The Brendan Iribe Center for Computer Science and Innovation

College Park, MD

Brendan Barrett

Structural Option

Advisor: Dr. Said

FINAL REPORT

April 3, 2017

Brendan Iribe CCSI

College Park, MD

Brendan Barrett **Structural** **Dr. Said**

Project Team

Owner: University of Maryland

Architect: HDR Architecture

General Contractor: Whiting-Turner

Structural Engineer: Hope Furrer Associates

MEP Engineer: Setty Associates

Building Information

Location: College Park, MD

Delivery Method: Design-Bid-Build

Construction Dates: 2016-2018

Project Cost: Cost is being withheld

Project Size: 215,600 Square Feet

Construction

Construction will begin in 2016 and conclude in 2018

Site currently consist of a parking lot and green space

Parking lot will be demolished and the flood-plain will be restored to a more natural condition

Structural

Concrete spread footings and foundation walls

8" thick slab on grade reinforced with 4 # 12 each way bottom and 6x6 w-2.9 x 2.9 WWR top

3 1/4" LW concrete on 3" 20 gage composite deck

Lateral system consists of moment frames and braced frames

Mechanical

Six air handling units ranging in size from 18,000-30,000 CFM located in penthouse

Two 530 ton chillers located in the basement

High Pressure steam, 125 PSI, generated at UMD College Park's central campus power/boiler plant

Electrical

Electrical distribution system includes 480Y/277 V 4000 A main switchgear located on first floor

Transformers located on the first floor provide 120/208 V for receptacles

480Y/277 V indoor natural gas generator located on first floor

http://bbarrett0825.wixsite.com/thesis

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Executive Summary

The Brendan Iribe Center for Computer Science and Innovation is a 215,600 square foot research building at the University of Maryland in College Park, Maryland. As one of the world's top computer science institutions, this building will provide state of the art virtual reality research labs, conference rooms, offices, and classrooms to the students and faculty. In addition, a 300 seat auditorium will help the University of Maryland showcase the latest innovations in the virtual reality industry.

The existing gravity system consists of a composite steel system with wide flange girders and columns, while the existing lateral system consists of ordinary moment frames and ordinary braced frames. The technical reports last semester have determined that the existing system meets all required code. The purpose of this report is to propose a new structural system and determine if it would be a feasible alternative design.

This structural redesign aims to reduce the depth of the structure while maintaining an open spacious floor plan, and reduce the overall cost of the building. After analyzing several different systems, a voided concrete slab with reinforced concrete shear walls was selected as the proposed system. A voided concrete slab reduces the depth of the structure while providing the ability to reach long spans due to the reduced self-weight. Slabs, columns, and shear walls were designed using hand calculations and RAM Structural System.

Due to changing the material from steel to concrete, the cost of the structure will change as well. In addition, the reduction in building height will reduce façade, ductwork, and piping costs. The construction management breadth compares the cost estimate of the existing steel system with the proposed concrete system, and determines the cost of the structure reduces by roughly 30%. With the change in material, the acoustical performance is also effected. The mechanical breadth calculates the Sound Transmission Class (STC) rating of the voided slab system and determines it improves over the existing composite steel system.

The report determines that a voided concrete slab with shear walls is a viable alternative system. Both preliminary goals of a reduced structural depth and reduced building cost have been met; however several drawbacks include a longer construction schedule and larger structural weight which would result in larger foundations. As both systems are acceptable and have their advantages and disadvantages, the decision whether to use the existing or proposed system would be up to the digression of the owner.

Acknowledgements

I would like to thank the following people for their help and support for the completion of my senior thesis:

- The engineers at Hope Furrer Associates, especially Hope Furrer, Tom Barabas, and Nicole Baer for providing me with an excellent building and offering assistance to help

- The AE Faculty, especially my advisor Dr. Aly Said, for providing a wealth of knowledge and teaching us the tools necessary to enter the industry

- My fellow AE friends for some much needed fun times

- My parents for their constant support throughout my college career

1.Introduction

1.1 Building Overview

Figure 1: Rendering from north-east

As one of the world's to computer science institutions, the University of Maryland continues to grow. There is no longer enough room in the existing facilities to keep up with the latest advancements in virtual reality. The Brendan Iribe Center for Computer Science and Innovation (ICCSI) will increase the number of classrooms available and help sustain the University of Maryland as the leader in virtual reality research.

The 7 story building will reach a height of 118'-8" and is scheduled to be complete in 2018. Students and faculty will be provided with a magnificent six story building that will house eight collaborative classrooms, thirteen research labs, five conference rooms, offices, tutoring centers, a café, as well as many common areas. These labs will support groundbreaking research in many virtual reality sectors such as artificial intelligence, robotics, cybersecurity, computational biology, and quantum computing. Adjacent to the boomerang shaped main tower will be the 300-seat Antonov Auditorium pictured below which will help the university showcase the latest advancements in the field of virtual reality.

With a main design goal of maximizing collaboration amongst classmates, the curtain wall façade will allow natural lighting to illuminate the buildings open floor plans and common spaces. Many students that are technologically advanced come up with innovative ideas outside of lectures, and the open floor plans and common spaces will provide students the opportunity to share these ideas.

1.2 Foundations

The foundation for this project consists of mat foundations and shallow spread footings. The bottoms of all exterior footings are 4' below finished grade to reach frost depth, and a minimum net allowable bearing capacity of 5000 PSF has been used for design. Due to the partial basement being located within 500 year flood plain, the walls and slab on grade are designed for hydrostatic pressure. As a result, a 48" thick mat slab is located 3' below the top of the finished basement floor. Continuous wall footings are 3' wide x 1'-6" deep and reinforced with 3 # 5 bars.

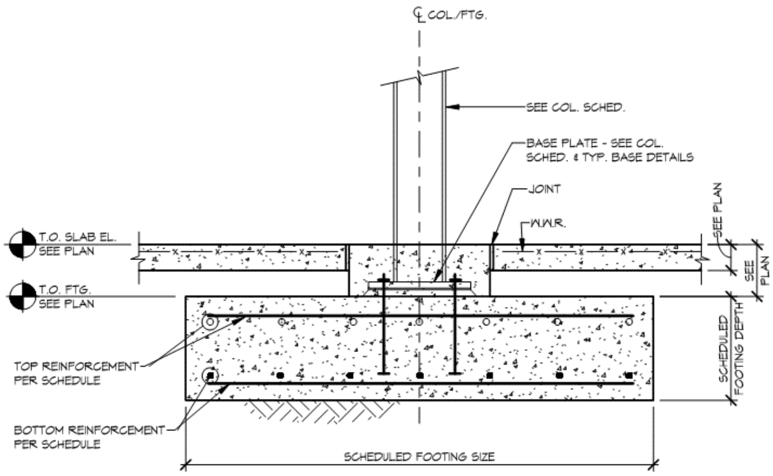


Figure 2: Typical interior column footing without pier

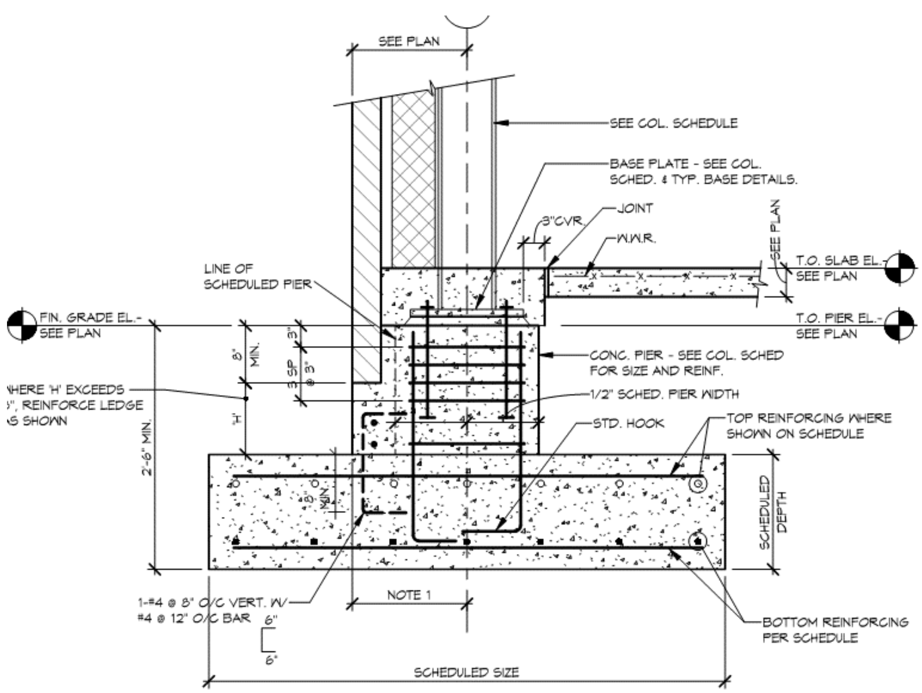


Figure 3: Typical column foundation at exterior wall

1.3 Gravity System

1.3.1 Typical Bay

As previously stated, the boomerang shaped building results in varied bay sizes along the building. At the far east and west ends, infill beams only span about 20'. However, at the center of the building where the north-south distance of the building is at its greatest, infill beams span up to 42'. *Figure 4* shows a bay at the east end of the building. Typical girders are 29' W 21X50 with 30 studs, while infill beams are W21's with 30 studs ranging from 16' to 22'. Infill beams are spaced about 10' o.c.

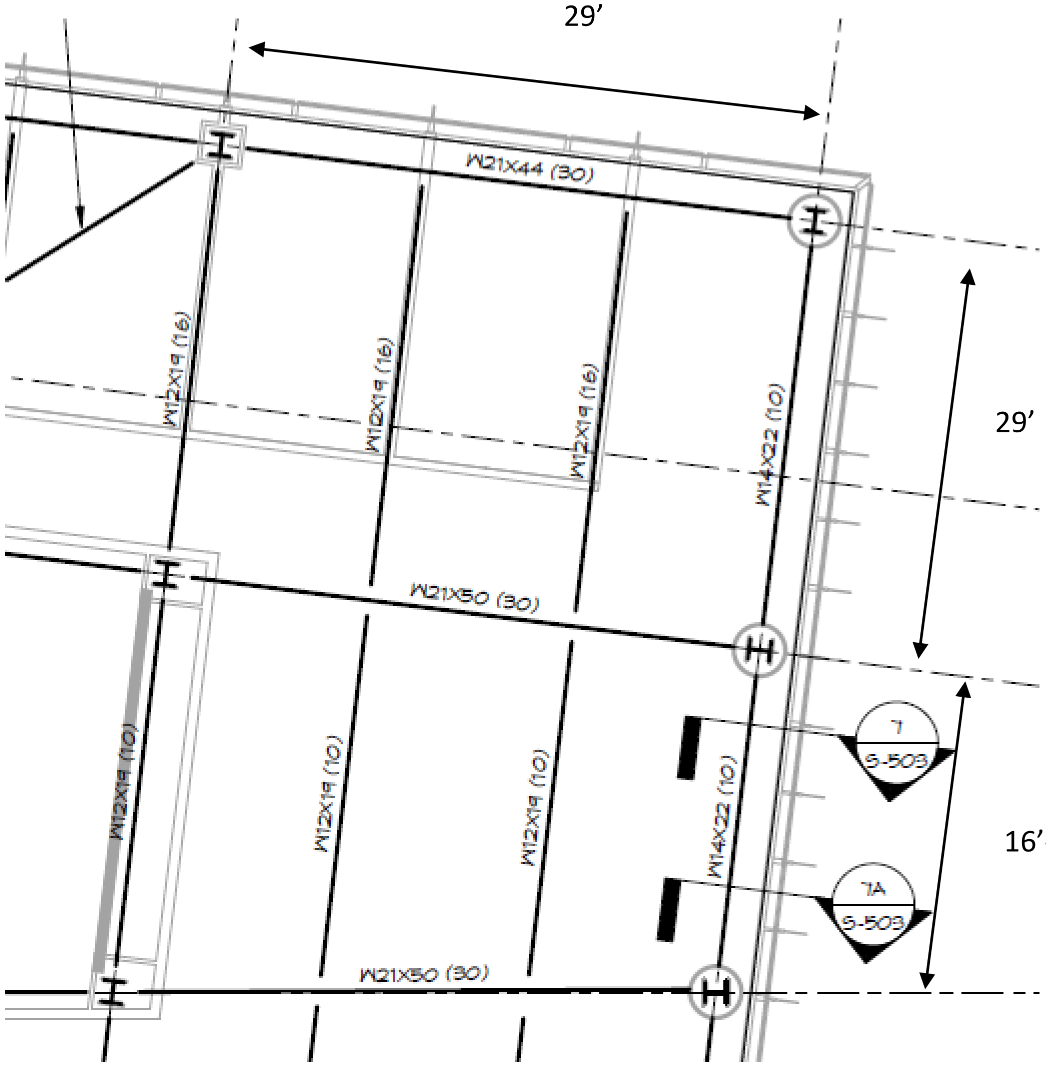


Figure 4: Bay in eastern wing

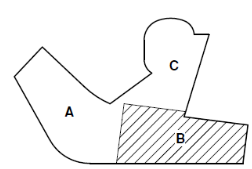
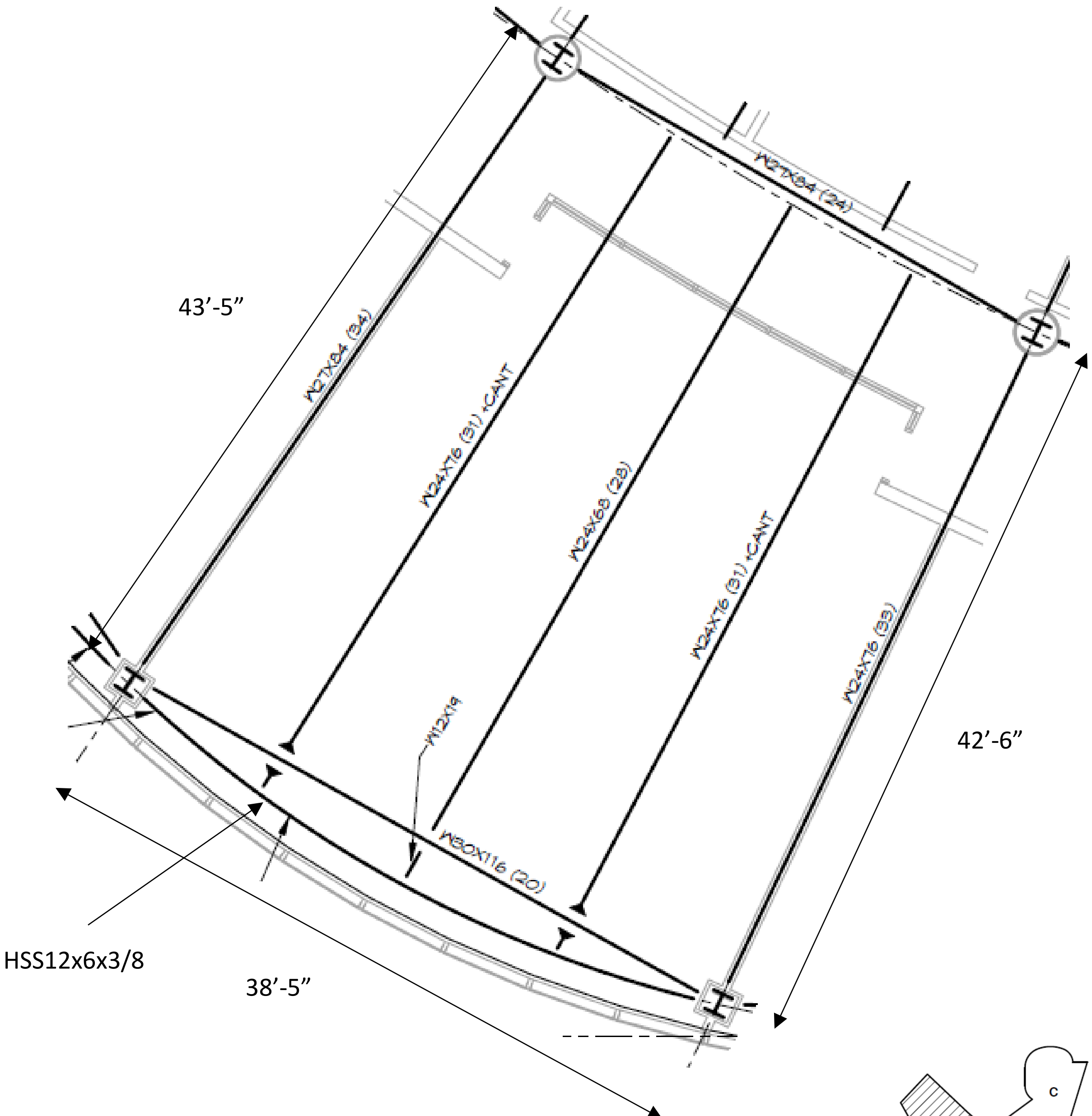


Figure 5 shows a bay close to the center of the building and western stairwell. At this bay, the girder along the curved wall is a W30x116 with 20 studs while the infill beams are W24's reaching spans up to 44'. Infill beams are spaced about 9' o.c. Due to the curve in the building, there is a curved HSS12x6x3/8 to match the radius of the grid arc.



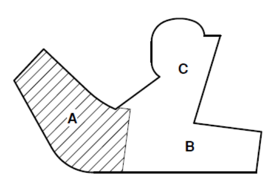


Figure 5: Bay in western wing

Framing for the Antonov Auditorium includes wide flange girders. *Figure 6* shows a bay at the north east corner of the auditorium. Girders are W24s and reach spans up to 32’ spaced at 10’. A 90’ truss supports the first floor and the roof in the north-south direction of the auditorium.

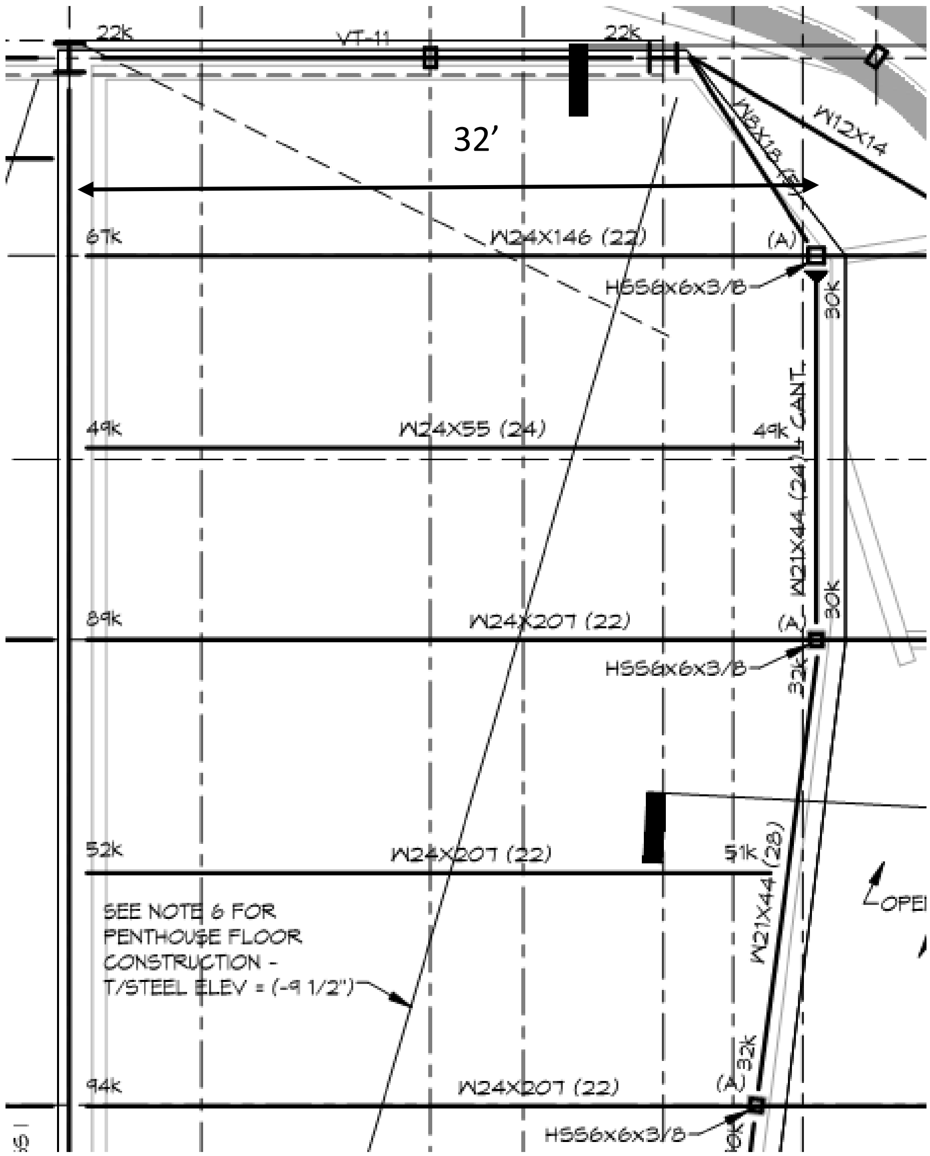
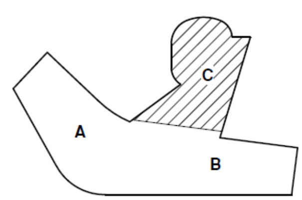


Figure 6: Bay in auditorium



1.3.2 Floor

The floor consists of 3 ¼” lightweight concrete on 3”x 20 gage galvanized metal deck (6 ¼” total thickness) reinforced with 6x6- W2.0 W.W.R. At the penthouse level, the slab is 4 ½” normal weight concrete on 3” x 18 gage galvanized metal deck (7 ½” total thickness) reinforced with 6x6- W2.9xW2.9 W.W.R. The increased thickness will provide additional dampening of the mechanical units to the floors below. Finally the roof level consists of 1 ½” x 20 gage Type B galvanized metal roof deck on steel filler beams and girders.

