural light, and aluminum spandrel panels to add a slightly modern flair. There is a glass and aluminum curtain wall at ground level on the east façade as this side of the building is adjacent to the existing main entrance. This side of the building is the most visible as it faces the main visitor pick up and drop off area. The curtain wall stands out from the other sides of the façade, which face mainly parking and service areas, to distinguish the entrance for patients and visitors.

As previously mentioned, the main roof is a lightweight concrete on composite deck with a total thickness of 5¼”. There is a large penthouse that occupies almost half of the building footprint at this level. The roof of that penthouse area is simply a 1½” galvanized metal roof deck. Both roof systems utilize 5/8” sheathing, rigid insulation, and multi-ply roofing atop the slab/deck. The main roof system provides R-20 insulation.

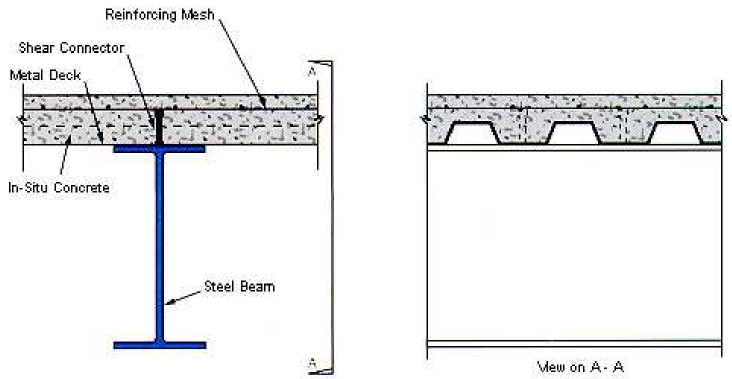
Project Team:

Owner	John Hopkins Medicine www.hopkinsmedicine.org/
Construction Manager	Whiting-Turner Contracting Co. www.whiting-turner.com/
Architects/Planners	Wilmot/Sanz Inc. www.wilmotsanz.com/
Structural Engineer	Rathgeber/Goss Associates www.rath-goss.com/
MEP Engineer	Leach Wallace Associates, Inc. www.leachwallace.com/
Civil Engineer	Joyce Engineering, Inc. www.joyceengineering.com/

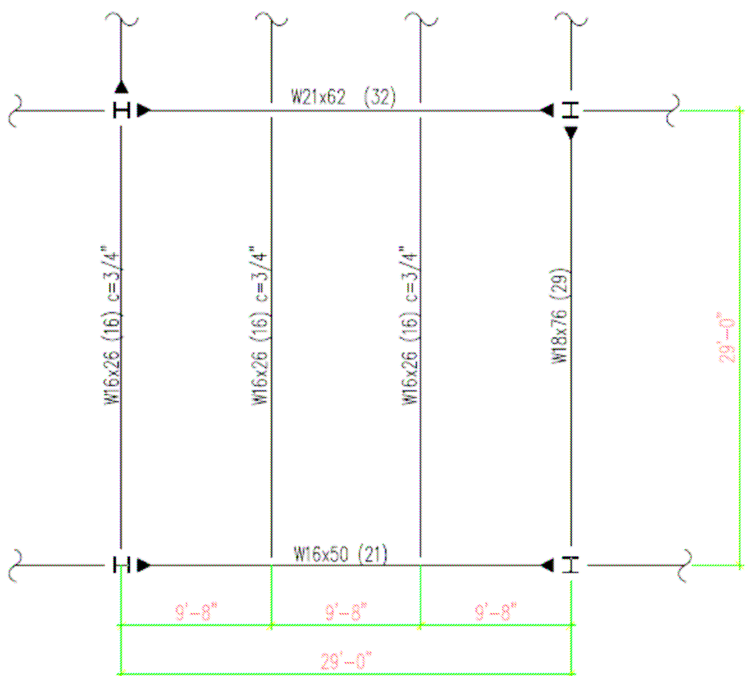
IV. EXISTING STRUCTURAL SYSTEMS AND MATERIALS

Floor System:

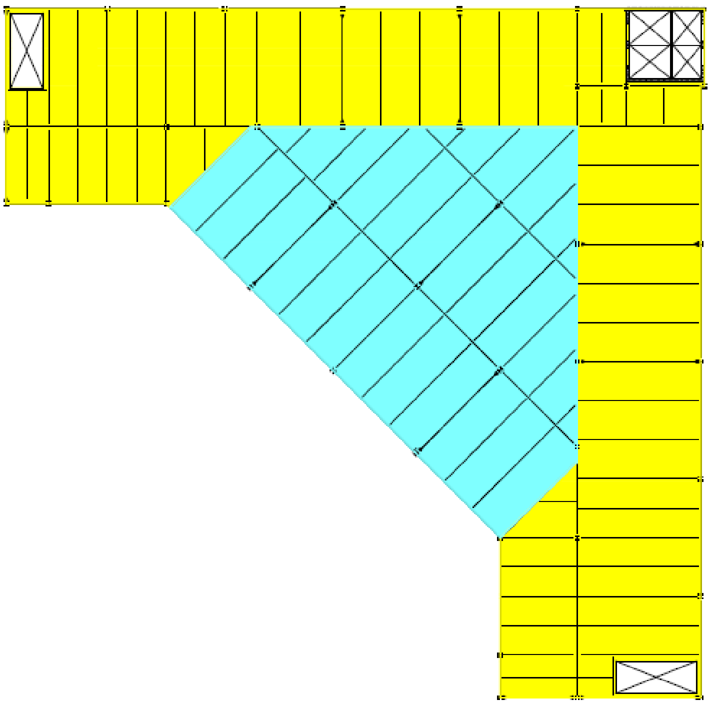
As briefly mentioned, the typical floor framing system is 3 ¼” lightweight concrete on 2” deep 18 gage composite metal deck for a total depth 5 ¼”. Composite action is achieved with ¾” diameter by 4” shear studs evenly spaced along the length of supported beams.



There are three typical infill beam sizes – W12x19, W14x22, and W16x26. These beams vary from 19 feet to 30 ½ feet in length and are usually spaced at 7’-3” or 9’-8”. The most typical bay is 29 by 29 feet with two infill beams, shown below.



The beams are supported by girders, which widely range in size from W16 shapes to W30 shapes. In addition to the standard composite slab, additional reinforcing of 5 foot long #4 top bars are specified at 16” on center over all interior girders.

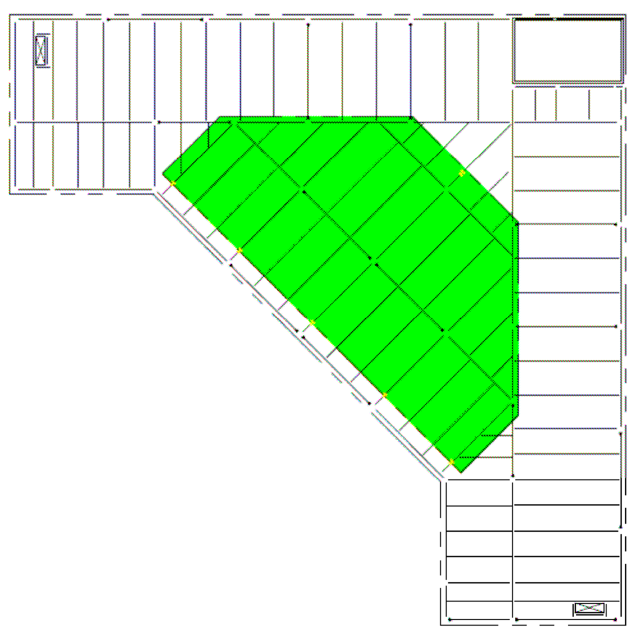


The new addition is a uniquely shaped structure, so the floors are framed out in two directions. As you can see in the adjacent figure, the "center" floor framing (shown in blue) is rotated at a 45 degree angle, while the framing along the outer "L" of the building (shown in yellow) is orthogonal. This required the composite deck to be oriented in two different directions as well.

The second, third, and fourth floors required 2" depressed slabs in the patient rooms to accommodate the prefabricated stall-less showers. The depressions are framed out with W12x19 beams, located at each of the thirty patient rooms on each of the three typical floors, second through fourth. This irregularity in the steel floor system resulted in a great number of additional members and some increased beam sizes, complicating the typical framing.

Full floor structural plans of the existing system are available upon request.

Roof System:



The main roof is also a composite system, since a considerable portion of it is occupied for the mechanical penthouse floor. This roof/floor system is composed of the same 3 ¼" lightweight concrete on 2" metal deck as the typical floors are. Infill beam sizes and lengths are similar to those mentioned above in the typical floor system. Transfer girders are also required at this level for 6 new columns that extend from the main roof/penthouse floor up to the penthouse roof. The portion of this level that is roof is shown in white in the adjacent figure, and the portion that is penthouse is shown in green.

The penthouse roof is the only floor system that varies from the typical composite system. It is made up of 1 ½" wide rib 20 gage metal roof deck supporting typical roof sheathing and materials. The infill beams are typically either 24'-9" long W10x19s or 35'-4" long W16x36s. The framing at the penthouse roof is at a forty-five degree angle, the same direction as that in the "center" framing area of the typical floors.

Exterior:

The exterior of the building is composed of precast, metal and glass panels, creating a striping effect. The precast panels are 8" thick. A rendered photograph of the exterior is shown to the right.

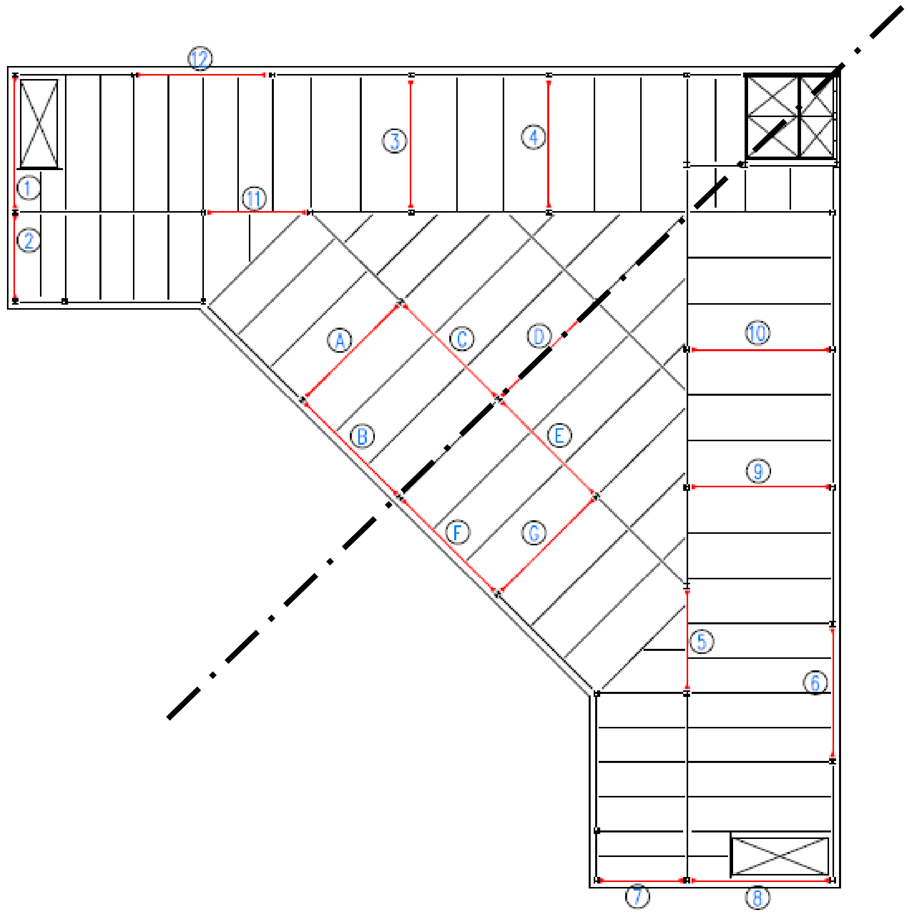
As mentioned previously, a curtain wall system is used at the first floor on the east side of the building, similar to the curtain wall used on the existing hospital.

The only other variation to the precast, metal, and glass striping pattern is that the 39.5 foot long true south and true north walls are made up of almost exclusively precast with a few punched out windows.

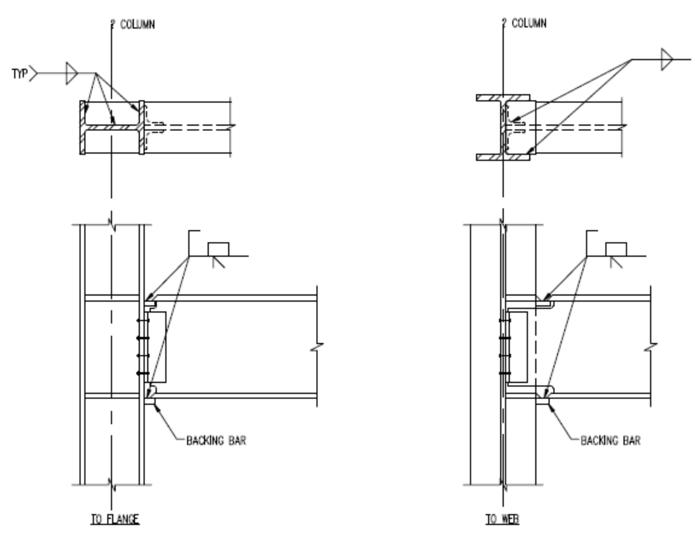
The walls that extend from the penthouse floor to the penthouse roof are composed of 6" metal studs at 16" on center with insulation. These walls have an exterior finish of "dryvit" on them for protection and aesthetics.

Lateral Load Resisting System:

Steel moment frames were used at each level to resist lateral loads. Each floor contains 19 moment frames, 8 of which are along the perimeter of the building and 11 are interior beams. The moment frames are symmetrical about the same 45 degree diagonal axis that the building is. These lateral force-resisting beams are highlighted in red in the diagram below, with the axis of symmetry shown as a dashed line. The frames numbered 1 through 12 directly resist load in the North-South or East-West direction, while the frames lettered A through G are at a 45 degree angle and resist lateral loads in both directions.



At each of these moment frames, end beam reactions are called out on the plans from which the moment connections can be designed. A double angle connection is used to connect the beam web to the column with the angle welded to the beam and bolted to the column. Stiffener plates are then added to the column at the same elevation and thickness as the beam flanges. Backing bars can be welded to connect the beam flanges to the face of the column or the column stiffeners, depending on the orientation of the column. To the right is the detail for this moment connection included in the structural plans.



1 BEAM TO COLUMN WELDED CONNECTION
TYPE 1 – FULLY RIGID
NTS

Although moment frames were an expensive option for the lateral system, they were selected in order to allow for floor plan flexibility. With the hospital constantly growing and the changing demands of various branches (i.e. surgery, physical therapy, rehabilitation, etc.), the space initially designed for patient rooms could have an alternate use sometime in the future. If trusses or braced frames were used, the location of these braces would greatly reduce the flexibility of the floor plan.

Foundation System:

Five soil test borings were taken at the site of the new patient tower. They were drilled to a depth of about 30 feet each according to ASTM D 1586 standards. It was found that the top layer of soil was fill soil consisting of sand and silt, but the basement floor elevation should generally fall below this layer of soil. Therefore, the geotechnical report specified an allowable bearing pressure of 6,000 psf for foundation design.

The footing sizes of the main addition vary from 8 foot by 8 foot to 11 foot by 11 foot square footings along with a few rectangular footings. Along the north wall of the building, there is an existing retaining wall footing. According to the plans, this footing is to be field verified during construction and any portions that interfere with the new footings are to be removed.

A 14” thick concrete foundation wall surrounds that building at the basement level. The wall is reinforced with #4 bars at 12” vertical on each face and #5 bars at 12” horizontal. Concrete piers protrude from the wall at the location of exterior columns from which steel columns extend from the first floor up.

The slab on grade is 5” thick reinforced with 6x6” WWF on a vapor retarder over a minimum 4” layer of clean, well graded gravel or crushed stone. There is a small area, approximately 20 by 40 feet, where the top of slab elevation is depressed one foot.

Codes and Standards:

The structural engineers, Rathgeber/Goss Associates designed the Howard County General Hospital patient tower, which began design in 2004, according to the 2000 International Building Code and ASCE 7-98. Concrete design specifically references ACI 318-98 while steel design followed the AISC Load and Resistance Factor Design, Third Edition 2001.

My analysis of the existing system utilized the more recent versions of the building codes, the 2006 International Building Code, which references ASCE 7-05. For concrete analysis, ACI 318-05 was used and for steel analysis, the Load and Resistance Factor Design portion of the AISC Thirteenth Edition Steel Manual was used.

Material Strengths:

Concrete

Application	f'c @ 28 days	Weight (pcf)
Slabs-on-grade	3000 psi	145
Fill on Metal Deck	3500 psi	110
Footings	3000 psi	145
Precast Units	5000 psi	145
Piers	4000 psi	145

Steel

Materials	Fy (ksi)
Wide-Flange Shapes	50
Channels, Angles, and Plates	36
Structural Pipe	35
Round HSS Shapes	42
Square/Rectangular HSS Shapes	46
Reinforcing Steel	60

V. PROPOSED PROBLEM AND SOLUTION

Problem Statement:

The existing composite steel system adequately performs as the building's structural system, proving to be a suitable design for the 100 psf live load and relatively large 29 by 29 foot column bays. However, a few issues were identified throughout various analyses in the fall semester, which suggest that another structural system could be designed to perform more efficiently.

Most notably, an in-depth lateral analysis proved wind drift to be an issue. Both total building drift and story-to-story drift exceeded the industry standard of H/400. Although wind drift this is not a strength requirement and is not specifically addressed in the code, it is still an issue that requires some improvement per engineering judgment and industry standards. Reducing this large wind drift is very important, especially because some of the hospital equipment may be sensitive to lateral movement.

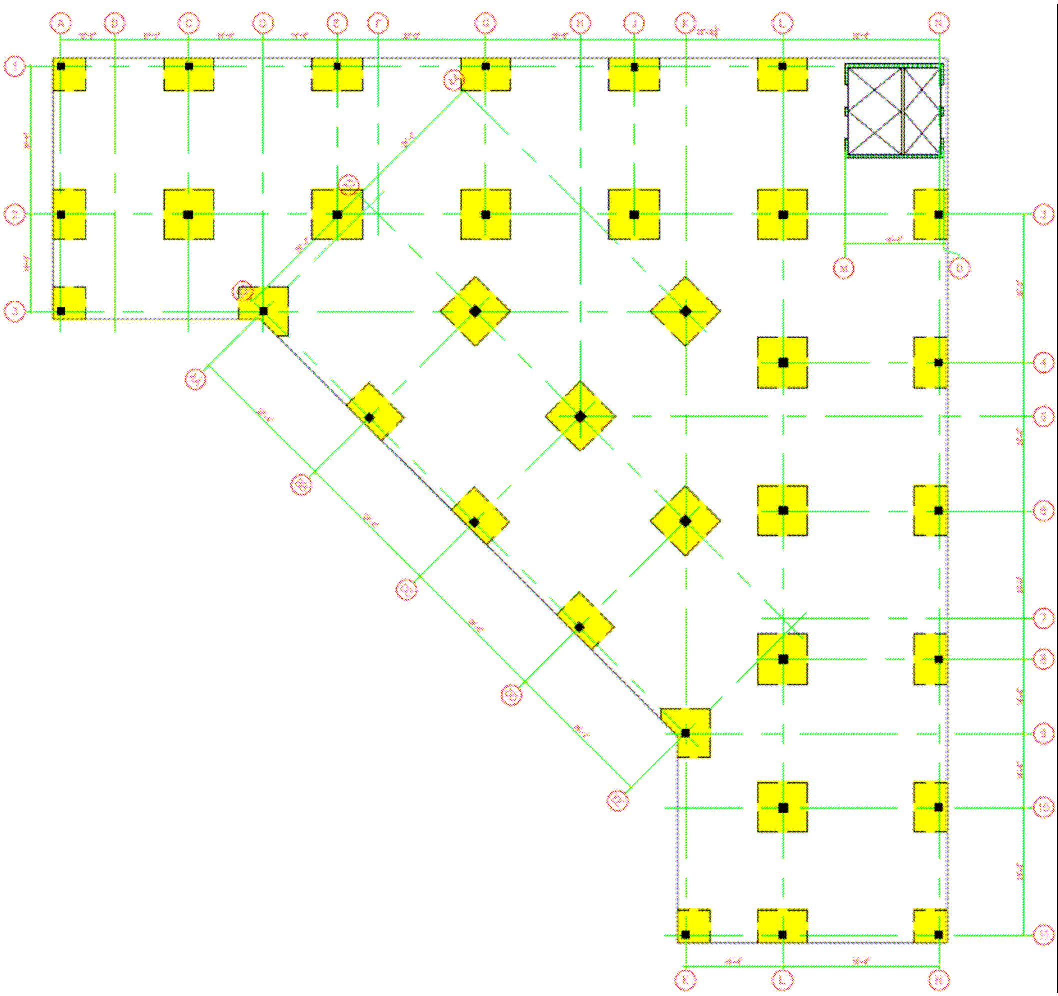
The existing lateral system resists all lateral loads with steel moment frames. Nineteen frames occur at each level and require substantially sized steel sections. This was desirable because of floor plan flexibility. However, moment connections are expensive to produce, so another lateral system could greatly decrease the total building costs, especially in terms of labor. It would be ideal to maintain the floor plan flexibility by avoiding braced frames or shear walls, which would reduce this flexibility.

Finally, for the composite system, each individual slab depression for the stall-less showers had to be framed out in steel members. Considering that there are 30 rooms on each typical floor, each room including a shower, this is a very costly, time consuming, and inconvenient task. This could be resolved by using concrete, as slab depressions are easily formed without much additional labor or expense.

It has been demonstrated that an alternate structural system could prove to be more effective for this building. Ideally, the optimal system would address all of the above issues without negatively affecting the cost or schedule.

Proposed Solution:

It has been determined that switching from a steel to concrete structure seems is a viable alternative that would address most or all of the issues outlined in the problem statement. It was previously determined in technical assignment 2 that a possible concrete floor system for this building would be a two-way concrete flat slab system with drop panels. A schematic plan developed in technical assignment two is shown below, based on preliminary sizes from the CRSI Handbook.



Initially in technical assignment 2, a concrete flat plate was ruled out because of the large column sizes required to resist punching shear. However, upon a number of discussions, it was discovered that using a flat plate system with stud rails could be another viable alternative. Because this system was not analyzed in technical assignment 2, additional research and investigation was required at the beginning of the spring semester. However, it was ultimately determined that punching shear at critical columns exceeded the maximum of $6*f'c^{1/2}$ specified in the ACI code. A flat plate with stud rails was therefore ruled out before design began.

Based on the preliminary analysis of a flat slab with drops performed in technical assignment 2, this floor system has many advantages, some of which are inherent to concrete. Concrete is more readily available and requires less lead-time than steel. The concrete slab provides the required 2-hour fire rating and the increase in mass and stiffness relieves the building of any vibratory or acoustical issues. Most importantly, concrete will address the drift problem previously mentioned.

According to preliminary sizes obtained in technical report 2 per the CRSI Handbook, the existing typical 29 by 29 foot column bays can essentially be maintained with a 10” slab, 8 ½” drop panels, and 16” to 19” square columns. Some slight changes to the architectural layout and/or column grid may be necessary. These sizes provided a starting point but will ultimately be adjusted as necessary for the final design.

The concrete columns and slab will be poured together, inherently forming concrete moment frames to resist lateral loads. Floor plan flexibility was an important issue in the existing design of the hospital, as previously mentioned, so all attempts will be made to maintain this flexibility. Hopefully, the inherent concrete moment frames will be adequate to resist lateral loads, though substantial reinforcing may be required. This cannot be determined until the analysis is completed and loads are obtained. At that point, the need for additional lateral resistance, most likely in the form of shear walls, can be assessed. If absolutely required, at that time optimal locations for these shear walls will be determined per the building plans.

By switching from steel to concrete, the building weight will greatly increase. For some issues such as drift, this is a positive change. However, it could have a negative effect on the foundation sizes and seismic loading. The existing spread footings will need to be evaluated and changes in size and/or reinforcing due to the increase dead load will most likely be necessary. Seismic forces will be thoroughly reevaluated as wind previously controlled.

It must be recognized that a redesign of the building structure will affect all other building systems. Efforts will be made to remain cognizant of this issue and briefly address any considerable impacts due to the change in structural system throughout the report. Two specific issues that will be analyzed and compared are the effect on the construction cost and scheduling and acoustical performance of the floor systems.

Criteria/Goals:

Certain criteria will be used to judge whether or not the concrete structure should be recommended over the steel structure. The final recommendation will be based on whether or not the proposed concrete design achieves the following goals:

- Perform adequately under all loading conditions with reasonably sized members and reinforcing
- Maintain floor plan flexibility
- Limit wind drift to H/400
- Avoid significant cost increases and find potential areas of savings
- Maintain or reduce the current schedule duration for structural construction
- Achieve target goals for acoustical performance of wall/floor assemblies and reverberation time

VI. DEPTH STUDY – CODES, MATERIALS, AND LOADS

This depth study includes the redesign of the Howard County General Hospital Patient Tower as a concrete flat plate system. Building codes and new design loads are outlined, followed by design methods and results for the two-way slab, concrete columns, concrete beams, and spread footings. The new floor system was modeled in RAM Structural System, and other design programs such as PCA Slab and PCA Column were also utilized. Supporting calculations and spreadsheets can be found in the Appendices at the end of the report.

Codes and Standards:

The existing hospital began design in 2004 and hence utilized earlier versions of the current building codes. For this redesign, the 2006 International Building Code was used, which references ASCE 7-05. All concrete design and analysis is in accordance with Building Code Requirements for Structural Concrete ACI 318-05.

Material Strengths:

Concrete

Application	f'c @ 28 days	Weight (pcf)
Slabs-on-grade	3000 psi	145
Concrete Slab	5000 psi	145
Footings	3000 psi	145
Columns	5000 psi	145
Beams	5000 psi	145

Steel

Materials	Fy (ksi)
Reinforcing Steel	60

Load Combinations:

The following load combinations were considered for design of the new concrete system in accordance with Chapter 9 of ACI 318-05:

(1) 1.4D
(2) 1.2D + 1.6L + 0.5L_r
(3) 1.2D + 1.6L_r +/- 0.8W
(4) 1.2D + + 0.5L + 0.5L_r +/- 1.6W
(5) 1.2D + 0.5L +/- 1.0E
(6) 0.9D +/- 1.6W
(7) 0.9D +/- 1.0E

These load combinations will later be referenced throughout the report, specifically for column design.

Dead Loads:

The floor dead load is composed of the 10" normal weight concrete slab and a 15 psf superimposed dead load to account for mechanical ductwork, floor finishes, suspended ceilings, etc. Therefore, a total uniform dead load of 140 psf was used for design. The 6" drop panels also contribute additional dead load at all column locations due to the additional concrete weight.

The total exterior dead load at the building perimeter consists mainly of the 8" precast panels with the lightweight glass and aluminum contributing minimally. Based on the typical façade configuration of precast, glass, and aluminum stripes, a line load of 1000 lb/ft for the exterior wall was assumed along the perimeter of the building.

The main roof slab is the same as the typical floor slab, so the dead load due to self weight is the same. However, a 5 psf superimposed dead load was assumed to account for roofing materials, for a total dead load of 130 psf. Drop panels once again contributed self weight at the column locations.

Live Loads:

Most of the design live loads were included on the structural general notes and were verified with the newer code, ASCE 7-05. Any live loads not listed in the structural general notes were taken from chapter 4 of ASCE 7-05. A live load of 100 was used for the hospital, though not required, for future flexibility reasons.

Location	Load	Comments
Framed Floor Areas	100 psf	80LL + 20 for Partitions
Lobbies/Stairs	100 psf	
Storage	125 psf	Unreducible
Penthouse	125 psf	Unreducible
Roof	30 psf	Unreducible

Snow Load:

Snow load is not typically greater than the 30 psf roof live load in the Mid-Atlantic area where the hospital is located. In this case, the ground snow load is 25 psf while the calculated flat roof snow load is 22 psf.

Snow drift must also be considered from the higher roofs, such as from the penthouse roof to main roof area. Refer to the Appendix D these snow drift diagrams and calculations. It was determined that leeward snow drift controlled for all drift conditions

Ground Snow Load (P_g)..........25 psf
Snow Exposure Factor (C_e)..........1.0
Importance Factor (I_s)..........1.1
Thermal Factor (C_t)..........1.0
Flat Roof Snow Load (P_f)..........19.25 psf

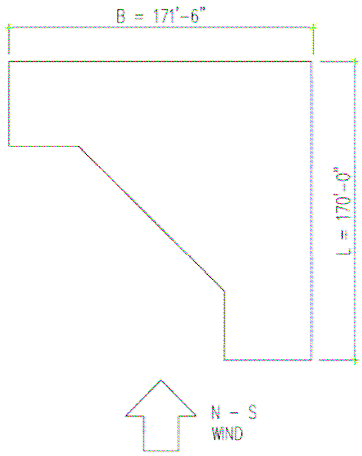
The flat roof snow load is less than the code minimum of

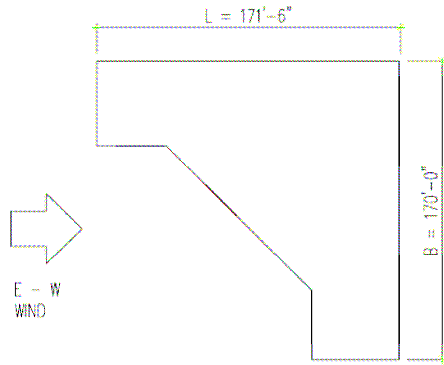
$P_{f,\ min}$ = I*20 psf = 1.1*20 psf = 22 psf

Therefore, the minimum flat roof snow load of 22 psf will be used. This will be added to the drift snow load where applicable. Otherwise, it is less than the 30 psf roof load, as expected, so the roof live load will control.

Wind Load:

Wind loads were determined in accordance with ASCE 7-05 and with the assumptions listed below. The building is enclosed and cannot use the simplified design procedure outline in ASCE 7-05 because the mean roof height is over 60 feet. Therefore, the more extensive analytical procedure must be used. Below is a diagram showing the direction of wind loading with respect to building orientation.





The following factors were obtained from Section 6 of ASCE 7-05 to calculate the applied wind pressures.

Basic Wind Speed (V)	90 mph
Importance Factor (I)	1.15
Wind Directionality Factor (K_d)	0.85
Exposure Category	B
Topographic Factor (K_{zt})	1.0
Enclosure Classification	Enclosed
Internal Pressure Coefficient (GC_{pi})	+/- 0.18

The factors listed above were input into RAM Frame so wind loads could be calculated and distributed accordingly. Hand calculations in Appendix A verify the applied forces shown below are accurate.