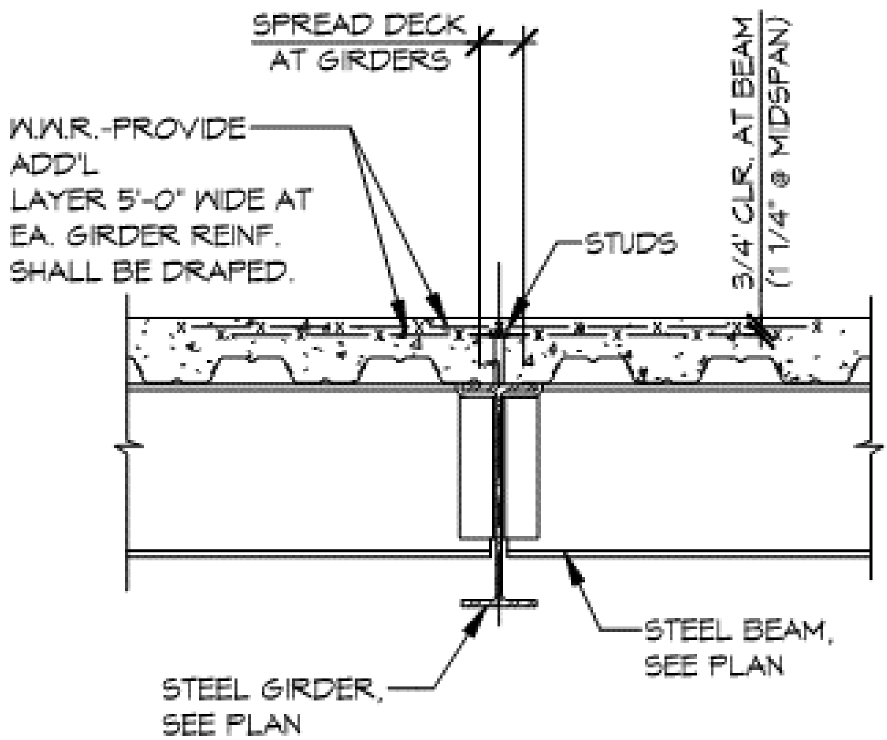


Figure 7: Typical composite floor construction

1.3.3 Columns

All columns in the Brendan Iribe CCSI are W12s or W14s spliced every two stories, usually 1'-6" above the finished floor slab. Splices can be welded or bolted as shown below. *Figure 8* shows the welded detail while *Figure 9* shows the bolted detail. Some columns can reach sizes up to W14x370 due to the high axial loads acting on it.

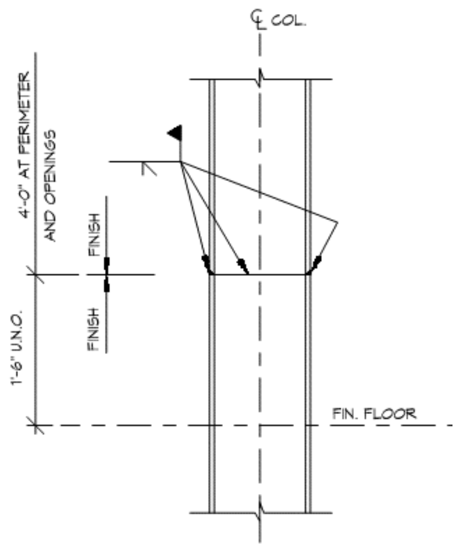


Figure 8: Typical welded detail

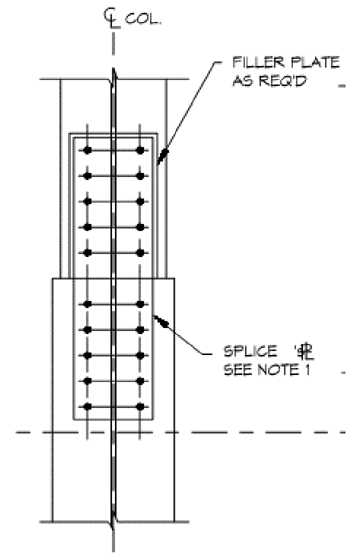


Figure 9: Typical bolted detail

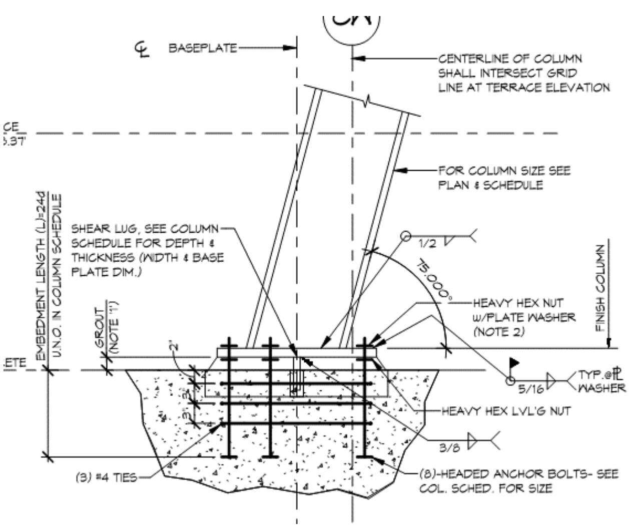


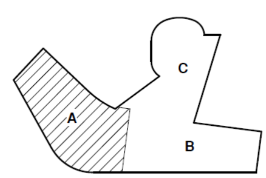
Figure 10: Sloped column foundation

The sloping columns located at the eastern cantilever require significantly larger sizes. As the sloping turns the column into a beam-column, a W14x730 must be used for two of these columns. This large size results in a 48" x 48" x 5" base plate which weighs over 3000 pounds. *Figure 10* shows a detail of the sloped column foundation.

1.4 Lateral System

The lateral force resisting system of the main tower consists of moment frames and braced frames located in the eastern and western wings of the building. The next two figures show the configuration on the structural plan where red designates moment frames and green designates vertical trusses. Girders and moment frames are W24's or W27's and range from 8' to 24' spans.

Figure 11: Lateral system in western wing



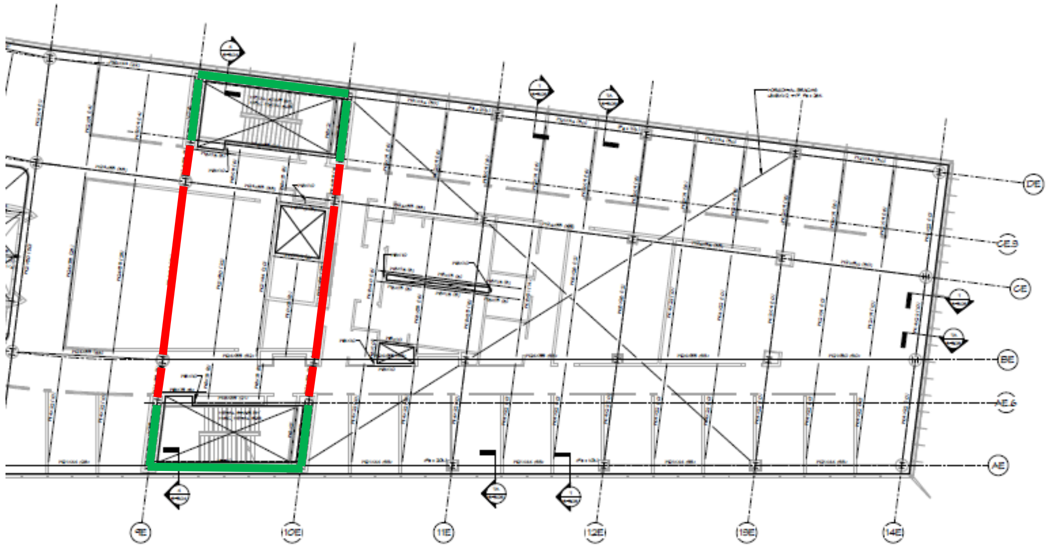


Figure 12: Lateral system in eastern

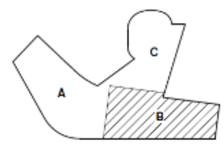
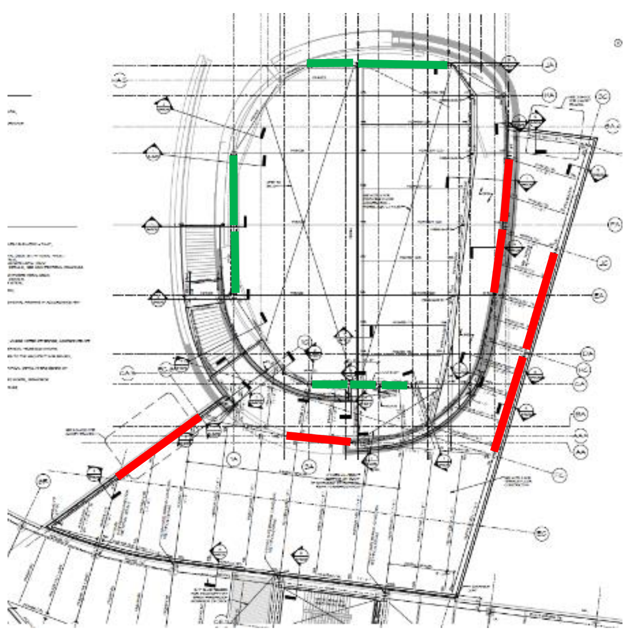


Figure 13 below shows the lateral system in the auditorium consisting of moment frames and vertical trusses. Due to the open floor plan, moment frames and vertical trusses are located along the perimeter of the auditorium.



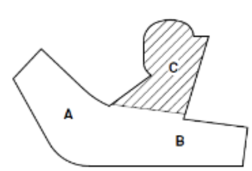


Figure 13: Lateral system in auditorium

There are thirteen separate braced frame configurations located throughout the building including diagonal, double diagonal, and chevron bracing (k-brace). The vertical trusses use W10x112, W12x120 and HSS 20x12x1/2 for the bracing members. *Figure 14* shows the elevation for Vertical Truss 1 which is located adjacent to the stairwell in the buildings western wing.

1.

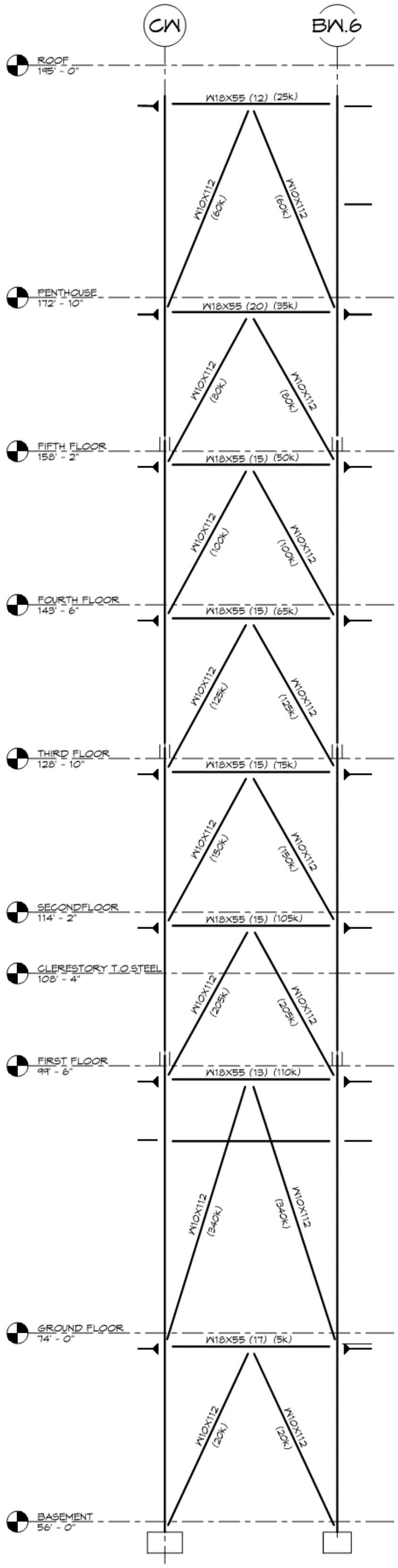


Figure 14: Typical braced frame elevation

1.5.1 Secondary Elements

Two architectural features on the Antonov Auditorium include canopies located beyond the southwest corner of the auditorium and at the northeast corner. The canopy consists of L2x2x1/4 kickers bolted to W12x19s with ¼" full depth stiffener plates at each side of the web and kicker. *Figure 15* below shows a detail of the northeast canopy.

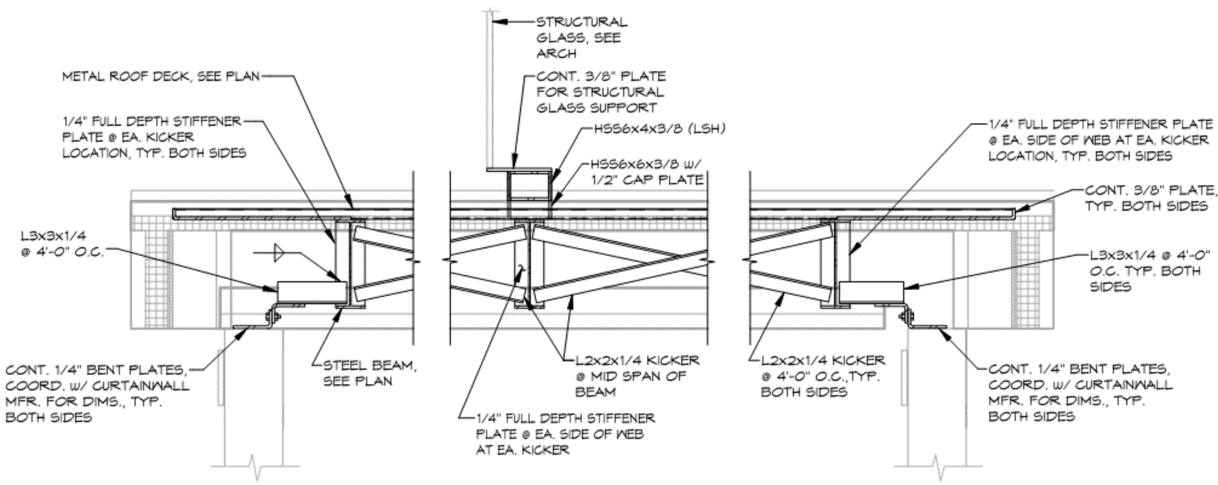


Figure 15: Northeast canopy detail

1.5.2 Joint Details

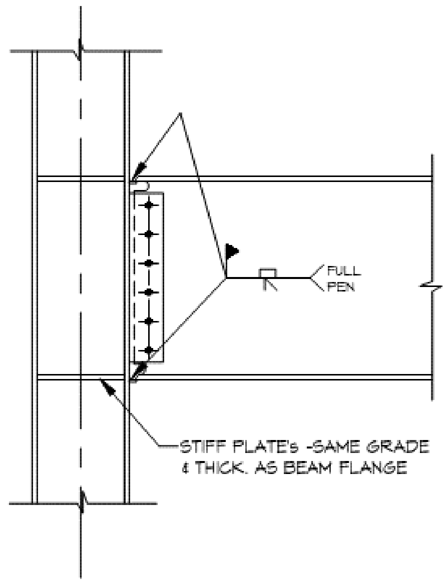


Figure 16: Typical moment connection to column flange

The Brendan Iribe CCSI has many cases where different connection details are required. Several cases include moment connections to wide flange columns, moment connections to HSS, vertical truss connections, and truss connections. All connections have ¾" A325 bolts using single angles unless otherwise noted. *Figure 16* shows a typical detail of a moment connection to a column flange. *Figure 17* on the following page shows a typical truss connection. A claw angle on each side of the gusset plate connects the diagonal member to the gusset plate.

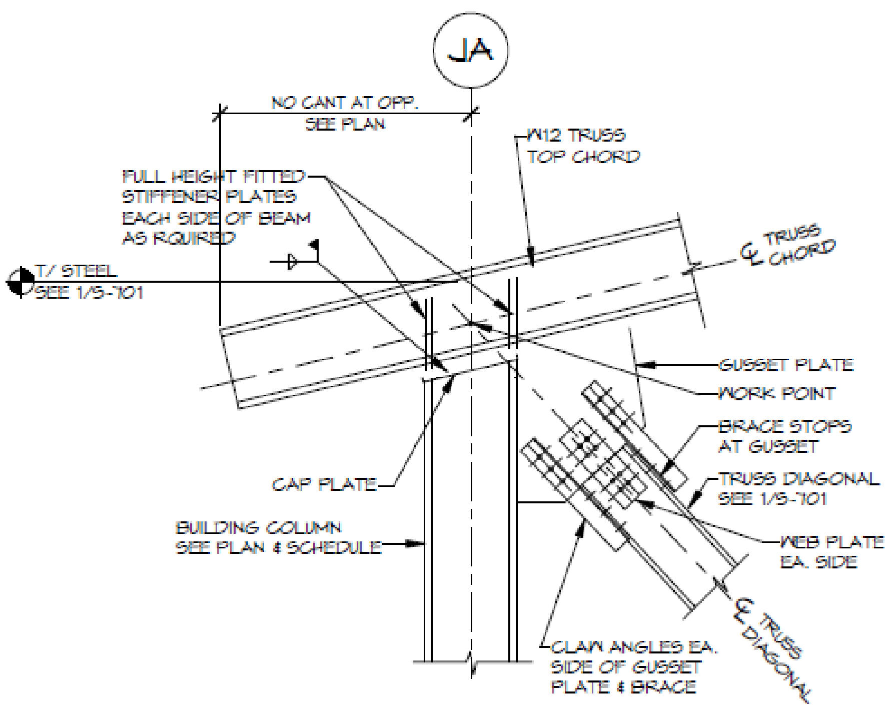


Figure 17: Typical truss

2. References, Codes, and Loading

2.1 References and Codes

The following codes, standards, and design guides apply to the design and construction of this project, and have been used and referenced throughout the report.

I. International Code Council
   a. 2015 International Building Code

II. American Society of Civil Engineers
   a. Minimum Design Loads for Buildings and Other Structures (ASCE 7-10)

III. American Concrete Institute
   a. Building Code Requirements for Structural Concrete (ACI 318-14)

IV. American Institute of Steel Construction
   a. Steel Construction Manual, 14th Edition

V. Concrete Reinforcing Institute
   a. Design Guide for Voided Concrete Slabs

VI. RS Means
   a. 2017 Building Construction Costs

VII. Madam Mehta, Jim Johnson, and Jorge Rocafort
   a. Architectural Acoustics: Principles and Design

VIII. Hope Furrer Associates
   a. Structural drawings and specifications

2.2 Gravity Loads

Dead Loads have been formulated by the engineers through office standards. Dead loads in the figure below do not include the self-weight of structural members. The dead loads used in design consists of the self-weight of the building including structural steel, decking, concrete slab, walls, and roofs. In addition, a superimposed dead load is added which accounts for MEP equipment, interior finishes, and any other miscellaneous load. Live loads are dependent on the occupancy of the room, and are determined from Chapter 4 of ASCE 7-10, and reduction has been included where applicable by code. Drifting and sliding snow loads are accounted for in the 2015 International Building Code, but not included in the figure below. *Figure 18* shows the loading schedule provided by Hope Furrer Associates, the structural engineer on this project.

LOADING SCHEDULE (PSF)

LOCATION / LOADING	BASEMENT	TYP. ELEVATED FLOOR (GROUND FLOOR TO SIXTH FLOOR)	PENTHOUSE (AREA A & B)	ROOF (AREA A, B)	ROOF (AREA C)	TERRACE	ELEVATED AUDITORIUM FLOOR	AUDITORIUM PENTHOUSE
CONCRETE SLAB	VARIES	46	75	63	-	75	46	100
METAL DECK	-	2	3	2	2	3	2	3
M/E/C/L	-	10	10	10	10	10	10	10
MEMBRANE	-	-	-	1	1	-	-	-
INSULATION	-	-	-	4	4	-	-	-
PARTITION	-	-	-	-	-	-	-	-
SOIL (GREEN ROOF)	-	-	-	40	-	200	-	-
TOTAL DEAD LOAD	VARIES	58	88	120	17	288	58	113
LIVE LOAD	100	100	150	30	30	100	100	150
TOTAL LOAD	VARIES	158	238	150	47	388	158	263

NOTES:

1. ALL LIVE LOADS ARE IN ACCORDANCE WITH INTERNATIONAL BUILDING CODE 2015 EDITION.
2. LIVE LOAD REDUCTION HAS BEEN INCLUDED IN THE DESIGN WHERE APPLICABLE AND ALLOWED BY CODE.
3. TOTAL DEAD LOADS DO NOT INCLUDE WEIGHT OF STEEL OR PRIMARY FRAMING MEMBERS.
4. LOADS IN SCHEDULE DO NOT INCLUDE WEIGHTS OF ROOF TOP MECHANICAL UNITS. THE PROVISION FOR THE SUPPORT OF THESE UNITS HAVE BEEN MADE ON AN INDIVIDUAL BASIS. ANY CHANGE FROM SPECIFIED MECHANICAL UNIT (SIZE, WEIGHT AND LOCATION) SHALL BE BROUGHT TO THE ATTENTION OF THE STRUCTURAL ENGINEER.
5. SEE PLANS FOR LOCALIZED CONCENTRATED LOADS.
6. DRIFTED AND SLIDING SNOW LOADS ARE ACCOUNTED FOR IN ACCORDANCE WITH INTERNATIONAL BUILDING CODE 2015 EDITION, BUT ARE NOT INCLUDED IN THE LIVE LOADS INDICATED ABOVE.

Figure 18: Loading schedule

From ASCE 7-10, the ground snow load for College Park, MD is 35 PSF with an exposure factor of 0.9, importance factor of 1.1, and thermal factor of 1.0. The flat roof snow load is 24 PSF plus unbalanced, drifting, and sliding where applicable.

2.3 Lateral Loads

2.3.1 Wind Loads

Wind loads were determined in accordance with ASCE 7-10. College Park, MD has an ultimate design wind speed of 120 mph and a nominal wind speed of 93 mph. The Brendan Iribe CCSI falls under exposure B and risk category III. An internal pressure coefficient of +/- 0.18 has been used. Components and cladding wind loads for parapets have also been determined in accordance with ASCE 7-10.

2.3.2 Seismic Loads

Seismic loads have been calculated using the equivalent lateral force procedure. A risk Category of III, Site Class D, and Seismic Design Category B have been used for these calculations. The basic seismic force resisting system is ordinary reinforced concrete shear walls.

2.4 Load Paths

Although construction starts at the foundation, design starts at the top of the building. All gravity loads act downwards, which is absorbed by the voided slab and transferred to the columns where the load travels to the foundation and is distributed at the ground.

Lateral loads can act horizontally and may even cause uplift. To negate this lateral load, reinforced concrete shear walls have been placed to resist the load.