North-South Wind

APPLIED STORY FORCES

Type: Wind_IBC03_1_Y

Level	Ht ft	Fx kips	Fy kips
penthouse roof	88.50	0.00	25.60
roof	70.50	0.00	73.21
fourth	54.00	0.00	66.03
third floor	36.00	0.00	64.42
second floor	18.00	0.00	59.15
		0.00	288.41

East-West Wind

APPLIED STORY FORCES

Type: Wind_IBC03_1_X

Level	Ht ft	Fx kips	Fy kips
penthouse roof	88.50	24.84	0.00
roof	70.50	61.09	0.00
fourth	54.00	55.56	0.00
third floor	36.00	54.10	0.00
second floor	18.00	49.54	0.00
		245.12	0.00

Seismic Loads:

A seismic analysis of the building was performed to determine the total base shear as well as the applied shear forces at each floor. The spectral response accelerations S_S and S_1 were obtained from the United States Government Seismic Design Values for Buildings (http://earthquake.usgs.gov/research/hazmaps/design) using the latitude and longitude of the Howard County General Hospital. The seismic loads were calculated using the equivalent lateral force method in accordance with ASCE 7-05. To determine the response coefficient, the seismic force system used was “Ordinary Concrete Moment Frames”.

A few important assumptions and/or decisions should be noted. The building is classified as Seismic Use Group III rather than IV because no surgery facilities are located within the new tower addition. This results in the importance factor of 1.25 rather than 1.5, which matches what the designer used in the existing design. Also, the total above grade height was taken to be 88.5 feet, which includes the penthouse, but does not take into consideration the basement, though it is above grade on one side. This assumption was made because it is assumed that the first floor cannot experience any significant lateral movement as it is braced in all but one direction.

Mapped Spectral Response Accelerations	$S_S = 0.160$ g
	$S_1 = 0.050$ g
Site Class	D
Seismic Use Group	III
Importance Factor (I)	1.25
Site Class Factors	Fa = 1.6
	Fv = 2.4

Adjusted Spectral Response Accelerations	S_{MS} = 0.256
	S_{M1} = 0.12
Design Spectral Response Accelerations	S_{DS} = 0.171
	S_{D1} = 0.08
Seismic Design Category	B
Response Modification Coefficient (R)	3.0
Approximate Fundamental Period (T_a)	0.9044
Fundamental Period (T)	1.538
Seismic Response Coefficient (C_s)	0.0217
Effective Seismic Weight (W)	16,108 k

This criterion was be utilized for hand calculations of story forces and shears which can be found in Appendix A. These forces were then compared to those obtained using the RAM Model, which are shown below. The hand calculated forces are very similar to those calculated in RAM, so RAM is considered to be accurate in its determination of seismic forces. Design in this report will utilize the loads calculated in RAM.

APPLIED STORY FORCES

Type: EQ_IBC03_X_+E_F

Level	Ht ft	Fx kips	Fy kips
penthouse roof	88.50	60.85	0.00
roof	70.50	118.76	0.00
fourth	54.00	93.74	0.00
third floor	36.00	50.64	0.00
second floor	18.00	17.68	0.00
		341.67	0.00

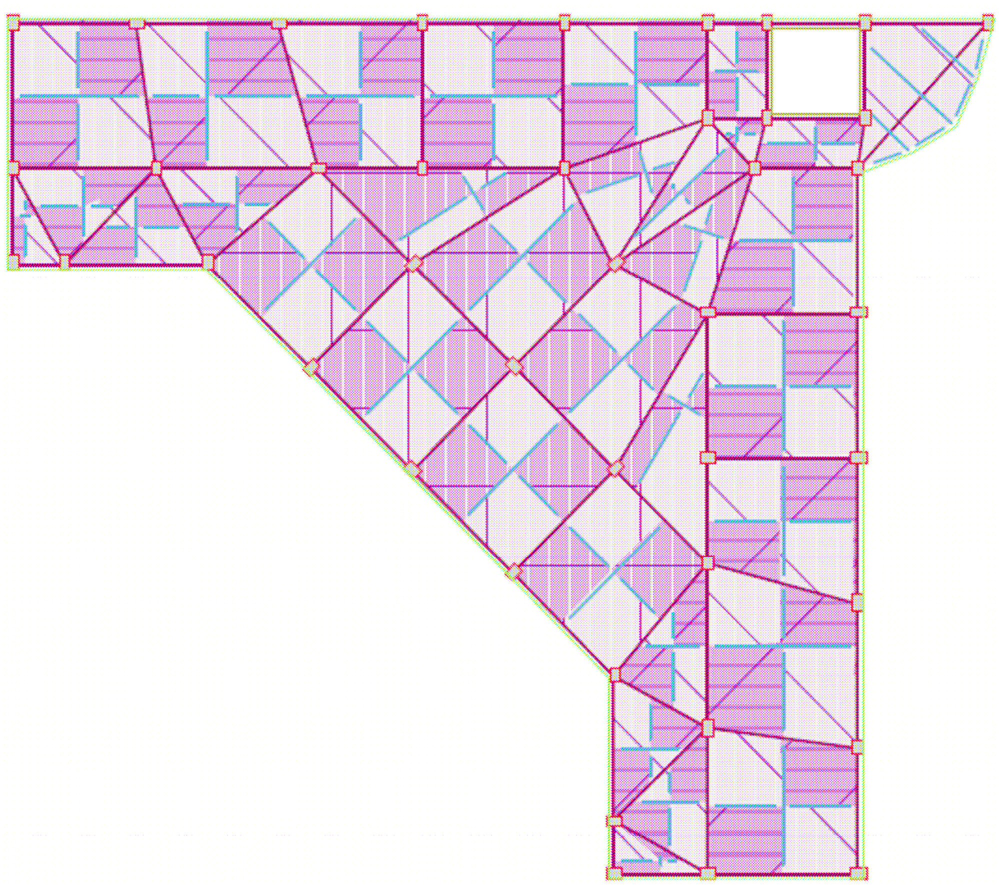
It is important to note that seismic loads control over wind for the unfactored base shear, but they may not control for individual members once factored. Slab design lateral moments will be input as unfactored loads for both wind and seismic, and PCA Slab will design for the controlling case. RAM will design the columns based on the controlling load combination as well, which for lateral loads may be either wind or seismic.

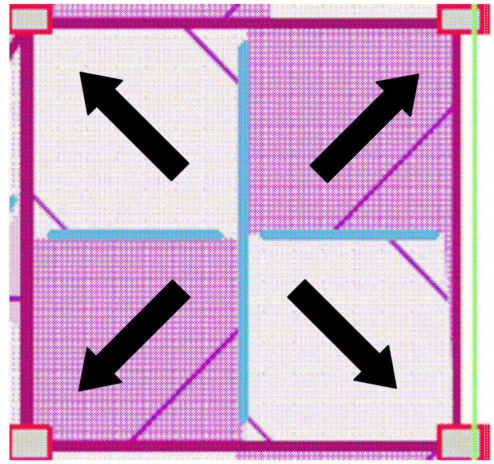
VII. DEPTH STUDY – RAM MODELING

RAM Structural System is a complex program that contains several modules with different design capabilities. After consulting with Rathgeber/Goss Associates, the structural engineer for Howard County Hospital patient tower, they suggested using RAM for this project. Having never used RAM for concrete design, it was necessary to learn the proper way to model a concrete flat slab. Modeling the structure effectively is vital to obtaining accurate loads and design results. It is important to understand the modeling process before moving on to the results and outputs.

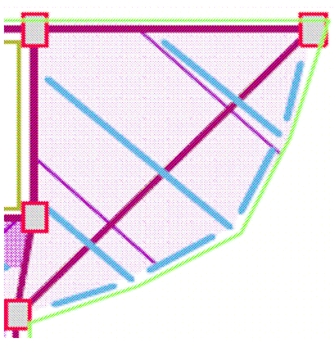
Slab Modeling:

As with any model built in RAM, the first step was to define grid lines for the columns. Very few column locations changed, so the existing grid line coordinates from the structural plans were input into RAM. Once the columns were inserted in their appropriate locations and connected with beams, the concrete slab could be created. A 10" concrete slab was specified in the property table, then assigned as shown below.





The dark reddish purple beams were defined as lateral beams and connect the lateral columns. This will later be explained in more detail. The blue beams are gravity beams and serve solely to divide each bay into four quadrants. These beams are specified as 10 inches deep, which is simply the slab thickness. It may be helpful to think of these beams as fictional since they are techncially just a portion of the 10" slab. The slab, shown in dark and light purple is then assigned at either 45 degrees or 135 degrees in each of the four quadrants. Modeling the slab in this manner simulates how a two-way slab would transfer the floor loads into the columns.



The quarter-circular area shown to the left is located at the top right of the floor plan and houses the waiting room/lobby. This portion of the building is framed using a one-way slab, cantilevering over a concrete girder. Once again the blue beams are simply gravity beams defined with a depth equal to the slab thickness. The curved concrete slab cantilevers over the girder a maximum of 8 feet.

Lateral Modeling:

Once the two-way slab is defined, the lateral system has to be modeled. As mentioned above, the reddish purple beams are specified to be lateral members. All of the columns are also defined as lateral members. In order for RAM Frame to run, all lateral members must be assigned sizes. For the columns, a 24" by 24" column was input in the property table and assigned to all columns. Defining sizes to the lateral beams is a more complex issue that requires further explanation.

It is important to understand that these beams, like the gravity beams, are fictional and used to model the slab as a lateral element. The final structural design will not include actual beams between all columns, but instead the slab itself will span from column to column and resist the lateral loads. Therefore, all of these beams will be defined as 10" deep, just like the gravity beams, to match the slab depth. The beam width is used model the stiffness of the slab so that the lateral loads are accurately distributed. Each beam is defined a width equal to 0.35 times its tributary width. This represents the cracked portion of the beam, which is specified in Section 10.11.1 of ACI 318-05 to be $0.35*I_g$. Defining the fictional beams in this manor allows RAM to model the distribution of lateral loads throughout the slab.

Running/Using the Model:

Once the slab and lateral system are modeled in the Modeler and the Data Check has verified that all errors or warnings are resolved, it is time to move forward with design. For the sake of this project, RAM will be used to obtain the lateral moments in the slab, to design the columns, and to size the foundations. On the main RAM Manager screen, certain criteria needs to be set for the model to run as desired. Here, the user can specify codes and tables used for design. Most importantly for this design is the self-weight criterion. RAM will automatically calculate self-weight for whichever members the user specifies. For this model, the self-weight of the slab/deck and columns should be included, but not the self weight of the beams. This is very important because if the beam self weight is included, the program will double count part of the slab weight, since the beams are fictional and actually just a part of the slab. Once this is specified, design can begin.

RAM Frame will calculate the slab lateral moments, which can then be input into PCA Slab to design the slab itself. Before running RAM Frame, all of the load cases are defined. When adding the wind and seismic load cases, the criteria and factors outlined in the loads portion of this report can be input. The load case analysis will run each of the load cases and provide unfactored member forces. At that point, the wind and seismic moments for each lateral beam can be determined. Once again, since these beams are simply modeling the slab, the "beam" moments are really the slab moments. Those are the moments that will be input into PCA Slab.

Once RAM Frame is run and all lateral load cases are analyzed, RAM Concrete will perform the concrete column design. First, a concrete gravity analysis is performed, then the user can switch to concrete column design mode. Column sizes have already been assigned, but now a bar pattern will be specified. The concrete column design will reference the gravity analysis and the lateral load cases defined in RAM Frame to determine the member design loads. All load combinations can be generated according to the selected code, which allows for the controlling load combination to be determined. The design results will include the bar pattern, maximum axial load, and maximum moments.

The final use of RAM for this project will be in determining foundation sizes. The Foundation Design module takes the column loads from all of the previous analyses and calculates the required footing size and reinforcing based on the input soil properties.

Although RAM's capabilities go much further than those outlined above, this is how RAM will be utilized for analyzing and designing the new concrete structure for Howard County General Hospital's patient tower addition.

VIII. DEPTH STUDY – CONCRETE DESIGN

In order to determine if the proposed concrete flat plate system is advantageous compared to the existing steel system, a full concrete redesign must occur. First, preliminary sizes were chosen and any required structural layout changes must be addressed. All loads have been determined, as included in the previous loads section. These loads will be used to design the slab and columns using computer software. Certain locations may require beams, which will be designed by hand. Foundations will be resized according to the new building loads. At that point it will be possible to compare the two systems structural performance and make a recommendation as to which system is more efficient.

Schematic Design:

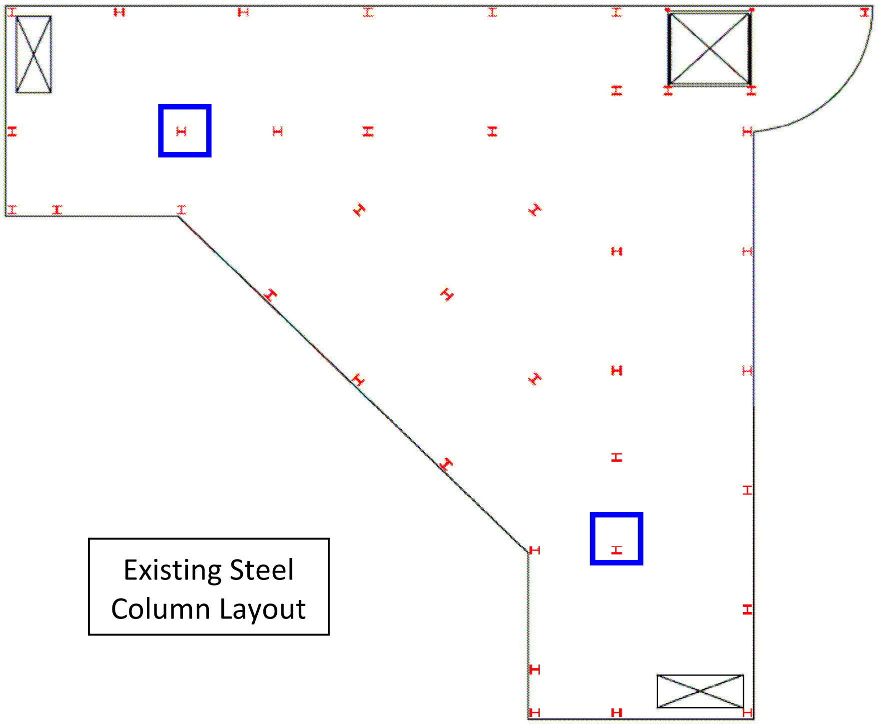
For a starting point, the 10" slab specified in the CRSI Handbook was maintained, but it was decided to increase the column sizes to 24" by 24". Upon review of the architectural plans, it was found that the existing steel columns were built up to 24" by 24" sections. Therefore, using 24" by 24" columns will minimize the impact on the architectural layout. Larger column sections will also decrease the amount of required reinforcing steel and reduce the likelihood of slender columns due to the large 18 foot floor to floor heights.

The 10" slab meets the requirements for minimum slab thickness in accordance with Table 9.5(c) from ACI 318-05. The typical 29 foot span is checked below, eliminating deflection requirements for typical conditions. However, deflections will be checked later in the report for the 30.5' end span.

TABLE 9.5(c)—MINIMUM THICKNESS OF SLABS WITHOUT INTERIOR BEAMS

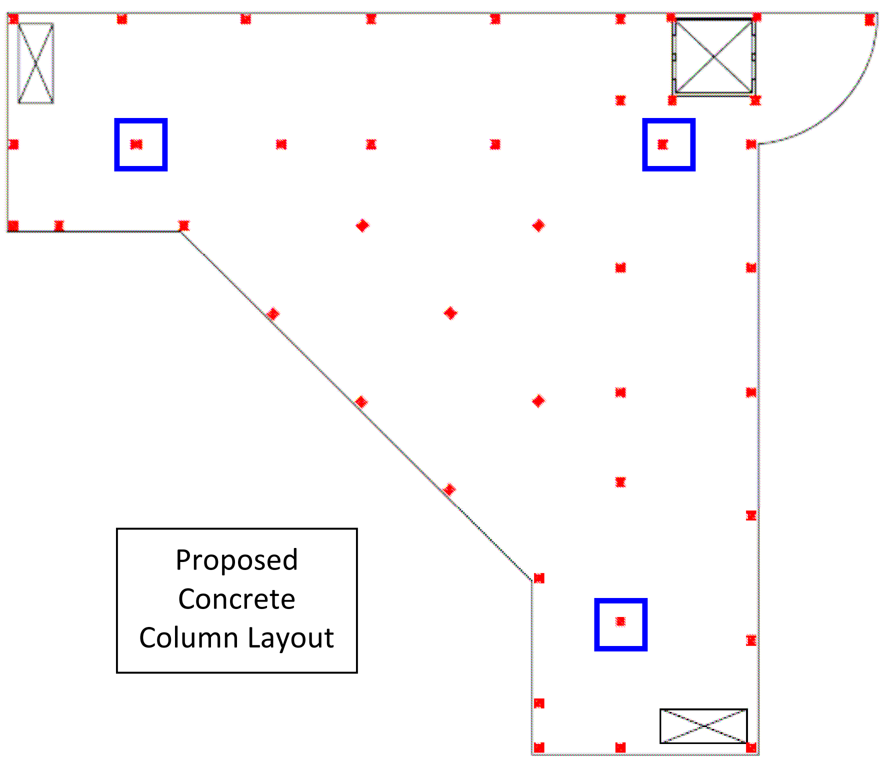
Yield strength, f_y, psi*	Without drop panels†			With drop panels†		
	Exterior panels		Interior panels	Exterior panels		Interior panels
	Without edge beams	With edge beams‡		Without edge beams	With edge beams‡	
40,000	$\frac{\ell_n}{33}$	$\frac{\ell_n}{36}$	$\frac{\ell_n}{36}$	$\frac{\ell_n}{36}$	$\frac{\ell_n}{40}$	$\frac{\ell_n}{40}$
60,000	$\frac{\ell_n}{30}$	$\frac{\ell_n}{33}$	$\frac{\ell_n}{33}$	$\boxed{\frac{\ell_n}{33}}$	$\frac{\ell_n}{36}$	$\boxed{\frac{\ell_n}{36}}$
75,000	$\frac{\ell_n}{28}$	$\frac{\ell_n}{31}$	$\frac{\ell_n}{31}$	$\frac{\ell_n}{31}$	$\frac{\ell_n}{34}$	$\frac{\ell_n}{34}$

$l_n = 29' - 24''/12 = 27'$
$l_n/33 = 27/33 = 9.8''$
Therefore 10" slab OK



Existing Steel Column Layout

Once preliminary sizes were determined, slight changes were made to the existing column layout to minimize the need for concrete beams and reduce the length of a few longer bays. All in all, one column was added and two columns were moved. The adjacent plan shows the existing column layout, while the one below shows the new concrete column layout. The two columns that were moved and the column that was added are highlighted in blue.

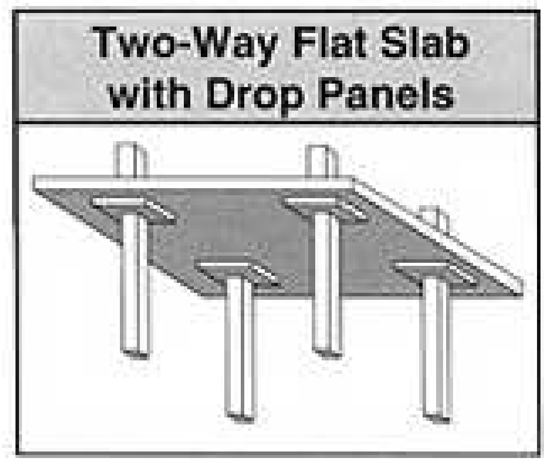


Proposed Concrete Column Layout

Moving and adding these columns reduced what were over 39 foot spans to approximately 29 foot spans. This allowed the two-way slab to support the loads without requiring concrete beams or continuous drop panels at these locations.

These few moves did slightly impact the floor plan, but not drastically. This portion of the floor plan contains two patient rooms and a conference room, which can easily be reconfigured to work with the new column layout.

Slab Design:



After obtaining all of the required loads, gravity and lateral, PCA Slab was used to design the two-way slab for a typical floor. As evident in the column layout diagrams above, there are some unusual conditions due to the building orientation. This proved difficult to create “typical” slab runs, so each column line had to be designed individually, resulting in approximately ten slab runs in each direction. This allowed for a complete and thorough design that could not have been accomplished by modeling only typical conditions. Best attempts were made to accurately model the slab in terms of span lengths and tributary widths. Since there are very few variations between the floor plans, the typical floor design is assumed to be sufficient at all levels. Due to time constraints, the roof slab was not fully designed. The 10” slab will be maintained at the roof to meet deflection requirements, but reinforcing would be reduced due to the decreased live load. The factored distributed load would decreased from the 328 psf typical floor load to 216 psf, therefore it can be estimated that the reinforcing would be reduced by approximately 30%.

Moving forward with a 10” slab thickness per the CRSI Handbook and 24” square columns for architectural reasons previously explained, drop dimensions were determined in accordance with ACI 318-05. From ACI 13.2.5, “When used to reduce the amount of negative moment reinforcement … a drop panel shall project below the slab at least one-quarter of the slab thickness and extend in each direction from the centerline of support a distance not less than one-sixth the span length measured from center-to-center of supports in that direction”. From these requirements, the typical drop panel for a typical 29 by 29 foot bay was determined to be 9’-8”x 9’-8”. Through trial and error, PCA Slab outputs proved that a 6” thick drop panel would provide sufficient capacity, which is greater than one-quarter of the slab thickness, and therefore acceptable.

Lateral moments were obtained in RAM for each span according to the wind and seismic loads previously mentioned. Since only a “typical” floor was being designed, the worst case lateral moments from each of the floors and all load cases were selected and input into PCA Slab under “lateral effects”. These were all unfactored moments, so PCA will factor them according to the specified load combinations. The gravity loads were input in pounds per square foot then calculated based on the span length and width. Since PCA Slab calculates the self weight of the slab and drops automatically, only a 15 psf superimposed dead load was input. The typical live load of 100 psf was used for the typical floor design. The exterior wall load of 1000 lb/ft was input as a line load along the exterior spans.

After careful modeling and interpretation of the output files, it was found that (14) #7 bars each way at the interior columns and (12) #7 each way at the exterior columns provided sufficient column strip top steel in most cases.

Middle strip top steel for the typical 29 foot tributary width was found to be (10) #7, spaced evenly within the 14.5 foot middle strip. For larger or smaller middle strips, bars should be spaced at 18" within the middle strip width.

Typical concrete floor systems utilize a bottom mat of equally spaced bars to resist positive moments. For this design, the bottom mat was determined to be #6 @ 10" on center each way. This was sufficient to resist the positive moments in most locations. Where there are longer spans and hence larger mid span moments, additional bars will be specified.

When comparing the typical required reinforcing to the preliminary reinforcing tabulated in the Chapter 10 of the CRSI Handbook, it can be concluded that this slab reinforcement is very reasonable.

f'_c = 4,000 psi
Grade 60 Bars

FLAT SLAB SYSTEM
SQUARE EDGE PANEL — With Drop Panels
No Beams

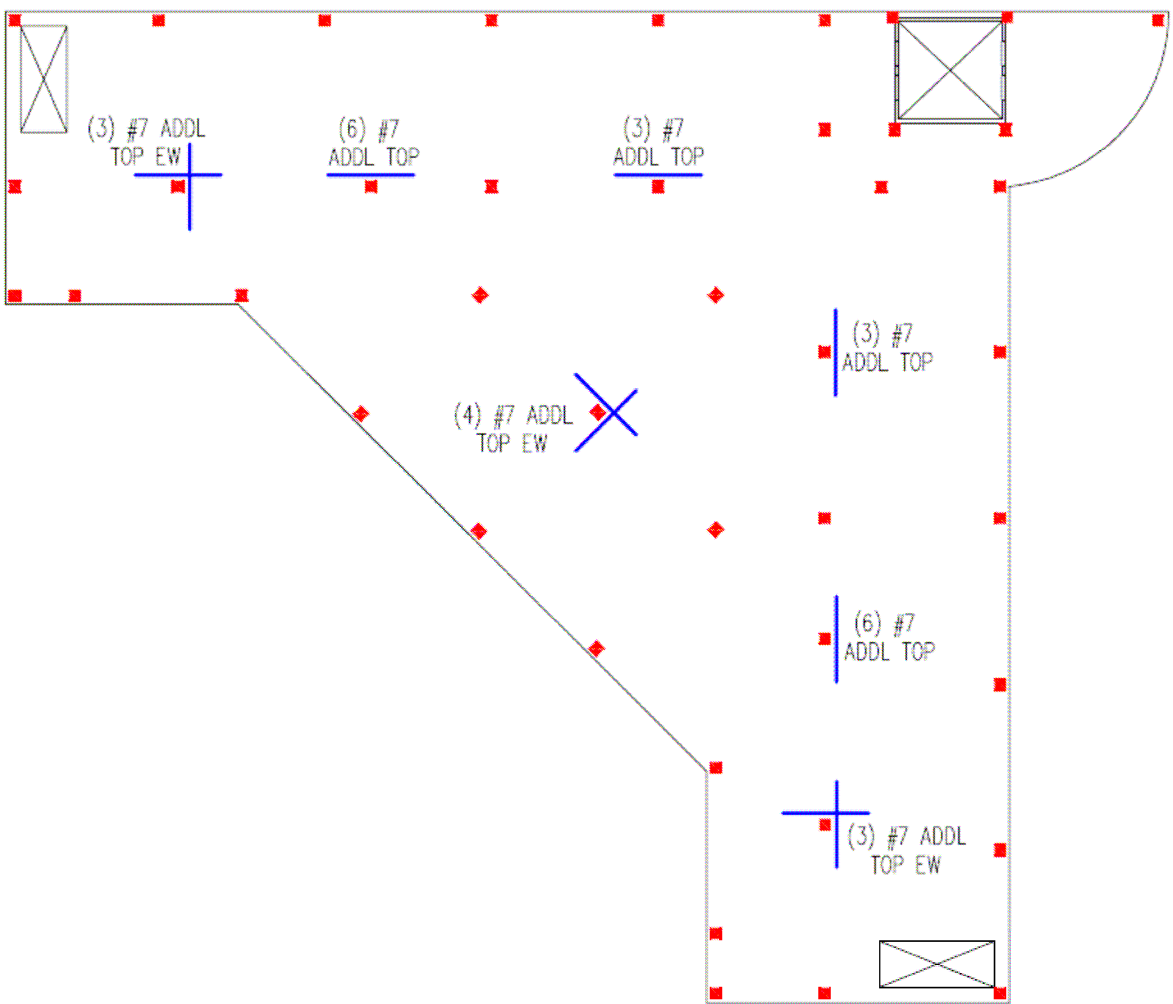
SPAN c.-c. $\ell_1 = \ell_2$ (ft)	Factored Superimposed Load (psf)	Square Drop Panel		(3) Square Column		REINFORCING BARS (E. W.)						MOMENTS		
						Column Strip (1)			Middle Strip		Total Steel (psf)			
		Depth (in.)	Width (ft)	Size (in.)	γ_f	Top Ext. +	Bottom	Top Int.	Bottom	Top Int.		Edge (−) (ft-k)	Bot. (+) (ft-k)	Int. (−) (ft-k)
h = 10 in. = TOTAL SLAB DEPTH BETWEEN DROP PANELS														
29	100	8.50	9.67	12	0.737	13-#5 2	22-#5	18-#5	15-#5	12-#5	2.91	206.7	413.4	556.5
29	200	8.50	9.67	16	0.758	13-#5 4	12-#8	13-#7	19-#5	16-#5	3.81	271.2	542.5	730.3
29	300	8.50	9.67	22	0.718	15-#5 4	20-#7	16-#7	10-#8	20-#5	4.92	334.3	668.6	900.1
29	400	8.50	11.60	28	0.639	17-#5 2	15-#9	14-#8	12-#8	10-#8	5.83	392.7	785.4	1057.3

SQUARE INTERIOR PANEL
With Drop Panels(2)
No Beams

Factored Superimposed Load (psf)	(3) Square Column	REINFORCING BARS (E. W.)					Concrete (cu. ft / sq. ft)
		Column Strip		Middle Strip		Total Steel (psf)	
	Size (in.)	Top	Bottom	Top	Bottom		
h = 10 in. = TOTAL SLAB DEPTH BETWEEN DROP PANELS							
100	12	17-#5	15-#5	12-#5	11-#5	2.58	0.912
200	19	16-#6	19-#5	15-#5	13-#5	3.27	0.912
300	21	15-#7	10-#8	10-#7	16-#5	4.34	0.912
400	26	13-#8	12-#8	12-#7	10-#7	5.06	0.947

To further verify the accuracy of PCA Slab's design, hand calculations using the direct design method for a sample slab strip are included in Appendix B. This method of analysis does not include lateral effects, so the hand calculated results were compared to a PCA Slab output without the inclusion of lateral effects. The lateral moments are already deemed accurate, as they were input based on the RAM results.

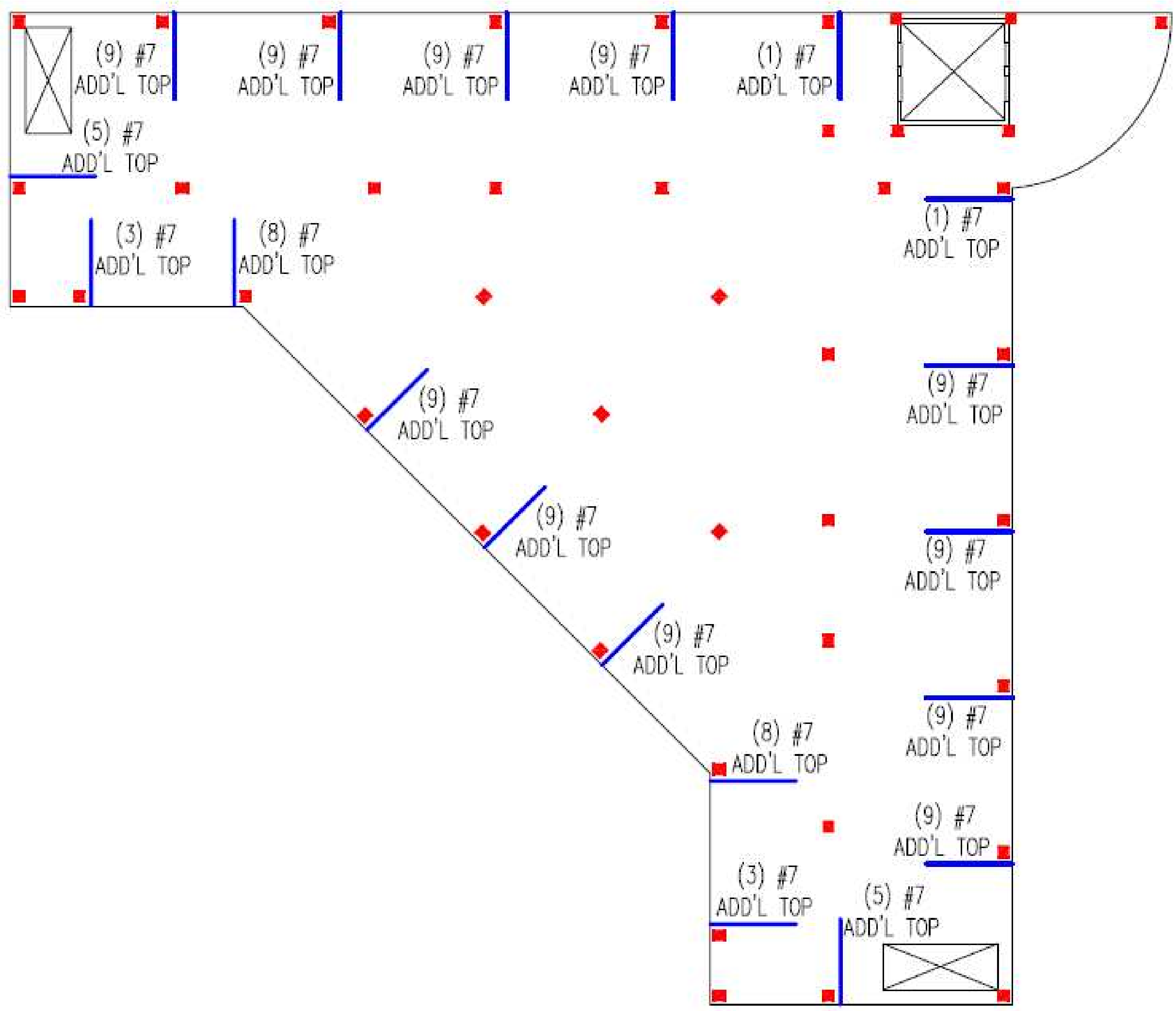
Although the reinforcing outlined above is sufficient in most cases, at some locations additional top steel was required to resist excessive negative moments at certain supports. The requirements for additional top steel at the interior columns are summarized in the diagram below.



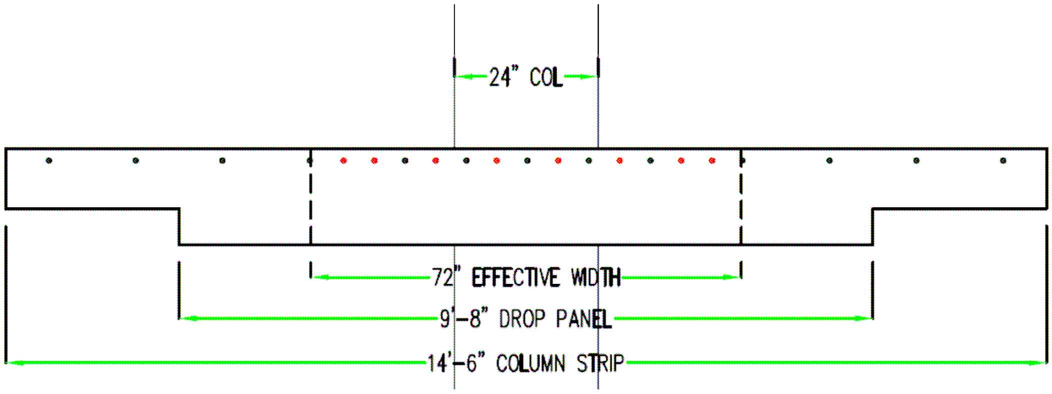
The exterior columns required additional steel within the effective width to transfer the negative unbalanced moment from the end span. The effective width for a typical exterior column is calculated to be:

Effective width = c + 2*1.5h
c = the column dimension
h = slab plus drop depth
Effective width = 24 + 2*(1.5*16) = 72"

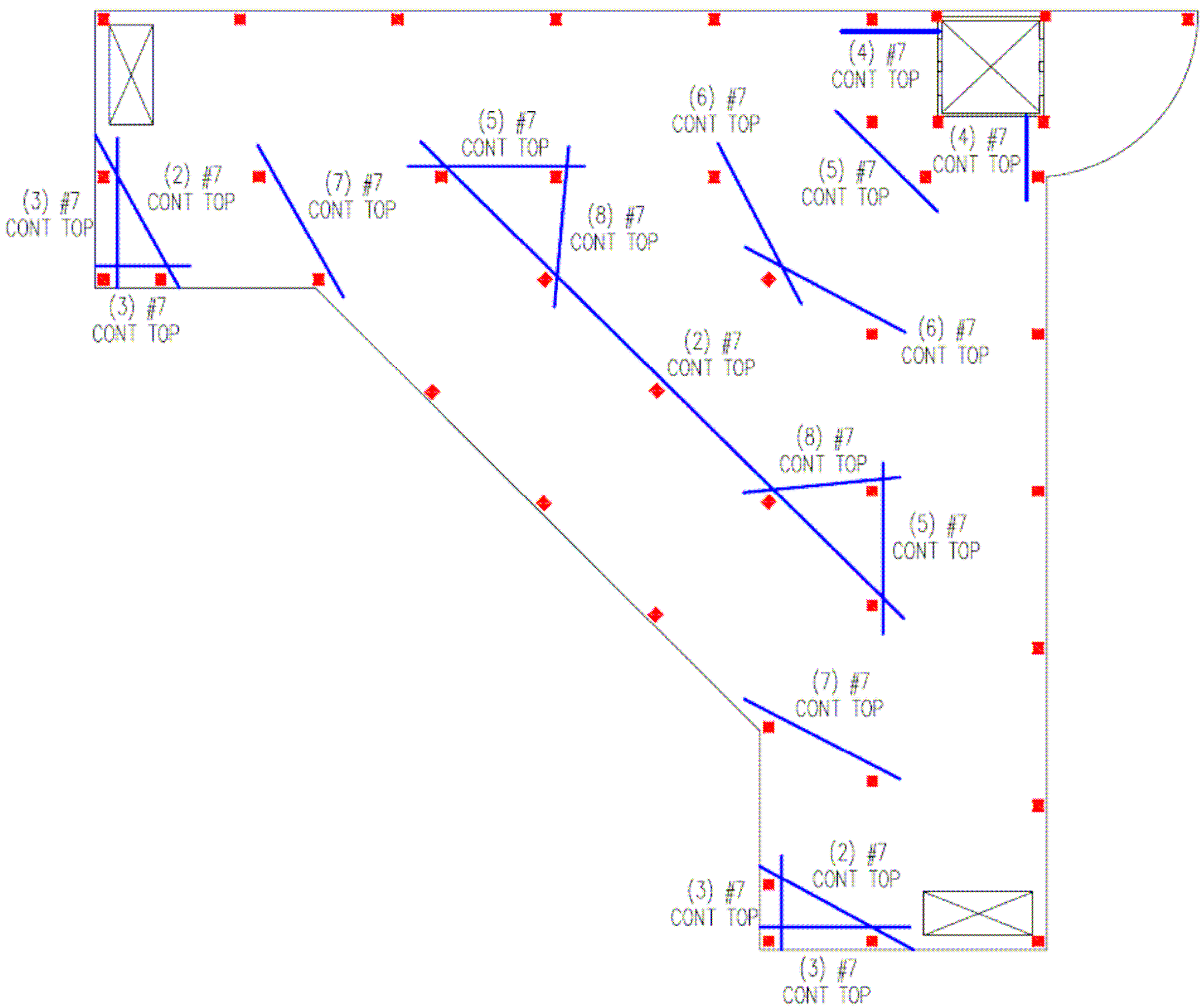
Below is a summary of the additional steel required within this 72" effective width at exterior column locations.



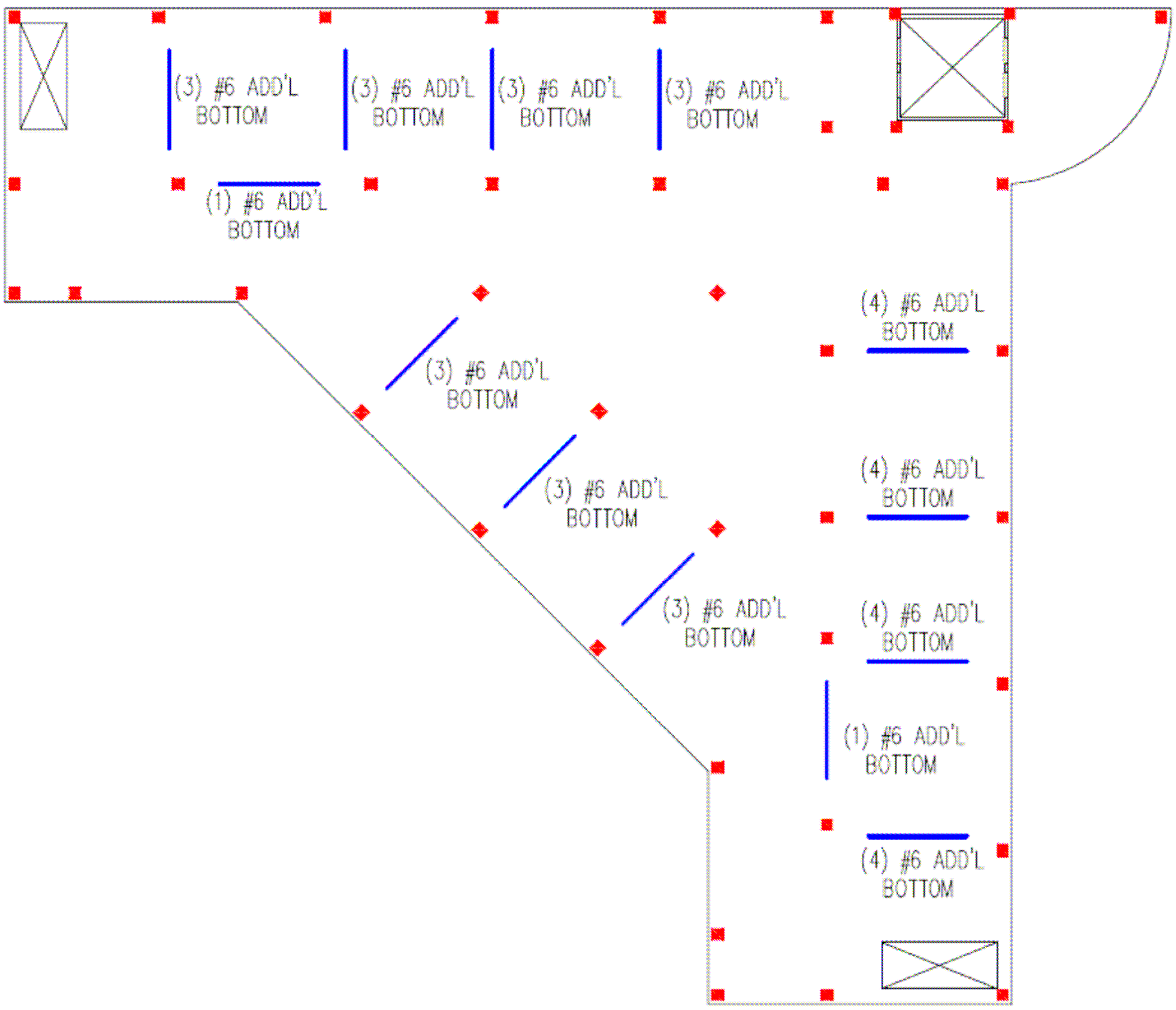
Below is a cross section demonstrating how the additional steel requirements would be achieved. The (12) #7 typical bars are shown in black while the additional (9) #7 bars are in red. The effective width is shown with a dashed line.



In some cases, continuous top steel was required from one support to the next. This is expected to occur where short spans are adjacent to longer spans, creating a need for negative reinforcement at mid span. Below is a diagram showing the requirements for continuous top steel.



As previously mentioned, a bottom steel mat of #6 bars @ 12" on center each way was sufficient for all middle strip bottom steel and most column strip bottom steel. For some of the longer spans, additional bottom steel was required to resist the higher mid span moments. This diagram shows where additional bottom steel was required, primarily at the 29 foot and 30.5 foot end spans with a 29 foot tributary width.



Although not included above, there would also be a requirement for additional bottom steel at the slab depressions occur on the second through fourth floors. These depressions typically occur at mid span, reducing the "d" with which bottom steel can resist the moment. Considering the slab depression of 2", the "d" would be reduced by 2" as well. The moment would increase as "d" decreases. Due to time provisions this was not calculated for every slab depression, but a sample calculation is included in Appendix B.

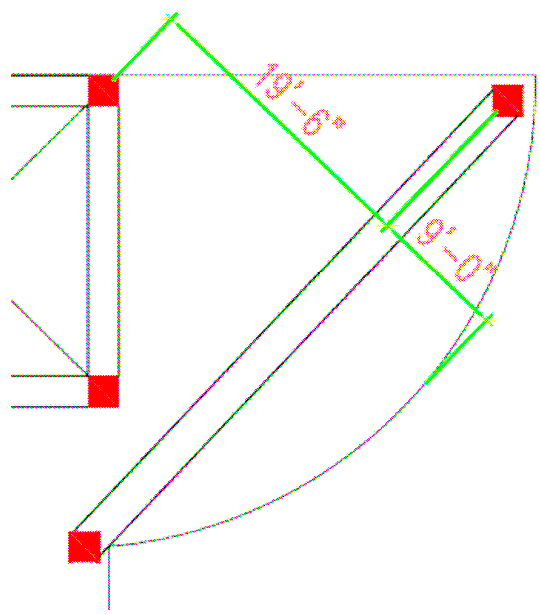
As previously mentioned, at the 30.5 foot spans the 10” slab does not meet the minimum slab thickness required per ACI 318-05. These spans must therefore be checked for deflection. I chose to limit the total load deflection to l/360 rather than l/240 because this is a hospital and large deflections could prove to be an issue for sensitive equipment.

$$\Delta_{allowable} = l/360 = (30.5*12)/360 = 1.017”$$

The maximum long term deflection, obtained in PCA Slab, was found to be 0.923”, which is less than the allowable deflection and therefore acceptable.

Live load deflection is assumed to be satisfactory as well, since the live load is less than half of the total load, and therefore accounts for less than half of the deflection.

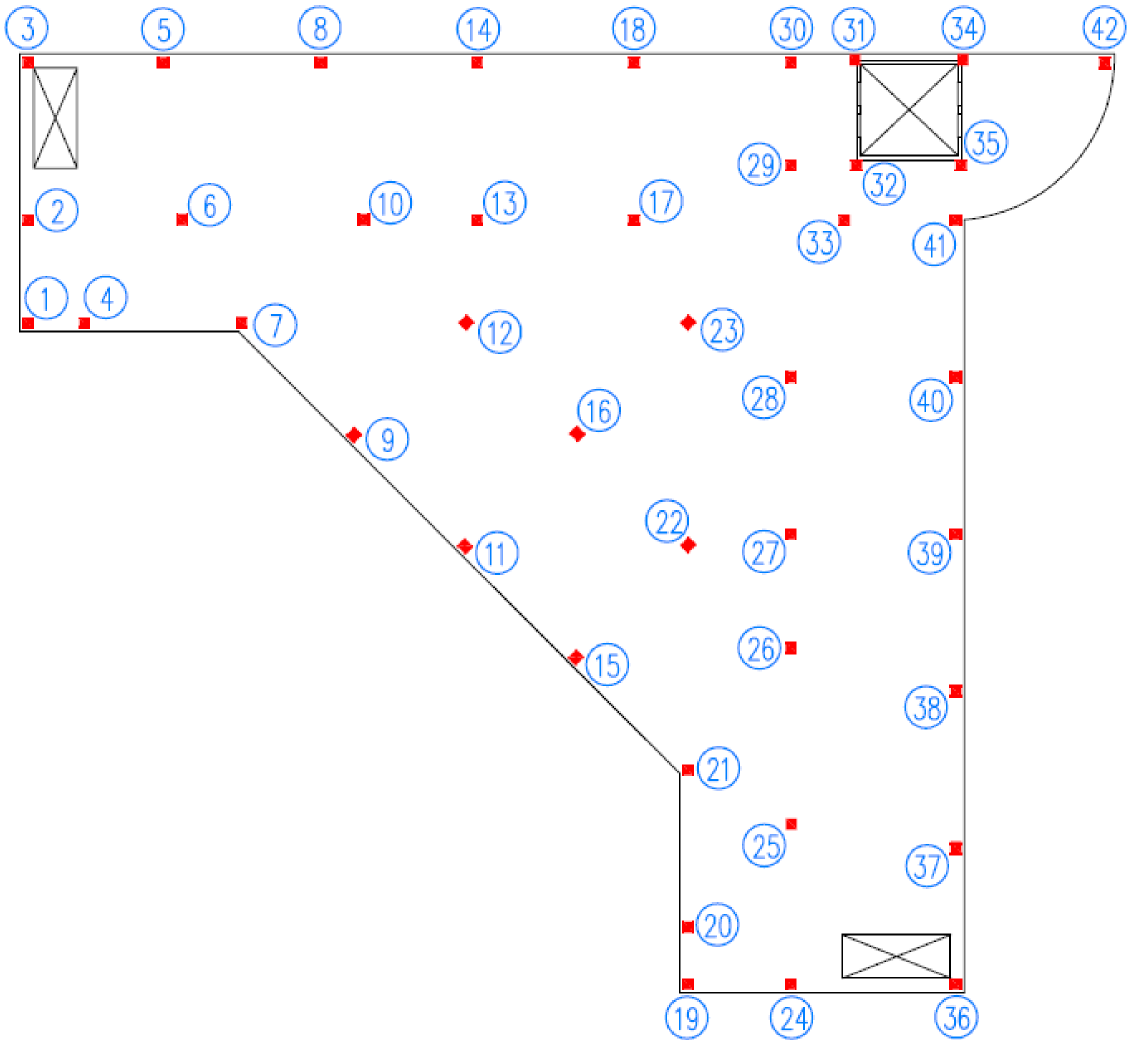
Finally, the one-way slab that houses the elevator lobby and waiting room must be designed. This slab will also be designed using PCA Slab. The curved portion of the slab cantilevers approximately 9 feet beyond the centerline of the beam. To the right is a diagram showing this portion of the plan with dimensions,



Using PCA, this slab was designed to be 8“ thick with #6 top bars at 12” on center. Bottom bars are only required in the 19’-6” span, not the cantilever, and should be #6 bars spaced at 18” on center. #5 bars could also be used at 14” on center, but since #5 bars are not used anywhere else in the building, keeping with #6 bars seems to be a better option. Maximum deflection at the cantilever is 0.21 inches, which is within the l/360 limit.

Column Design:

The columns must be designed for biaxial compression and flexure due to both gravity and lateral loads. Because of the level of complexity, they were designed using RAM Concrete, which directly references RAM Frame for the lateral load conditions. Below is a column layout plan identifying all column numbers. These column numbers will be referenced throughout the report.



Gravity loads were verified by performing takedowns on a few representative columns at the base. These calculations can be found in Appendix C, but below is a spreadsheet comparing the gravity loads calculated by hand to those obtained from RAM for the selected columns.

Column #16

	Dead Load	Live Load	Gravity Load
Hand Calc's	815	267	1082
RAM Loads	787	279	1066
% Error	3.6%	4.3%	1.5%

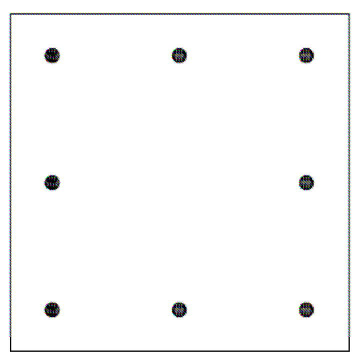
Column #6

	Dead Load	Live Load	Gravity Load
Hand Calc's	574	141	715
RAM Loads	602	145	747
% Error	4.7%	2.8%	4.3%

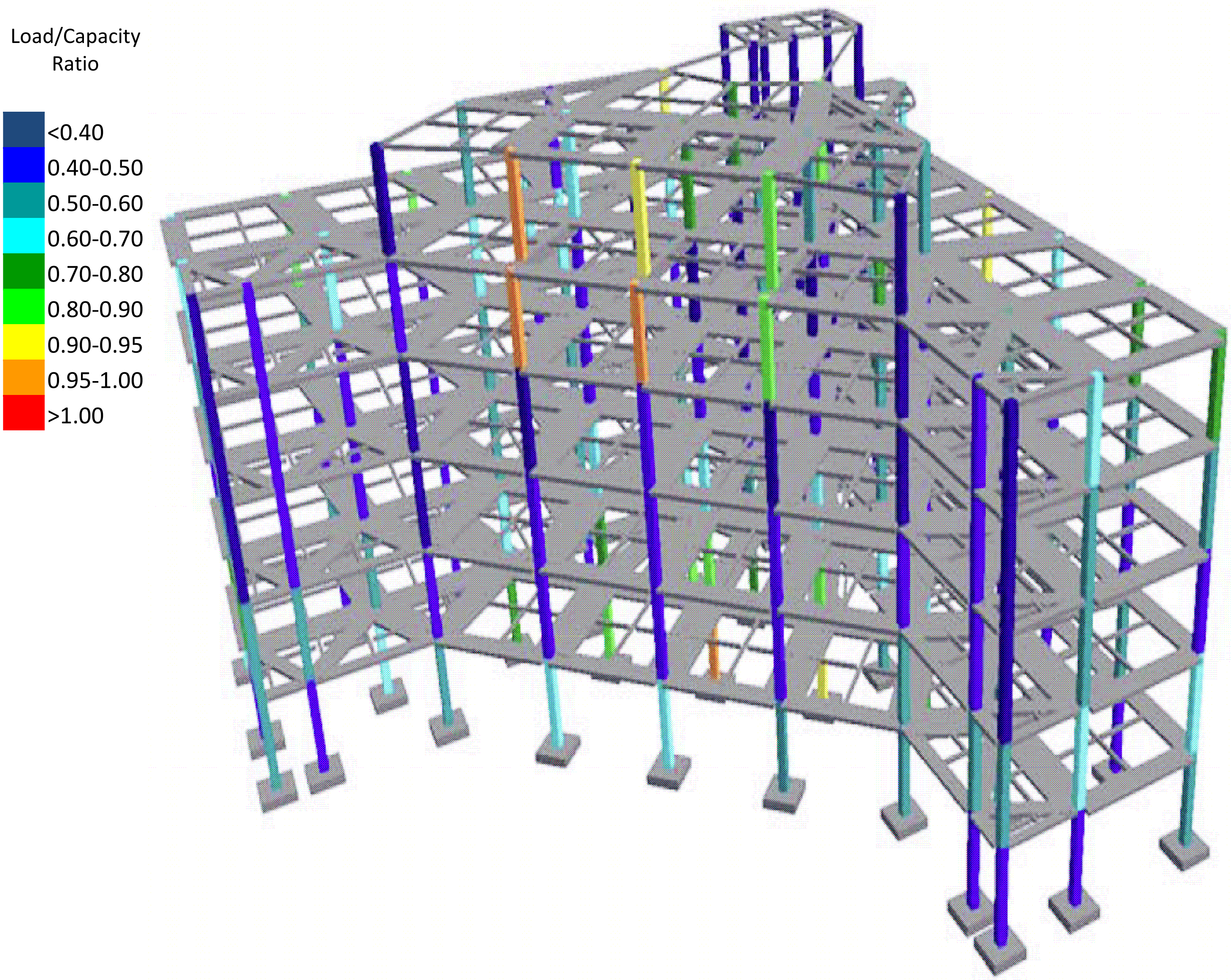
Column reinforcing was designed using RAM Concrete for combined gravity and lateral loads. The loads used for design reference the loads calculated in the frame analysis and concrete analysis modules. The column design tools allow the user to input the column size, 24" by 24" for this building, and specify a bar pattern. To begin, I input an 8 bar pattern, with equal spacing on all sides. RAM then specifies the required bar size based on the load combinations previously outlined. If the specified bar pattern is not adequate to resist the loads, RAM will identify that the column is overstressed and a new bar pattern can be specified.

A spreadsheet including the controlling load case, axial load, and major and minor moments for each column at each level can be found in Appendix C. The load combinations in the spreadsheet directly reference the previously included list of load combinations. The column numbers from the spreadsheet are consistent with the column plan above. It can be seen in this spreadsheet that in some cases, columns are primarily experiencing bending moments about one axis, with moments about the other axis being negligible. However, in other cases, columns are subjected to biaxial bending, most likely due to the irregular column layout. For the biaxial bending condition, select representative column designs were checked using PCA Column rather than hand calculations. For those columns subjected to primarily uniaxial bending, hand spot checks were performed. Both a sample PCA Column design and sample hand calculations are included in Appendix C.

It was concluded that all columns can sufficiently resist the maximum factored loads with (8) #8 bars. The fact that so little reinforcing is required is most likely because fairly large 24" by 24" cross sections were used. The (8) #8 reinforcing is slightly above the required minimum of $0.01*A_c$. The bar pattern for the columns is shown in the diagram to the right.



Below is a color coded diagram from RAM's column design module. This shows the load/capacity ratio based on the interaction equation for the controlling load combination for each column design.

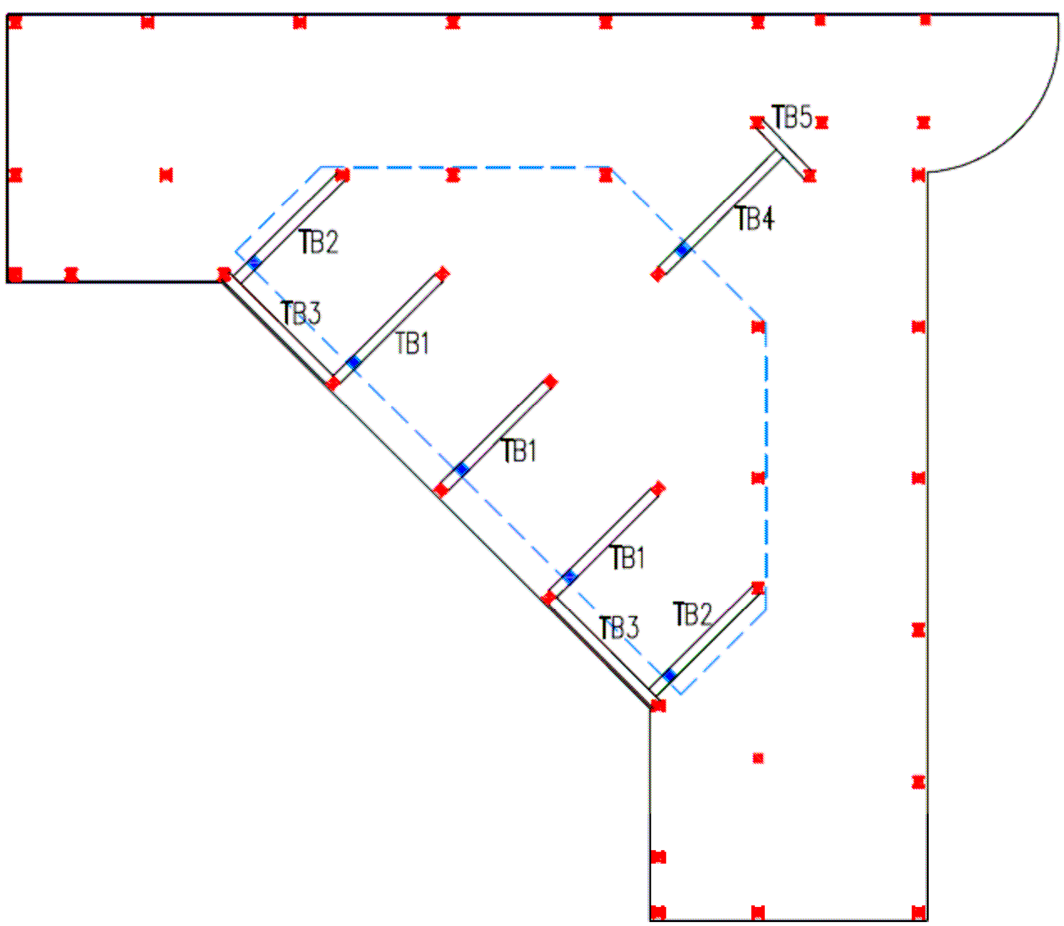


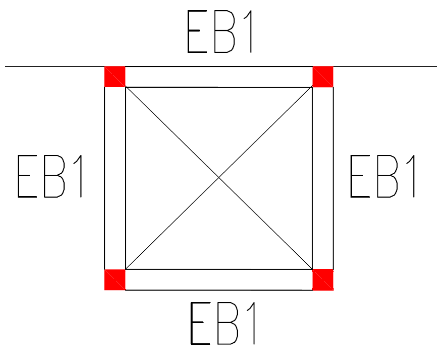
Many of the lower level columns experience primarily axial loads and minimal moments. On the contrary, the columns at the upper levels have very little axial load and higher moments due to lateral loads. The ratio of load to capacity is calculated using the interaction equation for the controlling load combination, which combines the effects of axial loads and moments. Spot checks on a representative column, including the interaction diagram, can be found in Appendix C.

Beam Design:

In designing the slab, consideration was given to including edge beams between the exterior columns. However, it was determined that drop panels were required not only for punching shear, but to resist a portion of the negative moment. Adding beams would not address this issue, as the beams would be spanning in the opposite direction of the moment. Therefore, drop panels would be required regardless. Although the exterior wall consists of some precast, which is rather heavy, there is also a lot of lightweight glass and aluminum, resulting in an approximate exterior line load of 1000 lb/ft. Upon investigation, I found the slab with drop panels could sufficiently support this load, therefore deeming beams unnecessary.

With the need for edge beams eliminated, there are only three locations where beams are required. Most importantly, they are needed at the main roof level, where the penthouse roof columns are transferred out. The plan below shows the main roof/penthouse floor plan with the labeled transfer beams. The columns in blue are the columns being transferred out that extend up to the penthouse roof, while the columns in red are columns below. The blue dashed line is the outline of the penthouse area.





Smaller concrete beams will also be used to frame out the slab openings at the elevator and stairwells. The load on these beams is minimal, supporting mainly the CMU wall surrounding these openings. To the left is a partial plan with beam labels for the elevator framing beams.

Finally, a beam is needed to support the one-way slab cantilever at the elevator lobby and waiting room. A partial plan of this area is included in the slab design section. That beam was labeled B1.

Full calculations for the beam designs are included in Appendix D. Deflections were checked in accordance with Table 9.5(a) of ACI 318-05 to a limit of l/16 for simply supported beams. All beam designs are summarized in the spreadsheet below.

Beam	B	H	Flex. Reinf.	Stirrups
TB1	24	32	(10) #9	#4
TB2	24	32	(10) #9	#4
TB3	24	32	(6) #9	#3
TB4	24	32	(8) #9	#3
TB5	24	32	(6) #7	#3
EB1	24	16	(4) #7	#3
B1	24	32	(8) #9	#3

The sizes above were chosen for specific reasons, most notably for ease of constructability. All beams were designed with a 24” width in order to match the column dimensions. For the transfer beams, TB1 and TB2 experienced the highest loading conditions. The 32” depth for these beams was chosen because it was the shallowest depth that provided reasonable reinforcement requirements. Since TB2 frames into TB3, it was desirable to maintain the 32” depth for each of connections. Therefore, the reinforcement was reduced accordingly for the required moment capacity. Similarly TB4 was designed with a 32” depth as it resulted in reasonable reinforcement and because a larger depth will limit deflections for the long span. TB4 frames into TB5, so once again the 32” depth was maintained for ease of construction. EB1 experiences lighter loads, but maintained the 24” width to match the column dimension. The 16” depth was chosen because that it matches the maximum slab thickness of the 10” slab plus 6” drop panel and it provided reasonable reinforcement requirements. This provides the maximum amount of space for mechanical and electrical space beneath the slab since the beams will not project below the slab.