3. Structural Design Proposal

The Brendan Iribe Center for Computer Science and Innovation consists of steel wide flange girders and columns to resists gravity loads, and moment frames and braced frames to resist lateral loads. The previous notebook submissions have determined that the structural system is acceptable and meets code. Although the current system is efficient, a study will be done to determine if a new system performs just as efficiently as the existing one.

3.1 Design Proposal

The proposed alternative system consists of a voided flat slab for the gravity system and reinforced concrete shear walls for the lateral system. A voided concrete slab removes concrete from the middle of the slab where it is not structurally efficient by placing plastic voids in the shape of spheres. Theses voids reduce the dead load by as much as 35% compared to a solid reinforced concrete slab, which allows for larger spans, lower floor to floor heights due to the reduced slab thickness, and thus a reduced height of the structure. This reduced height of the building can help reduce costs for the façade, pipes, and ductwork. *Figure 19* shows a side by side comparison of a conventional concrete slab system next to the voided concrete slab system. The redesign of the lateral system will consist of shear walls located in the same place as the current moment frames and braced frames. As shear walls have higher stiffness's than moment frames and braced frames, strength and drift should perform better for the proposed lateral system compared to the existing lateral system.

Figure 19: Conventional slab vs voided concrete slab

3.2 Construction Management Breadth

This alternate system will have an effect on the cost of construction. Since concrete is typically cheaper than steel, the overall cost of the building should be cheaper. In addition, the overall building height will be several feet shorter which will help reduce the cost. Although the current cost is being withheld from the owner, this breadth will determine if the new system will reduce the overall cost, and ultimately the feasibility of the alternate system.

3.3 Mechanical Breadth

As the structure changes from steel to concrete, the acoustical performance will be effected. The mechanical breadth will determine how changing the structure effects the Sound Transmission Class (STC) rating of the building. With many research labs, conference rooms, and classrooms, it is essential that sound does not travel through the slab to disturb students and faculty.

3.4 MAE Requirements

The graduate coursework that will be included into this report is from AE 530: Computer Modeling of Building Structures. RAM Structural System will be used to create a three dimensional model to design the new gravity and lateral system.

4. Structural Depth

The structural depth focuses on the redesign of the Brendan Iribe Center for Computer Science & Innovation. The gravity system will be a voided two-way concrete slab while the lateral system will consist of reinforced concrete shear walls. The main goal when considering which alternative gravity system to choose was reducing the depth of the structure. A concrete flat plate slab is the most effective in terms of reducing the depth; but due to the increased self-weight and large live load, it will be difficult for the slab to reach the longer spans of the building (roughly 40-45 feet). In addition to difficulty reaching longer spans, punching shear will most likely be an issue at most columns. The benefit to a voided slab is it reduces the self-weight by 30-35% compared to a solid slab which makes it easier to reach longer spans. The reduced self-weight and depth of the slab, as well as the ability to reach longer spans without beams makes the voided slab a viable option to look into for an alternative design.

Due to the irregular column layout, the building does not satisfy the requirements set forth in ACI 318-14 to use the direct design method for the design of the slab. Therefore a RAM Concept model will be used for the slab design. RAM Concept is a finite element analysis software which utilizes the equivalent frame method to design the two way slab. The corresponding gravity columns and concrete shear walls will be designed in RAM Structural System. In addition to the design from RAM, hand calculations will be conducted to determine if the RAM design is adequate.

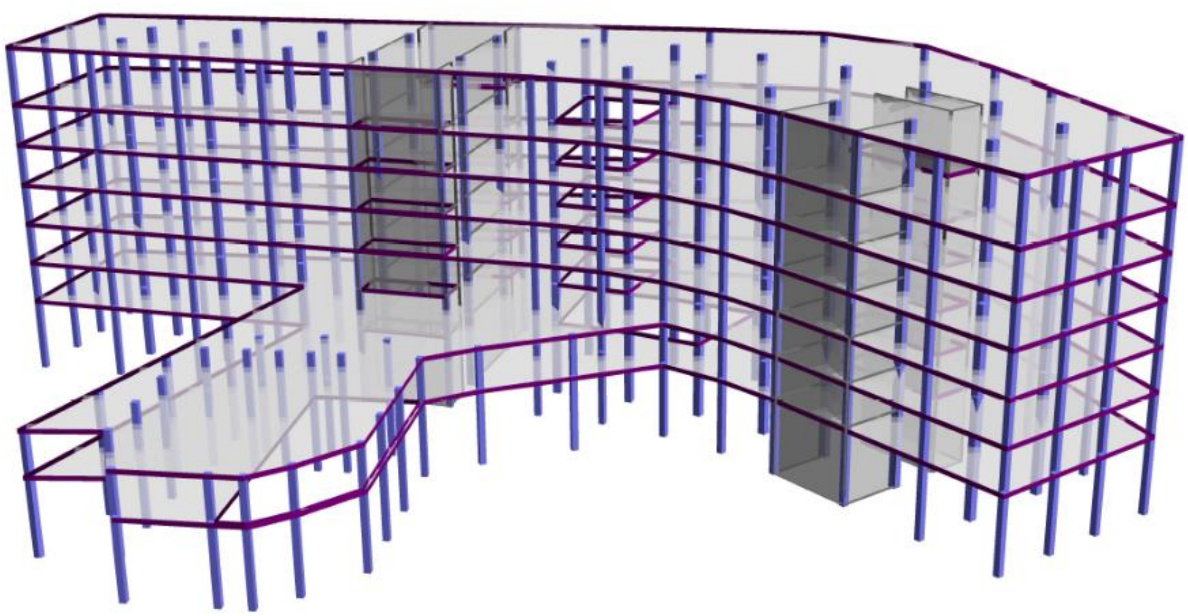


Figure 20: RAM model of voided concrete slab system

4.1 Gravity System Redesign

4.1.1 Gravity Columns

The main goal in redesigning the gravity columns was to keep the columns in the same locations, and to keep the columns sizes smaller than the existing column encasement of 30”. Nine columns that were part of the lateral system were removed as shear walls will be replacing them as the new lateral system. *Figure 21* shows these columns that were removed in red. The removal of these columns also introduce longer spans from the existing building; however, the reduction in self weight in the voided slabs makes it easier for the slab to reach these spans. In addition, the removal of these columns reduces the number of column lines in the east-west direction from eight to four.

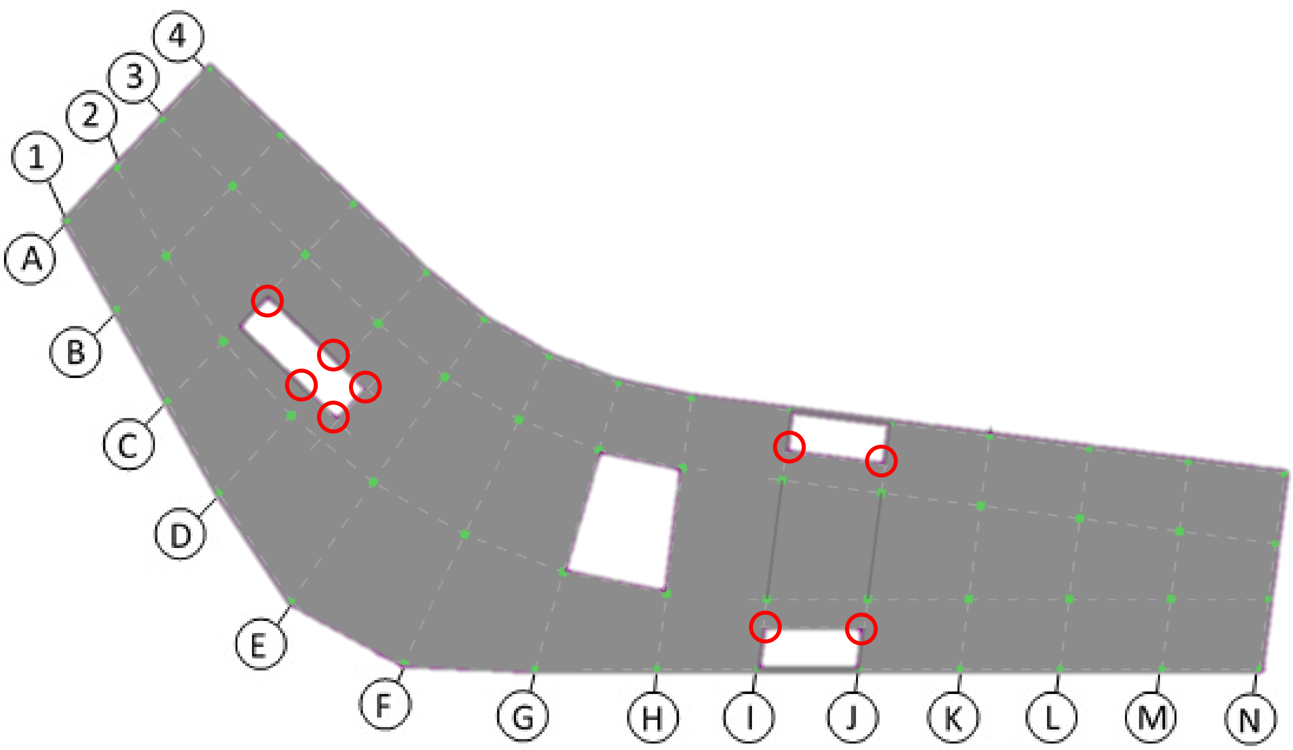


Figure 21: Proposed column layout

4.1.1.1 Interior Column

In order to determine a preliminary size for the columns, equation 22.4.2.2 from ACI 318-14 will be used. For constructability reasons, there will be only be one size for interior columns and one size for exterior columns. Column E2 from *Figure 21* on the previous page will be used to determine the interior column preliminary size as it has the largest tributary area. Table 1 shows the loading on this column throughout each floor. The total axial load at the ground floor is 3900 kips, which is very similar to the value from RAM of 3883 kips. The report for this column can be seen in Appendix A.

Table 1: Load calculation of column E2

	Dead (psf)	SW Slab (psf)	SW Column (k)	Live (psf)	Snow (psf)	1.4D	1.2D+1.6L+0.5Lr	Total (k)
Roof	73	172.5	17.65	30	24.25	**502.45**	497.394	502.45
Penthouse	10	172.5	12.8	100	0	373.07	**563.02**	1065.47
5th	10	172.5	12.8	100	0	373.07	**563.02**	1628.49
4th	10	172.5	12.8	100	0	373.07	**563.02**	2191.51
3rd	10	172.5	12.8	100	0	373.07	**563.02**	2754.53
2nd	10	172.5	12.5	100	0	372.65	**562.66**	3317.19
1st	10	172.5	30	100	0	397.15	**583.66**	3900.85

Originally an f’c value of 4000 psi was used; however column sizes were too large. Ultimately a value of 8000 psi was used to keep the column sizes reasonable. Additionally, a reinforcement ratio of 0.015 is used as a conservative estimate.

$$\phi P_n = 0.80\phi\left[0.85f'c\left(A_g - A_{st}\right) + f_y A_{st}\right]$$

$$3900000 = 0.80(0.75)\left[0.85(8000)\left(A_g - 0.015A_g\right) + 60000(0.015A_g\right]$$

$$A_g = 855\ in^2$$

From this calculation, a 30”x30” column should be appropriate for all interior columns. Trial and error was then used to determine the optimum reinforcement in these columns. After several iterations, it was determined that columns E2 and F2 require substantial more reinforcement due to the larger tributary area and larger axial load compared to the rest of the interior columns. As a result, these two columns will be different from the rest of the interior columns. For these two columns longitudinal reinforcement will consist of 28 #9 at the ground floor, 28 #6 at the 2nd floor, and 12 #8 at each remaining floor. From ACI 25.7.2.2, transverse reinforcement will include #3 bars as the longitudinal bars are smaller than #10 bars. The spacing of these ties shall not exceed 16 longitudinal bar diameters (16*9/8=18”), 48 tie bar diameters (48*3/8=18”), or the least dimension of the compression member (30”). Therefore, transverse reinforcement will be #3 ties @ 12”. *Figure 22* shows a cross section of interior columns E2 and F2 at the ground floor. The column summary of this column can be seen in Appendix A.

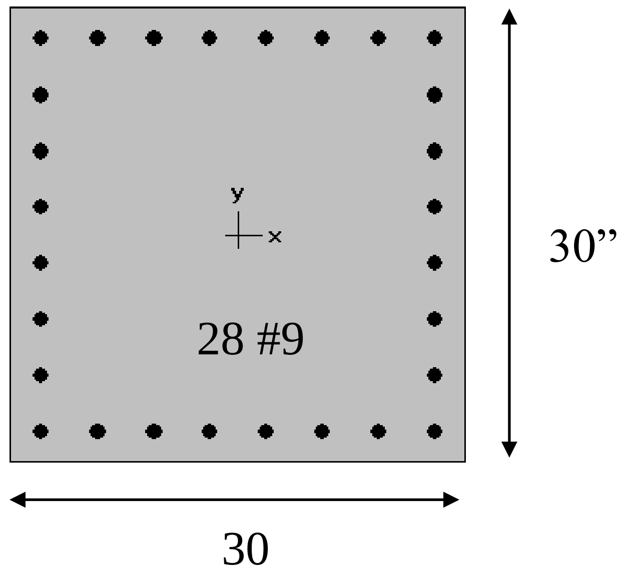


Figure 22: Cross section of columns E2 and F2

One of the main goals in designing is to keep dimensions, rebar quantity, and rebar spacing as consistent as possible to make it easier for the contractor. Therefore, the rest of the interior columns will also be 30”x30” but with reduced rebar. Column E3 from *Figure 21* will be analyzed for the typical interior column. From ACI 10.6.1.1, minimum longitudinal reinforcement in a column is $0.01A_g$. For a 30”x30” column, minimum reinforcement is 0.01*30*30 = 9 in^2. Therefore, 12 #8 (9.48 in^2) will be used as a trial longitudinal reinforcement. The spacing of the ties shall not exceed 16 longitudinal bar diameters (16*8/8=16”), 48 tie bar diameters (48*3/8=18”), or the least dimension of the compression member (30”). Therefore transverse reinforcement will be #3 ties @ 15”. After running the analysis, all interior columns passed with these parameters. *Figure 23* shows a cross section of a typical interior column, and the column summary for column E3 can be seen in Appendix A.

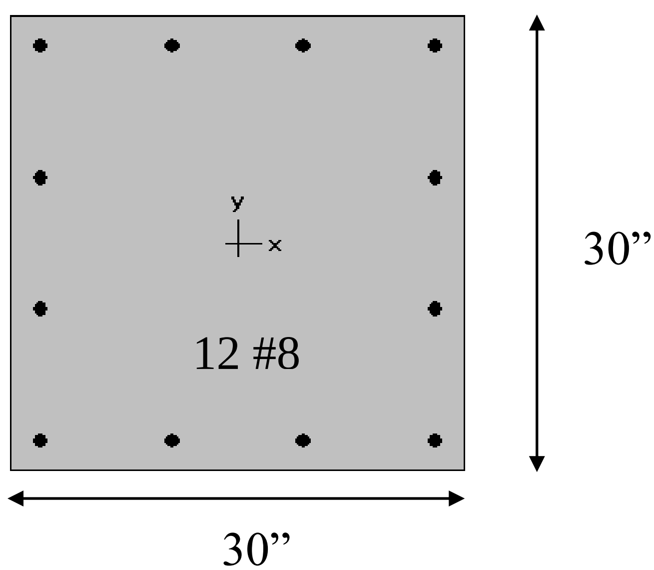


Figure 23: Cross section of typical interior column

4.1.1.2 Exterior Column

Since exterior columns have smaller tributary areas, the dimensions of these columns will be reduced. Column E1 in *Figure 21* will be used to determine the exterior column preliminary size. Table 2 shows the loading at each floor for this column. Once again, the total axial load at the ground floor is 1790 kips which is similar to the value from RAM of 1751 kips. The RAM report can also be seen in Appendix A.

Table 2: Load calculation of column E1

	Dead (psf)	SW Slab (psf)	SW Column (k)	Live (psf)	Snow (psf)	1.4D	1.2D+1.6L+0.5Lr	Total (k)
Roof	73	173.5	11.3	30	24.25	**240.14**	237.03	240.14
Penthouse	10	173.5	8.2	100	0	178.47	**256.97**	497.11
5th	10	173.5	8.2	100	0	178.47	**256.97**	754.08
4th	10	173.5	8.2	100	0	178.47	**256.97**	1011.05
3rd	10	173.5	8.2	100	0	178.47	**256.97**	1268.02
2nd	10	173.5	8.2	100	0	178.47	**256.97**	1524.99
1st	10	173.5	14.7	100	0	187.57	**264.77**	1789.76

$$\phi P_n = 0.80\phi\left[0.85f'c\left(A_g - A_{st}\right) + f_y A_{st}\right]$$

$$1790000 = 0.80(0.75)\left[0.85(8000)\left(A_g - 0.015A_g\right) + 60000(0.015A_g\right]$$

$$A_g = 393\ in^2$$

From this calculation, a 20”x20” column should work; however after running several iterations of design, a 24”x24” column is required. Minimum longitudinal reinforcement of a 24”x24” column is 0.01*24*24 = 5.76 in^2. Therefore 8#8 (6.32 in^2) will be used as a trial reinforcement. The spacing of the ties shall not exceed 16 longitudinal bar diameters (16*8/8=16”), 48 tie bar diameters (48*3/8=18”), or the least dimension of the compression member (24”). Thus, transverse reinforcement will be #3 ties @15”. After running the analysis, all exterior columns passed with these parameters. *Figure 24* shows a cross section of an exterior column, and the column summary for column E1 can be seen in Appendix A.

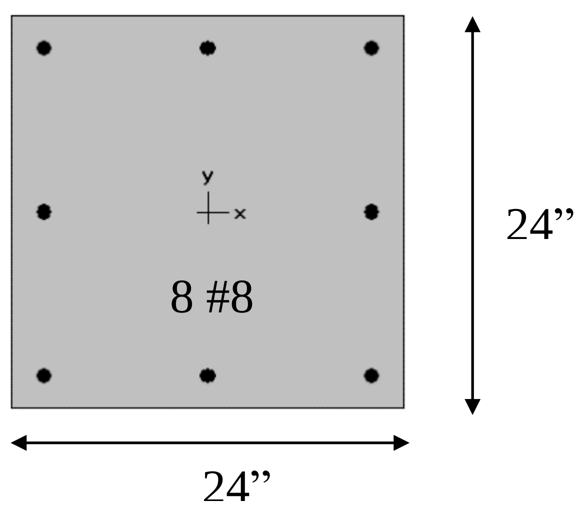


Figure 24: Cross section of typical exterior column

4.1.2 Voided Concrete Slab

As previously mentioned earlier, a voided concrete slab is a lightweight concrete system that utilizes plastic spheres to remove concrete from the middle of the slab where it is not structurally efficient. These voids reduce the dead load by 30-35%, allowing for longer spans without beams, reducing the structural depth as well as the overall height of the building, and reducing deflections. *Figure 25* shows an isometric view of the configuration of a voided concrete slab, while *Figure 26* shows a typical cross section of a voided slab.

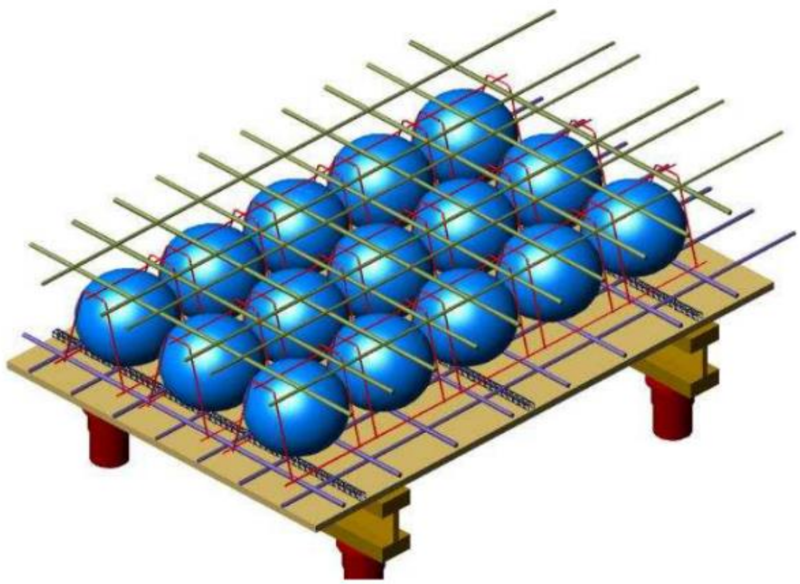


Figure 25: Isometric view of a voided slab

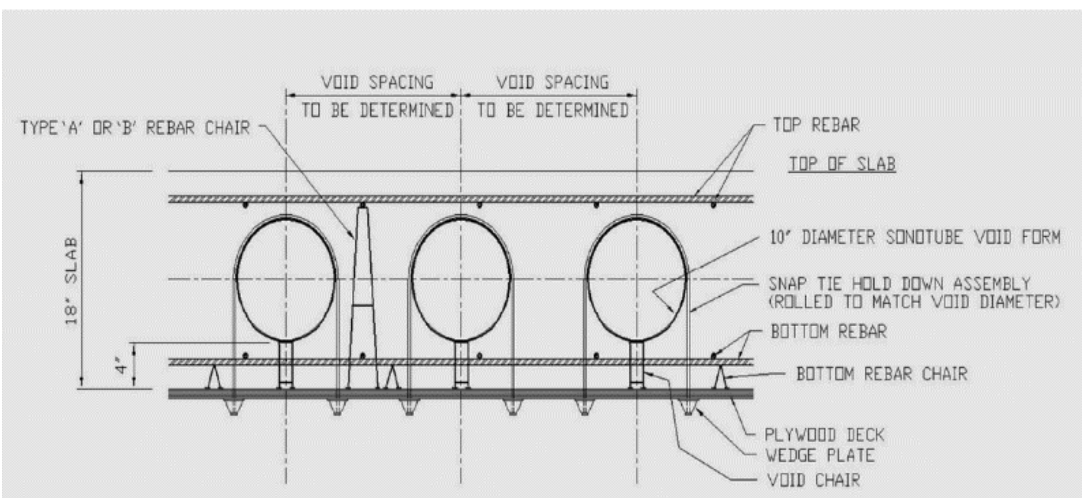


Figure 26: Typical cross section of a voided slab

The construction of a voided slab is similar to the construction of a typical cast in place concrete slab. The formwork is placed followed by the positioning of the bottom reinforcement bars. Next, the void formers are placed in 8 foot long cages and are set on top of the bottom reinforcement via void chairs. Due to the heavy shear forces around columns, the void formers are omitted in this region where a solid slab is required to resist the shear demand. In addition, the voids are omitted in solid strip along the perimeter of the floor plate. Lastly the top reinforcement is set using rebar chairs. The concrete is then poured in two separate stages. The first layer is intended to lock in the void formers and the cage, securing them from the buyout forces that are experienced during concrete placement. Once the concrete is set, the remainder of the concrete is poured and leveled at the top of the slab. *Figure 27* below shows the setup of the voided slab system on a construction site prior to the pouring of the first layer of concrete.

Figure 27: Voided concrete slab configuration before concrete placement

4.1.2.1 Slab Design

When designing the slab, a voided concrete slab is treated as a solid two way slab with less self-weight. The process of designing the slab will include:

- Determining a trial slab thickness and void properties
- Modeling the slab in RAM Concept with these properties
- Checking to see if the slab passes with these design parameters and adjusting if needed
- Designing a panel on a typical floor by hand to validate RAM's design.

As noted earlier, the Brendan Iribe Center for Computer Science and Innovation does not meet the requirements for the direct design method. Therefore, RAM Concept will design the slab using the equivalent frame method. In addition, only the slab in the tower will be designed. The auditorium features spans roughly 100', and cannot be designed unless there are columns added in the auditorium. One of the goals of this redesign is to not affect the architectural floor plan of the building. Therefore, the auditorium will not be included in the scope of the slab redesign. If this redesign were to happen in real life, the auditorium would most certainly have to remain as steel.

ACI 8.3.1 will be used to determine the minimum slab thickness for serviceability. From ACI Table 8.3.1.1, the governing slab thickness for an exterior panel without drop panels and without edge beams for 60,000 psi stress steel is $l_n/30$. The longest span in the building at 43-6" will be used to determine the slab thickness.

$$h_{min} = \frac{l_n}{30} = \frac{(43.5 * 12) - (\frac{30}{2} + \frac{24}{2})}{30} = 16.5"$$

Based off of Cobiax Eco-Line properties from the Design Guide for Voided Concrete Slabs, a 17.5" slab depth with 12 3/8" spherical void formers will be used. Table 3 shows the properties for a 12 3/8" void.

Table 3: Specifications of a 12 3/8" void

Slab depth (in)	17.5
Dead load reduction (psf)	-66
Stiffness correction factor	0.91
Shear reduction factor	0.55
Cage module support height (in)	12 5/8
Void former height (in)	12 3/8
Void former horizontal dimension (in)	12 3/8
Spacing between void formers (in)	1 3/8
Void formers center line spacing (in)	13 3/4
Number of void formers per sq ft	0.76
Concrete displacement per sq ft (cubic ft)	0.44
Void formers per cage module	7
Equivalent area per cage module (sq ft)	9.25

The table lists several important factors such as the spacing and dimensions of the void formers, and also shows the reduction of the dead load using a concrete density of 150 pcf. A voided slab system consists of voided slab areas where voids are spaced uniformly, and solid areas around the columns and solid strip areas around the perimeter of the floor which do not contain these voids. These areas are considered when determining the reduction of the average dead load. Several calculations will be performed to verify the numbers in the table and ultimately determine the self-weight of the new slab. The first step to determine the reduction in the voided slab areas is to determine the volume of one spherical void.

$$Volume = \frac{4\pi r^3}{3} = \frac{4\pi \left(\frac{12\ 3/8}{2 * 12}\right)^3}{3} = 0.574\ ft^3$$

Next, the amount of concrete that is displaced is equal to the volume of one void times the number of voids per square foot.

$$Concrete\ displacement = 0.574 * 0.76 = 0.436\ ft^3/ft^2$$

To determine the volume of concrete in the voided area of the slab, the concrete displacement is subtracted from the overall slab thickness.

$$\left(\frac{17.5}{12}\right) - 0.44 = 1.018\ ft^3/ft^2$$

The self-weight in the voided area of the slab is equal to the unit weight of concrete times the volume of concrete. For this design, normal weight concrete (150 pcf) will be used.

$$150\ pcf * 1.018 = 152.7\ psf$$

The dead load reduction corresponds to the average reduction in slab dead load based on the average volume of voids in the slab. In order to determine this, the slab weight in the voided area of the slab is subtracted from the weight of the solid slab.

$$\frac{17.5 * 150}{12} - 152.7 = 66\ psf$$

This matches the value given in Table 3. The average dead load of the slab takes into account the solid areas of that slab around the columns and perimeter of the floor plate. Since the solid areas of the slab have not yet been determined since they are governed by punching shear, the dead load reduction is reduced to 70% as a conservative estimate. Therefore, the average dead load of the new slab is

$$\frac{17.5 * 150}{12} - (0.7 * 66) = 172.5\ psf$$

Since the average dead load takes into consideration the solid parts of the slab, the 172.5 psf dead load is applied along the whole floor plan. In addition, a 10 psf superimposed dead load and 100 psf live load are applied to the slab. The slab was modeled in RAM with two-way slab behavior, 17.5" slab, and an f'c of 4000 psi. The slab was then imported into RAM concept from Ram modeler. Since the slab was imported from RAM, no loads need to be applied. Therefore the next step includes defining the latitude and longitude design strips. *Figure 28* shows the latitude design strips and *Figure 29* shows the longitude design strips. The light blue are column strips and the dark blue are middle strips. The properties that were specified for these design strips are a top and bottom cover of 0.75" and a #7 top and bottom bar.

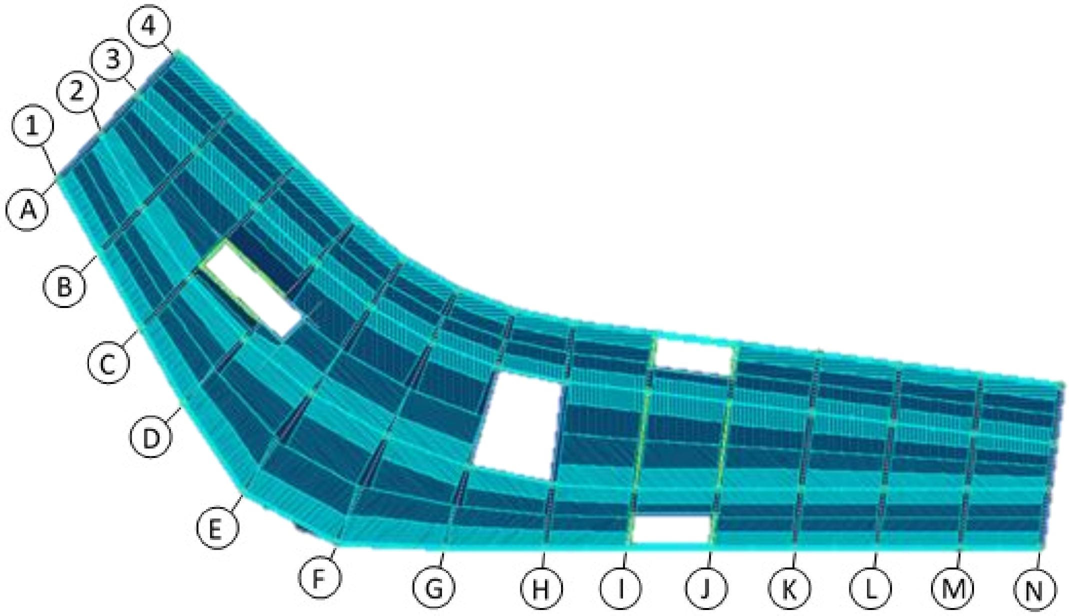


Figure 28: Latitude design strips in RAM Concept

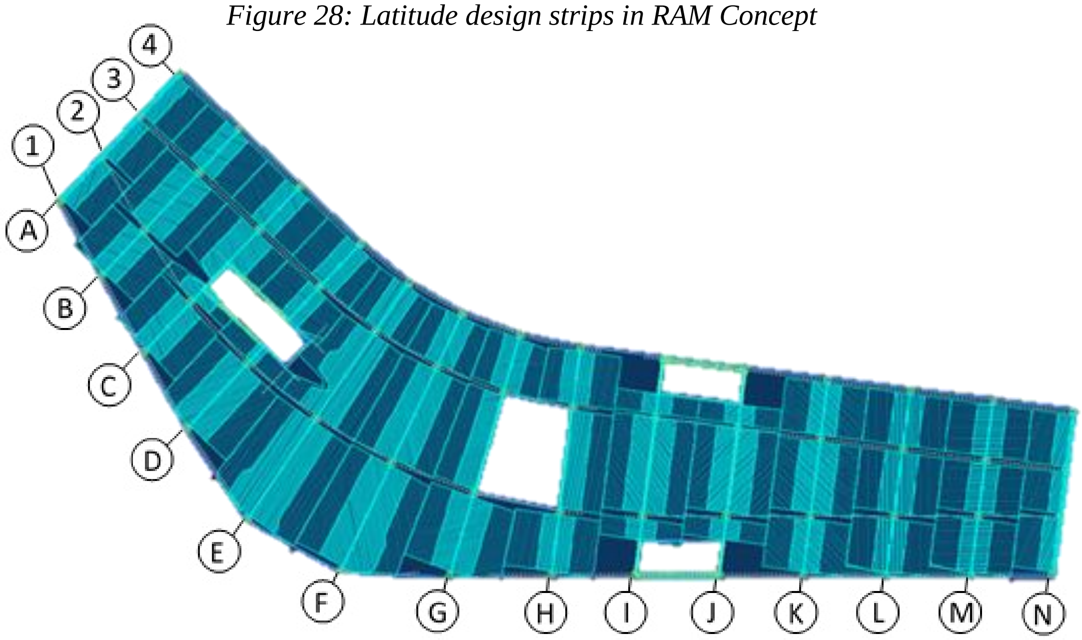


Figure 29: Longitude design strips in RAM Concept

4.1.2.2 Punching Shear

Since punching shear usually governs the depth of the slab, the first thing to check is the punching shear status plan. This plan checks the punching shear at each column, and displays it as red if it's failing and green if it passes. *Figure 30* below shows that each column passes for punching shear.

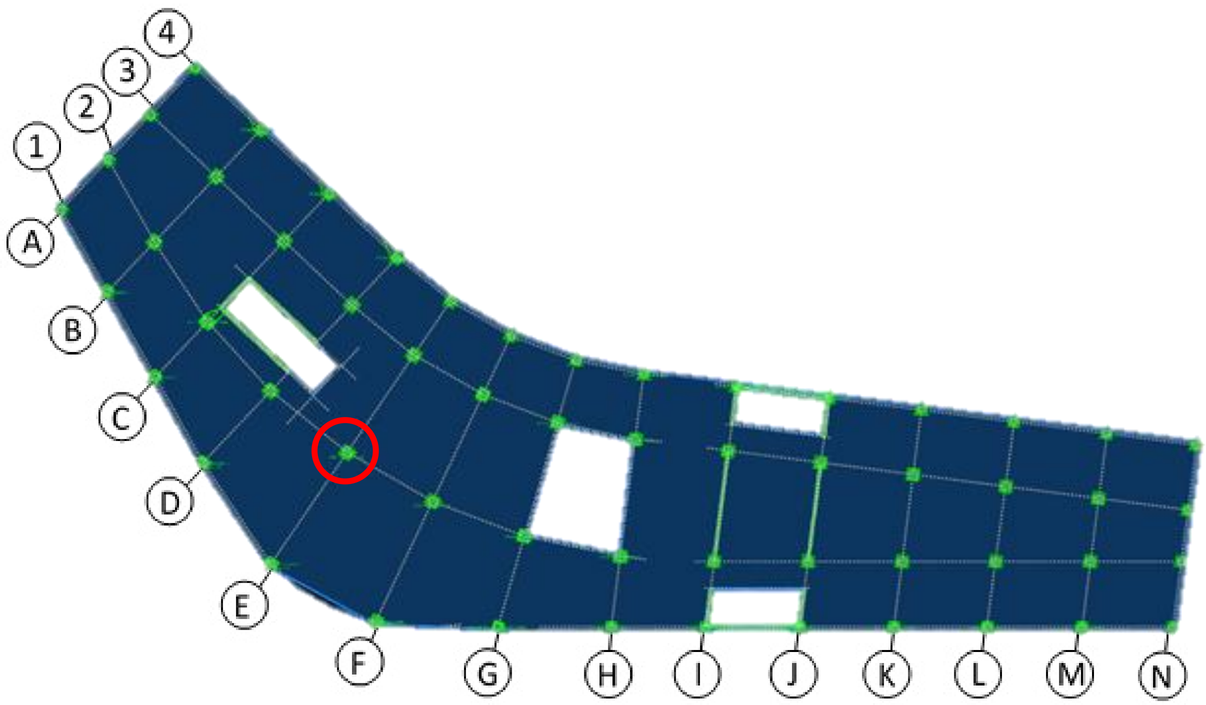


Figure 30: Punching shear status plan

Although RAM Concept displays all columns are passing, the column E2 will be checked by hand to confirm the results from RAM. This column was chosen as it has the largest tributary area, and will experience the largest shear forces. The factored loading is

$$q_u = 1.2(10 + 172.5) + 1.6(100) = 379\ psf$$

To determine d, a cover of 0.75" is bottom bars are # 6 bars.

$$d = 17.5 - 0.75 - \frac{0.75}{2} = 16.375"$$

$$b_0 = 2\big((30 + 16.375) + (30 + 16.375)\big) = 185.5"$$

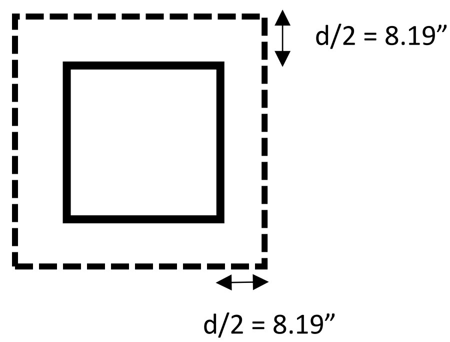


Figure 31: Critical section of column E2

Figure 31 shows the dimensions of the critical section d/2 for punching shear. The applied punching shear is

$$V_u = 0.379\left[\left(\left(\frac{43.67'}{2}+\frac{38'}{2}\right)*31.5'\right)-\left(\frac{30+2(8.19)}{12}*\frac{30+2(8.19)}{12}\right)\right] = 484.6\ kips$$

$$v_u = \frac{V_u}{b_0 d} = \frac{484.6*1000}{185.5(16.375)} = 159.5\ psi$$

From ACI 22.6.5.2, V_c shall be the smallest of

$$V_c = 4\lambda = 4$$

$$V_c = \left(2+\frac{4}{\mathrm{B}}\right)\lambda = 2+\frac{4}{1} = 6$$

$$V_c = \left(2+\frac{\alpha_s d}{b_0}\right)\lambda = \left(2+\frac{40*16.375}{185.5}\right) = 5.53$$

The first equation governs in this case. Therefore, the allowable punching shear stress is

$$\Phi V_c = 0.75(4)(1)\sqrt{4000} = 189.7\ psi\ > 159.5\ psi \therefore OK$$

4.1.2.3 Deflection

The maximum allowable deflection is in accordance with Table 24.2.2 of ACI 318-14. To be conservative, a deflection limit of l/480 is used. This deflection is for roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflections, and considers the sum of the long term deflection due to all sustained loads and the immediate deflection due to any additional live load.

Figure 32 shows the deflection diagram of a typical floor. The color scale shows the severity of deflection in inches across the floor plate. As expected, the most severe deflection is experienced in the southwestern part of the building where spans reach 43’-4”, with a maximum deflection of 0.77”. Using l/480, the allowable deflection in this region is 1.08” which is much greater than the maximum deflection. Other than this region, the deflection experienced is very small with values around 0.10”. Since there were no issues with punching shear, it makes sense that deflection would not be controlling for the slab design.

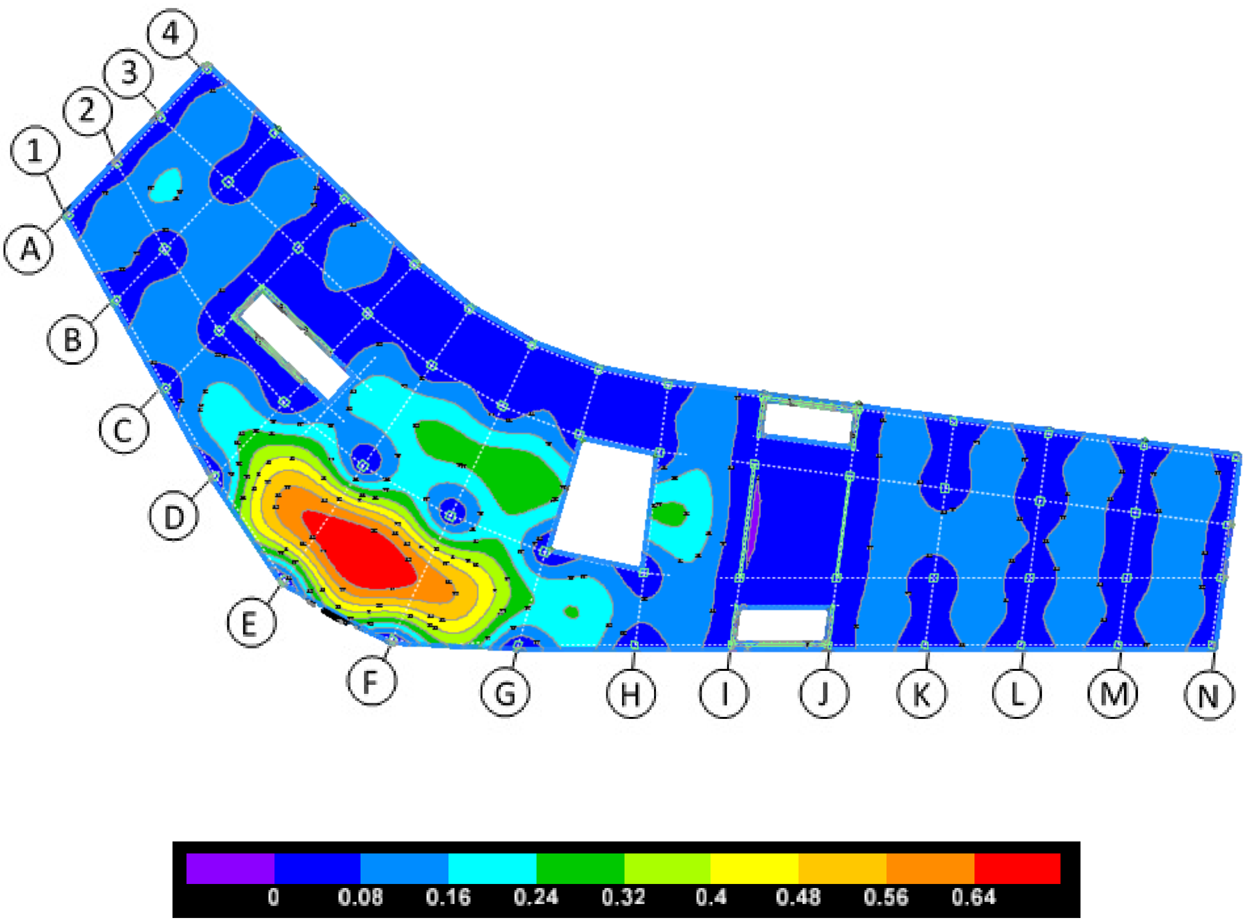


Figure 32: Deflection diagram of typical floor

4.1.2.4 Rebar Layout

Due to no punching shear failures, the 17.5” slab depth will be used. As mentioned earlier, the reinforcement has been specified as #7 bars. *Figure 33* shows the layout of the top reinforcement. The bar layout looks reasonable as there are top bars located along column strips and middle strips that are perpendicular to column lines. The reinforcement in the column strips consists anywhere from 5-14 #7 bars while reinforcement in the middle strips consists anywhere from 4-10 #7 bars. The one area where it is different is in the panel between E and F. Due to the large moments in this panel, a high number of bars are required resulting in very small spacing. To increase the spacing, the bars in the longitudinal direction were increased to #9 bars. *Figure 34* shows the layout of the bottom reinforcement. The bottom reinforcement will be a mat of #7@12” each way. Since bottom reinforcement is essentially needed everywhere, it is easier to specify a mat of bottom bars for easier construction. The exact reinforcement can be seen in Appendix B.