Drift Check:

As previously mentioned, wind drift proved to be of great concern for the existing steel design. Both total building drift and story drift considerably surpassed the accepted value of H/400. Resolving this wind drift issue was an important goal outlined in the proposal.

The new concrete structure provides additional stiffness, which should resolve the excessive drift issue. Using RAM, drift at each story height was calculated for each service wind load case. The controlling case proved to be wind in the East-West direction. Below is a summary of actual wind drift at each level along with the allowable calculated drift. Both total building drift and story drift are considered.

	Story Height (ft)	**Total Drift (in)**	**Allowable H/400 (in)**	**Floor to Floor Height (ft)**	**Story Drift (in)**	**Allowable H_{story}/400**
PH Roof	88.5	1.02	2.66	18	0.1	0.54
Main Roof	70.5	0.92	2.12	16.5	0.14	0.50
4th Floor	54	0.78	1.62	18	0.23	0.54
3rd Floor	36	0.55	1.08	18	0.3	0.54
2nd Floor	18	0.25	0.54	18	0.25	0.54

It can be seen above that all drift values for the controlling load case are in accordance with the engineering standard of H/400. Therefore, as expected, the increased stiffness of the new concrete structure in comparison to the existing steel structure resolves the wind drift issue.

Seismic drift was not of great concern for the steel structure, as all drift values were found to be acceptable. The new concrete structure, however, experiences increased seismic loads due to the increased building weight. Therefore, seismic drift will likely increase and must be checked. The allowable drift was calculated in accordance with ASCE 7-05 Chapter 12 from the table shown below.

TABLE 12.12-1 ALLOWABLE STORY DRIFT, $\Delta_a{}^{a,b}$

Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}{}^{c}$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures [d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$\boxed{0.015h_{sx}}$	$0.010h_{sx}$

[a] h_{sx} is the story height below Level x.
[b] For seismic force–resisting systems comprised solely of moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.
[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.

The actual seismic drift was calculated in RAM for each seismic drift load case. These values were calculated by elastic analysis and required amplification in accordance with ASCE 7-05 Section 12.8.6. Rather than calculating a new drift for each story, an adjusted drift ratio was calculated as shown below.

$\delta_x = (C_d * \delta_{xe})/I$
$0.015h_{sx} = (2.5 * \delta_{xe})/1.25$
drift ratio = $\delta_{xe}/h_{sx} = 0.015*1.25/2.5 = 0.0075$

	Story Height (ft)	Floor to Floor Height (ft)	Total Drift (in)	Story Drift (in)	Actual Drift Ratio	Allowable Drift Ratio
PH Roof	88.5	18	1.23	0.17	0.0008	0.0075
Main Roof	70.5	16.5	1.06	0.19	0.0010	0.0075
4th Floor	54	18	0.87	0.29	0.0013	0.0075
3rd Floor	36	18	0.58	0.33	0.0015	0.0075
2nd Floor	18	18	0.25	0.25	0.0012	0.0075

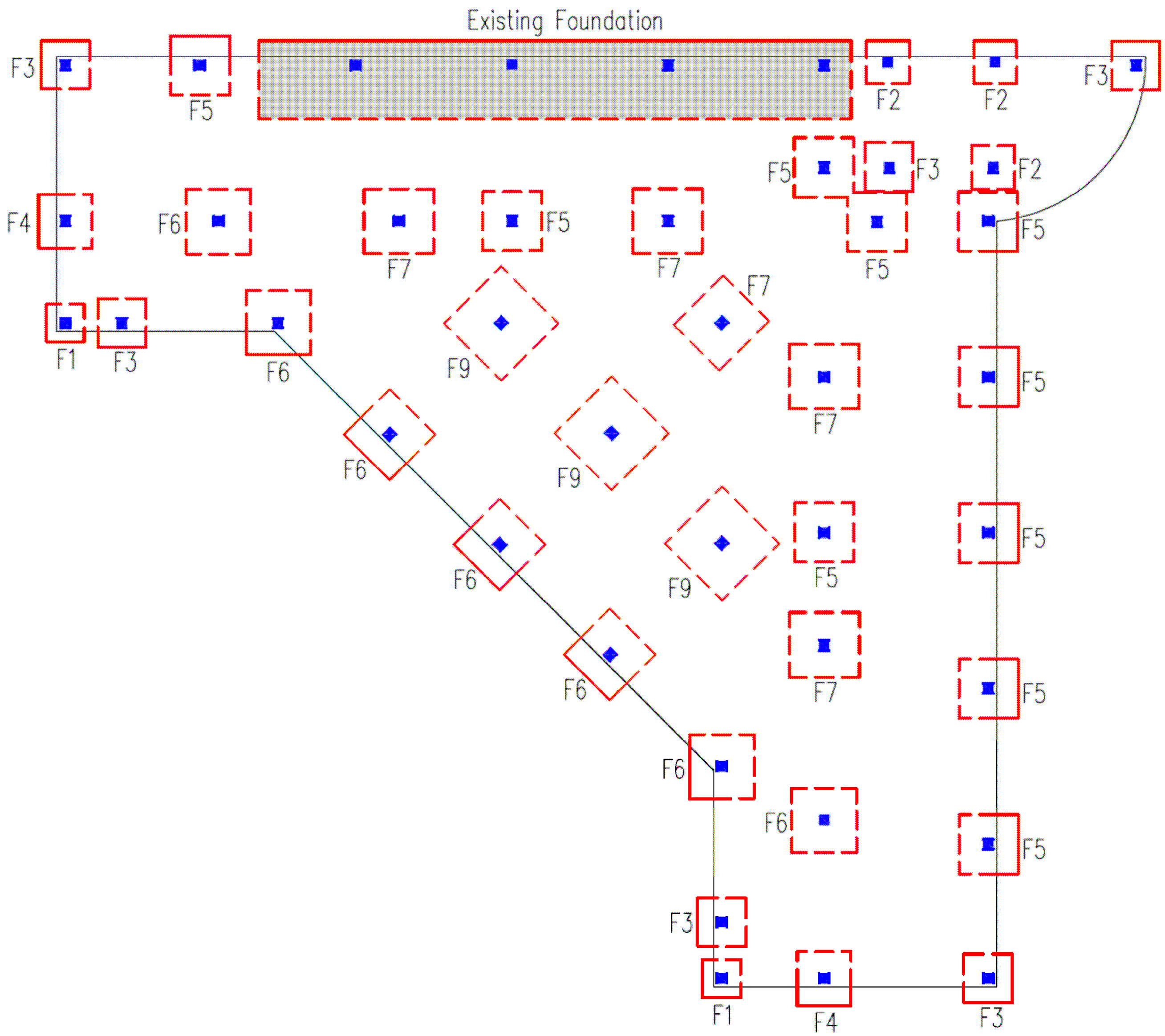
It can be seen above that seismic drift is also well within the acceptable limits. Therefore, the proposed concrete structure is sufficient in terms of lateral movement. This is extremely important as it addresses what was an area of great concern in the existing design.

Foundation Design:

The new concrete structure experiences increased dead loads in comparison to the existing steel structure, soit is expected that the footing sizes will increase. As previously mentioned, the existing structure utilized mainly 8 by 8 foot square footings to 11 by 11 foot square footings, with a select few rectangular footings due to special conditions. RAM Structural System has a Foundation module, which will be used to design the footings.

As mentioned prior, there is an existing retaining wall present at the north side of the new patient tower. The design of the existing steel building planned to remove the retaining wall foundation where it interfered with the new spread footings for the columns along the north side of the addition. However, it was found during construction that the columns could be tied into this existing footing so that it could be utilized. This method will be used for the footings of the concrete columns along the north side of the addition. The retaining wall projects 10 feet into the new tower and is over 100 feet long, which proved to provide sufficient capacity for the columns along that side.

The plan below shows the new footings at a relative scale. All concrete columns are shown in blue and the footings are shown in red. The footing labels reference the footing spreadsheet, which is included beneath the plan. This spreadsheet provides the length, width, thickness, and reinforcing for each footing. Also on this plan, shown in gray, is the existing retaining wall foundation.



Ftg #	L (ft)	W (ft)	t (ft)	Reinf
F1	7	7	1.5	#6@12" EW
F2	8	8	2	#6@10" EW
F3	9	9	2	#6@10" EW
F4	10	10	2	#6@8" EW
F5	11	11	2.5	#7@10" EW
F6	12	12	2.5	#7@8" EW
F7	13	13	3	#7@8" EW
F8	15	15	3.5	#8@10" EW

Some of the footings considerably increased in size from the steel to concrete building, resulting in additional concrete. This will be taken into consideration in the construction management cost portion of the report.

Design Summary:

This concludes the design of the new proposed concrete structural system. It has been proven that the concrete structure performs adequately under all design loads. The loading conditions produced reasonable member sizes and reinforcing, similar to what was outlined in the preliminary CRSI Handbook recommendations.

As desired, the concrete frame was able to take all lateral loads, with no requirement for shear walls. This maintains the floor plan flexibility that the composite steel system afforded. Wind drift was resolved because of the increased stiffness of the concrete system. Although seismic loads increased due to building weight, this did not prove to be an issue as the Mid-Atlantic region is not generally exposed to high seismic loads.

In order to determine whether the concrete system should be recommended over the steel system, other issues have to be considered. Most importantly, a cost and schedule analysis must be performed. If the concrete system proves to be more expensive and/or takes longer to construct, the steel system may be the better option.

IX. BREADTH STUDY – CONSTRUCTION COSTS AND SCHEDULE

Howard County General Hospital's patient tower addition was originally designed in structural steel, matching the existing hospital's skeleton. The desire to maintain this continuity in structure between the existing hospital and new addition ruled out concrete as an alternative material. However, as specified in the structural depth of this report, there are considerable advantages to a concrete system. It is also possible that a concrete structure could introduce savings in terms of cost or time, which are both extremely valuable in this industry.

Cost Estimate: Existing System

The existing steel system is expected to be costly due to the large W-shapes required for floor framing and moment connection costs. RS Means Building Construction Cost Data 2008 was used to estimate the cost of the structural steel, concrete slab on metal deck, fireproofing and foundations. All other costs such are considered to remain virtually the same between the existing steel system and the proposed concrete system. Therefore, this is not a comprehensive estimate for all building costs, but instead an estimate for the structural system for comparison sake only.

Upon consultation with the project manager at Whiting-Turner, the CM for this addition, it was determined that their estimates for steel structures are simply based off of total tonnage of steel. One advantage of using RAM Structural System is that it provides a takeoff report summing the lengths and weights of each steel cross section at each floor. A RAM model of the existing composite steel building was built during the fall semester, so this tool will be used to calculate the tonnage of steel from which a cost can be obtained. Using the takeoff report provided by RAM, spreadsheets were developed to tabulate the number, length, and weight of all beams at each level. Another summarizes the total length of each column cross section. All of these spreadsheets are included in Appendix E, while a tonnage summary is included below. The material, labor and equipment costs were taken from RS Means, as mentioned prior, and these tables are also available in Appendix E.

	Pounds of Steel	Tons of Steel	Material ($/ton)	Labor ($/ton)	Equip ($/ton)	Total $
Beams	631564	315.8	$2,250	$375	$130	$869,980
Columns	362834	181.4	$2,250	$375	$130	$499,803
SUM	994398	497.2				**$1,369,783**

The estimate for the slab/deck cost includes a variety of components. The total slab square footage for all floors was used to calculate the cost of lightweight concrete and metal deck. The cost of shear studs, which was obtained using the RAM takeoff tool explained above, and the welded wire fabric used for slab reinforcing are also included in the slab/deck estimate. Finally, the cost of the 1 ½" wide rib roof deck was added to this figure. Spreadsheets containing all slab costs can be found in Appendix E, but a summary is shown on the right right.

Slab Component	Total $
Metal Deck	$249,275
LW Concrete	$358,573
Shear Studs	$22,719
WWF	$36,097
Roof Deck	$10,046
SUM	**$676,710**

Finally, certain miscellaneous costs were based on consultations with industry professionals rather than RS Means. The cost of moment connections is a substantial figure to consider in the total steel building cost. There are typically 19 moment frames at each floor, totaling to 38 connections per floor. In addition, the quarter-circular slab on the northeast side of the building is supported by cantilevered beams that require moment connections. In total, there are 222 moment connections throughout the building. In consulting with the structural engineer and CM, a figure of $200 per moment connection for a standard W12 beam was obtained. Approximately $100 of this is for materials, and the other $100 for labor. Since most of the moment connected beams in this building are W16s or larger, a multiplier of 1.5 was recommended. Although a true estimate would require calculating the amount of welding steel, bolts, etc., this is an approximation specifically for this project and is assumed to be sufficient for this level of estimating. Also, the cost of fireproofing must be included in the steel estimate. The Construction Manager provided a figure of $2.00 per square foot of slab area for fireproofing costs for the beams and deck. In comparing this figure with those from RS Means, this estimate seems very reasonable.

Based on all of the estimates outlined above, the following figures were calculated for the existing building design.

Structural Steel	$1,369,783
Concrete Slab on Metal Deck	$676,710
Foundations	$61,580
Fireproofing	$203,500
Moment Connections	$66,600
Total Cost	**$2,378,173**

It is important to point out, once again, that this is by no means a total building cost estimate. This is a relative estimate including only the costs of structural elements that will be altered in the proposed concrete design. It is only to be used in comparing these structural costs to those in the new design.

Cost Estimate: Proposed System

The goal of the proposed concrete system is not only to improve the building's structural performance, but also to reduce costs if possible. RS Means was used to estimate the cost of the cast in place concrete, reinforcing, and foundations. All tables and values used for estimating included in this section and Appendix E are from RS Means Building Construction Cost Data 2008.

The total cubic yardage of concrete required to construct the number of columns proposed in the new concrete floor plan was calculated based on the column dimensions and heights. All columns are 24" by 24" and there are two different heights – the column height for the 18 foot floor to floor height and the column height for the 16.5 foot floor to floor heights. RS Means tabulates the cost of columns according to size and reinforcing. The cost in these tables includes forms (4 uses), reinforcing steel, concrete, placement, and finishing. Given that the typical reinforcing for the columns in this building is (8) #8, which is only slightly above the minimum reinforcing, concrete column costs will be estimated using data for 24" by 24" square columns with minimum reinforcing.

The cast in place concrete portion of RS Means was also used to estimate the cost of the flat slab floor system. The values for this section are tabulated according span. For this estimate, a 30 foot span was used, which is very close to the 29 foot span typical in this building. This value includes all components of the floor slab including reinforcing, forms, etc.

Since many of the footing sizes increased, the foundation costs for the concrete system are expected to be higher than for the steel building. For this comparison estimate, only the spread footings were considered. All other below grade work including excavation, underpinning, etc. is assumed to be the same for both systems. The footings were tabulated according to size and a total cubic yardage of concrete was calculated.

There are only 3 locations were beams are required for this building, so they are not expected to be very expensive in terms of overall cost. RS Means includes a cost per cubic yard of beams including forms, reinforcing steel, placement and finishing. This value was used to calculate the cost of the beams in terms of material, labor and equipment.

Below are the estimated figures for the various aspects of the proposed concrete design.

Two-Way Reinforced Flat Slab	$1,418,373
Reinforced Concrete Columns	$321,179
Reinforced Concrete Beams	$94, 523
Foundations	$101,702
Total Cost	**$1,935,777**

Cost Estimate: Conclusion

Based on the estimates from RS Means outlined above, it can be concluded that the concrete system provides almost $500,000 of savings in the structural system alone. The composite steel system has some unusual conditions that increase the cost of the existing design. For instance, framing out the individual slab depressions for the stall-less showers requires over 400 additional feet of W12x19 sections per floor. In addition, the desire to maintain floor plan flexibility and rely on moment connections for all lateral resistance is an expensive option. On the contrary, for the concrete system, the slab depressions are easily formed and the inherent concrete moment frames are inexpensive compared to steel moment frames. Therefore, it seems that the concrete system is a more efficient structural option in terms of cost, and hence resulted in savings.

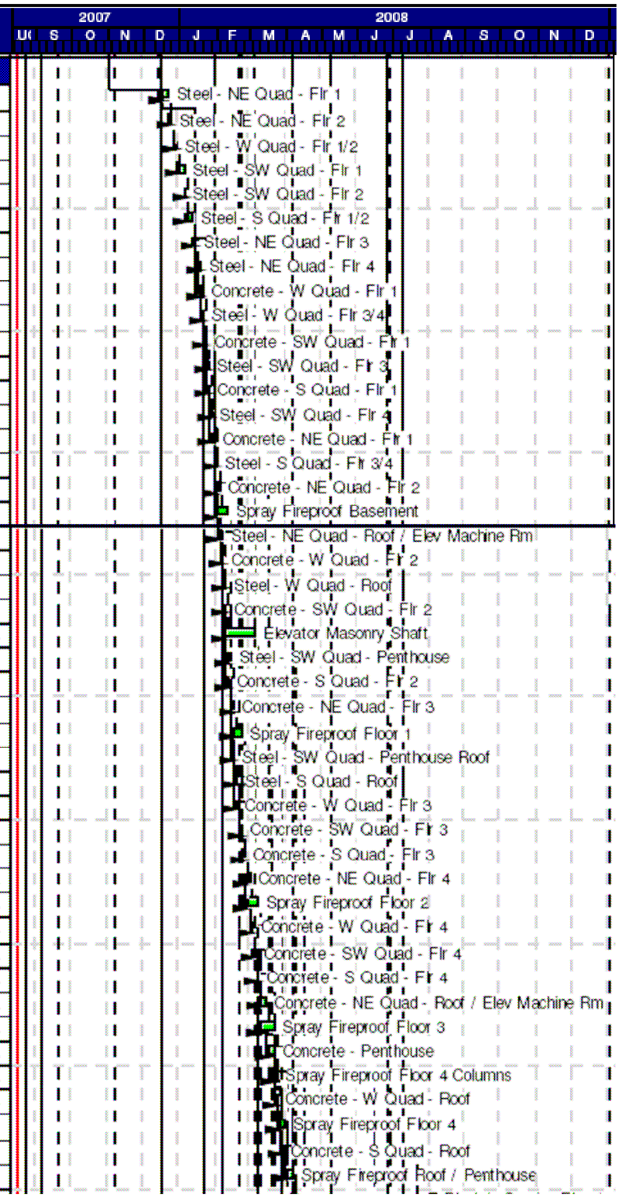
In addition to the savings outlined above, there are other potential savings possible with the new concrete system. The concrete system was designed for this report using the same floor-to-floor heights as the steel building, typically 18 feet. This was mainly because the steel system was designed to match the existing hospital's floor elevations, so the concrete system was design to do the same. However, for the new design the structural floor thickness decreased from approximately 3 feet to 16 inches, resulting in a savings of about 20" of thickness per floor. The architectural details already include a drop ceiling, providing plenty of room for mechanical and electrical equipment, so this additional space is not necessary. If the owner and architect decided to, they could maintain the typical 9 foot floor to ceiling heights with the same amount of mechanical space while reducing the overall building height by over 8 feet. This would provide considerable savings in many areas, most notably in terms of the building façade.

Schedule: Existing System

The existing schedule was provided by Whiting-Turner, the construction manager on the project. The entire schedule was included, but for this comparison only erection of the steel columns and beams, installing the metal deck, and pouring the floor slabs will be considered. All other task durations are expected to remain virtually the same, regardless of the structural system used.

Steel fabrication began in June of 2007 and erection began in January of 2008, finishing in March of 2008. To the right is a photo of the completed steel skeleton.

Act ID	Activity Description	Orig Dur	Early Start	Early Finish
Structure				
30270	Steel - NE Quad - Flr 1	3d	21 DEC 07	26 DEC 07
30275	Steel - NE Quad - Flr 2	2d	27 DEC 07	28 DEC 07
30280	Steel - W Quad - Flr 1/2	3d	31 DEC 07	03 JAN 08
30290	Steel - SW Quad - Flr 1	3d	04 JAN 08	08 JAN 08
30295	Steel - SW Quad - Flr 2	2d	09 JAN 08	10 JAN 08
30300	Steel - S Quad - Flr 1/2	3d	11 JAN 08	15 JAN 08
30310	Steel - NE Quad - Flr 3	3d	16 JAN 08	18 JAN 08
30315	Steel - NE Quad - Flr 4	2d	21 JAN 08	22 JAN 08
30400	Concrete - W Quad - Flr 1	3d	22 JAN 08	24 JAN 08
30320	Steel - W Quad - Flr 3/4	3d	23 JAN 08	25 JAN 08
30405	Concrete - SW Quad - Flr 1	2d	25 JAN 08	28 JAN 08
30330	Steel - SW Quad - Flr 3	3d	28 JAN 08	30 JAN 08
30410	Concrete - S Quad - Flr 1	2d	29 JAN 08	30 JAN 08
30335	Steel - SW Quad - Flr 4	2d	31 JAN 08	01 FEB 08
30415	Concrete - NE Quad - Flr 1	3d	31 JAN 08	04 FEB 08
30340	Steel - S Quad - Flr 3/4	3d	04 FEB 08	06 FEB 08
30420	Concrete - NE Quad - Flr 2	3d	05 FEB 08	07 FEB 08
30515	Spray Fireproof Basement	1w3d	05 FEB 08	14 FEB 08
30350	Steel - NE Quad - Roof / Elev Machine Rm	3d	07 FEB 08	11 FEB 08
30425	Concrete - W Quad - Flr 2	2d	08 FEB 08	11 FEB 08
30360	Steel - W Quad - Roof	2d	12 FEB 08	13 FEB 08
30430	Concrete - SW Quad - Flr 2	2d	12 FEB 08	13 FEB 08
30510	Elevator Masonry Shaft	4w	12 FEB 08	10 MAR 08
30370	Steel - SW Quad - Penthouse	3d	14 FEB 08	18 FEB 08
30435	Concrete - S Quad - Flr 2	2d	14 FEB 08	15 FEB 08
30440	Concrete - NE Quad - Flr 3	3d	18 FEB 08	20 FEB 08
30520	Spray Fireproof Floor 1	1w3d	18 FEB 08	27 FEB 08
30380	Steel - SW Quad - Penthouse Roof	2d	19 FEB 08	20 FEB 08
30390	Steel - S Quad - Roof	2d	21 FEB 08	22 FEB 08
30445	Concrete - W Quad - Flr 3	2d	21 FEB 08	22 FEB 08
30450	Concrete - SW Quad - Flr 3	2d	25 FEB 08	26 FEB 08
30455	Concrete - S Quad - Flr 3	2d	27 FEB 08	28 FEB 08
30460	Concrete - NE Quad - Flr 4	3d	29 FEB 08	04 MAR 08
30525	Spray Fireproof Floor 2	1w3d	29 FEB 08	11 MAR 08
30465	Concrete - W Quad - Flr 4	2d	05 MAR 08	06 MAR 08
30470	Concrete - SW Quad - Flr 4	2d	07 MAR 08	10 MAR 08
30475	Concrete - S Quad - Flr 4	2d	11 MAR 08	12 MAR 08
30480	Concrete - NE Quad - Roof / Elev Machine Rm	1w	13 MAR 08	19 MAR 08
30530	Spray Fireproof Floor 3	1w3d	13 MAR 08	24 MAR 08
30485	Concrete - Penthouse	1w	20 MAR 08	26 MAR 08
30535	Spray Fireproof Floor 4 Columns	3d	25 MAR 08	27 MAR 08
30490	Concrete - W Quad - Roof	2d	27 MAR 08	28 MAR 08
30540	Spray Fireproof Floor 4	1w	28 MAR 08	03 APR 08
30495	Concrete - S Quad - Roof	2d	31 MAR 08	01 APR 08
30545	Spray Fireproof Roof / Penthouse	1w	04 APR 08	10 APR 08

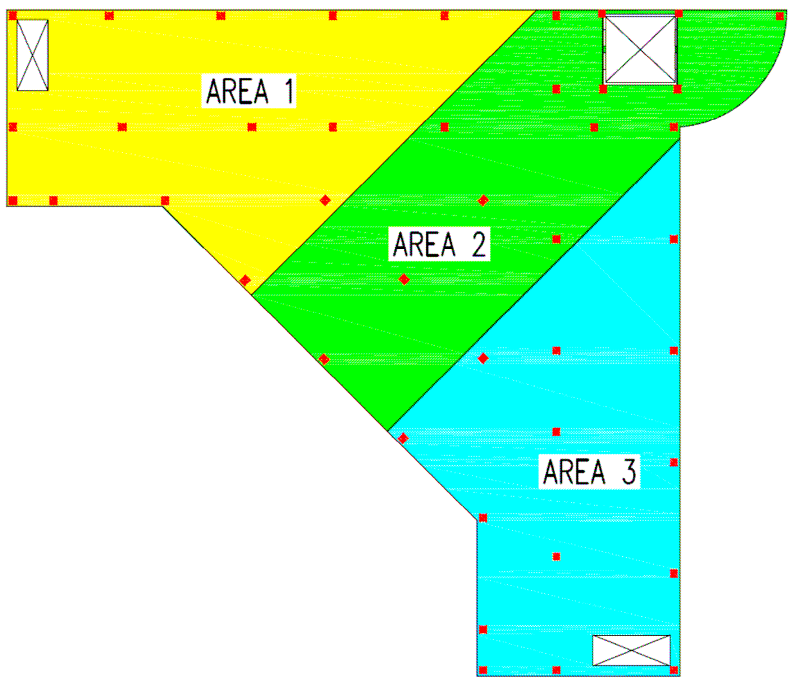


It is evident that steel erection started a little late, as it was scheduled to begin on December 21, 2007 and did not begin until January 2008. Still, the project's progression seems to be following the schedule relatively closely.

In the schedule above, it is clear that for erection purposes the building was divided into four quads – NE, W, SW, and S. Erection of a typical steel floor for one quad took 2 to 4 days. Pouring the concrete slab for a typical floor for one quad took 2 to 3 days. Finally, fireproofing for a whole floor was scheduled to take approximately 10 days. Overall the schedule shows the construction of the structural frame lasting from December 21, 2007 until April 10, 2008. This results in a total construction time of almost four months, or 16 weeks.

Schedule: Proposed System

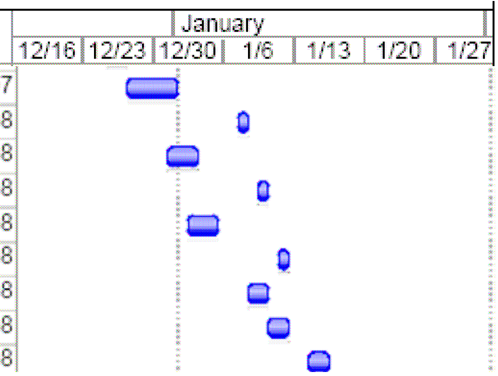
Construction scheduling of the new concrete system was developed based on sample schedules, a consultation with a local construction manager, and RS Means crew information. The foundation work including slab on grade will not be included in the schedule, as it is not expected to differ much from the foundation work for the composite steel schedule. Therefore, construction will begin with the columns extending from the basement to first floor level. For both column and slab construction, the floor plan will be divided into three areas. These areas are of approximately equal square footage and contain the same number of columns. Below is a diagram demonstrating how the three areas could be divided, each area shown in a different color.



Although the composite steel system was divided into four "quads", the construction manager advised that for scheduling the concrete system, using three areas would be more efficient. It seems that the four "quads" were required in the steel building because of the two different framing directions, which does not affect the concrete system.

It was determined that forming, reinforcing, and placing the columns for a single area for a single floor would take about two days. Similarly, forming, reinforcing, and placing one area of the flat slab would take three days. An additional day is needed later after the concrete has partially cured to strip the slab for each area. The schedule was developed in Microsoft Project, and is included in full in Appendix E. A sample of the schedule for just the first floor is shown below.

ID	Task Name	Duration	Start	Finish
4	F/R/P Flat Slab 1st Floor, 1	3 days	Thu 12/27/07	Mon 12/31/07
5	Strip Flat Slab 1st Floor, 1	1 day	Mon 1/7/08	Mon 1/7/08
6	F/R/P Flat Slab 1st Floor, 2	3 days	Mon 12/31/07	Wed 1/2/08
7	Strip Flat Slab 1st Floor, 2	1 day	Wed 1/9/08	Wed 1/9/08
8	F/R/P Flat Slab 1st Floor, 3	3 days	Wed 1/2/08	Fri 1/4/08
9	Strip Flat Slab 1st Floor, 3	1 day	Fri 1/11/08	Fri 1/11/08
10	F/R/P Columns 1st Floor, 1	2 days	Tue 1/8/08	Wed 1/9/08
11	F/R/P Columns 1st Floor, 2	2 days	Thu 1/10/08	Fri 1/11/08
12	F/R/P Columns 1st Floor, 3	2 days	Mon 1/14/08	Tue 1/15/08



There were a few important assumptions made in developing this schedule. Of the three days used to form, reinforce, and place the slab, the first two are prep work. Therefore, on the third day of the slab duration, when one area is being poured, the carpenters would have started prepping the next area. Hence, there is one day of overlap between one area and the next. It was also assumed that the slabs would cure for 7 days before being stripped, which was scheduled to take one day.

It can be seen that the total schedule duration is from December 21, 2007 to April 9, 2007. This is about 4 months, or 16 weeks, almost exactly the same as the existing steel schedule.

Schedule: Conclusion

There seems to be no real advantage in terms of either structural system's schedule. The steel schedule lasted 16 weeks, from December 21, 2007 to April 10, 2007. The concrete schedule lasted 16 weeks also, with the same start date but ending one day prior.

The only disadvantage to the steel system's schedule is that it does not include the amount of time required for fabrication, which would take several months. However, there is a multitude of work being done prior to the structural work including some demolition and excavation. Therefore, it is expected that the fabrication time would not drastically postpone the structural schedule.

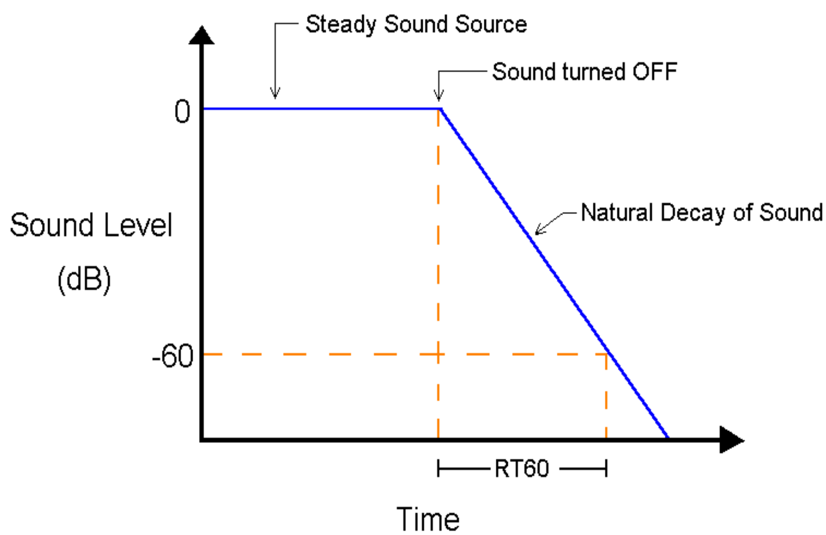
X. BREADTH STUDY – ACOUSTICS

The main reasons for this addition are to better serve the community and to provide more private patient rooms. It is very important for patients to feel that their medical conditions are kept private and to be comfortable while staying at the hospital. Acoustics could play a large part in whether this patient tower accomplishes the hospital's goal of serving the public to the best of their ability and maintaining privacy in accordance with medical confidentiality.

Many people are unaware of how important a role acoustics plays in hospitals. An article published in the Baltimore Sun stated that hospital noise levels have been rising steadily since the 1960s. Hospital surfaces tend to be tiled and bare and therefore reflect sound. Carpet, which improves acoustical performance, is often avoided because it is hard to clean and harbors bacteria. Hospitals are designed to be efficient for patient care but acoustical issues are often overlooked.

Limiting sound transmission between floors and walls is of primary concern so that no one can overhear medical discussions between doctors and patients. There are often very private, important, and personal matters being discussed between doctors and patients, and the patients must be confident that their privacy is maintained.

Reverberation time is the length of time it takes for a sound to naturally decay after it stops or is turned off. This concept is demonstrated in the diagram to the right. A long reverberation time degrades speech perception, especially for hearing-impaired persons. Since many hospital visitors are elderly and may have deteriorated hearing, a short reverberation time will be very important to them.



In addition to its importance for the elderly patients, reverberation time will be important for the hospital employees as well. A short reverberation time results in a smaller likelihood of misinterpretations. It is extremely important that doctors and nurses hear and understand every conversation very clearly. Hearing a drug name or dosage wrong can potentially result in a deadly situation. Therefore, for both of the reasons outlined above, reverberation time for the typical patient room will be calculated. If it is not up to the desirable standard, a solution for reducing reverberation time will be proposed.

Reverberation Time:

As stated above, a low reverberation time is desirable so that doctors and patients, even those who are hearing impaired, can clearly hear all conversations. After browsing many textbooks and other resources, I was unable to find a target reverberation time for hospitals. However, it seems reasonable that a hospital could be comparative to a classroom or conference room. Both of these spaces are intimate and require a high level of speech understandability. Therefore, a target value of 0.5-0.7 seconds will be used, which is the goal for intimate classrooms, studios, and conference rooms.

Achieving this short of a reverberation time may prove to be a challenge since most of the surfaces in the hospital are hard and reflective. The typical wall is composed of 5/8" gypsum board on 6" metal studs at 16" on center with insulation. There are 6 foot tall windows along the back wall of all patient rooms. An acoustical drop ceiling is suspended beneath the floor slab, resulting in a typical 9 foot floor to floor height. Floor finishes were not specified in the architectural plans, but it is likely that the concrete slab will be sealed, painted, or tiled. There is very little difference in the absorption coefficient between these finishes, 0.02 compared to 0.03. Therefore, 0.02 will be used for more conservative results, although this assumption will have very little effect on the overall calculation. The spreadsheet below summarizes absorption coefficients for each surface and the square footages from which the reverberation time can be calculated.

			Absorb. Coeff.		**Sα**	
Surface	**Material**	**Area (SF)**	**500 Hz**	**1000 Hz**	**500 Hz**	**1000 Hz**
Floor	Tile on Concrete	230	0.02	0.02	4.6	4.6
Wall	Gypsum Board	433	0.05	0.04	21.7	17.3
Door	Wood	56	0.09	0.06	5.0	3.4
Ceiling	Acoustical Tile	230	0.65	0.59	149.5	135.7
Window	Glass	75	0.18	0.12	13.5	9.0
				∑Sα =	194.3	170.0
Volume =	2070			**T_{60} = 0.05*(V/∑Sα) =**	0.53	0.61

It can be seen that the calculated reverberation times are within the goal of 0.5 to 0.7 seconds. It is possible that in the larger, more common areas, such as waiting rooms, hallways, and lobbies, there will be a higher reverberation time. This is expected simply because larger rooms generally have longer reverberation times. However, for this analysis, the concern was with patient rooms. Therefore, it is assumed that patient rooms are adequate for both patients and doctors.

James West, an acoustics professor at Johns Hopkins University, performed a study at Johns Hopkins Hospital in Baltimore, MD to determine the hospital's acoustical shortcomings and possible solutions. He made audio recordings to calculate reverberation time and found the hospital to have an approximate reverberation time of 1.2 seconds, which is significantly greater than what I calculated above. There are several explanations for why this variation could occur. Johns Hopkins Hospital does not use acoustical ceiling tile, which would drastically reduce the reverberation time. Also, West used actual recordings to determine the length of time for sound to decay rather than just calculations, which may provide somewhat different results. Finally, West's recordings were taken for various hospital "wings", which implies that he included hallways, common areas, etc. rather than just individual patient rooms. It seems reasonable to find such a discrepancy due to these varying circumstances, so Howard County Hospital is still assumed to be sufficient in terms of reverberation time. However, West's study provided insight into how much of an issue reverberation time can be in hospitals.

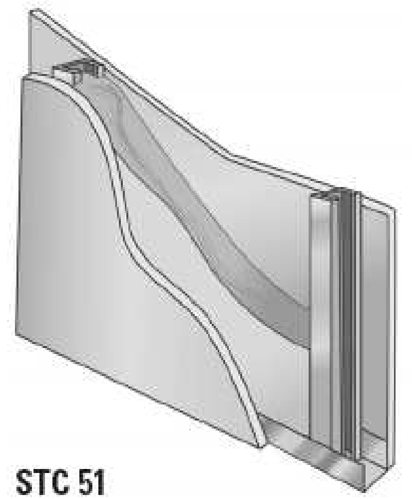
Sound Transmission:

The sound transmission class, commonly known as STC, is a rating scale that measures a wall, ceiling, or floor assembly's ability to block sound transmission. Higher STC ratings mean less sound can be heard through the assembly. One STC point is approximate equivalent to one decibel point. For this project, the biggest concerns in terms of blocking sound transmission are the wall and floor assemblies. This is primarily due to the fact that maintaining the patients' privacy is so important and no conversations should be overheard. The table below summarizes at what STC various levels of speech can or cannot be heard through walls or floors.

STC	*Speech Heard Through Wall or Floor*
30	Loud speech can be understood fairly well
35	Loud speech audible but not intelligible
42	Loud speech audible as a murmur
45	Some loud speech barely audible
48	Hearing strained to hear loud speech
50	Loud speech not audible

Based on this chart, the goal for this project will be to attain an STC rating of 50 or higher, which prevents even loud speech from being heard through the walls. That will eliminate any privacy issues for the patients and allow for the highest standard of care at Howard County General Hospital, which is the goal.

The partition walls separating patient rooms are made up of 6” metal studs placed at 16” on center with 5/8” gypsum board on each side. These walls do also contain insulation, which will improve their ability to block sound. This wall assembly is standard for both the existing composite steel design and the proposed concrete design. After thorough research, it proved difficult to find general data for this wall assembly. Most tabulated data was for 3 5/8” metal studs rather than 6”. However, the diagram on the right was taken from Johns Manville Insulation website, and demonstrates a similar wall achieving an STC of 51. Although this is for a specified manufacturer’s insulation, it is likely that any insulation will perform similarly in terms of sound transmission. Therefore, this wall assembly achieves an STC of 51, which is greater than the target STC of 50.

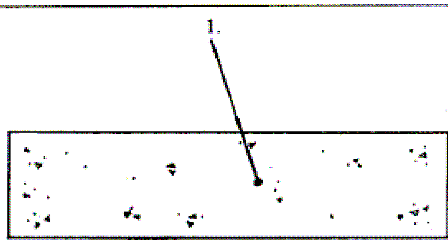


STC 51

6" (152 mm) 25 gauge steel studs: single layer 5/8" (16 mm) Type X gypsum board each side; one thickness 6 1/2" (165 mm) JM Formaldehyde-free thermal/acoustical fiber glass batts.

The existing steel design consists of 3 ¼” lightweight concrete on metal deck supported by steel beams and girders. It proved to be very difficult finding STC data for this whole floor assembly. However, upon consultation with Professor Ling, it was determined that the concrete is the main contributing factor and using the STC for a 3 ¼” concrete slab would be most accurate. From AE309 notes, the STC for a 3” concrete slab is 42. It is assumed that this whole assembly will achieve an STC in the mid to high 40s, which is slightly below the target STC of 50.

For the proposed concrete system, the floor is a 10” slab with a suspended ceiling beneath. However, in some areas, there is a 2” slab depression, so the STC of an 8” slab will be used in order to be conservative. Below you can see that an 8” concrete slab provides an STC rating of 58. Adding the suspended ceiling will only increase the STC rating, so it is assumed that the required STC rating of 50 is achieved with the proposed floor system.

1. 	1. 8" thick concrete slab, 95 psf.	...	Riverbank Acoustical Labs. TL 76-77 1977 16f Prestressed Concrete Inst.	58 NA	2.3.2.1.1.2

Acoustical Conclusions:

It was determined that a standard patient room experiences a reverberation time within the range of 0.5 to 0.7 seconds. This calculation is not affected by the structural system because it is based off of the absorption coefficients of the finished materials. Therefore, it is constant for the existing composite system and the proposed concrete system, and both are acceptable. Like previously mentioned, some of the larger, more public rooms may experience higher reverberation times. If reverberation time proved to be a problem, a few options could be considered. The most likely option would be to install acoustical wall panels where required. These panels are typically made of mineral wool or fiberglass on wood backing, resulting in a much higher absorption coefficient than the current gypsum board surface. Another option would be installing carpeting, but this may not be practical in some hospital areas since it is harder to clean and harbors germs. Regardless, there are possible solutions if reverberation time proved to be a problem, but based on the calculations, it is within the desireable range.

The STC ratings of the floor systems for both the existing composite and new concrete system were researched. It proved to be difficult finding a rating for the existing floor assembly, but the concrete slab alone achieved a rating of 42. The metal deck, steel beams, and suspended ceiling may increase this rating slightly, but most likely will not rival the STC rating of 58 achieved by the 8" concrete slab. In addition, using an 8" concrete slab is conservative because the slab is only 8" where the depressions occur and 10" at all other locations. In most areas, a 10" slab is present, increasing the STC rating even more. Therefore, it has been determined that the concrete slab performs better in preventing sound transmission through the floor system. However, the composite system still performs rather well and does not raise cause for great concern.

Both systems use the same partition walls of 6" metal studs with 5/8" gypsum board on each side and insulation. The STC rating for this wall was determined to be 51, which is above the desirable STC rating of 50. Therefore, these partition walls perform adequately to block all speech from transmitting from one room to another.

Based on these results, it can be seen that the concrete system has a slight advantage over the composite steel system in terms of acoustics. However, the difference is not so great that it should drive the design one way or the other. Both systems perform rather well acoustically and achieve the desired reverberation times and close to or above the recommended STC rating.

XI. CONCLUSIONS/RECOMMENDATIONS

This thesis study was intended to determine whether the Howard County General Hospital patient tower addition would perform more efficiently designed as a flat plate concrete system rather than the existing composite steel system. Through analysis of the existing system, concerns were raised about building drift due to wind loads. There were also special conditions, such as the recurring slab depressions, that proved to be rather expensive. However, the existing composite steel system with moment frames adequately performed under the heavy 100 psf building live loads and maintained floor plan flexibility, an extremely important asset for the hospital. The overall goal, therefore, was to maintain these positive features of the existing structural system while addressing the wind drift and cost issues. Ultimately, a concrete flat plate system proved to be of viable consideration.

The concrete flat plate system was designed using a variety of computer programs and hand calculations. A final design resulted in 24” by 24” columns and a 10” slab thickness with 6” standard sized drops. The concrete frame itself was designed to take all of the lateral load, eliminating the need for shear walls. Foundations experienced an increase in size due to the additional building weight.

It has been determined that this analysis was successful and designing the hospital addition as a two way concrete flat slab is recommended. Below are the reasons by which this has been concluded:

- Wind drift was greatly reduced to well below the H/400 limit due to increased stiffness.

- Shear walls were not required as the concrete columns and slab were designed to resist all of the lateral loads. Therefore, there are no infringements on the floor plan and the desired flexibility is maintained.

- The concrete design resulted in a savings of approximately $500,000 in comparison to the composite steel design. Additional savings are also possible due to changes such as reduced floor thickness. The concrete structure does not require an increased schedule duration.

- Framing out the individual slab depressions is no longer an inconvenience with the use of concrete and forms, therefore saving time and money.

Based on these results, it has been concluded that this Howard County General Hospital’s patient tower addition would be more efficiently designed as a concrete flat plate structure.

XII. REFERENCES

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APPENDIX A – LOAD CALCULATIONS

Wind Calculations:

North-South Wind

Dimensions and Period

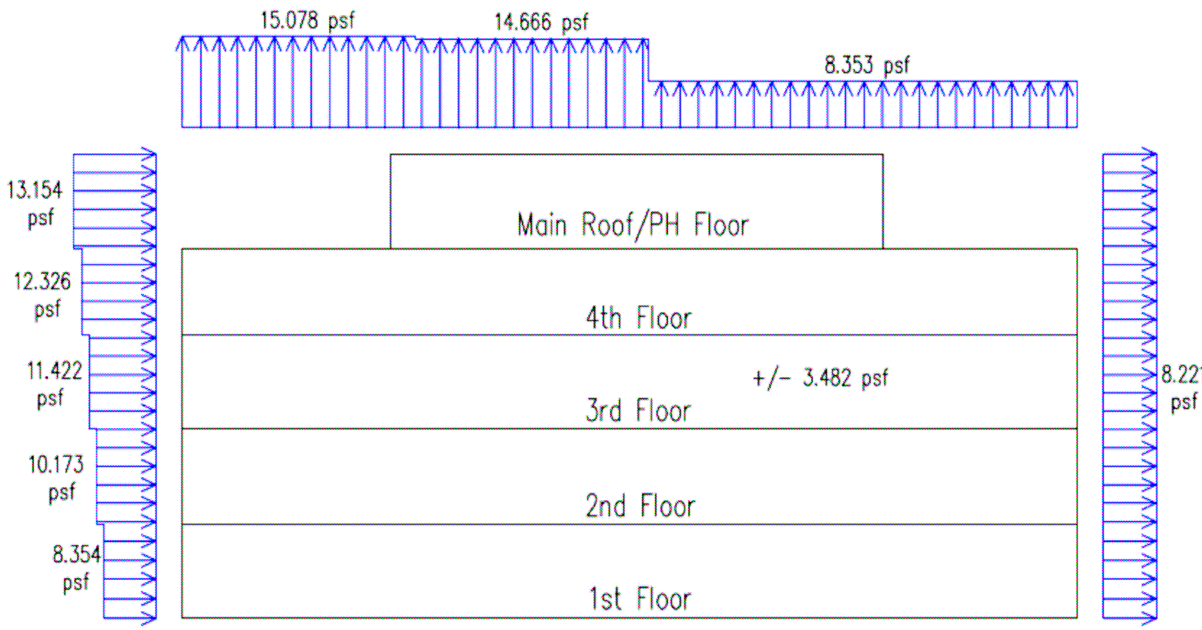
H = 88.5
L = 170
B = 171.5
L/B = 0.99
T_a = 0.904 <1 therefore rigid
*for calculation of Ta see Seismic calcs

Variable	Value	Fig/Table/Eqn
V =	90	Figure 6-1
I =	1.15	Table 6-1
K_{zt} =	1	Eqn 6-3
K_d =	0.85	Table 6-4
GC_{pi} =	0.18	Figure 6-5

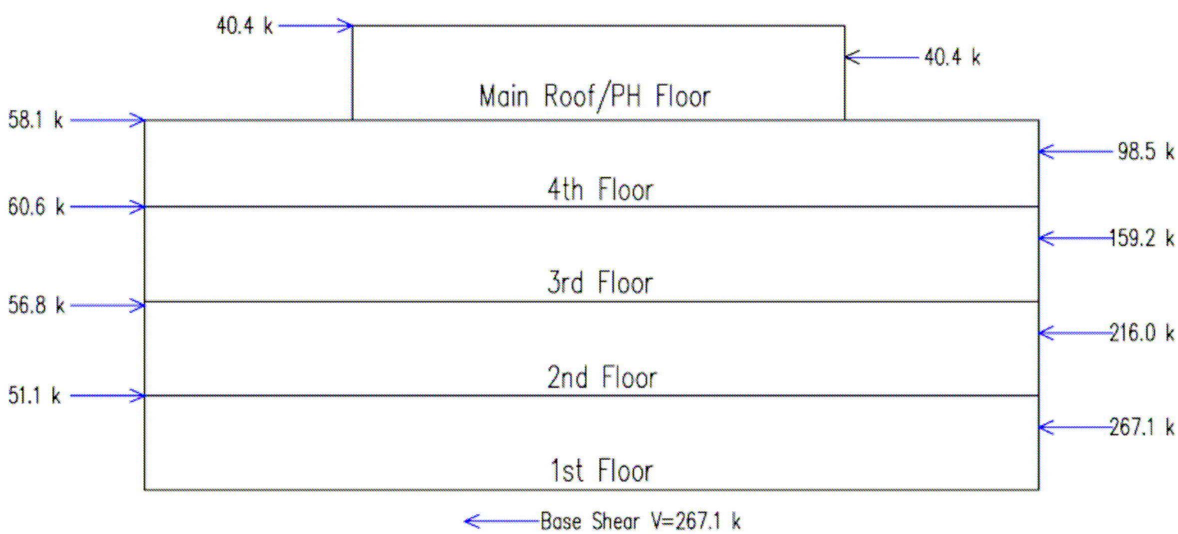
Calculate Pressures using Eqn			
C_p =	0.8	Figure 6-6	(Windward)
C_p =	-0.5	Figure 6-6	(Leeward)
C_p =	-0.7	Figure 6-6	(Sidewall)
G_f =	0.850		

z (ft)	K_z	q_z	P (leeward)	P (windward)	P (sidewall)	+/- qGC_{pi}
0-18	0.605	12.272	-8.221	8.345	-11.510	+/- 3.482
36	0.738	14.960	-8.221	10.173	-11.510	+/- 3.482
54	0.829	16.797	-8.221	11.422	-11.510	+/- 3.482
70.5	0.894	18.127	-8.221	12.326	-11.510	+/- 3.482
88.5	0.954	19.344	-8.221	13.154	-11.510	+/- 3.482
	q_h =	19.344				

Calculate Roof Pressure				
H/L =	0.521	requires interpolation		**P (roof)**
C_p =	-0.917	Figure 6-6	0 to 44.25 ft	-15.078
C_p =	-0.892	Figure 6-6	44.25 to 88.5 ft	-14.666
C_p =	-0.508	Figure 6-6	88.5 to 170 ft	-8.353



NORTH-SOUTH WIND



NORTH-SOUTH WIND

East-West Wind

Dimensions and Period

H =	88.5	
L =	171.5	
B =	170	
L/B =	1.01	
T_a =	0.904	<1 therefore rigid

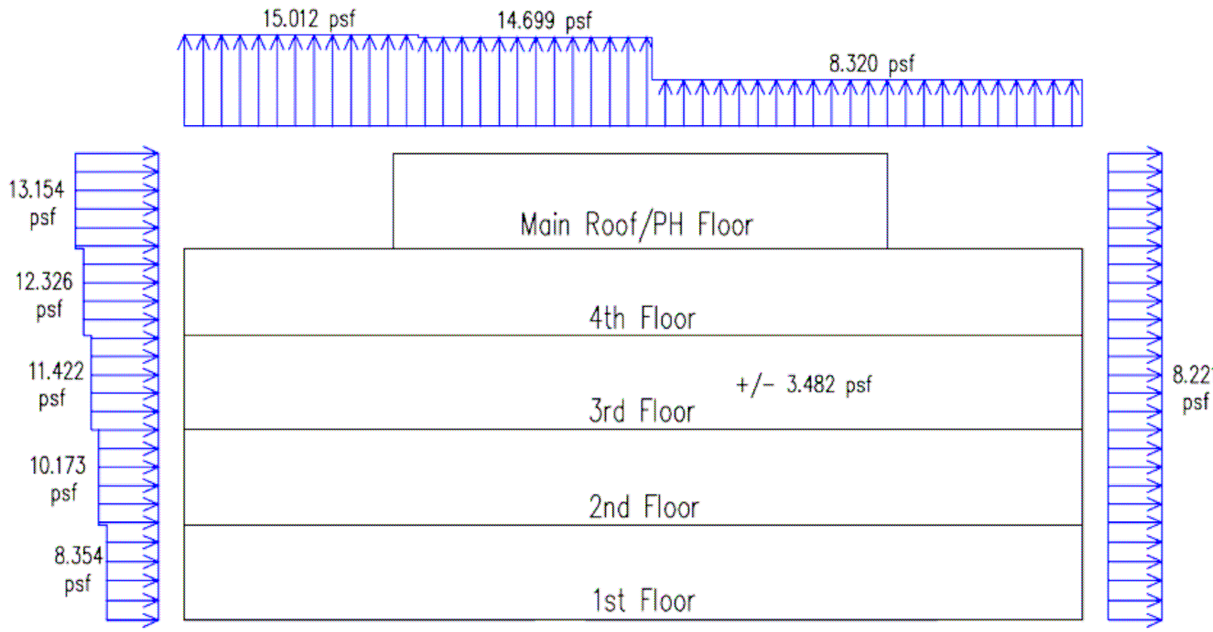
*for calculation of Ta see Seismic calcs

Variable	Value	Fig/Table/Eqn
V =	90	Figure 6-1
I =	1.15	Table 6-1
K_{zt} =	1	Eqn 6-3
K_d =	0.85	Table 6-4
GC_{pi} =	0.18	Figure 6-5

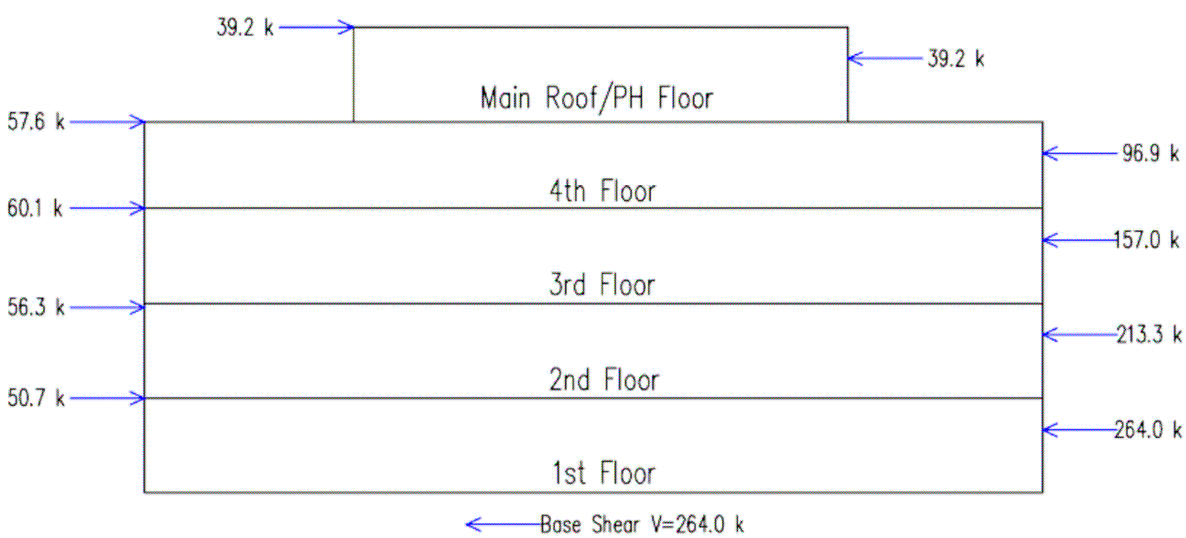
Calculate Pressures using Eqn			
C_p =	0.8	Figure 6-6	(Windward)
C_p =	-0.5	Figure 6-6	(Leeward)
C_p =	-0.7	Figure 6-6	(Sidewall)
G_f =	0.85		

z (ft)	K_z	q_z	P (leeward)	P (windward)	P (sidewall)	+/- qGC_{pi}
0-18	0.605	12.272	-8.221	8.345	-11.510	+/- 3.482
36	0.738	14.960	-8.221	10.173	-11.510	+/- 3.482
54	0.829	16.797	-8.221	11.422	-11.510	+/- 3.482
70.5	0.894	18.127	-8.221	12.326	-11.510	+/- 3.482
88.5	0.954	19.344	-8.221	13.154	-11.510	+/- 3.482
	q_h =	19.344				

Calculate Roof Pressure				
H/L =	0.516	requires interpolation		**P (roof)**
C_p =	-0.913	Figure 6-6	0 to 44.25 ft	-15.012
C_p =	-0.894	Figure 6-6	44.25 to 88.5 ft	-14.699
C_p =	-0.506	Figure 6-6	88.5 to 171.5 ft	-8.320



EAST–WEST WIND



EAST–WEST WIND

Seismic Calculations:

Factors/Coefficients

Mapped Spectral Response Accelerations	$S_S = 0.160$ g
	$S_1 = 0.050$ g
Site Class	D
Seismic Use Group	III
Importance Factor (I)	1.25
Response Modification Coefficient (R)	3.0
Site Class Factors	$F_a = 1.6$
	$F_v = 2.4$
Adjusted Spectral Response Accelerations	$S_{MS} = 0.256$
	$S_{M1} = 0.120$
$S_{MS} = F_a*S_S = 0.160*1.6 = 0.256$	
$S_{M1} = F_v*S_1 = 0.050*2.4 = 0.12$	
Design Spectral Response Accelerations	$S_{DS} = 0.171$
	$S_{D1} = 0.080$
$S_{DS} = (2/3)*S_{MS} = (2/3)*0.256 = 0.171$	
$S_{D1} = (2/3)*S_{M1} = (2/3)*0.12 = 0.08$	
Seismic Design Category	B
$0.167 \le S_{DS} < 0.33$ and $0.067 \le S_{D1} < 0.133$	
Therefore, Seismic Design Category B	
Approximate Period (T_a)	1.011
Ordinary Reinforced Concrete Moment Frames	
Therefore C = 0.016 and x = 0.9	
$T_a = C_t*h_n^x = 0.028*(88.5')^{0.8} = 0.9044$	
Fundamental Period (T)	1.719
$S_{D1} \le 0.1$ Therefore, $C_u = 1.7$	
$T = C_u*T_a = 1.7*0.9044 = 1.538$	
Seismic Response Coefficient (C_s)	0.0194

C_S = min
- $S_{DS}/(R/I) = 0.171/(3.0/1.25) = 0.0713$
- $S_{D1}/[T*(R/I)] = 0.08/[1.538*(3.0/1.25)] = 0.0217$ ← controls
- $(S_{D1}*T_L)/[T^2*(R/I)] = (0.08*8)/[1.538*(3.0/1.25)] = 0.1128$

Building Weight (in kips)

Floor	Slab	Drops	Columns	CMU Wall	Exterior	Partitions	Openings	Depressions	Total
1st	-	-	-	-	-	-	-	-	0
2nd	2413	210	464	50	650	187	-75	-53	3847
3rd	2413	210	464	50	650	187	-75	-53	3847
4th	2413	210	464	50	650	187	-75	-53	3847
PH/Roof	2413	210	426	-	650	-	-	-	3699
PH Roof	734	45	162	-	-	-	-	-	941
SUM	10387	885	1981	151	2600	561	-225	-158	16183

Building Weight calculated in RAM = 16,154 kips (% Error = 0.18%)

Story Forces, Shears, and Moments

Story	h_x (ft)	total W (k)	$h_x^k W_x$	C_{vx}	$F_x = C_{vx}*V$	V_x (k)	M_x (ft-k)
PH Roof	88.5	941	857267.6	0.139	48.7	0	4313.5
Roof/PH	72	3699	2461553	0.399	140.0	48.7	10076.5
4	54	3847	1653539	0.268	94.0	188.7	5076.6
3	36	3847	892802.9	0.145	50.8	282.7	1827.4
2	18	3847	311307.8	0.050	17.7	333.5	318.6
1	0	0	0	0.000	0.0	351.2	0.0
	SUM	16182.6	6176470	1	351.2		21612.6

Story Forces Calculated in RAM

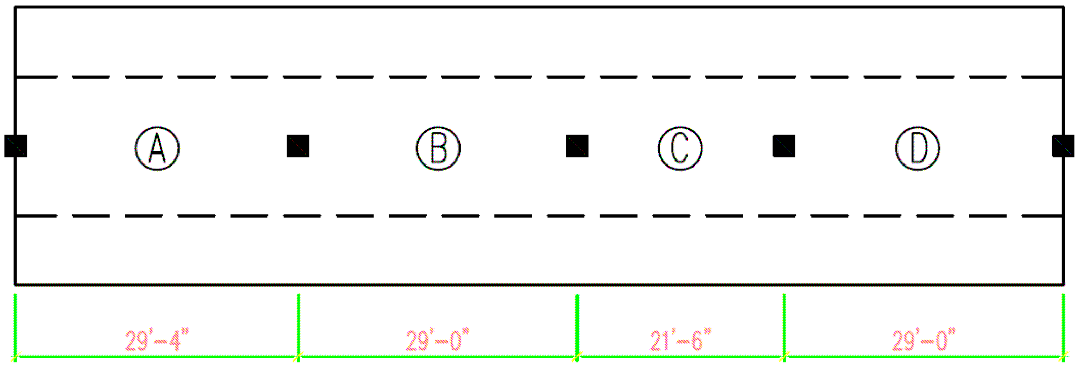
APPLIED STORY FORCES

Type: EQ_IBC03_X_+E_F

Level	Ht	Fx	Fy
	ft	kips	kips
penthouse roof	88.50	60.85	0.00
roof	70.50	118.76	0.00
fourth	54.00	93.74	0.00
third floor	36.00	50.64	0.00
second floor	18.00	17.68	0.00
		341.67	0.00

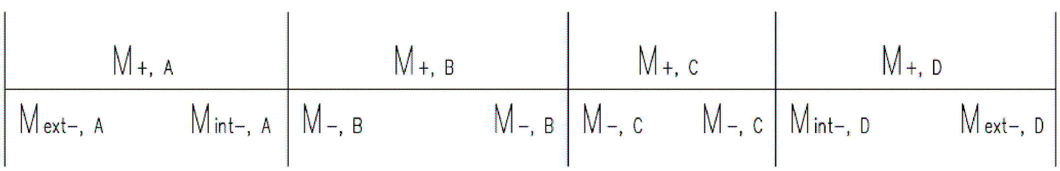
APPENDIX B – SLAB CALCULATIONS

Slab Line J: Hand Design



w_D = 150 pcf*(10”/12) + 15 psf SDL = 140 psf
w_L = 100 psf
w_u = 1.2*140 psf + 1.6*100 psf = 328 psf = 0.328 ksf