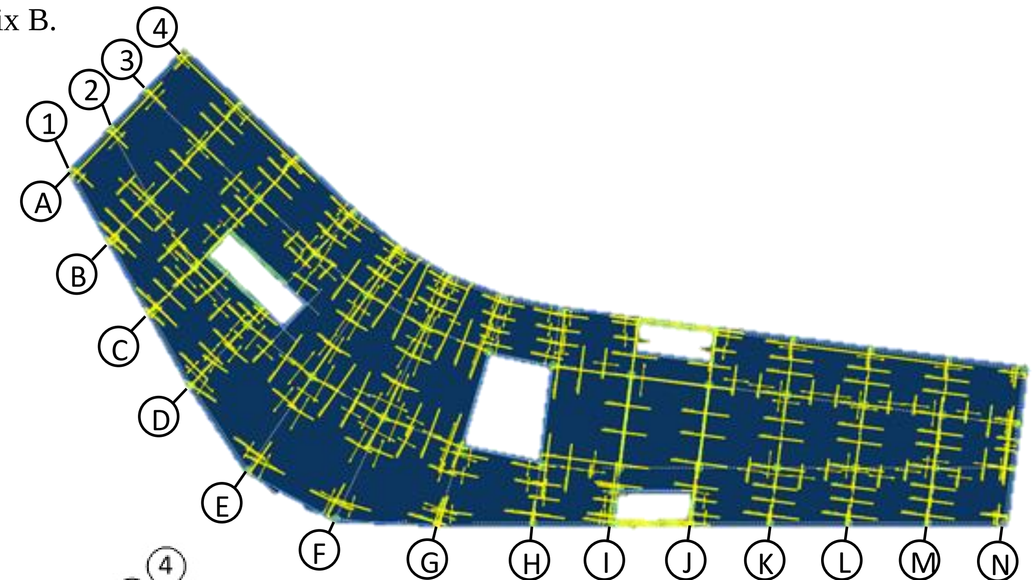


Figure 33: Top rebar layout

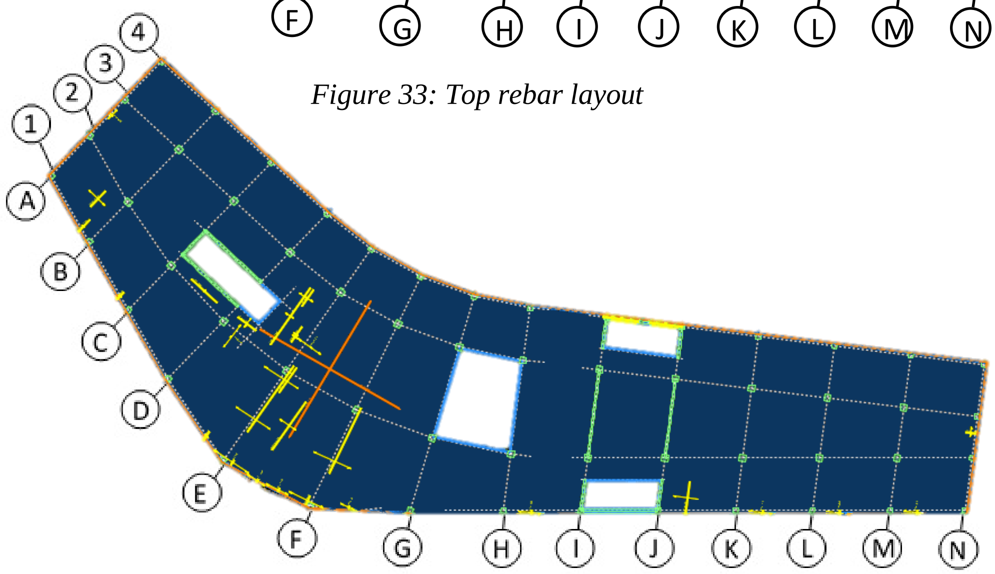


Figure 34: Bottom rebar layout

Although RAM states the design is acceptable, the panel between E and F will be further analyzed in Appendix B. This section of the building has been selected as it contains the longest spans of the building, and therefore will experience the largest moments. As noted earlier, the building does not meet the requirements of the direct design method. These requirements from ACI 8.10.2 include:

- There shall be a minimum if three continuous spans
  - The spans are not along a continuous column line in the horizontal direction therefore failing this requirement
- Panels shall be rectangular, with a ratio of longer to shorter span center-to-center of supports within a panel not greater than two
  - All panels in this group have ratios of longer to shorter spans less than 2, but they are not rectangular therefore failing this requirement
- Successive span lengths center-to-center of supports in each direction shall not differ by more than one-third the longer span
  - The vertical dimension of the lower panel is 42'-6" and the vertical dimension of the top panel is 21'-2". These differ by more than one-third of 42'-6" therefore failing this requirement
- Offset of columns by a maximum of 10 percent of the span (in direction of offset) from either axis between centerlines of successive columns shall be permitted
  - The column in the bottom left corner is offset by more than 10 percent of the vertical dimension therefore failing this requirement
- All loads shall be due to gravity only and uniformly distributed over an entire panel. The unfactored live load shall not exceed two times the unfactored dead load
  - All loads are due to gravity and distributed uniformly; and the service live load (100 psf) is less than 2 times the service dead load (2*182.5 psf) therefore passing this requirement
- For a panel with beams between supports on all sides, equation 13-2 shall be satisfied for beams in the two perpendicular directions
  - There are no beams in this building so this requirement is negligible

Only one of the six requirements are met for the panel that is being subjected to further analysis. Although it does not meet the requirements, the direct design method can still be used to get a rough approximation of the moments and required area of rebar. Even when a building does meet these requirements, they are still an approximation and will not provide exact numbers. Therefore, this panel will be transformed into an orthogonal panel using the actual vertical dimensions and using the largest horizontal dimension amongst all three bays. This will ensure that the design will be on the conservative side. The process will include analyzing this panel in both the latitude and longitude direction to determine the moments in the column and middle strips. Then the required reinforcement will be calculated and compared to the RAM model. If the required reinforcement from the hand calculations is less than the reinforcement provided in RAM, then the RAM design can be deemed appropriate.

4.1.2.5 Solid Areas of Slab

As mentioned earlier, a voided slab system consists of three area: the voided slab area where the voids are placed uniformly, solid areas around the columns where a solid slab is needed to resist the shear forces, and a solid strip around the perimeter of the floor plate. The solid area around a column is dependent on the location where the shear strength of the voided area can resist the total shear stress alone. A shear reduction factor is used to account for the reduced shear strength in the voided area of the slab. Column E2 is used to determine the solid area of slab required.

$$Solid\ area\ around\ column = Tributary\ area\ of\ column - \frac{(Shear\ reduction\ factor)(Allowable\ direct\ shear\ force)}{Total\ factored\ unifromly\ distributed\ load}$$

$$\Phi V_c = \Phi 4\lambda\sqrt{f'c}b_0 d = 0.75(4)(1)\sqrt{4000} * 2[(30 + 16.375) * 2] * \left(\frac{16.375}{1000}\right) = 576.3\ kips$$

$$Solid\ area\ arond\ columm = 1286 - \frac{0.55 * 576.3}{\frac{379}{1000}} = 449.6\ ft^2$$

This means 450 ft^2 of solid slab around the column is needed to adequately resist the shear force. Appendix A shows the calculation of the solid area required at each column. Due to the high shear capacities, a solid slab is only required at four columns: E1, E2, F1, and F2. This means the voided area of the slab is capable of resisting the total shear stress at all other columns. In addition to the area around columns, the perimeter of the floor plate is also solid. According to the design guide, this width is typically two feet. *Figure 35* shows the areas where the slab is solid in red.

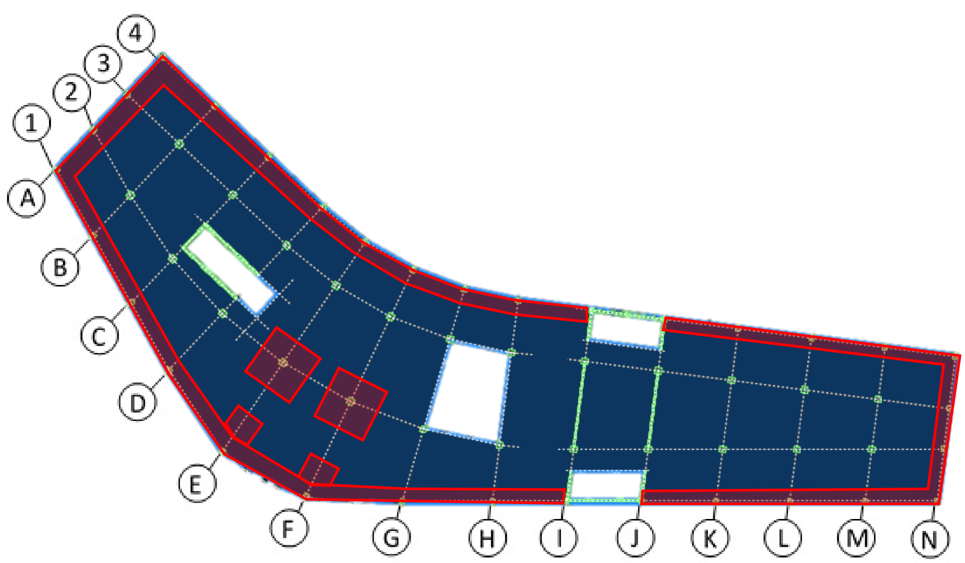


Figure 35: Areas where solid slab is required

4.1.2.6 Changes to Height of Structure

As mentioned earlier, one of the biggest benefits to the voided slab is it reduces the depth of the structure. The depth of the current structure is 30”, and the depth of the voided slab is 17.5”. This means the structural depth at each floor is reduced by a foot. With seven stories, this reduces the overall height of the building from 118’-8” to 111’-8”. *Figure 36* shows a side by side comparison of the existing buildings floor heights and the proposed floor heights with the voided slab. Reducing the height by 7’ reduces the overall cost of the building as there is less material for façade, pipes, and ductwork. This will be further analyzed in the construction management breadth.

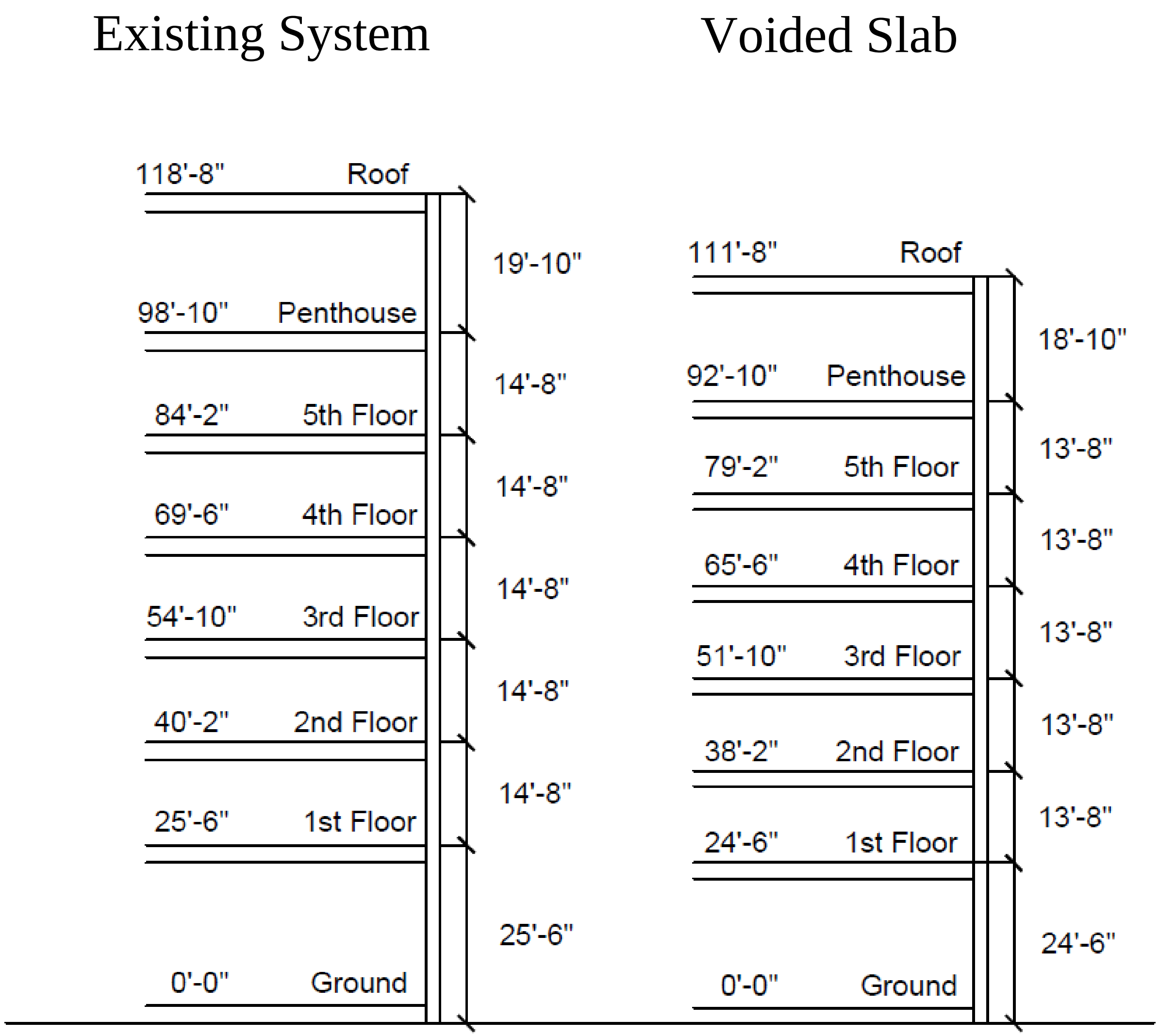


Figure 36: Height comparison between existing and proposed system

4.1.2.7 Final Slab Design

To summarize, a 17.5” slab with 12 3/8” void formers will be used for the slab. After checking punching shear, deflection, and reinforcement, the slab has been deemed an acceptable design. The top reinforcement is shown in Appendix B, and the bottom reinforcement is a mat of #7@12” each way. *Figure 37* shows a cross section along a column strip, and *Figure 38* shows a cross section along a middle strip.

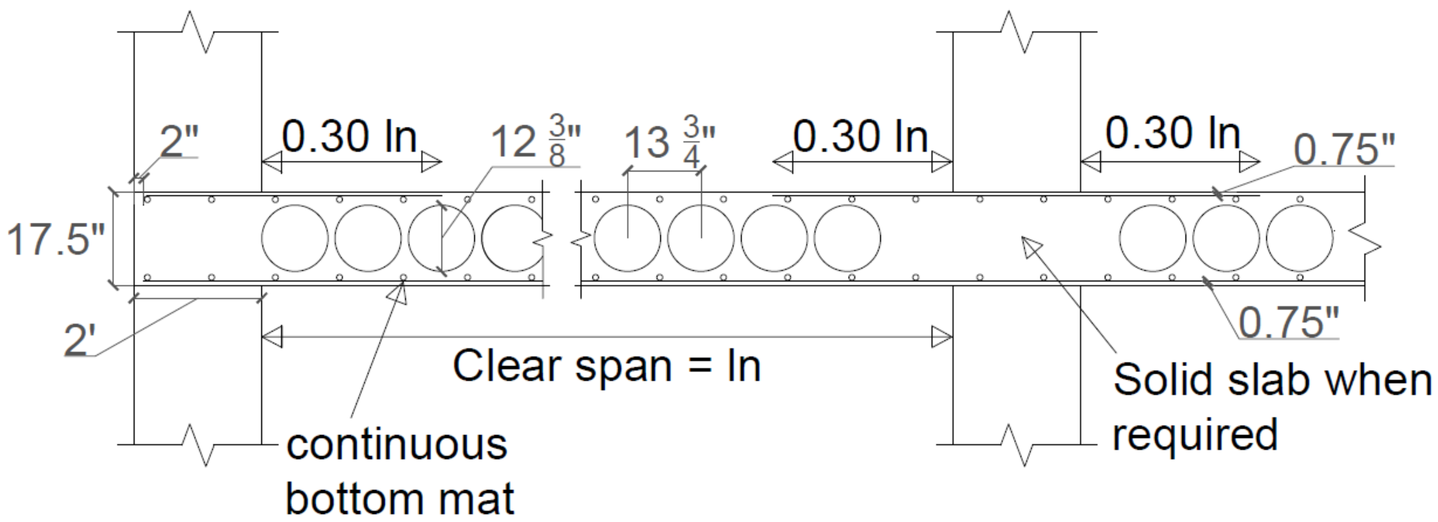


Figure 37: Column strip cross section

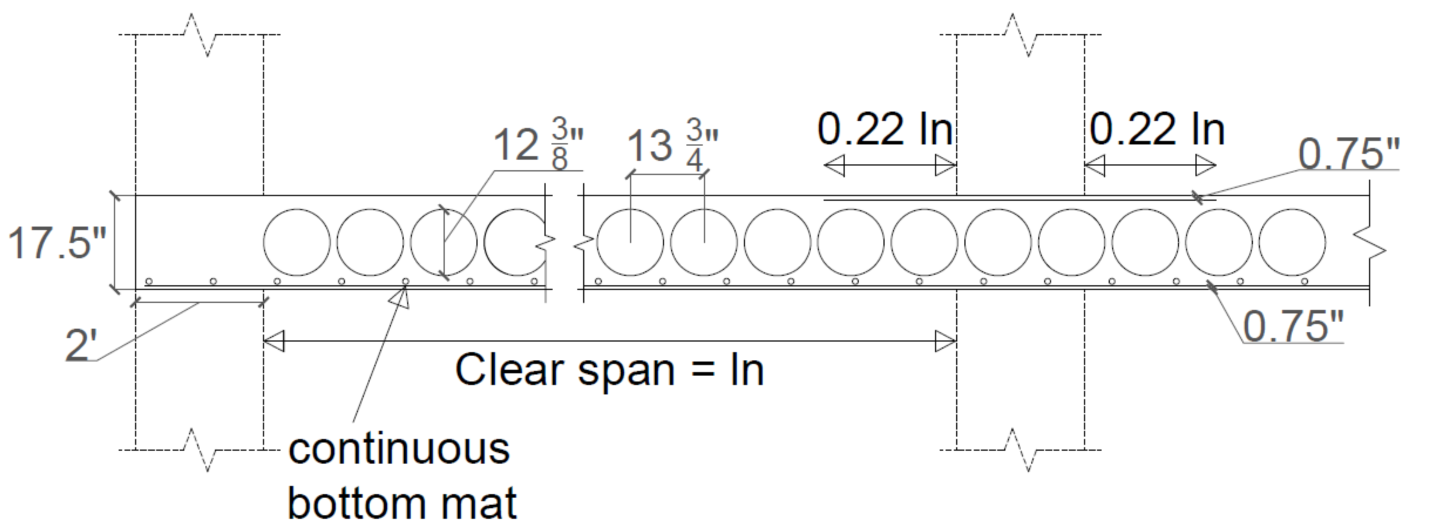


Figure 38: Middle strip cross section

4.2 Lateral System Redesign

Ordinary reinforced concrete shear walls were chosen to resist the lateral loads in this redesign. The location of the shear walls is important as it determines where the lateral loads are applied on the building. Wind loads are a function of pressure and act on the center of pressure, while seismic loads are a function of mass and thus act at the center of mass. The center of rigidity is the geometric stiffness center of the shear walls throughout the building. The goal in designing the lateral system is to minimize the eccentricity between the center of mass and center of rigidity which reduces torsional deformations on the building and ultimately reduces the design forces and moments in these shear walls.

Since the strength and drift requirements were adequate for the existing lateral system, the shear walls remain in the same locations as the existing lateral system; however not all are needed as shear walls provide more stiffness than braced frames and moment frames. *Figure 39* below shows the locations of the shear walls. Refer to Notebook Submission C to compare the shear wall locations vs. the braced/moment frame locations. After finalizing the layout of the shear walls, wind/seismic loads and the center of rigidity are recalculated by hand. Then these numbers are compared to RAM's results to validate the computer model. If these numbers are relatively close, then RAM's forces will be used to design the shear walls and check drift requirements.

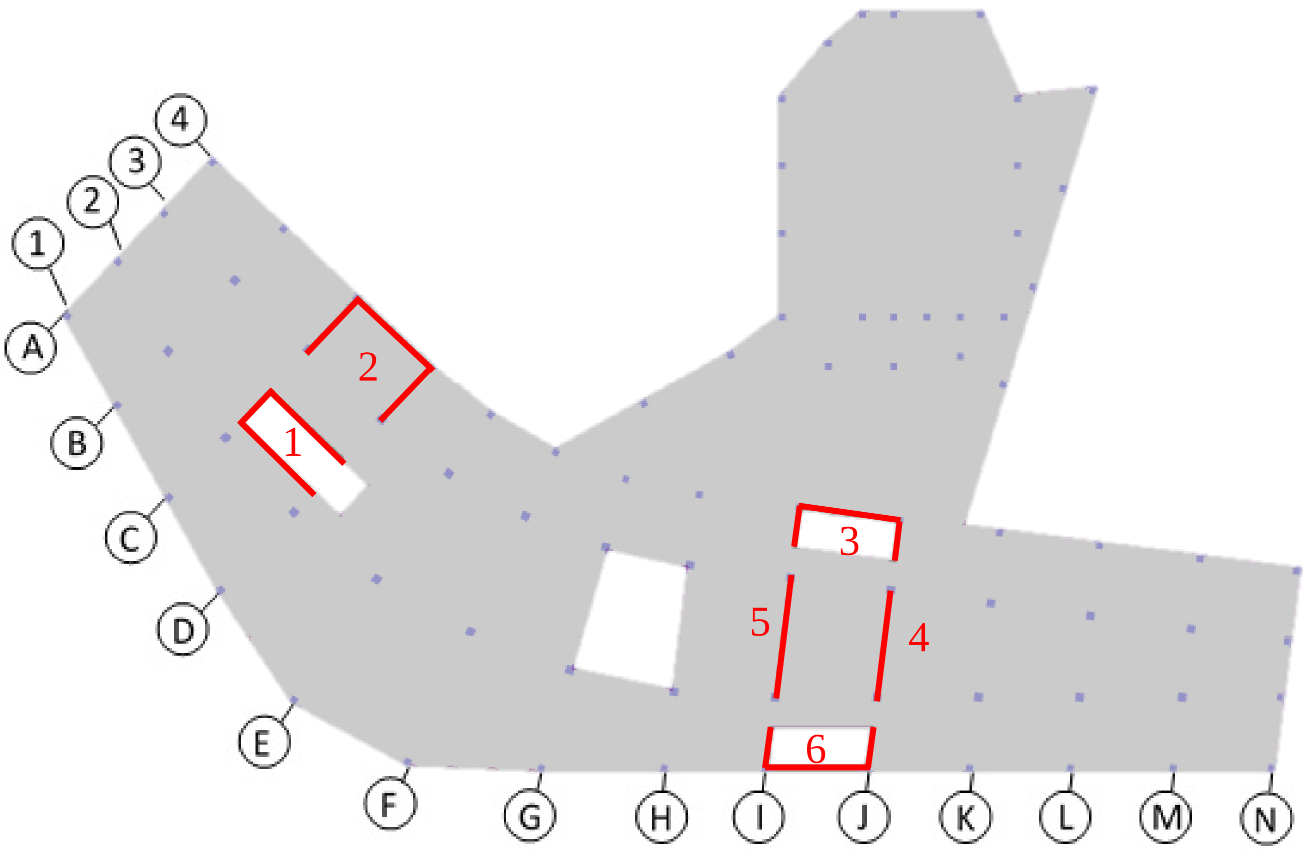


Figure 39: Shear wall layout

4.2.1 Wind Loads

Since the overall building height has been reduced due to the reduction in the slab depth, new wind loads were calculated. Table 4 and Table 5 show the total wind pressures for each floor in the north-south and east-west direction respectively. Appendix C has the spreadsheet showing the parameters that were used to calculate these pressures.