$M_{o,A}$ = [0.328*29’*(29.3’ – 24”/12)^2]/8 = 886 ft-k
$M_{o,B}$ = $M_{o,D}$ = [0.328*29’*(29’ – 24”/12)^2]/8 = 867 ft-k
$M_{o,C}$ = [0.328*29’*(21.5’ – 24”/12)^2]/8 = 452 ft-k



$M^-_{ext,A}$ = 0.26$M_{o,A}$ = 0.26*886 = 230.4 ft-k
M^+_A = 0.52$M_{o,A}$ = 0.52*886 = 460.7 ft-k
$M^-_{int,A}$ = 0.70$M_{o,A}$ = 0.70*886 = 620.2 ft-k
M^-_B = 0.65$M_{o,B}$ = 0.65*867 = 563.6 ft-k
M^+_B = 0.35$M_{o,B}$ = 0.35*867 = 303.5 ft-k
M^-_C = 0.65$M_{o,C}$ = 0.65*452 = 293.8 ft-k
M^+_C = 0.35$M_{o,C}$ = 0.35*452 = 158.2 ft-k
$M^-_{int,D}$ = 0.70$M_{o,D}$ = 0.70*867 = 606.9 ft-k
M^+_D = 0.52$M_{o,D}$ = 0.52*867 = 450.8 ft-k
$M^-_{ext,D}$ = 0.26$M_{o,D}$ = 0.26*867 = 225.4 ft-k

460.7		303.5		158.2		450.8	
230.4	620.2	563.6	563.6	293.8	293.8	606.9	225.4

% of Moment to CS:

- $M^-_{ext,A}$ → 100%
- M^+_A → 60%
- $M^-_{int,A}$ → 75%
- M^-_B → 75%
- M^+_B → 60%
- M^-_C → 75%
- M^+_C → 60%
- $M^-_{int,D}$ → 75%
- M^+_D → 60%
- $M^-_{ext,D}$ → 100%

Column Strip Moment	276.4		182.1		94.9		270.5	
	230.4	465.2	422.7	422.7	220.4	220.4	455.2	225.4

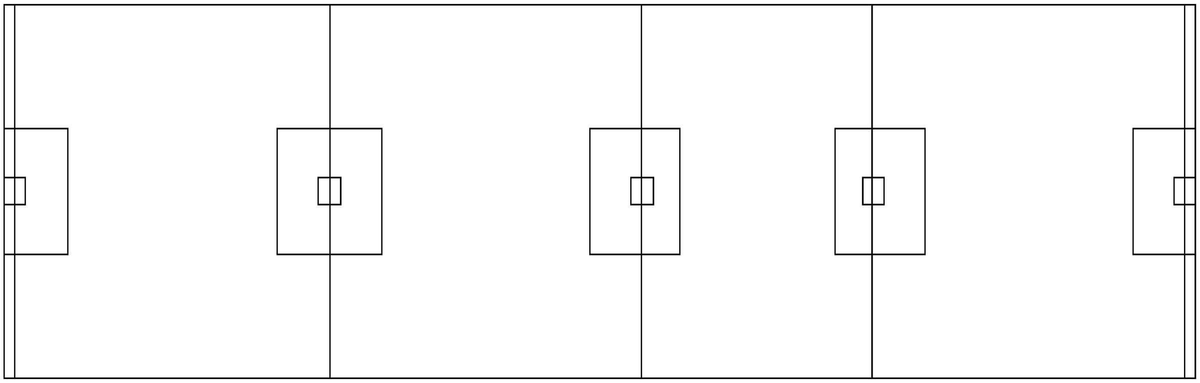
Middle Strip Moment	184.3		121.4		63.3		180.3	
	0	155.0	140.9	140.9	73.4	73.4	151.7	0

	Span A CS			Span A MS			Span B CS		Span B MS	
	M-ext	M+	M-int	M-ext	M+	M-int	M-	M+	M-	M+
Moment	230.4	276.4	465.2	0	184.3	155	422.7	182.1	140.9	121.4
Width "b"	116	174	116	2@87	2@87	2@87	116	174	2@87	2@87
Eff d	14.37	8.5	14.37	8.37	8.5	8.37	14.37	8.5	8.37	8.5
Mn = Mu/phi	256.0	307.1	516.9	0.0	204.8	172.2	469.7	202.3	156.6	134.9
R = Mn/bd2	128	293	259	0	195	170	235	193	154	129
rho	0.0022	0.0051	0.0045	0	0.0033	0.0029	0.004	0.0033	0.0026	0.0022
As = rhobd	3.67	7.54	7.50	0.00	4.88	4.22	6.67	4.88	3.79	3.25
As,min	3.71	3.48	3.71	3.48	3.48	3.48	3.71	3.48	3.48	3.48
N = As/As,bar	6	17	13	6	11	7	11	11	6	8
Nmin = w/2t	10	10	10	10	10	10	10	10	10	10

	Span C CS		Span C MS		Span D CS			Span D MS		
	M-	M+	M-	M+	M-ext	M+	M-int	M-ext	M+	M-int
Moment	220.4	94.9	73.4	63.3	225.4	270.5	455.2	0	180.3	151.7
Width "b"	116	129	2@109	2@109	116	174	116	2@87	2@87	2@87
Eff d	14.37	8.5	8.37	8.5	14.37	8.5	14.37	8.37	8.5	8.37
Mn = Mu/phi	244.9	105.4	81.6	70.3	250.4	300.6	505.8	0.0	200.3	168.6
R = Mn/bd2	123	136	80	67	125	287	253	0	191	166
rho	0.0021	0.0017	0.0013	0.0011	0.0021	0.005	0.0044	0	0.0033	0.0028
As = rhobd	3.50	1.86	1.89	1.63	3.50	7.40	7.33	0.00	4.88	4.08
As,min	3.71	3.48	3.48	3.48	3.71	3.48	3.71	3.48	3.48	3.48
N = As/As,bar	6	8	6	8	6	17	12	6	11	7
Nmin = w/2t	10	10	12	12	10	10	10	10	10	10

The above values were calculated for #7 top bars and #6 bottom bars as used in the design.

Slab Line J: PCA Slab Design



For dimensions, see the diagram included in “Slab Line J: Hand Design”. For the following output results, spans 1 and 6 are the slight cantilevers which were ignored in the hand calculations for simplicity. These results are based on gravity loading only and do not include the lateral moments. This is for the sake of comparison because the direct design method used above is valid for gravity loads only. However, the final slab design results outlined in the report include gravity and lateral loads.

[2] DESIGN RESULTS*

*Unless otherwise noted, all results are in the direction of analysis only. Another analysis in the perpendicular direction has to be carried out for two-way slab systems.

Top Reinforcement:

Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)

	Span	Strip	Zone	Width	Mmax	Xmax	AsMin	AsMax	SpReq	AsReq	Bars
	1	Column	Left	14.50	0.00	0.000	3.132	32.584	17.400	0.000	10-#7
			Middle	14.50	0.00	0.000	3.132	54.769	17.400	0.000	10-#7
			Right	14.50	0.00	0.000	3.132	54.769	17.400	0.000	10-#7
		Middle	Left	14.50	0.00	0.000	3.132	32.584	17.400	0.000	10-#7
			Middle	14.50	0.00	0.000	3.132	32.584	17.400	0.000	10-#7
			Right	14.50	0.00	0.000	3.132	32.584	17.400	0.000	10-#7
	2	Column	Left	14.50	321.89	1.000	4.385	36.513	17.400	4.929	10-#7
			Middle	14.50	0.00	14.666	0.000	32.584	0.000	0.000	---
A			Right	14.50	604.22	28.333	4.385	36.513	10.875	9.430	16-#7
		Middle	Left	14.50	-0.00	1.000	3.132	32.584	17.400	0.000	10-#7
			Middle	14.50	0.00	14.666	0.000	32.584	0.000	0.000	---
			Right	14.50	201.41	28.333	3.132	32.584	17.400	5.204	10-#7
	3	Column	Left	14.50	566.17	1.000	4.385	36.513	10.875	8.813	16-#7
			Middle	14.50	0.00	14.500	0.000	32.584	0.000	0.000	---
B			Right	10.75	364.99	28.000	3.575	36.513	12.900	5.605	10-#7
		Middle	Left	14.50	188.72	1.000	3.132	32.584	17.400	4.868	10-#7
			Middle	14.50	0.00	14.500	0.000	32.584	0.000	0.000	---
			Right	18.25	121.66	28.000	3.942	41.011	16.846	3.103	13-#7
	4	Column	Left	10.75	283.31	1.000	3.575	36.513	12.900	4.327	10-#7
			Middle	10.75	61.86	13.675	2.322	24.157	16.125	1.575	8-#7
C			Right	10.75	356.83	20.500	3.575	36.513	9.923	5.477	13-#7
		Middle	Left	18.25	94.44	1.000	3.942	41.011	16.846	2.402	13-#7
			Middle	18.25	20.62	13.675	3.942	41.011	16.846	0.521	13-#7
			Right	18.25	118.94	20.500	3.942	41.011	16.846	3.033	13-#7
	5	Column	Left	10.75	500.89	1.000	3.575	36.513	9.923	7.762	13-#7
			Middle	14.50	0.00	14.500	0.000	32.584	0.000	0.000	---
D			Right	14.50	353.87	28.000	4.385	36.513	17.400	5.430	10-#7
		Middle	Left	18.25	166.97	1.000	3.942	41.011	16.846	4.277	13-#7
			Middle	14.50	0.00	14.500	0.000	32.584	0.000	0.000	---
			Right	14.50	-0.00	28.000	3.132	32.584	17.400	0.000	10-#7
	6	Column	Left	14.50	0.00	1.000	3.132	54.769	17.400	0.000	10-#7
			Middle	14.50	0.00	1.000	3.132	54.769	17.400	0.000	10-#7
			Right	14.50	0.00	1.000	3.132	32.584	17.400	0.000	10-#7
		Middle	Left	14.50	0.00	1.000	3.132	32.584	17.400	0.000	10-#7
			Middle	14.50	0.00	1.000	3.132	32.584	17.400	0.000	10-#7
			Right	14.50	0.00	1.000	3.132	32.584	17.400	0.000	10-#7

Bottom Reinforcement:

Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)

	Span	Strip	Width	Mmax	Xmax	AsMin	AsMax	SpReq	AsReq	Bars
	1	Column	14.50	0.00	0.000	0.000	32.815	0.000	0.000	---
		Middle	14.50	0.00	0.000	0.000	32.815	0.000	0.000	---
A	2	Column	14.50	208.55	12.810	3.132	32.815	13.385	5.353	13-#6
		Middle	14.50	139.03	12.810	3.132	32.815	17.400	3.538	10-#6
B	3	Column	14.50	164.05	15.244	3.132	32.815	17.400	4.188	10-#6
		Middle	14.50	109.37	15.244	3.132	32.815	17.400	2.774	10-#6
C	4	Column	10.75	66.83	10.256	2.322	24.329	16.125	1.691	8-#6
		Middle	18.25	44.55	10.256	3.942	41.302	16.846	1.120	13-#6
D	5	Column	14.50	221.65	15.739	3.132	32.815	13.385	5.698	13-#6
		Middle	14.50	147.77	15.739	3.132	32.815	17.400	3.765	10-#6
	6	Column	14.50	0.00	1.000	0.000	32.815	0.000	0.000	---
		Middle	14.50	0.00	1.000	0.000	32.815	0.000	0.000	---

It can be seen that the required steel is similar, and the design moments, though not exact, are consistent. This verifies the PCA Slab design output for the required amount of reinforcing.

Punching Shear @ Column 2:

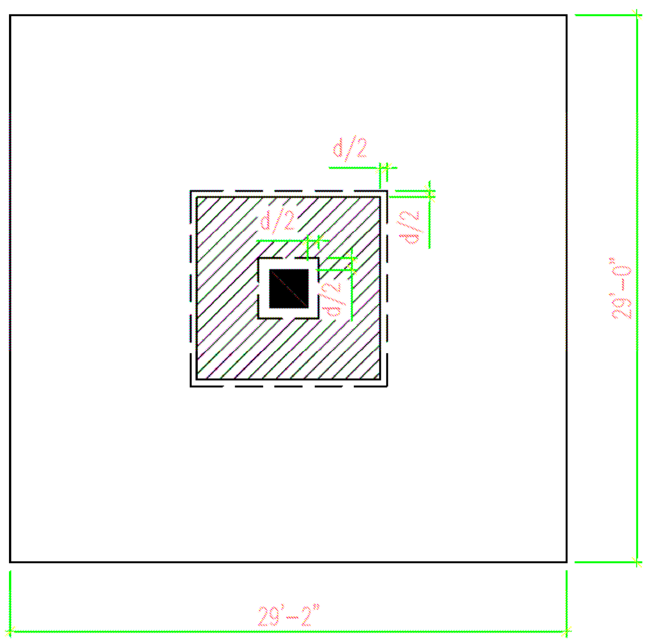
The diagram to the right shows the locations at which punching shear shall be checked for Column 2 of the slab line above. The tributary area for this column is 29'-2" by 29'.

The solid square is the column, with the critical punching shear perimeter shown in the dashed line offset d/2 from the column.

The diagonally hatched area is the drop panel, with its critical perimeter also shown as a dashed line.

$b_{o,\,col}$ = (24 + 14.37")*4 = 153.5"

$b_{o,\,drop}$ = (9.67*12 + 8.37")*4 = 497.6"



@ Column:

V_u = wu*Area = 0.328*[(29.33'*29') – (24"/12 + 14.37"/12)^2]
= 279 kips
V_c = 4*5000$^{1/2}$*153.5"*14.37"
= 624 kips
ϕV_c = 0.75*624 kips = 468 kips > V_u = 279 kips, therefore OK

@ Drop Panel:

V_u = wu*Area = 0.328*[(29.33'*29') – (9.67' + 8.37"/12)^2]
= 244 kips
V_c = 4*5000$^{1/2}$*497.6"*8.37"
= 1178 kips
ϕV_c = 0.75*1178 kips = 884 kips > V_u = 244 kips, therefore OK

Bottom Steel @ Slab Depression in Span A

M_u = 208.55 ft-k from PCA Slab output above
$d_{adjusted}$ = 10" – 0.75" cover – 0.375" – 2" depression = 7.25"
a = (60*A_s)/(0.85*5*174") = 0.08114*A_s
ϕM_n = 208.55*12 = 0.9* $A_{s,\ adjusted}$*60*(7.25 – 0.08114*$A_{s,\ adjusted}$/2)
$A_{s,\ adjusted}$ = 6.64 in^2

The required steel from the PCA Slab output above for the unadjusted d is 5.35 in^2.

6.64 in^2/5.35 in^2 = 1.24 → This means that approximately 25% more steel is required where slab depressions occur. This ratio could be used to adjust the required steel at all other slab depressions.

The bottom steel mat of #6@12" should be checked so additional bottom steel can be required if necessary. For this span with a 14.5 foot column strip width, the provided reinforcing is:

14.5 ft*12 in/ft = 174"
174"*0.44 in^2/12" spacing = 6.38 in^2 of steel provided
$d_{additional}$ = 6.64 in^2 – 6.38 in^2 = 0.26 in^2

Therefore, (1) additional #6 bar would provide sufficient bottom steel to account for the slab depression.

APPENDIX C – COLUMN CALCULATIONS

Column Moments from RAM:

	Base/1st Floor						1st Floor/2nd Floor					
Col #	**LC**	**P_U**	**$M_{T,maj}$**	**$M_{B,maj}$**	**$M_{T,min}$**	**$M_{B,min}$**	**LC**	**P_U**	**$M_{T,maj}$**	**$M_{B,maj}$**	**$M_{T,min}$**	**$M_{B,min}$**
C1	4	184.9	177.4	-88.9	27.9	-14.3	4	141.7	38.3	-202.5	12.5	-41
C2	4	528.9	199.1	-95.3	-78.6	36.8	5	247.8	135	-269.8	-16.5	7.1
C3	4	369	-267.9	130.7	-28.1	14.7	4	303.5	-122.4	306.5	-37.1	25.2
C4	5	207.9	164.9	-83.1	-6.3	3.2	4	286.3	67.4	-164.8	-93	127.9
C5	4	551.6	92.8	-40.4	-193.7	94.1	4	454.5	79.6	-112	-125.6	236.5
C6	2	927.8	54.4	-27.7	-12.5	42.2	2	745.3	53.9	-35.6	-10	84
C7	2	788.9	34.5	-29.7	-39.6	56.7	5	329.8	-86.1	141.8	-80.4	130.9
C8	2	694	25.7	9.3	-44.8	36.4	4	506.1	1.2	-60.3	-132.9	186.2
C9	2	896.8	121.8	-40	-18.2	31.4	4	454.2	79.6	-112	-125.6	236.5
C10	2	1134	-71.6	59.9	-15.2	41.3	2	916.8	-105.9	52.8	11.5	46.7
C11	2	889.8	30.7	14	-48	23.8	2	729.3	-31.7	-32.7	-52.1	48.3
C12	2	1304.9	61.7	5.4	-37.5	36.1	2	1083.1	-5	-62	-89.4	37.4
C13	2	705.6	119	-42.5	-33.6	28.7	4	482	151.2	-183.3	-91.1	135.6
C14	2	666.8	-118.9	45.7	17.7	-29.4	4	440.6	-149.5	186	38.5	-126.2
C15	2	882.1	65.3	-20	15.8	8.2	2	722.3	70	-39.1	17.9	17.2
C16	2	1378.9	-30.6	-16.4	-64.6	32.5	2	1167.2	44.8	31.2	-4.5	64.8
C17	2	1177	50.8	-4.3	5.5	-44.4	2	960.1	26.4	-50.8	33.1	-5.4
C18	2	669.9	-114.6	45.1	-15.8	27.3	4	458.1	-129.3	219.8	-19.8	11.1
C19	4	186.5	157.1	-78.3	157.1	19.2	4	136.4	36.2	-180.5	-16.5	54.4
C20	5	207.9	153.7	-77.5	-1.7	0.9	4	290.1	77.3	-165.7	91.6	-144.7
C21	2	792	-37.2	-2.2	55.3	-31.4	4	475.4	89.6	-165	78.3	-170.7
C22	2	1373	61	-43.4	7.4	-38.4	2	1176.2	48.5	-61	147.2	-5.3
C23	2	1034	-48.5	-25.1	6.7	-5.6	2	854.1	64.5	14.2	7.9	9.5
C24	4	517	168.3	-78.9	123.6	-59.4	5	251.8	135.7	-250.5	13.7	5.8
C25	2	966.9	82.9	-6.4	-46.4	33.1	4	640	84.6	-151.8	-123.4	195.9
C26	2	1144.4	-109.5	21.8	-23.9	-4.7	2	934.2	-128.7	42.8	-17.8	-9.4
C27	2	742.7	179.2	-64.5	49.6	-33.5	4	483.9	168.5	-225	107.8	-169.4
C28	2	1260.1	100.9	-13.3	-58.4	52.4	2	1049.3	75.5	-24.9	22.9	104.7
C29	2	667.4	-77.5	16.9	108.1	-36.5	4	379.7	-103	156.1	130.7	-172.9
C30	4	399.3	-81.9	41.5	122.5	-57.4	4	353.5	-58.4	114.8	105.6	-152.8
C31	2	415.7	-33.5	12.3	-7.4	7.8	7	180.7	-18.4	21.9	-79.4	133.4

C32	2	464.8	27.9	-5.5	27.9	12.8	4	216.1	80.5	130.2	-69.3	102.8
C33	2	604	-67.8	18	8.1	-4.7	4	317.3	-165.6	176.4	87.6	-101.7
C34	2	440.1	-27.2	10.1	-1.9	3.1	5	242.7	-70.2	123.7	50.5	-86.5
C35	2	421.3	15.9	-6.8	35	-12.9	5	189.9	95.1	-148.2	5.7	10.7
C36	4	353.2	-239.5	117.6	49.3	27.2	4	607.4	-130.2	276.3	33.8	-50.7
C37	2	677.9	76.4	-24.1	86.3	-64.5	4	454.2	71.2	-120.3	141.1	-231.5
C38	2	751.9	-47.9	-0.7	99.7	-64	4	479.8	-73.6	136.7	133.4	-202
C39	2	686.8	-184.9	65.5	1	-0.8	4	459.4	-165.8	227.7	-50.3	114.3
C40	2	682.4	-183	63	-0.4	0.7	4	450.5	-126	223.2	-10.9	99.8
C41	2	702.4	25	1.1	-57.7	32.7	4	542.4	133.1	-187.3	-207.8	200.1
C42	4	246.2	-142.4	69.8	17.4	-7.3	4	303.1	-208.9	240	157.9	-177.8

	2nd Floor/3rd Floor						3rd Floor/4th Floor					
Col #	**LC**	P_U	$M_{T,maj}$	$M_{B,maj}$	$M_{T,min}$	$M_{B,min}$	**LC**	P_U	$M_{T,maj}$	$M_{B,maj}$	$M_{T,min}$	$M_{B,min}$
C1	4	103.1	76.6	-54.7	19.7	-14.6	4	64.2	83.3	-16.2	17.1	-4.1
C2	4	272.5	206.2	-180.1	-33.2	28.7	4	183.7	183.1	-116.8	-39.2	-37
C3	4	235	-173.8	143	-27.2	30.3	4	151.2	-173.3	99.4	-26.8	32.5
C4	6	185	42.9	-45.6	-124.6	138.2	6	130.6	46.2	-40.4	-130.6	99
C5	4	355.5	96.5	-76.7	-165.3	142.6	4	228.7	91.4	-65.9	-156.7	105.7
C6	5	339.5	184.9	-178.4	5.3	-5.8	4	320.2	222.6	-199.5	-24.3	10.7
C7	4	363.1	-69.9	112.4	-102.8	112.3	4	300.7	155.6	-100.6	-10.9	8.3
C8	4	369	-44	29.2	-167.4	138.9	4	231.3	-43.7	19.9	-159.7	106.6
C9	2	592	118.3	-50.2	-19.7	32.1	6	356.2	133.5	-101	-80.3	59.1
C10	4	588.3	-203.7	129.7	111.3	-100.2	6	382.1	-232.7	192.5	4.2	-1
C11	2	587.4	73.1	35.1	-85.4	52.6	4	359.2	99.6	-65.6	-136.9	97.6
C12	2	890	-5.9	-42	-87.5	30	2	685.3	-11.4	-85.3	-88.5	58.7
C13	4	357.8	137.3	-158.6	-160.8	99.3	6	248.2	85.5	-77.8	-182.1	143.6
C14	4	335.3	-152.6	193.4	-22.4	-14.1	4	223.7	-154.9	107.1	-30.3	30.2
C15	2	581.5	99.8	-65.5	-21.1	33.7	4	351.8	137.7	-97.2	-87.9	69.4
C16	2	938.8	1.3	-48.9	-28.1	5.4	2	715.1	-0.8	-75.1	-25.3	30.4
C17	2	744	-38	-53.7	120.7	15.1	6	393.3	-12.2	-0.9	186.5	-135.6
C18	6	350.8	-96.3	148.4	5.5	35.4	4	225.7	-111	69.3	22.3	-15.9
C19	4	99.6	73.2	-52	-26.4	20	4	62.4	79.1	-16	-23.4	6.3
C20	4	216.7	78.3	-68.9	131.5	-108.5	4	151.5	94.7	-52.7	105.9	-74.6
C21	4	379.6	-117.8	94.5	114.2	-110.2	4	290.3	-93.8	58.9	112.1	-70.1
C22	2	950.3	134.6	11.4	3.7	-127.4	2	714.7	-17.7	-138	21.3	0.9
C23	2	695.7	-62.3	-95.4	3.7	19.3	2	512.7	50.4	69.3	6.2	-4.2

C24	4	284.9	207.2	-117.1	29.4	-27	4	190.3	185.7	-117.9	35.6	-36.9
C25	4	479.5	121.2	-91.3	-154.6	139.7	4	310.1	107.3	-68.5	-135.5	90.1
C26	2	729.7	-37.9	132.8	13.3	19.3	4	402.2	-150.8	113.6	-94.9	55.3
C27	4	373.5	189.6	-171.8	139.7	-115.9	4	269.7	181.6	-146.5	124.3	-87.4
C28	2	800.1	67	-8.5	-6.2	64.1	2	560.4	-10.6	-73.1	-136.8	-0.1
C29	6	271.9	-27	35.6	176.1	-203.3	6	190.7	-40.8	26.5	177	-149.3
C30	6	230	-15.4	16.5	131.4	-152.7	6	173.9	-15	16.2	141.7	-107
C31	6	200.8	-24.9	29	-111.4	114.5	6	173.2	-28.4	24	-106.8	85.2
C32	4	214.2	101.8	-87.3	-78.7	76.7	4	177.7	85.8	-56.3	-66.1	53.7
C33	4	241.1	-174.3	188.9	101.5	-107.7	6	168.5	-183.7	151.7	43.5	-39.8
C34	6	227.5	-21.6	23.5	-108.4	116	6	170.9	-24.7	20.3	-108.7	80.9
C35	4	197.4	120.7	-112.5	7.9	-11.4	6	167.8	108.4	-83.9	23.4	-22.5
C36	4	240.3	-177.6	149.4	32.3	-27.9	4	154.6	-177.6	106.3	31.7	-33.7
C37	4	362.6	107.8	-69.2	167	-157.1	4	229.2	101.1	-71.4	159.3	-112
C38	4	371.8	-87.9	78.7	151.3	-118.7	4	247.6	-80.2	39.7	147.4	-102.9
C39	6	341.3	-184.8	213	-7.8	9	2	223.1	-123.4	209.8	0.6	0.7
C40	6	339.1	-143	163.6	34	-41.5	6	227	-160.5	122.4	26.1	-24
C41	5	291.8	170.7	-168.9	-211.3	210.6	4	248.5	123.5	-113.3	-203.5	182.8
C42	4	223.3	-229.2	239.7	183.8	-189.7	4	147.8	-206.4	177.6	147.7	-140.9

	4th Floor/Main Roof					
Col #	**LC**	**P_U**	**$M_{T,maj}$**	**$M_{B,maj}$**	**$M_{T,min}$**	**$M_{B,min}$**
C1	4	25.4	55.1	6.9	8.6	2.6
C2	4	82.7	189.4	-82.4	-86.5	42.4
C3	3	71.1	-206.5	81.1	-67.1	32.8
C4	6	55.9	61.1	-37.2	-134.1	77
C5	3	116.8	55.6	-62.4	-291.8	82.9
C6	4	141.9	227.6	-98.3	33.8	4.8
C7	2	194.4	19.8	-114.6	-78.3	101.9
C8	3	118.3	61.1	16.4	-287.1	90.5
C9	2	270.5	390	-74.9	6.4	37.2
C10	2	245.5	-55.1	252.8	-2.6	-8.2
C11	2	266.5	-10.4	-40.2	-391.1	78
C12	2	337.2	-29.2	-59.6	34.6	167.7
C13	2	183.1	93.3	-146.6	-87	185.6
C14	3	119.7	-198.2	84.5	-79.4	36.5
C15	2	251.1	303.9	-75.2	-32.9	74.3

C16	2	374.1	-66.1	-35.1	137.6	122.6
C17	2	228.5	-11.6	57.2	55.7	-225.3
C18	3	116.5	-117.3	49.3	40	-11.4
C19	4	24.5	52	4.7	-11	-3.2
C20	6	57.9	58.8	-33.8	140	-79.6
C21	4	167.9	-79.8	30.3	105.7	-47.2
C22	2	414.1	-73	-160.5	103.5	23.1
C23	2	265.9	-132	-99.1	9.5	-9.1
C24	4	83.8	200.7	-89.4	78.1	-43.7
C25	4	147.6	41.9	-0.1	-175.8	107.9
C26	2	259.9	0.9	180.4	54	46.9
C27	2	185.5	148.7	-249.3	53	-116.2
C28	2	225.9	27.1	11.3	-25.5	175.3
C29	2	169	1.1	92.7	52.4	-154.9
C30	2	134	-25.4	26.5	50.8	-118.5
C31	2	182.4	-49.1	39	-80.1	3.4
C32	4	130.6	89.5	-34.7	-64	37.4
C33	3	99.1	-172.1	89.7	100.5	-47.4
C34	6	111	-17.3	25.6	83.8	-33.4
C35	6	106.9	97.9	-51.6	46.9	-22.9
C36	3	72.7	-224.2	90.7	64.5	-33
C37	3	118.8	136.9	-65.1	207.7	-92.5
C38	3	128.2	-47.5	18.5	233.5	-97.3
C39	3	121.3	-314.8	137.4	-2.7	1.6
C40	3	120.6	-226.6	98.9	60	-28.2
C41	4	118.5	75.2	-66.5	-217	141.4
C42	4	60.3	-193.2	145.7	104	-97.6

Gravity Load Takedowns: Column 16

Tributary Area = 29’*29.2’ = 847 SF/floor

Dead Load:

Floor = 150 pcf*(10”/12”) + 15 SDL = 140 psf = 0.14 ksf
Drop Panel = 150 pcf*9.67’*9.67’*6”/12 = 7.0 k/drop
Column = 150 pcf*2’*2’ = 0.6 k/ft

P_D = 0.14*6 floors*847 + 7.0*6 drops + 0.6*[5*(18'-16"/12) + (16.5'-16"/12)]
P_D = 812.6 kips

Live Load:

Typical Floor = 100 psf = 0.10 ksf
Penthouse Floor (Main Roof) = 125 psf = 0.125 ksf (Unreducible)
Roof = 30 psf = 0.03 ksf (Unreducible)
A_T = 847 per floor*4 floors = 3388 SF
A_I = 4*3388 = 13552 SF
$L_R = 100*[0.25 + 15/(13552)^{1/2}]$= 37.8 < 0.4*100 = 40 therefore, use L_R = 0.04 ksf

P_L = 0.04*4 floors*847 + 0.125*847 + 0.03*847
P_L = 266.8 kips

Gravity Load Takedowns: Column 6

Tributary Area = 31'*24' = 744 SF/floor

Dead Load:

Floor = 150 pcf*(10"/12") + 15 SDL = 140 psf = 0.14 ksf
Drop Panel = 150 pcf*10.33'*8'*6"/12 = 6.2 k/drop
Column = 150 pcf*2'*2' = 0.6 k/ft

P_D = 0.14*5 floors*744 + 6.2*5 drops + 0.6*[4*(18'-16"/12) + (16.5'-16"/12)]
P_D = 600.9 kips

Live Load:

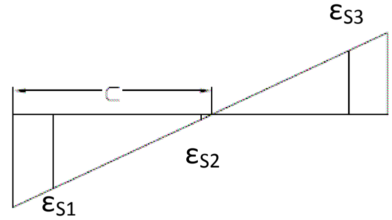
Typical Floor = 100 psf = 0.10 ksf
Roof = 30 psf = 0.03 ksf (Unreducible)
A_T = 744 per floor*4 floors = 2976 SF
A_I = 4*2976 = 11904 SF
$L_R = 100*[0.25 + 15/(11904)^{1/2}]$= 38.7 < 0.4*100 = 40 therefore, use L_R = 0.04 ksf

P_L = 0.04*4 floors*744 + 0.03*744
P_L = 145.0 kips

Interaction Diagram for 24x24 Column with (8) #8 bars:

Pure Compression:

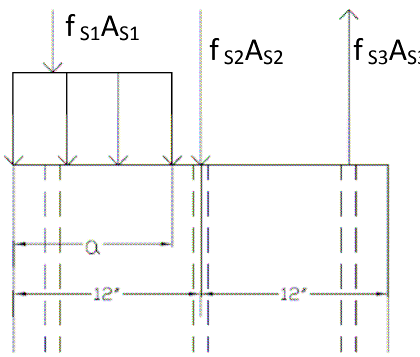
$P_o = 0.85f'c*(A_c\text{-}A_s) + A_s*f_s$
$= 0.85*5*(24*24 + 8*0.79*60)$
$= 2800$ kips



Balanced Point:

$\varepsilon_y = 60/29000 = 0.00207$

$c = [0.003/(0.003 + 0.00207)]*21.5'' = 12.72''$
$a = 0.80*c = 0.80*12.72 = 10.18''$
$\varepsilon_{S1} = 0.003*(12.72''\text{-}2.5'')/12.72'' = 0.00241 > \varepsilon_y$
$f_{S1} = 60$ ksi
$\varepsilon_{S2} = 0.003*(12.72''\text{-} 12'')/12.72'' = 0.00017 < \varepsilon_y$
$f_{S2} = 0.00017*29000 = 4.9$ ksi
$\varepsilon_{S3} = 0.003*(12.72''\text{-}21.5'')/12.72'' = 0.00207 = \varepsilon_y$
$f_{S3} = -60$ ksi



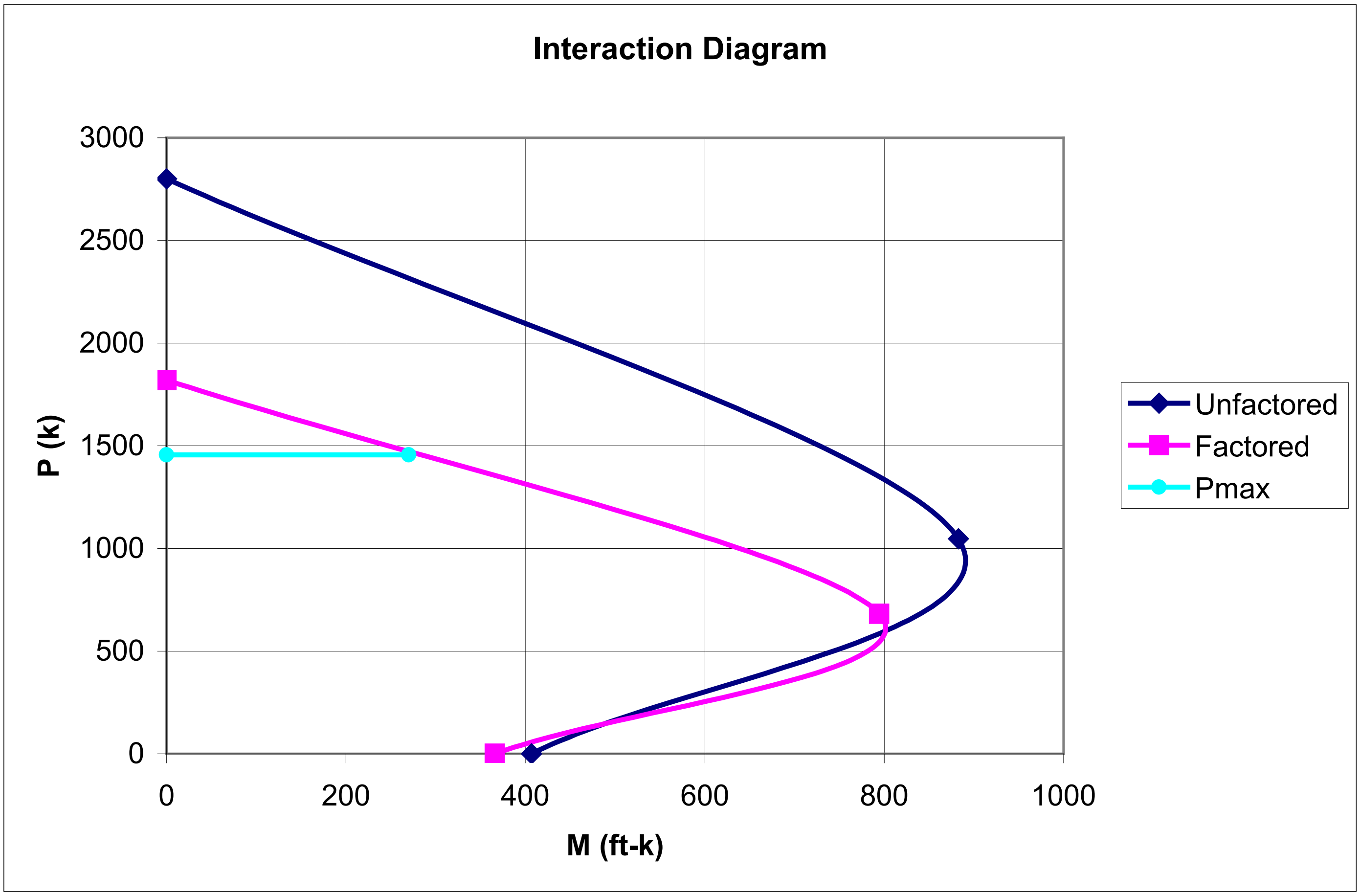
$P_b = (0.85*5*24''*10.18'') + 3*60 \text{ ksi} + 2*4.9 \text{ ksi} + 3*\text{-}60 \text{ ksi}$
$= 1047.8$ kips
$M_b = 0.85*5*24*10.18''*[24''/2 – 10.18''/2] + 3*60*(12''\text{-} 2.5'') + 3*\text{-}60*(12''\text{-} 21.5'')$
$= 10595$ in-k $= 882.9$ ft-k

Pure Flexure:

Assume ε_{S1} does not yield but ε_{S2} and ε_{S3} do yield
$f_{S1} = (0.003/c)*(c - 2.5'')*29000$
$f_{S2} = -60$ ksi
$f_{S3} = -60$ ksi

$\Sigma F = 0 = 0.85*5*24''*0.80c + 3*f_{S1} + 2*f_{S2} + 3*f_{S3}$
$= 81.6c + 3*(87 – 217.5/c) - 120 - 180$
Solving for c, $c = 3.08''$
$a = 0.80*c = 0.80*3.08'' = 2.46''$
$f_{S1} = (0.003/3.08'')*(3.08'' - 2.5'')*29000 = 16.4$ ksi < 60 ksi, therefore assumption OK
$\varepsilon_{S2} = 0.003*(3.08''\text{-} 12'')/3.08'' = -0.0087 < \varepsilon_y$, therefore assumption OK
$\varepsilon_{S3} = 0.003*(3.08''\text{-} 21.5'')/3.08'' = -0.0179 < \varepsilon_y$, therefore assumption OK

$M_o = 0.85*5*24''*2.46''*(24''/2 – 2.46''/2) + 3*16.4*(12'' – 2.5'') + 3*\text{-}60*(12'' – 21.5'')$
$= 4879.8$ in-k $= 406.7$ ft-k



This interaction diagram will be used for the hand column checks.

PCA Column Check for Biaxial Bending:

For biaxial bending, there is a 3-dimensional interaction diagram because P, M_x and M_y must all be plotted together. To view the interaction diagrams in PCA Column for the various load combinations, one variable must be constant, and the other two will be plotted, to create a 2 dimensional interaction diagram that is viewable on the screen. It can be difficult to and interpret this kind of interaction diagram, which is why the columns were designed in RAM. However, to demonstrate an understanding of how RAM designed the columns, these sample runs were performed.

Column 12:

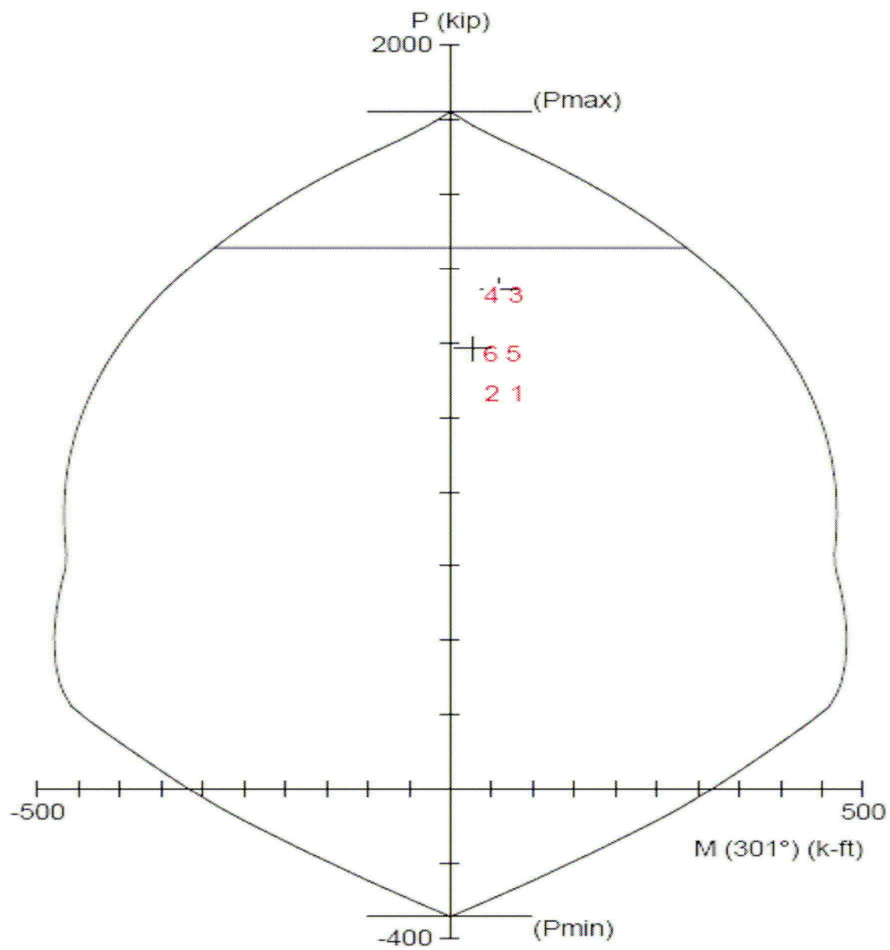
The loads for column 12 at the first floor were obtained from the RAM analysis. The controlling unfactored loads are summarized in the table below.

Column #: 12
Floor: 1st

Load Case	P	$M_{maj,T}$	$M_{maj,B}$	$M_{min,T}$	$M_{min,B}$
Dead	769.7	23.3	-11.4	-35.8	18.2
Live	238	1.56	-0.8	-4.5	2.3
Wind	5.2	41.2	-20.2	36	-18.3
Seismic	6.5	50.1	-24.6	44.3	-22.5

This column was selected to show the interaction of a column with significant axial loads and small moments, which is common for the lower levels of this building. Once these loads were input into PCA Column and a 24” by 24” column with (8) #8 bars was specified, the program was run. Below are samples of the results plotted on the viewable 2-dimensional interaction diagrams.

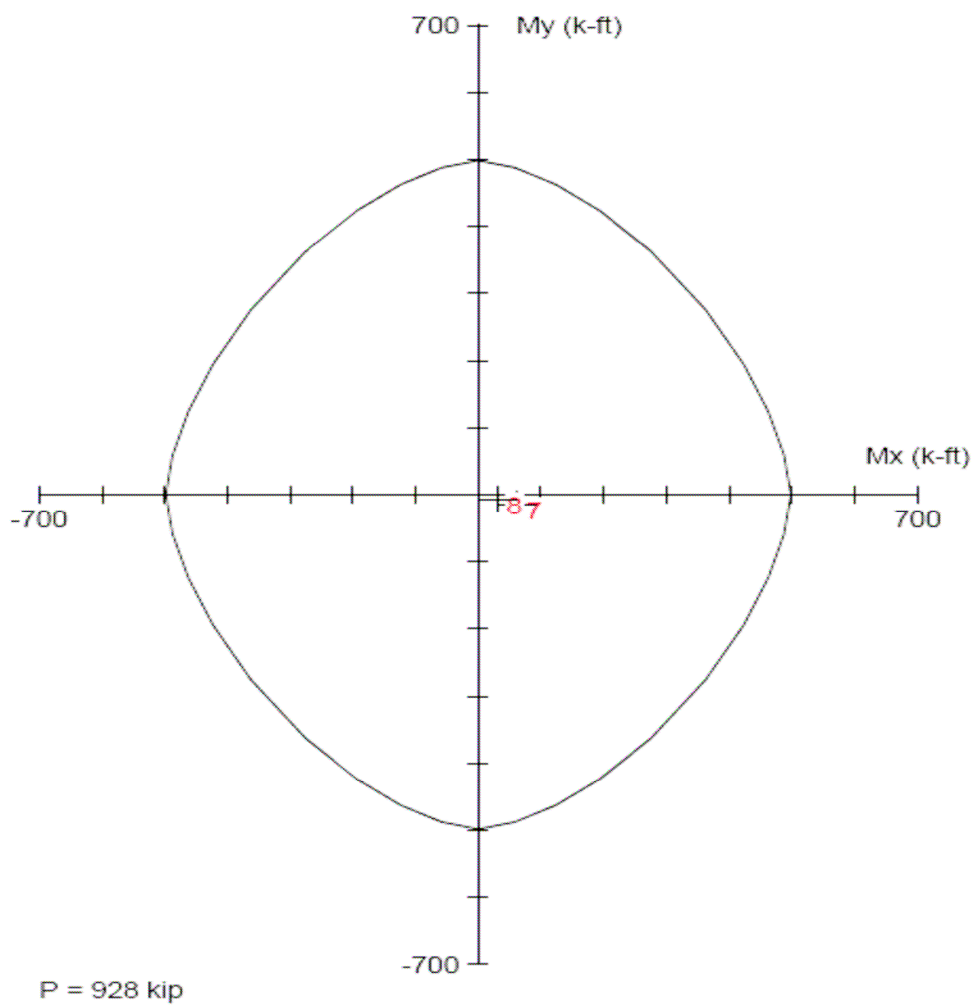
P-M plot at M (301 degrees)



You can see here the axes are labeled P and M. To imagine what this interaction diagram would look like in a 3 dimensional sense, the diagram above would be rotated about the P axis and form an almost egg-like shape.

It can be seen here that the axial loads are significantly larger than the moments, as expected. Therefore, the points are plotted with small x values (moments) and large y values (axial loads).

M_x-M_y plot at P=928 kips



You can see here the axes are labeled M_x and M_y. To understand how this diagram relates to the one above, imagine horizontally "slicing" the rotated 3-dimensioinal egg-like diagram at P=928 kips.

Since this diagram is taken at a particular axial load value, it doesn't show the relationship between P and M like the prior diagram, but instead the relationship between M_x and M_y. Neither moment is very large, but for the load combinations plotted above, it can be seen that M_x is larger than M_y. Different load combinations will be visibly plotted for different values of P.

Hand Column Checks for Uniaxial Bending:

Column #: 6
Floor: 1st

Load Case	P	M_{top}	M_{bottom}
Dead	602.4	31.4	-16
Live	120.5	10.4	-5.3
Live Roof	24.3	0.1	-0.1
Wind	16.2	62.1	-31.5
Seismic	21.7	72.1	-36.6

Controlling Load Combination: $1.2D + 1.6L + 0.5L_r$

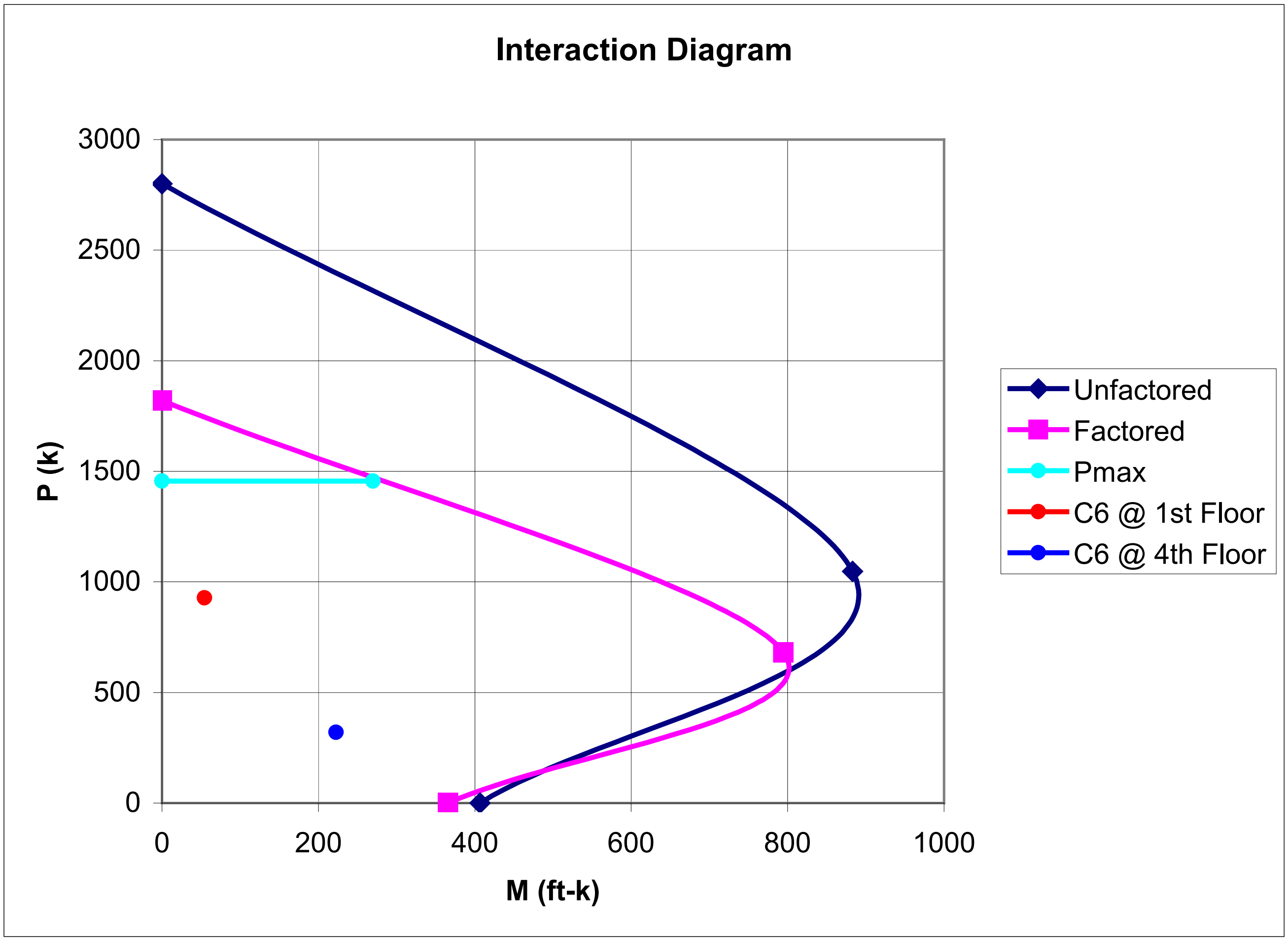
$P_u = 1.2*602.4 + 1.6*120.5 + 0.5*24.3 = 927.8$ k
$M_{u,top} = 1.2*31.4 + 1.6*10.4 + 0.5*0.1 = 54.4$ ft-k
$M_{u,bott} = 1.2*-16.0 + 1.6*-5.3 + 0.5*-0.1 = -27.7$ ft-k

Column #: 6
Floor: 4th

Load Case	P	M_{top}	M_{bottom}
Dead	231	43.8	-39.8
Live	39	14.8	-15
Live Roof	24.5	-3.4	1.1
Wind	7	-102.7	90.5
Seismic	-4.8	79.6	-55.8

Controlling Load Combination: 1.2D + 0.5L +/- 1.6W

Pu = 1.2*231 + 0.5*(39 + 24.5) + 1.6*7 = 320.2 k
Mu,top = 1.2*43.8 + 0.5*(14.8 + -3.4) + 1.6*102.7 = 222.6 ft-k
Mu, bott = 1.2*-39.8 + 0.5*(-15 + 1.1) + 1.6*-90.5 = -199.5 ft-k



It can be seen that this column is OK at both levels because the points are within the interaction diagram. According to RAM, this column is loaded to approximately 60% of its capacity. This interaction diagram supports that, proving the RAM results to be correct.

It is important to note that there are slight moments about the minor axis, approximately 5-10% of the major moments. The moments about the minor axis were neglected for these hand calculations for simplicity. However, the final RAM designs consider both major and minor moments.

APPENDIX D – BEAM CALCULATIONS

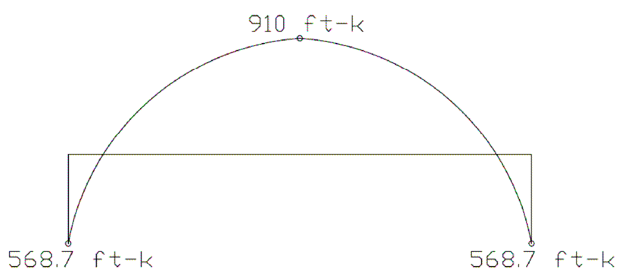
Beam Design:

TB1

w_D = 150 pcf*(10"/12)*29' = 3.6 k/ft *1.1 for self weight = 4.0 k/ft
w_L = 125 psf*29' = 3.6 k/ft
w_u = 1.2*4.0 + 1.6*3.6 = 10.6 k/ft

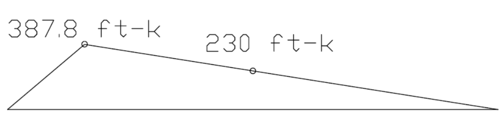
P_D = 70 k (from column above)
P_L = 10.5 k (from column above)
P_u = 1.2*70 + 1.6*10.5 = 100 k @ 4.6' from left end

Moment Diagram from Distributed Load



Note: This moment diagram is reflective of a beam supported with columns on each side forming a "medium restraint".
Negative moments = $wl^2/16$
Positive moment = $wl^2/10$

Moment Diagram from Point Load



Note: This moment diagram is reflective of a simply supported beam for simplicity due to the point load. It is conservative.

M_u = 910 + 230 = 1140 ft-k

Try a 24" by 32" concrete beam w/ (10) #9 bars (A_s = 10.0 in^2) and #4 stirrups
d = 32" – 1.5" – 4/8" – (9/8")/2 = 29.44"
a = (10*60)/(0.85*5*24) = 5.88"

ϕM_n = 0.9*10.0*60*(29.44" – 5.88"/2) = 14310 in-k = 1193 ft-k
ϕM_n = 1198 ft-k > M_u = 1140 ft-k, therefore OK

V_u = (155.3 – 10.6*29.44/12) + 84.30 = 213.6 k
$\phi V_c = 0.75*2*(5000)^{1/2}*24"*29.44"$ = 75.0 k
s = (0.75*0.40*60*29.44")/(213.6 – 75.0) = 4.1"
Use #4 stirrups @ 4"

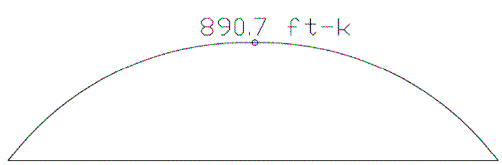
l/16 = 29.3*12/16 = 22 < h = 32" therefore deflections OK

TB2

w_D = 150 pcf*(10"/12)*29' = 3.6 k/ft *1.1 for self weight = 4.0 k/ft
w_L = 125 psf*29'/2 + 30 psf*24'/2' = 2.2 k/ft
w_u = 1.2*4.0 + 1.6*2.2 = 8.3 k/ft

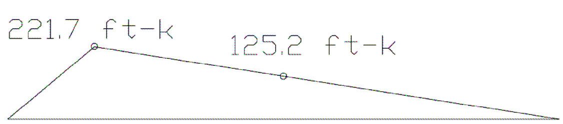
P_D = 42.3 k (from column above)
P_L = 4.0 k (from column above)
P_u = 1.2*42.3 + 1.6*4.0 = 57.2 @ 4.6' from left end

Moment Diagram from Distributed Load



Note: This moment diagram is reflective of a simply supported beam because it is only supported by another beam on one side, resulting in little restraint. It is conservative

Moment Diagram from Point Load



Note: This moment diagram is reflective of a simply supported beam for simplicity due to the point load. It is conservative.

M_u = 890.7 + 125.2 = 1015.9 ft-k

Use the same 24" by 32" concrete beam w/ (10) #9 bars (A_s = 10.0 in^2) and #4 stirrups as TB1

ϕM_n = 1193 ft-k > M_u = 1015.9 ft-k, therefore OK

V_u = (121.6 – 8.3*29.44/12) + 48.2 = 149.4 k
ϕV_c = 0.75*2*$(5000)^{1/2}$*24"*29.44" = 75.0 k

s = (0.75*0.40*60*29.44")/(149.4 – 75.0) = 7.1"
Use #4 stirrups @ 7"

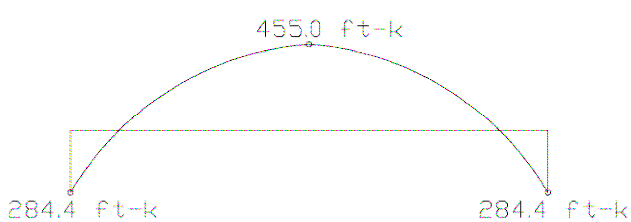
l/16 = 29.3*12/16 = 22 < h = 32" therefore deflections OK

TB3

w_D = 150 pcf*(10"/12)*29'/2 = 1.8 k/ft *1.1 for self weight = 2.0 k/ft
w_L = 125 psf*29'/2 = 1.8 k/ft
w_u = 1.2*2.0 + 1.6*1.8 = 5.3 k/ft

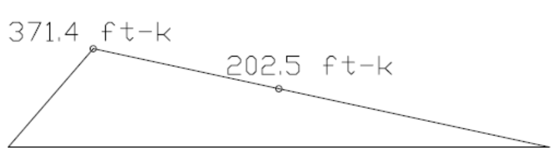
P_D = 42.3 k*(24.7'/29.3') + 3.6 k/ft*(29.3'/2) = 88.4 k (from beam)
P_L = 4.0 k*(24.7'/29.3') + 1.2 k.ft*(29.3'/2) = 35.2 k (from beam)
P_u = 1.2*88.4 + 1.6*35.2 = 100 k @ 4.6' from left end

Moment Diagram from Distributed Load



Note: This moment diagram is reflective of a beam supported with columns on each side forming a "medium restraint".
Negative moments = $wl^2/16$
Positive moment = $wl^2/10$

Moment Diagram from Point Load



Note: This moment diagram is reflective of a simply supported beam for simplicity due to the point load. It is conservative.

M_u = 455.0 + 202.5 = 657.5 ft-k

Try a 24" by 32" concrete beam w/ (6) #9 bars (A_s = 6.0 in^2) and #3 stirrups
d = 32" – 1.5" – 3/8" – (9/8")/2 = 29.56"
a = (6.0*60)/(0.85*5*24) = 3.53"

ϕM_n = 0.85*6.0*60*(29.56” – 3.53”/2) = 8505 in-k = 708.8 ft-k
ϕM_n = 708.8 ft-k > M_u = 657.5 ft-k, therefore OK

V_u = (77.6 – 5.3*29.56/12) + 84.3 = 148.2 k
ϕV_c = 0.75*2*$(5000)^{1/2}$*24”*29.56” = 75.3 k
s = (0.75*0.22*60*29.56”)/(148.2 – 75.3) = 4.0”
Use #3 stirrups @ 4”

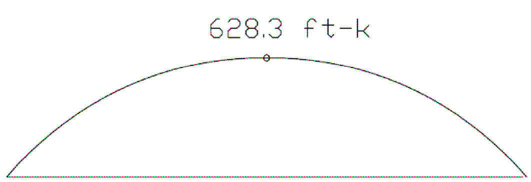
l/16 = 29.3*12/16 = 22 < h = 32” therefore deflections OK

TB4

w_D = 150 pcf*(10”/12)*41’/2 = 2.6 k/ft *1.1 for self weight = 2.9 k/ft
w_L = 30 psf*41’/2 = 0.6 k/ft
w_u = 1.2*2.9 + 1.6*0.6 = 4.4 k/ft

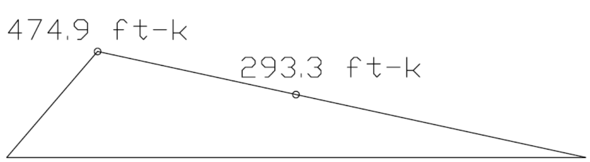
P_D = 60.8 k (from column above)
P_L = 11.6 k (from column above)
P_u = 1.2*60.8 + 1.6*11.6 = 91.5 k @ 6.4’ from left end

Moment Diagram from Distributed Load



Note: This moment diagram is reflective of a simply supported beam because it is only supported by another beam on one side, resulting in little restraint. It is conservative

Moment Diagram from Point Load



Note: This moment diagram is reflective of a simply supported beam for simplicity due to the point load. It is conservative.

M_u = 628.3 + 293.3 = 921.6 ft-k

Try a 24” by 32” concrete beam w/ (8) #9 bars (A_s = 8.0 in^2) and #3 stirrups
d = 32” – 1.5” – 3/8” – (9/8”)/2 = 29.56”
a = (8.0*60)/(0.85*5*24) = 4.71”

ϕM_n = 0.85*8.0*60*(29.56" – 4.71"/2) = 11099 in-k = 925 ft-k
ϕM_n = 925 ft-k > M_u = 921.6 ft-k, therefore OK

V_u = (74.4 – 4.4*29.56/12) + 74.2 = 137.8 k
ϕV_c = 0.75*2*$(5000)^{1/2}$*24"*29.56" = 75.3 k
s = (0.75*0.22*60*29.56")/(137.8 – 75.3) = 4.7"
Use #3 stirrups @ 4"

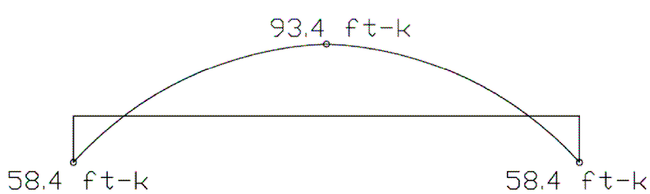
l/16 = 33.8*12/16 = 25 < h = 32" therefore deflections OK

TB5

w_D = 150 pcf*(10"/12)*2.5'/2 = 2.7 k/ft *1.1 for self weight = 3.0 k/ft
w_L = 30 psf*42.5'/2 = 0.6 k/ft
w_u = 1.2*3.0 + 1.6*0.6 = 4.6 k/ft

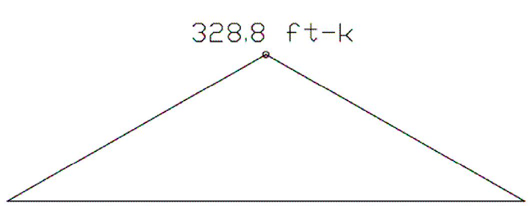
P_D = 60.8 k*(6.4'/33.8') + 2.9 k/ft*(33.8'/2) = 60.5 k (from beam)
P_L = 11.3 k*(6.4'/33.8') + 0.6 k.ft*(33.8'/2) = 12.3 k (from beam)
P_u = 1.2*60.5 + 1.6*12.3 = 92.3 k @ midspan

Moment Diagram from Distributed Load



Note: This moment diagram is reflective of a beam supported with columns on each side forming a "medium restraint".
Negative moments = $wl^2/16$
Positive moment = $wl^2/10$

Moment Diagram from Point Load



Note: This moment diagram is reflective of a simply supported beam for simplicity due to the point load. It is conservative.