Table 4: Wind pressures in the north-south direction

Level	Height (ft)	Kz	qz (psf)	$p_{winward}$ (psf)	$p_{leeward}$ (psf)	Total Pressure (psf)
Ground	0	0.570	17.9	11.50	-12.85	24.35
1st Floor	24.5	0.656	20.6	13.24	-12.85	26.09
2nd Floor	38.17	0.749	23.5	15.11	-12.85	27.97
3rd Floor	51.8	0.817	25.6	16.49	-12.85	29.35
4th Floor	65.6	0.872	27.3	17.60	-12.85	30.45
5th Floor	79.17	0.927	29.0	18.70	-12.85	31.55
Penthouse	92.83	0.968	30.3	19.54	-12.85	32.40
Roof	111.68	1.019	31.9	20.57	-12.85	33.42

Table 5: Wind pressures in the east-west direction

Level	Height (ft)	Kz	qz (psf)	$p_{winward}$ (psf)	$p_{leeward}$ (psf)	Total Pressure (psf)
Ground	0	0.570	17.9	11.72	-13.10	24.83
1st Floor	24.5	0.656	20.6	13.49	-13.10	26.60
2nd Floor	38.17	0.749	23.5	15.41	-13.10	28.51
3rd Floor	51.8	0.817	25.6	16.81	-13.10	29.91
4th Floor	65.6	0.872	27.3	17.94	-13.10	31.04
5th Floor	79.17	0.927	29.0	19.06	-13.10	32.16
Penthouse	92.83	0.968	30.3	19.92	-13.10	33.02
Roof	111.68	1.019	31.9	20.96	-13.10	34.07

Table 6 and Table 7 show the total base shear in the north-south and east-west direction respectively. Similar to the existing building, base shear controls in the north-south direction as there is greater surface that the wind loads will be acting on.

Table 6: Base shear in the north-south direction

Level	Height (ft)	Trib Height (ft)	Trib Width (ft)	Total Pressure (psf)	Story Force (kips)
1st Floor	24.5	19.09	380	26.09	189.26
2nd Floor	38.17	13.67	380	27.97	145.28
3rd Floor	51.8	13.67	380	29.35	152.44
4th Floor	65.6	13.67	380	30.45	158.17
5th Floor	79.17	13.67	380	31.55	163.90
Penthouse	92.83	16.25	380	32.40	200.04
Roof	111.68	9.415	380	33.42	119.56
				Base Shear (kips)	1128.66

Table 7: Base shear in the east-west direction

Level	Height (ft)	Trib Height (ft)	Trib Width (ft)	Total Pressure (psf)	Story Force (kips)
1st Floor	24.5	19.09	245	26.60	124.39
2nd Floor	38.17	13.67	245	28.51	95.48
3rd Floor	51.8	13.67	185.5	29.91	75.85
4th Floor	65.6	13.67	185.5	31.04	78.71
5th Floor	79.17	13.67	185.5	32.16	81.56
Penthouse	92.83	16.25	185.5	33.02	99.54
Roof	111.68	9.415	185.5	34.07	59.50
				Base Shear (kips)	615.03

4.2.2 Seismic Loads

As a concrete slab weighs more than composite steel, the overall weight of the structure has increased significantly (~30%). Therefore the seismic forces and base shear increase as well. Table 8 shows the seismic design parameters used to calculate the base shear. Appendix C shows a spreadsheet which calculates the seismic weight of the building. All parameters remain the same from the lateral system except for the Response Modification Coefficient (R), Overstrength Factor (Ω), and Deflection Amplication Factor (C_d) as the seismic force resisting system consists of ordinary reinforced concrete shear walls instead of ordinary braced frames and ordinary moment frames. Table 9 shows the calculation of the seismic story shear at each level.

Table 8: Seismic design parameters

Risk Category	III
S_s	0.119g
S_1	0.051g
S_{DS}	0.127g
S_{D1}	0.081g
Seismic Design Category	B
Site Class	D
R	4
Ω	2.5
C_d	4
Seismic Importance Factor	1.25
T_a	0.687
C_S	0.037
W	50,310.08 kips
Seismic Base Shear	1861.47 kips

Table 9: Seismic story shears

Level	h_x (ft)	W_K (k)	h_x^k	$W_K h_x^k$	C_{vx}	F_x (k)	V_x (k)
1st	24.5	9061.22	40.22	364479.75	0.06	108.60	1861.47
2nd	38.17	8913.30	67.13	598311.69	0.10	178.28	1752.87
3rd	51.83	6083.65	95.57	581441.04	0.09	173.25	1574.59
4th	65.5	6083.65	125.24	761944.06	0.12	227.04	1401.34
5th	79.17	6083.65	155.90	948422.68	0.15	282.60	1174.30
Penthouse	92.83	6154.96	187.36	1153201.16	0.18	343.62	891.70
Roof	111.68	7929.65	231.96	1839355.23	0.29	548.08	548.08
Total		50310.08		6247155.62	1.00	1861.47	

As expected, seismic base shear controls for the lateral loads. This makes as the increased structural weigh results in higher seismic loads. Tables 10, 11, and 12 compare hand calculated vs. RAM story shears at each level. For each case, the numbers are fairly similar, with the maximum percent error of 16.94% for base shear for wind in the east-west direction.

Table 10: Wind in north-south direction comparison

		Calculated	RAM	
Level	Height (ft)	Fy (kips)	Fy (kips)	% error
1st Floor	24.5	189.26	193.82	2.35
2nd Floor	38.17	145.28	148.92	2.44
3rd Floor	51.8	152.44	156.53	2.61
4th Floor	65.6	158.17	162.71	2.79
5th Floor	79.17	163.9	167.99	2.43
Penthouse	92.83	200.04	205.85	2.82
Roof	111.68	119.56	121.55	1.64
	Base Shear	1128.65	1157.37	2.48

Table 11: Wind in east-west direction comparison

		Calculated	RAM	
Level	Height (ft)	Fx (kips)	Fx (kips)	% error
1st Floor	24.5	124.39	105.32	18.11
2nd Floor	38.17	95.48	72.98	30.83
3rd Floor	51.8	75.85	66.41	14.21
4th Floor	65.6	78.71	68.88	14.27
5th Floor	79.17	81.56	71.57	13.96
Penthouse	92.83	99.54	88.21	12.84
Roof	111.68	59.5	52.57	13.18
	Base Shear	615.03	525.94	16.94

Table 12: Seismic story shear comparison

		Calculated	RAM	
Level	Height (ft)	Fx (kips)	Fx (kips)	% error
1st Floor	24.5	108.6	114.57	5.21
2nd Floor	38.17	178.28	188.93	5.64
3rd Floor	51.8	173.25	190.59	9.10
4th Floor	65.6	227.04	246.17	7.77
5th Floor	79.17	282.6	302.86	6.69
Penthouse	92.83	343.62	373.14	7.91
Roof	111.68	548.08	574.59	4.61
	Base Shear	1861.47	1990.85	6.50

4.2.3 Center of Rigidity/Center of Mass

As mentioned earlier, the center of rigidity and its respective distance from the center of mass is important in minimizing torsional deformations throughout the building. The center of rigidity for the new lateral system has been recalculated by hand, with the shear wall stiffness and center of rigidity calculations in Appendix C. Due to the irregular geometry and the difficulty in calculating the center of mass by hand, the center of mass will not be recalculated. In *Figure 40* below, the red dot represents the center of mass from RAM, the blue dot represents the center of rigidity from RAM, and the black dot represents the hand calculated center of rigidity. These values are also shown in Table 13. From looking at the plan, these locations look reasonable. The hand calculated center of rigidity is fairly similar to RAM as it is 12' different in the x direction and 14' different in the y direction. In addition, there is very little eccentricity between RAM's center of rigidity and center of mass, reducing the design forces in the shear wall. Based off this analysis, the shear walls appear to be in optimum locations.

Table 13: COR/COM

	COR_x (ft)	COR_y (ft)	COM_x (ft)	COM_y (ft)	e_x (ft)	e_y (ft)
Calculated	260.31	189.5	-	-	-	-
RAM	272.97	175.51	256.65	179.42	16.32	3.91
Difference	12.66	13.99	-	-	-	-

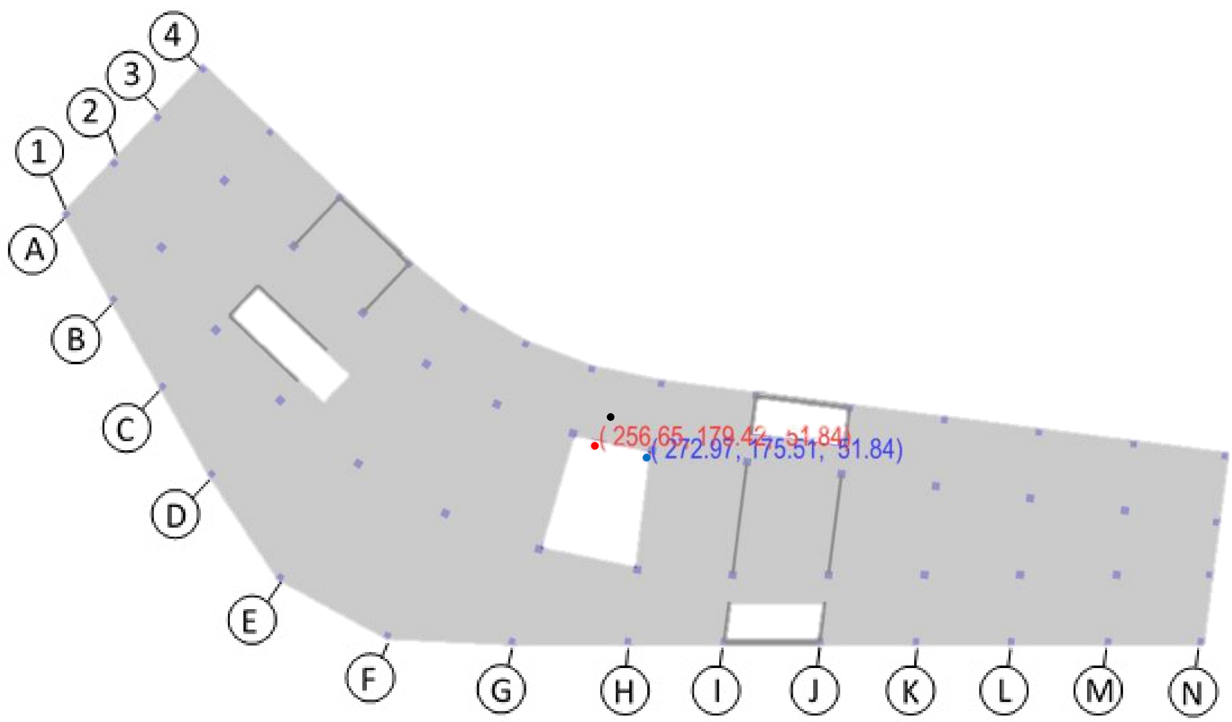


Figure 40: COR/COM location

4.2.5 Shear Wall Design

The previous few sections have successfully compared hand calculations to RAM's results. Since the numbers are similar enough, RAM's results have been deemed appropriate to use going forward. Instead of calculating the total shear into each wall, RAM's forces are used to design the shear walls. To prevent tedious work, only shear wall 5 is designed by hand. This wall was also chosen as it is primarily in the north-south direction meaning it will be resisting largest loads throughout the building. Refer to *Figure 39* for shear wall locations. Once shear wall 5 is designed, the remaining shear walls are designed in RAM.

The minimum thickness for shear wall 5 is 12" based off ACI 11.3.1.1. Therefore all shear walls have been modeled as 12" thick, f'c = 4000 psi, fy = 60000 psi, clear cover of 3" at the end of the wall, and clear cover of 0.75" to the horizontal reinforcement. The hand calculations for shear wall 5 can be found in Appendix C, and *Figure 41* displays a cross section of this shear wall. The reinforcement for the remaining shear walls is shown in Table 14. All shear walls have adequate shear and axial/flexural strength, and the summary for shear wall 5 can be found in Appendix C. As a note, boundary elements contain the axial/flexural reinforcement that are tied together with transverse reinforcement. These boundary elements are located at the wall edge on both sides. Therefore c shaped walls have 6 boundary elements as the 3 walls that make up the c shape each have two boundary elements.

Table 14: Shear wall summary

Shear Wall	Length	Horizontal RFT	Vertical RFT	Flexural/Axial RFT
1	Short- 13.17' Long-30'	#5@12"	#5@12"	8 #7@9"
2	Short- 21' Long- 30'	#5@12"	#5@12"	8 #10@9"
3	Short- 12.5' Long- 30'	#5@12"	#5@12"	8 #8@9"
4	32'	#4@12"	#4@12"	18 #10@10"
5	35.83'	#4@12"	#4@12"	14 #10@9"
6	Short- 12.67' Long- 30.25'	#5@12"	#5@12"	10 #9@9"

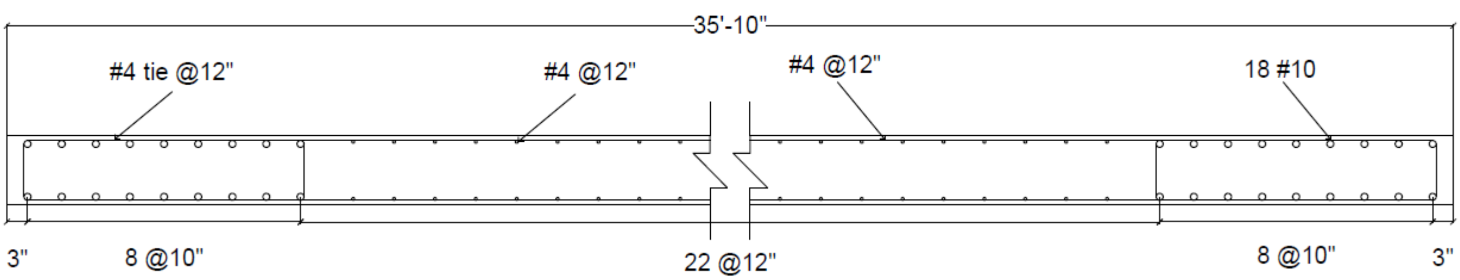


Figure 41: Cross section of shear wall 5

4.2.6 Drift Check

Per ASCE 7-10, the allowable drift for wind loading is set at a limit of H/400 where H is the total height of the story. From Table 12.12-1 of ASCE 7-10, the allowable drift for seismic loading for risk category III building is $0.015h_{sx}$ where h_{sx} is the story height. *Figure 42* displays the maximum drift from wind loading on the left and maximum drift from seismic loading on the right.

Story	LdC	Displacement X in	Displacement Y in
Roof	W15	0.3678	-0.7119
	W16	0.3193	-0.1761
	W17	0.0078	2.2212
	W18	0.2158	-0.0929
	W19	0.4554	0.6201
	W20	0.2317	-1.5082
	W21	0.4377	-0.6036
	W22	0.2453	1.5338
	W23	0.2700	-2.1998
	W24	0.0776	-0.0624

Story	LdC	Displacement X in	Displacement Y in
Roof	E5	3.0337	2.2321
	E6	2.4186	1.0743
	E7	0.1192	2.6197
	E8	1.3476	4.9320

Figure 42: Maximum drift experienced

$$\Delta_{allowable,wind} = \frac{111.67' x\ 12"/1'}{400} = 3.35" > 2.22" \therefore OK$$

$$\Delta_{allowable,seismic} = 0.015 * (111.68' * 12/1') = 20.1" > 4.93" \ \therefore\ OK$$

After calculating the allowable drift, it has been determined that this building passes both wind and seismic drift requirements. It also makes sense that the most severe drift is in the y direction due to the higher wind and seismic loads. The proposed reinforced concrete shear wall system passes strength and drift requirements, deeming it an acceptable design.

5. Construction Management Breadth

This construction management breadth compares the cost of the existing composite steel system with the proposed voided concrete system. This cost estimate only considers the structural system, such as concrete, rebar, and formwork for the voided slab and decking, steel framing, and shear studs for the composite system. Since the auditorium was not designed with the voided concrete slab, this breadth only considers the structure from the main tower towards the total cost.

The takeoffs for the voided concrete slab were provided from RAM Concrete Column, RAM Concrete Shear Wall, and RAM Concept. The takeoffs for the composite steel system were provided by RAM Structural System. RS Means 2017 was used for the cost analysis, and a location multiplier of 0.936 was used for Washington D.C. Table 15 is the cost estimate for the existing composite steel system and Table 16 is a cost estimate for the proposed voided concrete slab system.

Table 15: Existing cost estimate

Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip Unit Cost	Equip Cost	Total
05 31 13	Metal Decking									
	3" 20 gage decking	SF	31270	$ 2.41	$ 75,360.70	$ 7.50	$ 234,525.00	$ -	$ -	$ 309,885.70
					$ -		$ -		$ -	$ -
03 31 13	Concrete Decking				$ -		$ -		$ -	$ -
	Elevated Slab, less than 6" pumped	CY	11123	$ -	$ -	$ 19.25	$ 214,117.75	$ 6.15	$ 68,406.45	$ 282,524.20
					$ -		$ -		$ -	$ -
03 22 11	Welded Wire Fabric Reinforcing				$ -		$ -		$ -	$ -
	6x6-W2.1xW2.1	CSF	21200	$ 18.80	$ 398,560.00	$ 28.00	$ 593,600.00	$ -	$ -	$ 992,160.00
					$ -		$ -		$ -	$ -
05 05 23	Shear Studs				$ -		$ -		$ -	$ -
	3/4" diamter 5-3/16" long shear studs	each	1939	$ 0.73	$ 1,415.47	$ 0.96	$ 1,861.44	$ 0.43	$ 833.77	$ 4,110.68
					$ -		$ -		$ -	$ -
05 12 23	Structural Steel Members				$ -		$ -		$ -	$ -
Columns	W14x68	LF	4462	$ 107.00	$ 477,434.00	$ 3.07	$ 13,698.34	$ 1.70	$ 7,585.40	$ 498,717.74
	W14x145	LF	11424	$ 254.00	$ 2,901,696.00	$ 3.31	$ 37,813.44	$ 1.83	$ 20,905.92	$ 2,960,415.36
Beam	W10x15	LF	224	$ 21.50	$ 4,816.00	$ 5.05	$ 1,131.20	$ 2.79	$ 624.96	$ 6,572.16
	W12x19	LF	5397	$ 23.00	$ 124,131.00	$ 3.43	$ 18,511.71	$ 1.90	$ 10,254.30	$ 152,897.01
	W14x22	LF	4403	$ 37.50	$ 165,112.50	$ 3.05	$ 13,429.15	$ 1.69	$ 7,441.07	$ 185,982.72
	W16x26	LF	3227	$ 37.50	$ 121,012.50	$ 3.02	$ 9,745.54	$ 1.67	$ 5,389.09	$ 136,147.13
	W18x40	LF	154	$ 57.50	$ 8,855.00	$ 4.52	$ 696.08	$ 1.88	$ 289.52	$ 9,840.60
	W18x55	LF	301	$ 79.50	$ 23,929.50	$ 4.76	$ 1,432.76	$ 1.97	$ 592.97	$ 25,955.23
	W21x44	LF	2513	$ 63.50	$ 159,575.50	$ 4.08	$ 10,253.04	$ 1.69	$ 4,246.97	$ 174,075.51
	W21x50	LF	976	$ 72.00	$ 70,272.00	$ 4.08	$ 3,982.08	$ 1.69	$ 1,649.44	$ 75,903.52
	W24x55	LF	9194	$ 79.50	$ 730,923.00	$ 3.91	$ 35,948.54	$ 1.62	$ 14,894.28	$ 781,765.82
	W24x68	LF	1473	$ 98.00	$ 144,354.00	$ 3.91	$ 5,759.43	$ 1.62	$ 2,386.26	$ 152,499.69
	W24x76	LF	1923	$ 110.00	$ 211,530.00	$ 3.91	$ 7,518.93	$ 1.62	$ 3,115.26	$ 222,164.19
	W27x84	LF	217	$ 121.00	$ 26,257.00	$ 3.64	$ 789.88	$ 1.51	$ 327.67	$ 27,374.55
	W27x94	LF	255	$ 136.00	$ 34,680.00	$ 3.64	$ 928.20	$ 1.51	$ 385.05	$ 35,993.25
	W30x116	LF	819	$ 167.00	$ 136,773.00	$ 3.74	$ 3,063.06	$ 1.55	$ 1,269.45	$ 141,105.51
	HSS16x8x1/2	LF	420	$ 1,550.00	$ 651,000.00	$ 67.00	$ 28,140.00	$ 37.00	$ 15,540.00	$ 694,680.00
	HSS20x12x1/2	LF	688	$ 1,550.00	$ 1,066,400.00	$ 67.00	$ 46,096.00	$ 37.00	$ 25,456.00	$ 1,137,952.00
	Subtotals				$ 5,816,687.17		$ 1,208,805.57		$ 150,597.83	$ 7,176,090.57
	Sales Tax (6%)				$ 349,001.23				$ 9,035.87	$ 358,037.10
	Overhead & Profit (assume 20%)				$ 1,233,137.68		$ 241,761.11		$ 31,926.74	$ 1,506,825.53
	Subtotal				$ 7,398,826.08		$ 1,450,566.68		$ 191,560.44	$ 9,040,953.20
	Contingency (0% for C/O's)				$ -		$ -		$ -	$ -
	Adjustments		1.048- time	1.218 - location	$ (473,524.87)		$ (92,836.27)		$ (12,259.87)	$ (578,621.01)
	Total Cost				$ 6,925,301.21		$ 1,357,730.42		$ 179,300.57	**$ 8,462,332.20**

Table 16: Proposed cost estimate

Cost Code	Item	Units	Quantity	Mat'l Unit Cost	Mat'l Cost	Labor Unit Cost	Labor Cost	Equip Unit Cost	Equip Cost	Total
03 11 13	Formwork									
	30x30- 4 use	SFCA	31270	$ 2.41	$ 75,360.70	$ 7.50	$ 234,525.00	$ -	$ -	$ 309,885.70
	24x24- 4 use	SFCA	25016	$ 2.71	$ 67,793.36	$ 7.85	$ 196,375.60	$ -	$ -	$ 264,168.96
	Flat slab 4 use	SFCA	205870	$ 1.19	$ 244,985.30	$ 4.11	$ 846,125.70	$ -	$ -	$ 1,091,111.00
	Wall, job built plywood 4 use	SFCA	75823.9	$ 0.97	$ 73,549.18	$ 4.55	$ 344,998.75	$ -	$ -	
					$ -		$ -		$ -	$ -
03 21 11	Reinforcement				$ -		$ -		$ -	
	Columns #3-#7	ton	13.6	$ 940.00	$ 12,784.00	$ 1,150.00	$ 15,640.00	$ -	$ -	$ 28,424.00
	Columns #8-#9	ton	90.15	$ 940.00	$ 84,741.00	$ 755.00	$ 68,063.25	$ -	$ -	$ 152,804.25
	Elevated Slab #4-#9	ton	673.68	$ 940.00	$ 633,259.20	$ 600.00	$ 404,208.00	$ -	$ -	$ 1,037,467.20
	Walls #3-#7	ton	75	$ 940.00	$ 70,500.00	$ 580.00	$ 43,500.00	$ -	$ -	
	Walls #8-#10	ton	38.8	$ 940.00	$ 36,472.00	$ 435.00	$ 16,878.00	$ -	$ -	$ 53,350.00
					$ -		$ -		$ -	$ -
03 31 13	Concrete				$ -		$ -		$ -	$ -
	Conrete Material 8000 psi	CY	1291.3	$ 142.00	$ 183,364.60	$ -	$ -	$ -	$ -	$ 183,364.60
	Concrete Material 4000 psi	CY	12462	$ 125.00	$ 1,557,750.00	$ -	$ -	$ -	$ -	$ 1,557,750.00
	Column Pumped 30x30	CY	620.4	$ -	$ -	$ 19.25	$ 11,942.70	$ 6.15	$ 3,815.46	$ 15,758.16
	Column Pumped 24x24	CY	670.8	$ -	$ -	$ 29.50	$ 19,788.60	$ 9.40	$ 6,305.52	$ 26,094.12
	Slab over 10" thick pumped	CY	11123	$ -	$ -	$ 15.00	$ 166,845.00	$ 4.79	$ 53,279.17	$ 220,124.17
	Wall 12" thick pumped	CY	1338.9	$ -	$ -	$ 24.50	$ 32,803.05	$ 7.85	$ 10,510.37	$ 43,313.42
	Subtotals				$ 3,040,559.34		$ 2,401,693.65		$ 73,910.52	$ 4,983,615.58
	Sales Tax (6%)				$ 182,433.56				$ 4,434.63	$ 186,868.19
	Overhead & Profit (assume 20%)				$ 644,598.58		$ 480,338.73		$ 15,669.03	$ 1,034,096.75
	Subtotal				$ 3,867,591.48		$ 2,882,032.37		$ 94,014.18	$ 6,204,580.52
	Contingency (0% for C/O's)				$ -		$ -		$ -	$ -
	Adjustments		1.048- time	1.218 - location	$ (247,525.85)		$ (184,450.07)		$ (6,016.91)	$ (397,093.15)
	Total Bid				$ 3,620,065.63		$ 2,697,582.30		$ 87,997.27	**$ 5,807,487.37**

As expected, the total cost from switching from steel to concrete has decreased. The cost for the existing composite steel system is $8,462,332.20 and the cost for the proposed voided slab is $5,807,487.37 which results in a 31% reduction in cost. Due to the reduction of the overall height, the façade, ductwork, and pipes will also experience a cost reduction. From a cost standpoint, switching to a voided concrete slab would be a good alternate solution as the cost of the structure alone and the overall cost of the building would decrease. This would however increase the project schedule as concrete takes longer to construct compared to steel, so that is something that would need to be taken into consideration.

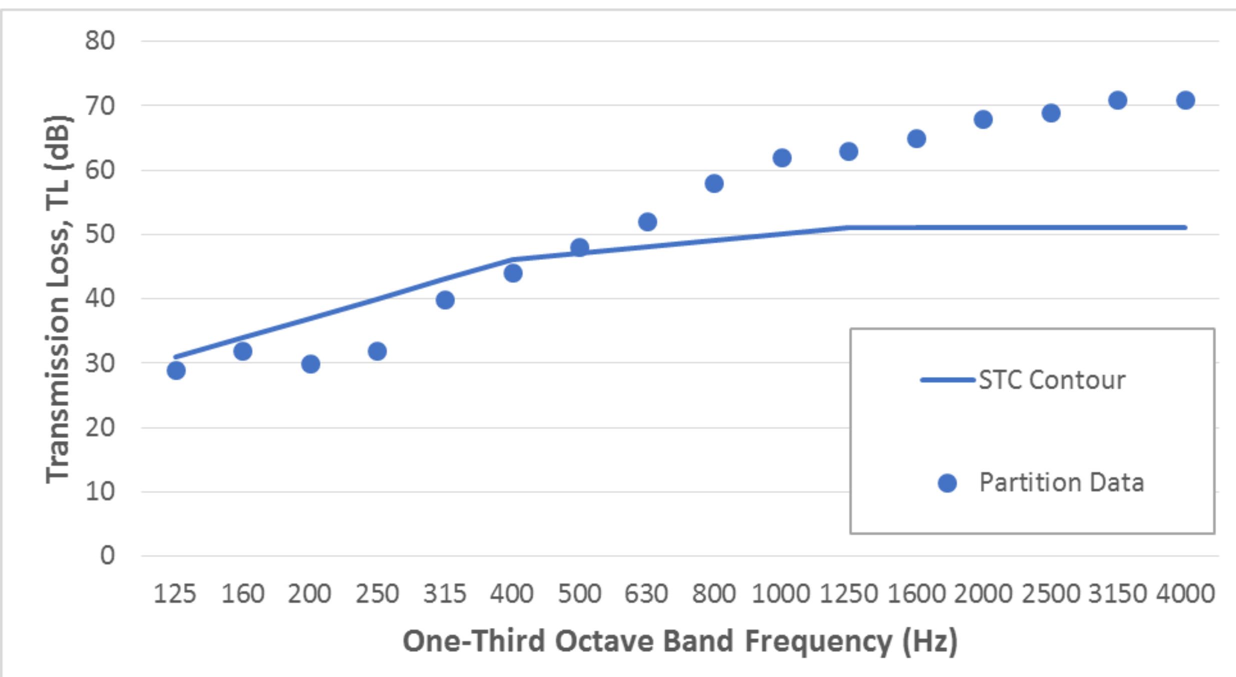
6. Mechanical Breadth

This mechanical breadth investigates how the acoustical performance is affected by changing the structure from steel to concrete. Sound Transmission Class (STC) is a rating that shows how well a building partition or floor/ceiling absorbs sound. The larger the rating, the more sound that is attenuated. With all buildings, it is important that the STC is high enough so that way people can't hear people talking or walking above or below them. Section 1207 of the 2012 IBC requires a STC rating of at least 50 as code minimum.

Architectural Acoustics: Principles and Design by Madam Mehta, Jim Johnson, and Jorge Rocafort is used to determine the sound transmission loss data for the two systems. Appendix J provides data for many different walls, slabs, and roofs, however does not have data for the existing deck (3 ¼" LW concrete topping on 3" 20 gage metal deck) or the proposed system (voided concrete slab). Therefore the assembly which most closely represents the existing and proposed system is used. For the existing system, a 22 gage corrugated steel deck is used, and for the proposed system, a 6" solid concrete slab is used.

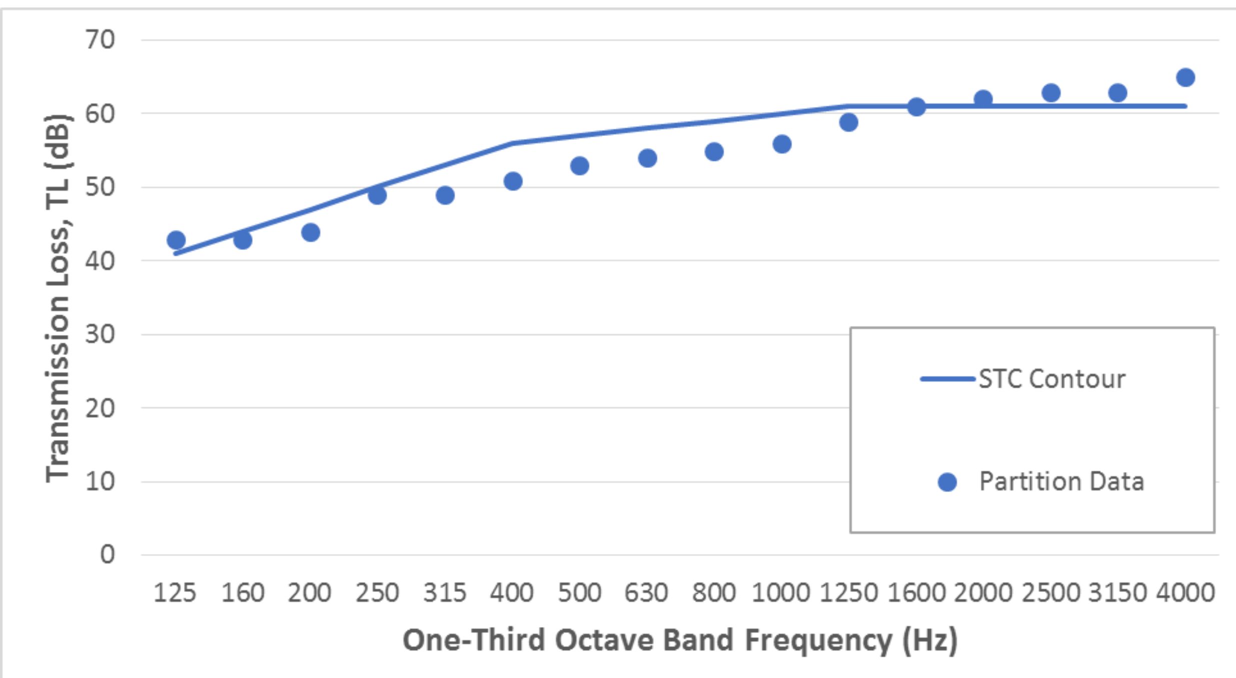
1/3 Octave-Band Frequency (Hz)	Contour Level (dB)	TL Partition (dB)	Deficiency	Exceeds Max Def?
125	31	29.0	2	NO
160	34	32.0	2	NO
200	37	30.0	7	NO
250	40	32.0	8	NO
315	43	40.0	3	NO
400	46	44.0	2	NO
500	47	48.0		NO
630	48	52.0		NO
800	49	58.0		NO
1000	50	62.0		NO
1250	51	63.0		NO
1600	51	65.0		NO
2000	51	68.0		NO
2500	51	69.0		NO
3150	51	71.0		NO
4000	51	71.0		NO

TOTAL DEFICIENCIES:	24
HOW MANY EXCEED?:	0
PARTITION STC IS:	47



1/3 Octave-Band Frequency (Hz)	Contour Level (dB)	TL Partition (dB)	Deficiency	Exceeds Max Def?
125	41	43.0		NO
160	44	43.0	1	NO
200	47	44.0	3	NO
250	50	49.0	1	NO
315	53	49.0	4	NO
400	56	51.0	5	NO
500	57	53.0	4	NO
630	58	54.0	4	NO
800	59	55.0	4	NO
1000	60	56.0	4	NO
1250	61	59.0	2	NO
1600	61	61.0		NO
2000	61	62.0		NO
2500	61	63.0		NO
3150	61	63.0		NO
4000	61	65.0		NO

TOTAL DEFICIENCIES:	32
HOW MANY EXCEED?:	0
PARTITION STC IS:	57



As stated at the beginning of this section, the sound transmission loss data for the exact systems were not provided in the textbook. Therefore these STC ratings are only rough estimates. The existing composite deck has an STC of 47 while the voided slab has an STC of 57. The voided slab has about 6" of solid concrete above and below the voids, but the presence of these voids will help alter the sound wave propagation and provide more absorption. As a result, the voided slab would have an STC greater than 57. The exact rating is unknown as there would have to be test data specifically for a 17.5" voided concrete slab to determine it.

Although this analysis is not an exact measurement, it is a safe assumption to make that the STC will increase with the voided concrete slab. In a building with many classrooms, research labs, and offices, it is important to minimize sound transmission between floors. Therefore, the acoustical performance would increase with the introduction of a voided slab.

7. Summary

The voided concrete slab is an innovative slab system that presents many advantages over a solid slab and steel system. The flat plate reduces the depth of the structure and the overall height compared to a steel building. In addition, the voids reduce the self-weight of the slab and allow it to span longer distances helping reduce the number of columns throughout the building. This reduction of columns also creates more open spacious floor plans.

The gravity system includes a 17.5” thick voided slab with 12 3/8” void formers. The top reinforcement consist of mostly #7 bars in each direction, with #8 and #9 bars at longer spans which experience greater moment. The bottom reinforcement consists of a mat of #7@12” each way. Flexure, punching shear, and deflections have been determined to be adequate through RAM and hand calculations. Exterior columns are 30”x30” and interior columns are 24”x24”. The depth at each level was reduced by 1’, reducing the overall height of the building from 118’-8” to 111’-8”.

The lateral system consists of 12” thick reinforced concrete shear walls. Seismic loads were the controlling lateral load case. The shear walls were needed to resist more load in the north-south direction as wind controlled in that direction over east-west. One shear wall was designed by hand and determined to have adequate axial/flexural and shear strength while the remaining walls were designed in RAM. The shear walls were also determined to be within acceptable drift limits.

The cost of the structure decreased by 31% while switching from steel to concrete. In addition to the reduction of cost of the structure, the costs of façade, ductwork, and piping also decreases due to the reduction of the overall height of the building. The acoustical performance of the slab also increases resulting in less sound transmission loss.

The primary goal with this redesign was to reduce the depth of the structure and the overall cost of the building, and both of these goals have been met. However there are several drawbacks to this system. An increase in the total weight of the structure will result in larger foundations, and the project would take longer to construct. After extensive analysis, the voided concrete slab and shear wall system would be an acceptable alternate design. The decision to use the existing or proposed system would ultimately come down to the digression of the owner.

Appendix A: Column References

Concrete Column Design

COLUMN INFORMATION:

Level	1st Floor		
Column Number:	19 Column E2	Grid Location:	(182.41ft-173.79ft)
Size:	**30x30**	Depth x Width (in)	30.00x30.00
Reinforcement			
Longitudinal:	**28-#9 (8 x 6)**	As (in^2)	28.00 (3.11%)
Transverse:	**#3@ 18.0" 0'-0"-24'-6"**		
Confinement	Tie	Clear Cover (in)	1.50
Shear Legs Major	2	Shear Legs Minor	2

Longitudinal Bars Max Tension Stress Ratio: 0.00

MATERIAL PROPERTIES:

f'c (ksi):	8.00	fy Long (ksi):	60.00
fct (ksi):	0.00	fyt Trans (ksi):	60.00
Conc. Weight (pcf):	145.00	Conc. Type:	NWC
Conc. Modulus (ksi):	5153.60	Reinf. Modulus (ksi):	29000.00

DESIGN PARAMETERS:

	Major	Minor
Unbraced Length (ft)	24.50	24.50
K	1.91	1.91
Braced Against Sidesway	No	No

LONGITUDINAL REINFORCEMENT:

Controlling Load Combination: (296) 1.200 D + 1.600 Lp

Axial		Load (kip)	3883.08
Moment	Top	Major(kip-ft)	-134.09
		Minor(kip-ft)	-38.85
Moment	Bottom	Major(kip-ft)	39.36
		Minor(kip-ft)	11.22

Calculated Parameters (Angle = 16.16 degrees): Ld/Cap = 0.98

0.65 Pn(kip): 3883.08

0.65 Mn Major(kip-ft): 1018.17 | 0.65 Mn Minor(kip-ft): 295.02

	Major	Minor
Kl/r	64.85	64.85
Slender	Yes	Yes
10.13.5: lu/r > limit	No	No

TRANSVERSE REINFORCEMENT:

Controlling Load Combination: (2) 1.200 D + 1.600 Lp

	Vu (kip)	Vc (kip)	Vs (kip)	ϕ	ϕ (Vc + Vs) (kip)	Ld/Cap
1 Major:	8.25	348.60	20.90	0.75	277.12	0.03
1 Minor:	2.37	348.60	20.90	0.75	277.12	0.01

TORSION CAPACITY:

Controlling Load Combination: (2) 1.200 D + 1.600 Lp

0.75 Tn (kip-ft) 136.36 | Tu (kip-ft) 0.05

Concrete Column Design

COLUMN INFORMATION:

Level	1st Floor		
Column Number:	18 Column E1	Grid Location:	(157.87ft-138.00ft)
Size:	**24x24**	Depth x Width (in)	24.00x24.00
Reinforcement			
Longitudinal:	**8-#8 (3 x 1)**	As (in^2)	6.32 (1.10%)
Transverse:	**#3@ 15.0" 0'-0"-24'-6"**		
Confinement	Tie	Clear Cover (in)	1.50
Shear Legs Major	2	Shear Legs Minor	2

Longitudinal Bars Max Tension Stress Ratio: 0.00

MATERIAL PROPERTIES:

f'c (ksi):	8.00	fy Long (ksi):	60.00
fct (ksi):	0.00	fyt Trans (ksi):	60.00
Conc. Weight (pcf):	145.00	Conc. Type:	NWC
Conc. Modulus (ksi):	5153.60	Reinf. Modulus (ksi):	29000.00

DESIGN PARAMETERS:

	Major	Minor
Unbraced Length (ft)	24.50	24.50
K	1.91	1.91
Braced Against Sidesway	No	No

LONGITUDINAL REINFORCEMENT:

Controlling Load Combination: (296) 1.200 D + 1.600 Lp

Axial		Load (kip)	1750.96
Moment	Top	Major(kip-ft)	269.68
		Minor(kip-ft)	-39.21
Moment	Bottom	Major(kip-ft)	-77.09
		Minor(kip-ft)	11.34

Calculated Parameters (Angle = 8.27 degrees): Ld/Cap = 0.79

0.65 Pn(kip):	1750.96		
0.65 Mn Major(kip-ft):	640.34	0.65 Mn Minor(kip-ft):	93.10

	Major	Minor
Kl/r	81.06	81.06
Slender	Yes	Yes
10.13.5: lu/r > limit	No	No

TRANSVERSE REINFORCEMENT:

Controlling Load Combination: (2) 1.200 D + 1.600 Lp

	LC	Vu (kip)	Vc (kip)	Vs (kip)	ϕ	ϕ (Vc + Vs) (kip)	Ld/Cap
1 Major:	0	16.43	233.95	19.03	0.75	189.74	0.09
1 Minor:	0	2.40	233.95	19.03	0.75	189.74	0.01

TORSION CAPACITY:

Controlling Load Combination: (2) 1.200 D + 1.600 Lp

0.75 Tn (kip-ft)	59.54	Tu (kip-ft)	0.01

Column E2 Summary

No.	Level	Section	f'c	Longitudinal	Rho %	Ld/Cap	Transverse	Ld/Cap
19	Roof	30x30	8.00	28-#6 (8 x 6)	1.37	0.25	#3@ 12.0" 0'-0"-18'-10"	0.12
19	Penthouse	30x30	8.00	28-#6 (8 x 6)	1.37	0.31	#3@ 12.0" 0'-0"-13'-8"	0.13
19	5th	30x30	8.00	28-#6 (8 x 6)	1.37	0.47	#3@ 12.0" 0'-0"-13'-8"	0.11
19	4th	30x30	8.00	28-#6 (8 x 6)	1.37	0.62	#3@ 12.0" 0'-0"-13'-8"	0.09
19	3rd	30x30	8.00	28-#6 (8 x 6)	1.37	0.78	#3@ 12.0" 0'-0"-13'-8"	0.08
19	2nd Floor	30x30	8.00	28-#6 (8 x 6)	1.37	0.94	#3@ 12.0" 0'-0"-13'-8"	0.08
19	1st Floor	30x30	8.00	28-#9 (8 x 6)	3.11	0.98	#3@ 18.0" 0'-0"-24'-6"	0.02

Column E3 Summary

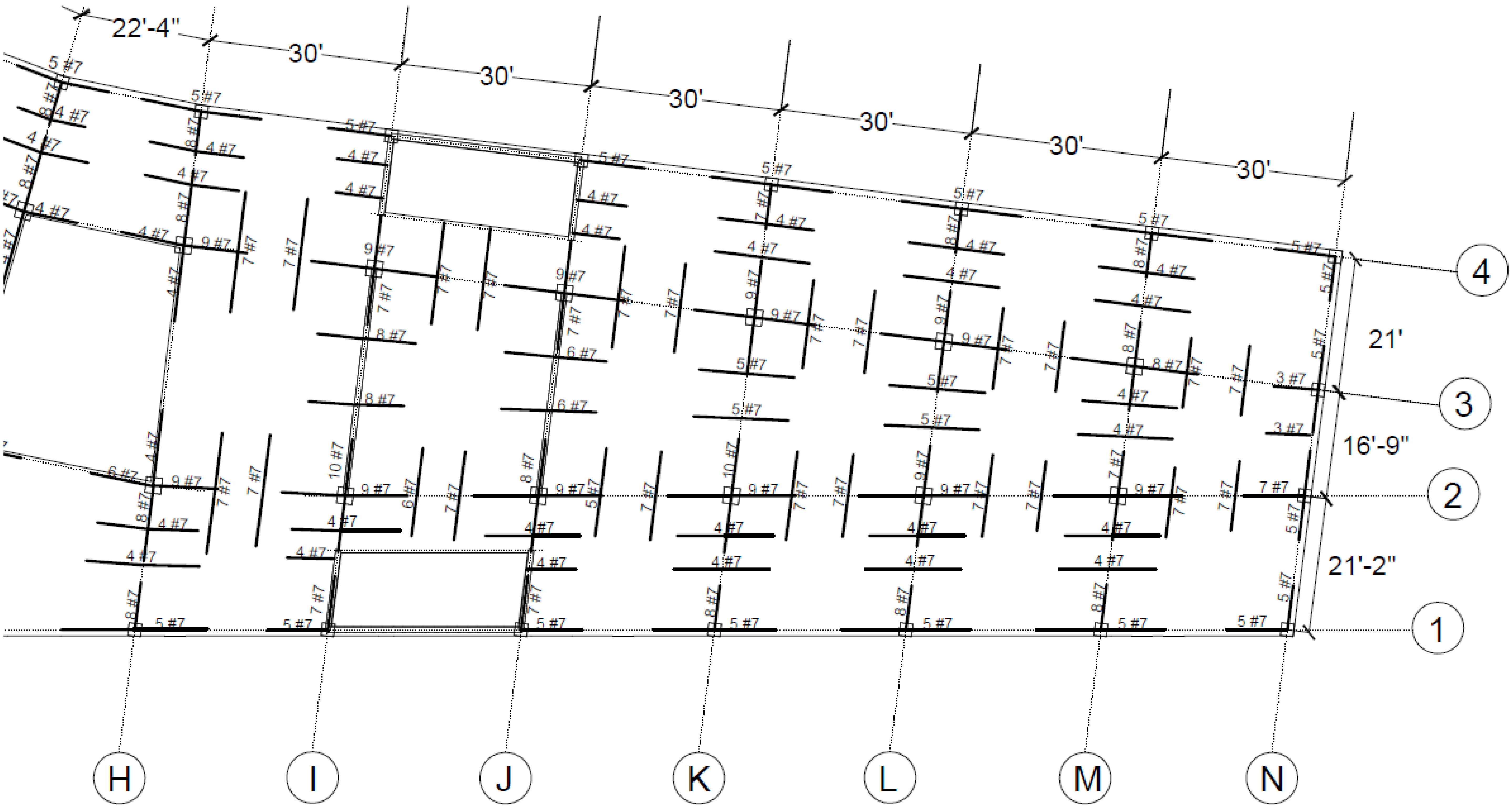
No.	Level	Section	f'c	Longitudinal	Rho %	Ld/Cap	Transverse	Ld/Cap
20	Roof	30x30	8.00	12-#8 (4 x 2)	1.05	0.17	#3@ 15.0" 0'-0"-18'-10"	0.07
20	Penthouse	30x30	8.00	12-#8 (4 x 2)	1.05	0.17	#3@ 15.0" 0'-0"-13'-8"	0.09
20	5th	30x30	8.00	12-#8 (4 x 2)	1.05	0.26	#3@ 15.0" 0'-0"-13'-8"	0.08
20	4th	30x30	8.00	12-#8 (4 x 2)	1.05	0.35	#3@ 15.0" 0'-0"-13'-8"	0.07
20	3rd	30x30	8.00	12-#8 (4 x 2)	1.05	0.44	#3@ 15.0" 0'-0"-13'-8"	0.07
20	2nd Floor	30x30	8.00	12-#8 (4 x 2)	1.05	0.53	#3@ 15.0" 0'-0"-13'-8"	0.08
20	1st Floor	30x30	8.00	12-#8 (4 x 2)	1.05	0.62	#3@ 15.0" 0'-0"-24'-6"	0.02