M_u = 93.4 + 328.8 = 422.2 ft-k

Try a 24" by 32" concrete beam w/ (6) #7 bars (A_s = 3.60 in^2) and #3 stirrups

d = 32" – 1.5" – 3/8" – (7/8")/2 = 29.69"

a = (3.60*60)/(0.85*5*24) = 2.12"

ϕM_n = 0.85*3.60*60*(29.69" – 2.12"/2) = 5256 in-k = 438 ft-k

ϕM_n = 438 ft-k > M_u = 422.2 ft-k, therefore OK

$A_{s,\,min}$ = [3*(5000)$^{1/2}$/60000]*24"*29.69" = 2.52 in^2 < 3.60 in^2 , therefore OK

V_u = (32.8 – 4.4*29.56/12) + 46.2 = 68.2 k

ϕV_c = 0.75*2*(5000)$^{1/2}$*24"*29.56" = 75.3 k

$V_u < \phi V_c$ therefore no stirrups required, use #3 at maximum spacing if desired

l/16 = 14.25*12/16 = 11 < h = 32" therefore deflections OK

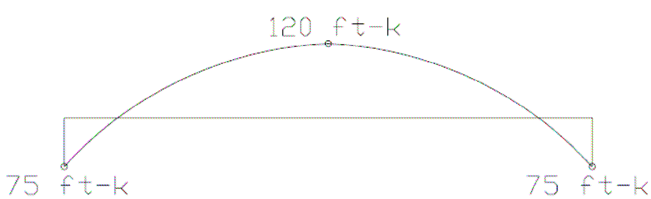
EB1

w_D = 150 pcf*(10"/12)*12'/2 + 40 psf*18'= 1.5 k/ft *1.1 for self weight = 1.7 k/ft

w_L = 100 psf*12'/2 = 0.6 k/ft

w_u = 1.2*1.7 + 1.6*0.6 = 3.0 k/ft

Moment Diagram from Distributed Load



Note: This moment diagram is reflective of a beam supported with columns on each side forming a "medium restraint".
Negative moments = wl^2/16
Positive moment = wl^2/10

Try a 24" by 16" concrete beam w/ (4) #7 bars (A_s = 2.40 in^2) and #3 stirrups

d = 16" – 1.5" – 3/8" – (7/8")/2 = 13.69"

a = (2.40*60)/(0.85*5*24) = 1.41"

ϕM_n = 0.85*2.40*60*(13.69" – 2.12"/2) = 1546 in-k = 129 ft-k

ϕM_n = 129 ft-k > M_u = 120 ft-k, therefore OK

$A_{s,\,min}$ = [3*(5000)$^{1/2}$/60000]*24"*13.69" = 1.16 in^2 < 2.40 in^2 , therefore OK

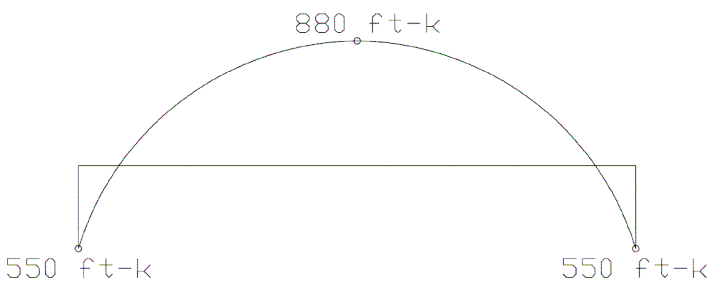
V_u = 30.0 – 3.0*13.69/12 = 26.6 k
ϕV_c = 0.75*2*$(5000)^{1/2}$*24"*13.69" = 34.8 k
$V_u < \phi V_c$ therefore no stirrups required, use #3 at maximum spacing if desired

l/16 = 20*12/16 = 15" < h = 16" therefore deflections OK

B1

w_D = 150 pcf*(8"/12)*(19.5'/2 + 9') = 1.9 k/ft *1.1 for self weight = 2.1 k/ft
w_L = 100 psf*(19.5'/2 + 9') = 1.9 k/ft
w_u = 1.2*2.1 + 1.6*1.9 = 5.5 k/ft

Moment Diagram from Distributed Load



Note: This moment diagram is reflective of a beam supported with columns on each side forming a "medium restraint".
Negative moments = $wl^2/16$
Positive moment = $wl^2/10$

M_u = 880 ft-k

Use the same 24" by 32" concrete beam w/ (8) #9 bars (A_s = 8.0 in^2) as TB4

ϕM_n = 925 ft-k > M_u = 880 ft-k, therefore OK

V_u = 110.0 – 5.5*29.56/12 = 96.5 k
ϕV_c = 0.75*2*$(5000)^{1/2}$*24"*29.56" = 75.3 k
s = (0.75*0.22*60*29.56")/(96.5 – 75.3) = 13.8"
Use #3 stirrups @ 12"

l/16 = 40*12/16 = 30 < h = 32" therefore deflections OK

APPENDIX E – CM CALCULATIONS

Existing Steel System: Cost Estimate

Steel Beams: 1st Floor

Size	# of Beams	Length (ft)	Weight (lbs)
W8x10	2	19	191
HSS8x2x3/16	1	19	212
W12x14	5	59.52	843
W8x15	2	23.33	352
W12x19	24	415.39	7873
W14x22	14	396.66	8760
W12x26	8	84.36	2196
W16x26	39	1118.12	29220
W10x39	1	18.05	706
W16x40	3	79	3160
W18x40	5	139.99	5621
W21x44	6	166.73	7376
W14x48	8	216.5	10392
W16x50	2	58	2900
W21x55	3	87	1608
W24x55	2	58	3190
W21x62	2	58	3596
W24x62	1	29	1806
W14x74	1	29.08	2152
W18x76	2	58.67	4459
W24x76	2	79	6022
W27x84	1	39	3291
SUM	134	3251	**105926**

Steel Beams: 2nd Floor

Size	# of Beams	Length (ft)	Weight (lbs)
W8x10	9	91.81	925
HSS8x2x3/16	1	19	212
W12x14	2	21.83	309
W8x15	2	20	302

W12x19	78	903.18	17118
W14x22	7	168.4	3719
W12x26	6	81.08	2111
W16x26	13	309.04	8076
W16x31	12	331.34	10294
W18x35	18	528.28	18515
W18x40	6	167.56	6728
W14x43	1	7.5	322
W21x44	11	313.67	13801
W18x46	2	59.83	2748
W21x48	3	90	4320
W18x50	1	30.5	1458
W14x53	2	38	2014
W24x55	2	68.17	3781
W24x62	1	29	1806
W18x65	2	58	3770
W16x67	3	86	5762
W14x68	3	87.08	5921
W21x68	2	58	3944
W21x73	2	58	4234
W18x76	3	83.67	6359
W24x84	1	39	3278
W27x94	2	79	7446
SUM	195	3827	**139274**

Steel Beams: 3rd Floor

Size	# of Beams	Length (ft)	Weight (lbs)
W8x10	4	43.15	435
HSS8x2x3/16	1	19	212
W10x12	1	25	301
W12x14	3	31.83	451
W8x15	3	30.5	461
W12x19	75	806.37	15284
W14x22	12	227.9	5033
W12x26	6	78.15	2034
W16x26	16	417.33	10906
W16x31	10	274.34	8523
W18x35	19	561.33	19674
W18x40	10	286.06	11486

W21x44	5	143.59	6352
W18x46	3	88.83	4081
W14x48	2	59.5	2856
W21x48	1	30.5	1463
W18x50	1	29.14	1458
W14x53	2	38	2014
W24x55	2	68.17	3781
W24x62	1	29	1806
W16x67	2	50	3350
W14x68	3	87.75	5967
W21x68	2	58	3944
W18x71	2	58	4118
W14x74	4	119	8806
W16x77	1	29	2233
W24x84	1	39	3278
W16x89	1	29	2581
W30x90	1	39.5	3548
W30x99	1	39.5	3911
SUM	195	3836	**140347**

Steel Beams: 4th Floor

Size	# of Beams	Length (ft)	Weight (lbs)
W8x10	4	43.15	435
HSS8x2x3/16	1	19	212
W10x12	1	25	301
W12x14	3	31.83	451
W8x15	3	30.5	461
W12x19	75	806.37	15284
W14x22	12	227.9	5033
W12x26	6	78.15	2034
W16x26	16	417.33	10906
W16x31	10	274.34	8523
W18x35	19	561.33	19674
W18x40	10	286.06	11486
W21x44	5	143.59	6352
W18x46	3	88.83	4081
W14x48	2	59.5	2856
W21x48	1	30.5	1463
W18x50	1	29.14	1458

W14x53	2	38	2014
W24x55	2	68.17	3781
W24x62	1	29	1806
W16x67	2	50	3350
W14x68	3	87.75	5967
W21x68	2	58	3944
W18x71	2	58	4118
W14x74	4	119	8806
W16x77	1	29	2233
W24x84	1	39	3278
W16x89	1	29	2581
W30x90	1	39.5	3548
W30x99	1	39.5	3911
SUM	195	3836	**140347**

Steel Beams: Main Roof

Size	# of Beams	Length (ft)	Weight (lbs)
W10x12	1	7.53	1226
W12x14	2	20	280
W8x15	1	7.53	820
W12x19	26	466.46	360
W14x22	16	422.15	2164
W12x26	7	90.43	1089
W16x26	32	894.07	845
W14x30	2	38	1140
W16x31	5	132.03	1119
W18x35	9	258.84	1438
W18x40	4	111.32	2153
W21x44	5	132.8	5329
W14x48	11	315.58	15148
W21x48	1	20	1003
W21x50	1	39.17	5220
W24x55	2	58	3190
W24x62	1	39.5	2449
W16x67	2	58	3886
W14x68	2	48.67	3310
W21x68	2	58	3944
W24x68	2	78.5	5338
SUM	134	3297	**61450**

Steel Beams: Penthouse Roof

Size	# of Beams	Length (ft)	Weight (lbs)
W8x10	5	50.85	512
W10x12	6	101.8	1226
W8x15	6	54.24	820
W12x19	1	19	360
W10x19	17	424.98	8127
W14x22	5	98	2164
W12x26	2	41.85	1089
W16x26	1	32.34	845
W14x26	2	53.61	1403
W12x30	2	50.05	1497
W14x34	1	32.87	1119
W18x35	2	41.02	1438
W16x36	9	302.21	10901
W18x40	2	53.61	2153
W18x46	4	116	5329
W12x50	2	20.05	1003
W14x90	2	58	5220
SUM	69	1550.48	**45206**

Steel Beam Weight Summary

Floor	Steel (lbs)
1st	105926
2nd	139274
3rd	140347
4th	140347
Main Roof	60464
PH Roof	45206
SUM	**631564**

Steel Columns

Size	Height (ft)	# of Cols	Total Length (ft)	Weight (lbs)
W14x109	18	49	882	96138
W14x109	16.5	13	214.5	23381

W14x159	18	14	252	40068
W14x159	16.5	4	66	10494
W14x193	18	6	108	20844
W14x132	18	20	360	47520
W14x132	16.5	5	82.5	10890
W14x145	18	13	234	33930
W14x145	16.5	5	82.5	11963
W12x65	18	7	126	8190
W12x53	18	21	378	20034
W12x53	16.5	9	148.5	7871
W12x45	18	4	72	3240
W12x40	18	2	36	1440
W12x40	16.5	1	16.5	660
W14x120	18	6	108	12960
W14x120	16.5	2	33	3960
W12x79	18	4	72	5688
W10x33	18	6	108	3564
	SUM	**191**	**3379.5**	**362834**

Structural Steel: Total Cost

	Steel (lbs)	**Steel (tons)**	**Material ($/ton)**	**Labor ($/ton)**	**Equip ($/ton)**	**Total $**
Beams	631564	315.8	$2,250	$375	$130	$869,980
Columns	362834	181.4	$2,250	$375	$130	$499,803
SUM	994398	497.2				**$1,369,783**

Footings

Length (ft)	**Width (ft)**	**Thick (ft)**	**# of Ftgs**	**CY of Conc**
10	10	2.83	6	62.97
9	9	2.50	12	90.01
8	8	2.33	11	60.84
11	11	2.33	3	31.37
8	12	2.67	1	9.48
7	6	1.67	3	7.78
10	7	1.67	1	4.32
			Total	266.77

Shear Studs

Floor	# Studs	Material ($/Stud)	Labor ($/Stud)	Equip ($/Stud)	Total $
1st	2601	$0.51	$0.74	$0.38	$4,240
2nd	2717	$0.51	$0.74	$0.38	$4,429
3rd	2901	$0.51	$0.74	$0.38	$4,729
4th	2901	$0.51	$0.74	$0.38	$4,729
Main Roof	2818	$0.51	$0.74	$0.38	$4,593
SUM	**13938**				**$22,719**

Composite Slab

Floor	Slab Area (SF)
1st	19175
2nd	19175
3rd	19175
4th	19175
Roof/PH	19175
SUM	**95875**

Slab Component	Material ($/SF)	Labor ($/SF)	Equip ($/SF)	Total $
Metal deck	2.15	0.41	0.04	$249,275
LW Concrete [1]	2.02	1.24	0.48	$358,573
WWF [2]	15.65	22	-	$36,097
SUM	19.82	23.65	0.52	**$643,944**

Notes: (1) LW concrete $/SF adjusted from 2.5" slab thickness to 4.25" slab thickness (average concrete thickness for 3 ¼" LW concrete on 2" metal deck)
(2) WWF cost is per 100 SF

Roof Deck

Roof Deck SF	Material ($/SF)	Labor ($/SF)	Equip ($/SF)	Total $
5875	1.36	0.32	0.03	**$10,046**

Fireproofing

Floor	Slab Area (SF)	Fireproofing $
1st	19175	$38,350
2nd	19175	$38,350
3rd	19175	$38,350
4th	19175	$38,350
Roof/PH	19175	$38,350
PH Roof	5875	$11,750
SUM	101750	**$203,500**

Moment Connections

Floor	# of Connections	Cost
1st	38	$11,400
2nd	44	$13,200
3rd	44	$13,200
4th	44	$13,200
Main Roof/PH	44	$13,200
PH Roof	8	$2,400
SUM	222	**$66,600**

Proposed Concrete System: Cost Estimate

Columns

Base (ft)	Width (ft)	Height (ft)	# of Col's	CY of Conc
2	2	16.67	183	451.85
2	2	15.17	42	94.37
			Total	**546.22**

Footings

Length (ft)	Width (ft)	Thick (ft)	# of Ftgs	CY of Conc
7	7	1.50	2	5.44
8	8	2.00	3	14.22
9	9	2.00	6	36.00
10	10	2.00	2	14.81
11	11	2.50	9	100.83
12	12	2.50	7	93.33
13	13	3.00	5	93.89
15	15	3.50	3	87.50
			Total	**440.59**

Slabs

Floor	Slab Thickness	Slab Area	Drop Thickness	Drop Area	CY of Conc
1st	0.83	19175	0.50	2244	633.38
2nd	0.83	19175	0.50	2244	591.82
3rd	0.83	19175	0.50	2244	591.82
4th	0.83	19175	0.50	2244	591.82
Roof/PH	0.83	19175	0.50	2244	591.82
PH Roof	0.83	5875	0.50	748	181.33
				Total	**3181.99**

Beams

Beam #	Width (in)	Depth (in)	Length (ft)	# of Beams	CY of Conc
TB1	24	32	29.3	3	17.36
TB2	24	32	29.3	2	11.58
TB3	24	32	29.3	2	11.58
TB4	24	32	33.75	1	6.67
TB5	24	32	14.25	1	2.81
EB1	24	16	20	16	31.60
B1	24	32	40	5	39.51
				Total	**121.11**

Concrete Summary

	CY of Conc	Material ($/CY)	Labor ($/CY)	Equip ($/CY)	Total $
Columns	546.22	231.00	325.00	32.00	$321,179
Footings	440.59	176.00	54.50	0.33	$101,702
Slabs	3181.99	275.00	156.00	14.75	$1,418,373
Beams	121.11	325.00	415.00	40.50	$94,523
				SUM	**$1,935,777**

RS Means Tables:

Structural Steel Costs

05 12 23.77 Structural Steel Projects										
0010	**STRUCTURAL STEEL PROJECTS**	R050516-30								
0020	Shop fab'd for 100-ton, 1-2 story project, bolted connections									
0200	Apartments, nursing homes, etc., 1 to 2 stories	R050523-10	E-5	10.30	7.767	Ton	2,200	330	165	2,695
0300	3 to 6 stories		"	10.10	7.921		2,250	335	168	2,753
0400	7 to 15 stories	R051223-10	E-6	14.20	9.014		2,300	380	132	2,812
0500	Over 15 stories		"	13.90	9.209		2,375	390	134	2,899
0700	Offices, hospitals, etc., steel bearing, 1 to 2 stories – CN	R051223-20	E-5	10.30	7.767		2,200	330	165	2,695
0800	3 to 6 stories		E-6	14.40	8.889		2,250	375	130	2,755
0900	7 to 15 stories	R051223-25		14.20	9.014		2,300	380	132	2,812
1000	Over 15 stories			13.90	9.209		2,375	390	134	2,899
1100	For multi-story masonry wall bearing construction, add	R051223-30						30%		

Cast in Place Concrete Costs

03 30 53.40 Concrete In Place		Crew	Daily Output	Labor Hours	Unit	Material	2008 Bare Costs Labor	Equipment	Total
1900	Elevated slabs, flat slab with drops, 125 psf Sup. Load, 20' span	C-14B	38.45	5.410		263	207	19.60	489.60
1950	30' span		50.99	4.079		275	156	14.75	445.75
2100	Flat plate, 125 psf Sup. Load, 15' span		30.24	6.878		242	264	25	531
2150	25' span		49.60	4.194		249	161	15.20	425.20
2300	Waffle const., 30" domes, 125 psf Sup. Load, 20' span		37.07	5.611		375	215	20.50	610.50
2350	30' span		44.07	4.720		335	181	17.10	533.10
2500	One way joists, 30" pans, 125 psf Sup. Load, 15' span		27.38	7.597		450	291	27.50	768.50
2550	25' span		31.15	6.677		410	256	24	690
2700	One way beam & slab, 125 psf Sup. Load, 15' span		20.59	10.102		264	385	36.50	685.50
2750	25' span		28.36	7.334		246	281	26.50	553.50
2900	Two way beam & slab, 125 psf Sup. Load, 15' span		24.04	8.652		253	330	31.50	614.50
2950	25' span		35.87	5.799		216	222	21	459
3100	Elevated slabs including finish, not								
3110	including forms or reinforcing								
3150	Regular concrete, 4" slab	C-8	2613	.021	S.F.	1.36	.73	.28	2.37
3200	6" slab		2585	.022		2.02	.73	.28	3.03
3250	2-1/2" thick floor fill		2685	.021		.87	.71	.27	1.85
3300	Lightweight, 110# per C.F., 2-1/2" thick floor fill		2585	.022		1.19	.73	.28	2.20
3400	Cellular concrete, 1-5/8" fill, under 5000 S.F.		2000	.028		.79	.95	.36	2.10
3450	Over 10,000 S.F.		2200	.025		.76	.86	.33	1.95
3800	Footings, spread under 1 C.Y.	C-14C	28	4	C.Y.	173	146	.88	319.88
3825	1 C.Y. to 5 C.Y.		43	2.605		192	95.50	.57	288.07
3850	Over 5 C.Y.		75	1.493		176	54.50	.33	230.83

03 30 53.40 Concrete In Place

0010	**CONCRETE IN PLACE**									
0020	Including forms (4 uses), reinforcing steel, concrete, placement,									
0050	and finishing unless otherwise indicated	R033053-50								
0300	Beams, 5 kip per L.F., 10' span		C-14A	15.62	12.804	C.Y.	315	490	48.50	853.50
0350	25' span		"	18.55	10.782		325	415	40.50	780.50
0500	Chimney foundations, industrial, minimum		C-14C	32.22	3.476		145	127	.76	272.76
0510	Maximum		"	23.71	4.724		172	173	1.04	346.04
0700	Columns, square, 12" x 12", minimum reinforcing		C-14A	11.96	16.722		335	640	63	1,038
0720	Average reinforcing	R033105-85		10.13	19.743		530	755	74.50	1,359.50
0740	Maximum reinforcing			9.03	22.148		795	850	83.50	1,728.50
0800	16" x 16", minimum reinforcing			16.22	12.330		269	475	46.50	790.50
0820	Average reinforcing			12.57	15.911		455	610	60	1,125
0840	Maximum reinforcing			10.25	19.512		700	750	73.50	1,523.50
0900	24" x 24", minimum reinforcing			23.66	8.453		231	325	32	588
0920	Average reinforcing			[illegible]	11.293		[illegible]	[illegible]	42.50	[illegible]
0940	Maximum reinforcing			14.15	14.134		645	540	53.50	1,238.50
1000	36" x 36", minimum reinforcing			33.69	5.936		205	228	22.50	455.50
1020	Average reinforcing			23.32	4.576		360	330	32.50	722.50
1040	Maximum reinforcing			17.82	11.223		605	430	42.50	1,077.50

Shear Stud Costs

05 05 23.85 Weld Shear Connectors

0010	**WELD SHEAR CONNECTORS**								
0020	3/4" diameter, 3-3/16" long	E-10	960	.017	Ea.	.46	.73	.37	1.56
0030	3-3/8" long		950	.017		.48	.74	.37	1.59
0200	3-7/8" long		945	.017		.51	.74	.38	1.63
0300	4-3/16" long		935	.017		.54	.75	.38	1.67
0500	4-7/8" long		930	.017		.60	.76	.38	1.74
0600	5-3/16" long		920	.017		.62	.77	.39	1.78
0800	5-3/8" long		910	.018		.63	.77	.39	1.79
0900	6-3/16" long		905	.018		.69	.78	.39	1.86

WWF Costs

0010	**WELDED WIRE FABRIC** ASTM A185	R032205-30								
0050	Sheets									
0100	6 x 6 - W1.4 x W1.4 (10 x 10) 21 lb. per C.S.F.	***CN***	2 Rodm	35	.457	C.S.F.	13.25	19.65		32.90
0200	6 x 6 - W2.1 x W2.1 (8 x 8) 30 lb. per C.S.F.			31	.516		15.65	22		37.65
0300	6 x 6 - W2.9 x W2.9 (6 x 6) 42 lb. per C.S.F.			29	.552		20	23.50		43.50
0400	6 x 6 - W4 x W4 (4 x 4) 58 lb. per C.S.F.			27	.593		29	25.50		54.50

Metal Deck Costs

05 31 13.50 Floor Decking

0010	**FLOOR DECKING**	R053100-10								
5200	Non-cellular composite deck, galv., 2" deep, 22 gauge		E-4	3860	.008		1.53	.36	.03	1.92
5300	20 gauge			3600	.009		1.69	.39	.04	2.12
5400	18 gauge			3380	.009		2.15	.41	.04	2.60
5500	16 gauge			3200	.010		2.69	.44	.04	3.17
5700	3" deep, galv., 22 gauge			3200	.010		1.67	.44	.04	2.15
5800	20 gauge			3000	.011		1.86	.46	.04	2.36
5900	18 gauge	CN		2850	.011		2.29	.49	.05	2.83
6000	16 gauge			2700	.012		3.06	.52	.05	3.63
[illegible]	**ROOF DECKING**									
2100	Open type, galv., 1-1/2" deep wide rib, 22 gauge, under 50 squares		E-4	4500	.007	S.F.	1.61	.31	.0[illegible]	[illegible]
2200	50-500 squares	CN		4900	.007		1.25	.28	[illegible]	[illegible]
2400	Over 500 squares			5100	.006		1.16	.27	.0[illegible]	1.46
2600	20 gauge, under 50 squares			3865	.00[illegible]		1.8[illegible]	.3[illegible]	.0[illegible]	[illegible]
[illegible]	50-500 squares			4170	.00[illegible]		1.51	.3[illegible]	[illegible]	[illegible]
2700	Over 500 squares			4300	.007		1.36	.32	.0[illegible]	[illegible]
2900	18 gauge, under 50 squares			3800	.008		2.45	.37	.03	2.85
2950	50-500 squares			4100	.008		1.96	.34	.03	2.33
3000	Over 500 squares			4300	.007		1.76	.32	.03	2.11
3050	16 gauge, under 50 squares			3700	.009		3.30	.38	.04	3.72
3060	50-500 squares			4000	.008		2.64	.35	.03	3.02
3100	Over 500 squares			4200	.008		2.37	.33	.03	2.73

Schedule:

ID	Task Name	Duration	Start	Finish
1	F/R/P Columns Basement, 1	2 days	Fri 12/21/07	Mon 12/24/07
2	F/R/P Columns Basement, 2	2 days	Mon 12/24/07	Tue 12/25/07
3	F/R/P Columns Basement, 3	2 days	Tue 12/25/07	Wed 12/26/07
4	F/R/P Flat Slab 1st Floor, 1	3 days	Thu 12/27/07	Mon 12/31/07
5	Strip Flat Slab 1st Floor, 1	1 day	Mon 1/7/08	Mon 1/7/08
6	F/R/P Flat Slab 1st Floor, 2	3 days	Mon 12/31/07	Wed 1/2/08
7	Strip Flat Slab 1st Floor, 2	1 day	Wed 1/9/08	Wed 1/9/08
8	F/R/P Flat Slab 1st Floor, 3	3 days	Wed 1/2/08	Fri 1/4/08
9	Strip Flat Slab 1st Floor, 3	1 day	Fri 1/11/08	Fri 1/11/08
10	F/R/P Columns 1st Floor, 1	2 days	Tue 1/8/08	Wed 1/9/08
11	F/R/P Columns 1st Floor, 2	2 days	Thu 1/10/08	Fri 1/11/08
12	F/R/P Columns 1st Floor, 3	2 days	Mon 1/14/08	Tue 1/15/08
13	F/R/P Flat Slab 2nd Floor, 1	3 days	Wed 1/16/08	Fri 1/18/08
14	Strip Flat Slab 2nd Floor, 1	1 day	Fri 1/25/08	Fri 1/25/08
15	F/R/P Flat Slab 2nd Floor, 2	3 days	Fri 1/18/08	Tue 1/22/08
16	Strip Flat Slab 2nd Floor, 2	1 day	Tue 1/29/08	Tue 1/29/08
17	F/R/P Flat Slab 2nd Floor, 3	3 days	Tue 1/22/08	Thu 1/24/08
18	Strip Flat Slab 2nd Floor, 3	1 day	Thu 1/31/08	Thu 1/31/08
19	F/R/P Columns 2nd Floor, 1	2 days	Mon 1/28/08	Tue 1/29/08
20	F/R/P Columns 2nd Floor, 2	2 days	Wed 1/30/08	Thu 1/31/08
21	F/R/P Columns 2nd Floor, 3	2 days	Fri 2/1/08	Mon 2/4/08
22	F/R/P Flat Slab 3rd Floor, 1	3 days	Tue 2/5/08	Thu 2/7/08
23	Strip Flat Slab 3rd Floor, 1	1 day	Thu 2/14/08	Thu 2/14/08
24	F/R/P Flat Slab 3rd Floor, 2	3 days	Thu 2/7/08	Mon 2/11/08
25	Strip Flat Slab 3rd Floor, 2	1 day	Mon 2/18/08	Mon 2/18/08
26	F/R/P Flat Slab 3rd Floor, 3	3 days	Mon 2/11/08	Wed 2/13/08
27	Strip Flat Slab 3rd Floor, 3	1 day	Wed 2/20/08	Wed 2/20/08
28	F/R/P Columns 3rd Floor, 1	2 days	Fri 2/15/08	Mon 2/18/08
29	F/R/P Columns 3rd Floor, 2	2 days	Tue 2/19/08	Wed 2/20/08
30	F/R/P Columns 3rd Floor, 3	2 days	Thu 2/21/08	Fri 2/22/08
31	F/R/P Flat Slab 4th Floor, 1	3 days	Mon 2/25/08	Wed 2/27/08
32	Strip Flat Slab 4th Floor, 1	1 day	Wed 3/5/08	Wed 3/5/08
33	F/R/P Flat Slab 4th Floor, 2	3 days	Wed 2/27/08	Fri 2/29/08
34	Strip Flat Slab 4th Floor, 2	1 day	Fri 3/7/08	Fri 3/7/08
35	F/R/P Flat Slab 4th Floor, 3	3 days	Fri 2/29/08	Tue 3/4/08
36	Strip Flat Slab 4th Floor, 3	1 day	Tue 3/11/08	Tue 3/11/08
37	F/R/P Columns 4th Floor, 1	2 days	Thu 3/6/08	Fri 3/7/08
38	F/R/P Columns 4th Floor, 2	2 days	Mon 3/10/08	Tue 3/11/08
39	F/R/P Columns 4th Floor, 3	2 days	Wed 3/12/08	Thu 3/13/08
40	F/R/P Flat Slab Main Roof, 1	3 days	Fri 3/14/08	Tue 3/18/08
41	Strip Flat Slab Main Roof, 1	1 day	Tue 3/25/08	Tue 3/25/08
42	F/R/P Flat Slab Main Roof, 2	3 days	Tue 3/18/08	Thu 3/20/08
43	Strip Flat Slab Main Roof, 2	1 day	Fri 3/28/08	Fri 3/28/08
44	F/R/P Flat Slab Main Roof, 3	3 days	Thu 3/20/08	Mon 3/24/08
45	Strip Flat Slab Main Roof, 3	1 day	Mon 3/31/08	Mon 3/31/08
46	F/R/P Columns Main Roof	2 days	Wed 3/26/08	Thu 3/27/08
47	F/R/P Flat Slab PH Roof	3 days	Mon 3/31/08	Wed 4/2/08
48	Strip Flat Slab PH Roof	1 day	Wed 4/9/08	Wed 4/9/08