Column E1 Summary

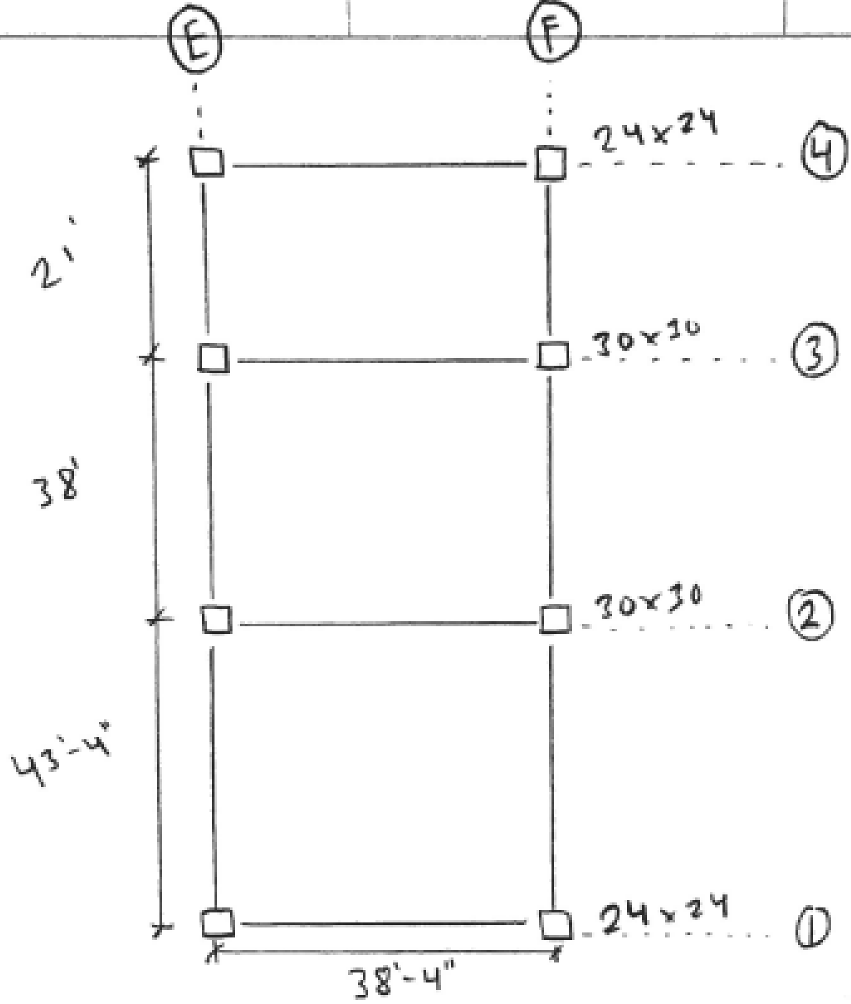
No.	Level	Section	f'c	Longitudinal	Rho %	Ld/Cap	Transverse	Ld/Cap
18	Roof	24x24	8.00	8-#8 (3 x 1)	1.10	1.00	#3@ 6.0" 0'-0"-18'-10"	0.36
18	Penthouse	24x24	8.00	8-#8 (3 x 1)	1.10	0.76	#3@ 6.0" 0'-0"-13'-8"	0.44
18	5th	24x24	8.00	8-#8 (3 x 1)	1.10	0.69	#3@ 15.0" 0'-0"-13'-8"	0.44
18	4th	24x24	8.00	8-#8 (3 x 1)	1.10	0.62	#3@ 15.0" 0'-0"-13'-8"	0.39
18	3rd	24x24	8.00	8-#8 (3 x 1)	1.10	0.56	#3@ 15.0" 0'-0"-13'-8"	0.35
18	2nd Floor	24x24	8.00	8-#8 (3 x 1)	1.10	0.69	#3@ 15.0" 0'-0"-13'-8"	0.36
18	1st Floor	24x24	8.00	8-#8 (3 x 1)	1.10	0.79	#3@ 15.0" 0'-0"-24'-6"	0.09

Appendix B: Slab References

Top Reinforcement- Right Wing



Top Reinforcement-Left Wing



Longiture Direction

Slab thickness = 17.5" with 12 7/8" void formers

Total factored moment:

$$M_o = \frac{q_u \ell_2 \ell_n^2}{8}$$

$q_u = 1.2(10 + 172.5) + 1.6(100) = 379$ psf

$\ell_n = 38' - \frac{30}{12} = 35.5'$

$\ell_2 = 38.33'$

$$M_o = \frac{379 \times 38.33 \times 35.5^2}{8} = 2288.5 \text{ ft-kips}$$

Summary of Design Strip Moments

End Span Moments			Interior Span Moments	
Exterior Negative	Positive	Interior Negative	Positive	Interior Negative
$0.26 M_o = 595^{1k}$	$0.52 M_o = 1190^{1k}$	$0.70 M_o = 1602^{1k}$	$0.35 M_o = 801^{1k}$	$0.65 M_o = 1487.5^{1k}$

* Factors from ACI 13.6.3

Table 13.6.4.1 Portion of Interior Negative M_u in Column Strip

$\alpha_{f2} l_2 / l_1 = 0 \Rightarrow 0.75$

Table 13.6.4.2 Portion of Exterior Negative M_u in Column strip

$\beta_t = 0 \Rightarrow 1.0$

Table 13.6.4.4 Portion of Positive M_u in Column Strip

$\alpha_{f2} l_2 / l_1 = 0 \Rightarrow 0.60$

	End Span Moments			Interior Span Moments	
	Exterior Negative	Positive	Interior Negative	Positive	Interior Negative
Column Strip	$0.26 M_o = 595^{1k}$	$0.31 M_o = 709.4^{1k}$	$0.53 M_o = 1212.9^{1k}$	$0.21 M_o = 480.6^{1k}$	$0.49 M_o = 1121.4^{1k}$
Middle Strip		$0.21 M_o = 480.6^{1k}$	$0.17 M_o = 389^{1k}$	$0.14 M_o = 320.4^{1k}$	$0.16 M_o = 366.2^{1k}$

$$\text{Width of Strips} = b = \frac{38'\text{-}4''}{2} = 19'\text{-}2'' = 230''$$

$$d = 17.5'' - 0.75 - \frac{0.75}{2} = 16.375''$$

End Span

Column Strip : Exterior Negative

$$R_n = \frac{M_n}{\phi b d^2} = \frac{595 \times 12{,}000}{0.9(230)(16.375)^2} = 128.6 \text{ psi}$$

$$\rho = \frac{0.85 f'_c}{f_y}\left[1-\sqrt{1-\frac{2R}{0.85 f'_c}}\right] = \frac{0.85(4)}{60}\left[1-\sqrt{1-\frac{2(128.6)}{0.85(4000)}}\right]$$

$$= 0.0022$$

$$A_s = \rho b d = 0.0022(230)(16.375) = 8.29 \text{ in}^2$$

Column Strip : Positive

$$R_n = \frac{709.4 \times 12000}{0.9(230)(16.375)^2} = 153.4 \text{ psi}$$

$$\rho = \frac{0.85(4)}{60}\left[1-\sqrt{1-\frac{2(153.4)}{0.85(4000)}}\right] = 0.0026$$

$$A_s = 0.0026(230)(16.375) = 9.79 \text{ in}^2$$

Column Strip : Interior Negative

$$R_n = \frac{1212.9 \times 12000}{0.9(230)(16.375)^2} = 262.2 \text{ psi}$$

$$\rho = \frac{0.85(4)}{60}\left[1-\sqrt{1-\frac{2(262.2)}{0.85(4000)}}\right] = 0.0046$$

$$A_s = 0.0046(230)(16.375) = 17.3 \text{ in}^2$$

Middle Strip: Positive

$$R_n = \frac{480.6 \times 12000}{0.9(230)(16.375)^2} = 103.9 \text{ psi}$$

$$\rho = \frac{0.85(4)}{60}\left[1 - \sqrt{1 - \frac{2(103.9)}{0.85(4000)}}\right] = 0.0018$$

$$A_s = 0.0018(230)(16.375) = 6.78 \text{ in}^2$$

$$A_{s\,min} = 0.0018bh = 0.0018(230)(17.5) = 7.25 \text{ in}^2 \Leftarrow \text{governs}$$

Middle Strip: Interior Negative

$$R_n = \frac{389 \times 12000}{0.9(230)(16.375)^2} = 84.1 \text{ psi} < 103.9 \text{ psi}$$

$\therefore$ use $A_{s\,min}$

$$A_s = A_{s\,min} = 7.25 \text{ in}^2$$

Interior Span

Column Strip: Positive

$$R_n = \frac{480.6 \times 12000}{0.9(230)(16.375)^2} = 103.9 \text{ psi}$$

$A_s = A_{s\,min} = 7.25 \text{ in}^2$

Column Strip: Negative

$$R_n = \frac{1121.4 \times 12000}{0.9(230)(16.375)^2} = 242.4 \text{ psi}$$

$$\rho = \frac{0.85(4)}{60}\left[1 - \sqrt{1 - \frac{2(242.4)}{0.85(4000)}}\right] = 0.0042$$

$A_s = 0.0042(230)(16.375) = 15.8 \text{ in}^2$

Middle Strip: Positive

$$R_n = \frac{320.4 \times 12000}{0.9(230)(16.375)^2} = 69.3 \text{ psi} < 103.9 \text{ psi}$$

$\therefore$ use $A_{s\,min}$

$A_s = A_{s\,min} = 7.25 \text{ in}^2$

Middle Strip: Negative

$$R_n = \frac{366.2 \times 12000}{0.9(230)(16.375)^2} = 79.2 \text{ psi} < 103.9 \text{ psi}$$

$\therefore$ use $A_{s\,min}$

$A_s = A_{s\,min} = 7.25 \text{ in}^2$

Latitude Direction

$q_u = 379$ psf

$\ell_n = 38.33 - \frac{30}{12} = 35.83'$

$\ell_2 = 38'$

$$M_o = \frac{379 \times 38 \times 35.83^2}{8} = 2311.1 \text{ ft-kips}$$

* No end spans in the latitude direction so only interior spans analyzed

	Positive	Negative
Column Strip	$0.21 M_0 = 485.3^{1K}$	$0.49 M_0 = 1132.4^{1K}$
Middle Strip	$0.14 M_0 = 323.6^{1K}$	$0.16 M_0 = 369.8^{1K}$

$$\text{width of strip} = b = \frac{38'}{2} = 19' = 228''$$

Column Strip: Positive

$$R_n = \frac{485.3 \times 12000}{0.9(228)(16.375)^2} = 105.8 \text{ psi}$$

$$\rho = \frac{0.85(4)}{60}\left[1 - \sqrt{1 - \frac{2(105.8)}{0.85(4000)}}\right] = 0.0018$$

$A_s = 0.0018(228)(16.375) = 6.72 \text{ in}^2$

$A_{s\,min} = 0.0018bh = 0.0018(228)(17.5) = 7.18 \text{ in}^2 \Leftarrow$ governs

Column Strip: Negative

$$R_n = \frac{1132.4 \times 12000}{0.9(228)(16.375)^2} = 247.0 \text{ psi}$$

$$\rho = \frac{0.85(4)}{60}\left[1 - \sqrt{1 - \frac{2(247.0)}{0.85(4000)}}\right] = 0.0043$$

$$A_s = 0.0043(228)(16.375) = 16.1 \text{ in}^2$$

Middle Strip: Positive

$$R_n = \frac{323.6 \times 12000}{0.9(228)(16.375)^2} = 70.6 \text{ psi} < 105.8 \text{ psi}$$

$\therefore$ use $A_{s\,min}$

$$A_s = A_{s\,min} = 7.18 \text{ in}^2$$

Middle Strip: Negative

$$R_n = \frac{369.8 \times 12000}{0.9(228)(16.375)^2} = 80.7 \text{ psi} < 105.8 \text{ psi}$$

$\therefore$ use $A_{s\,min}$

$$A_s = A_{s\,min} = 7.18 \text{ in}^2$$

Reinforcement Summary

Longitude Direction

			M_u	A_s	Ram Reinforcing	OK?
End span	Column Strip	Exterior negative	-595	8.29	14#9 = 14.0	OK
		Positive	709.4	9.79	mat** + 6#7 = 15.0	OK
		Interior negative	-1212.9	17.3	19#9 =19.0	OK
	Middle Strip	Exterior negative	0	0	-	
		Positive	480.6	7.25*	mat** + 1#7 = 12.0	OK
		Interior negative	-389	7.25*	12#9 =12.0	OK
Interior span	Column Strip	Positive	480.6	7.25*	mat** + 1#7 = 12.0	OK
		Negative	-1121.4	15.8	24#9 = 24.0	OK
	Middle Strip	Positive	320.4	7.25*	mat** = 11.4	OK
		Negative	-366.2	7.25*	22#9 =22.0	OK

Latitude Direction

			M_u	A_s	Ram Reinforcing	OK?
Interior span	Column Strip	Positive	485.3	7.18*	mat** = 11.4	OK
		Negative	-1132.4	16.1	31#7 = 18.6	OK
	Middle Strip	Positive	323.6	7.18*	mat** = 11.4	OK
		Negative	-369.8	7.18*	17#6 = 10.2	OK

* denotes A_s min is used

** mat consits of #7@12" each way

Solid Slab Required

Column	Column size	Tributary Area (ft^2)	b_0 (ft)	ΦV_C	Vu	Solid Area (ft^2)
A1	24x24	216	161.5	501.77	79.31	n/a
A2	30x30	395	185.5	576.34	146.78	n/a
A3	30x30	374	185.5	576.34	138.82	n/a
A4	24x24	182	161.5	501.77	66.43	n/a
B1	24x24	405	161.5	501.77	150.94	n/a
B2	30x30	763	185.5	576.34	286.25	n/a
B3	30x30	754	185.5	576.34	282.84	n/a
B4	24x24	330	161.5	501.77	122.52	n/a
C1	24x24	501	161.5	501.77	187.33	n/a
C2	30x30	670	185.5	576.34	251.00	n/a
C3	30x30	833	185.5	576.34	312.78	n/a
C4	24x24	338	161.5	501.77	125.55	n/a
D1	24x24	623	161.5	501.77	233.57	n/a
D2	30x30	749	185.5	576.34	280.94	n/a
D3	30x30	769	185.5	576.34	288.52	n/a
D4	24x24	303	161.5	501.77	112.29	n/a
E1	24x24	929	161.5	501.77	349.54	200.84
E2	30x30	1286	185.5	576.34	484.46	449.63
E3	30x30	757	185.5	576.34	283.97	n/a
E4	24x24	270	161.5	501.77	99.78	n/a
F1	24x24	894	161.5	501.77	336.28	165.84
F2	30x30	1286	185.5	576.34	484.46	449.63
F3	30x30	774	185.5	576.34	290.42	n/a
F4	24x24	272	161.5	501.77	100.54	n/a
G1	24x24	710	161.5	501.77	266.54	n/a
G2	30x30	690	185.5	576.34	258.58	n/a
G3	30x30	542	185.5	576.34	202.49	n/a
G4	24x24	238	161.5	501.77	87.65	n/a

Solid Slab Required (Cont.)

Column	Column size	Tributary Area (ft^2)	b_0 (ft)	ΦV_C	Vu	Solid Area (ft^2)
H1	24x24	497	161.5	501.77	185.81	n/a
H2	30x30	582	185.5	576.34	217.65	n/a
H3	30x30	602	185.5	576.34	225.23	n/a
H4	24x24	291	161.5	501.77	107.74	n/a
I1	24x24	233	161.5	501.77	85.76	n/a
I2	30x30	836	185.5	576.34	313.91	n/a
I3	30x30	732	185.5	576.34	274.50	n/a
I4	24x24	230	161.5	501.77	84.62	n/a
J1	24x24	220	161.5	501.77	80.83	n/a
J2	30x30	815	185.5	576.34	305.96	n/a
J3	30x30	825	185.5	576.34	309.75	n/a
J4	24x24	223	161.5	501.77	81.97	n/a
K1	24x24	445	161.5	501.77	166.10	n/a
K2	30x30	614	185.5	576.34	229.78	n/a
K3	30x30	622	185.5	576.34	232.81	n/a
K4	24x24	307	161.5	501.77	113.80	n/a
L1	24x24	427	161.5	501.77	159.28	n/a
L2	30x30	595	185.5	576.34	222.58	n/a
L3	30x30	706	185.5	576.34	264.64	n/a
L4	24x24	318	161.5	501.77	117.97	n/a
M1	24x24	433	161.5	501.77	161.56	n/a
M2	30x30	571	185.5	576.34	213.48	n/a
M3	30x30	559	185.5	576.34	208.93	n/a
M4	24x24	393	161.5	501.77	146.40	n/a
N1	24x24	231	161.5	501.77	85.00	n/a
N2	30x30	292	185.5	576.34	107.74	n/a
N3	30x30	315	185.5	576.34	116.46	n/a
N4	24x24	176	161.5	501.77	64.15	n/a

Appendix C: Lateral References

Wind Load Pressures- North-South Direction

Step 1:	Risk Category	III
Step 2:	V	120
Step 3:	K_d	0.85
	Exposure Category	B
	Kzt	1.00
	G	0.805
	Enclosed	
	Gcpi = +/-	0.18

Building Geometry

l	245
B	380
h	111.68

Gust Effect Factor Calculation

Iz calculation

c	0.3
z bar	67.008
Iz	0.2666

g_r calculation

g_r	4.102213575

Q calculation

l	245
z bar	67.008
ε	0.333333333
L_z bar	310.2441894
B	380
h	111.68
Q	0.736803572

G_f	**0.805**

R calculation

α bar	0.25
b bar	0.45
V_z	94.54278
β	0.015
B	380
L	245
h	111.68
N_1	2.284
R_n	0.082
η_h	3.783
R_h	0.229
η_B	12.870
R_B	0.075
η_L	27.780
R_L	0.035
R	0.227

Story	Height z (ft)	Story Height (ft)	Kz	Kd	Kzt	qz (psf)
Ground	0	24.5	0.570	0.85	1	**17.9**
1	24.5	13.67	0.656	0.85	1	**20.6**
2	38.17	13.67	0.749	0.85	1	**23.5**
3	51.83	13.67	0.817	0.85	1	**25.6**
4	65.5	13.67	0.872	0.85	1	**27.3**
5	79.17	13.67	0.927	0.85	1	**29.0**
Penthouse	92.83	18.83	0.968	0.85	1	**30.3**
Roof	111.68		1.019	0.85	1	**31.9**

Wind Load Pressures- East-West Direction

Step 1:	Risk Category	III
Step 2:	V	120
Step 3:	K_d	0.85
	Exposure Category	B
	Kzt	1.00
	G	0.821
	Enclosed	
	Gcpi = +/-	0.18

Building Geometry

l	380
B	245
h	111.68

Gust Effect Factor Calculation

Iz calculation

c	0.3
z bar	67.008
Iz	0.2666

g_r calculation

g_r	4.377650709

Q calculation

l	380
z bar	67.008
ε	0.333333333
L_z bar	481.1950693
B	245
h	111.68
Q	0.810656148

G_f	**0.821**

R calculation

α bar	0.25
b bar	0.45
V_z	94.54278
β	0.015
B	245
L	380
h	111.68
N_1	11.407
R_n	0.030
η_h	12.178
R_h	0.079
η_B	26.715
R_B	0.037
η_L	138.720
R_L	0.007
R	0.055

Story	Height z (ft)	Story Height (ft)	Kz	Kd	Kzt	qz (psf)
Ground	0	24.5	0.570	0.85	1	**17.9**
1	24.5	13.67	0.656	0.85	1	**20.6**
2	38.17	13.67	0.749	0.85	1	**23.5**
3	51.83	13.67	0.817	0.85	1	**25.6**
4	65.5	13.67	0.872	0.85	1	**27.3**
5	79.17	13.67	0.927	0.85	1	**29.0**
Penthouse	92.83	18.83	0.968	0.85	1	**30.3**
Roof	111.68		1.019	0.85	1	**31.9**

Seismic Weight Calculation

Area A & B

Level	Story Height (ft)	Area (ft^2)	Perimeter (ft)	Total Dead Load (PSF)	Exterior Wall Load (PSF)	Story Weight W (kips)
1st	13.67	32300	921.25	182.5	15	6083.65
2nd	13.67	32300	921.25	182.5	15	6083.65
3rd	13.67	32300	921.25	182.5	15	6083.65
4th	13.67	32300	921.25	182.5	15	6083.65
5th	13.67	32300	921.25	182.5	15	6083.65
Penthouse	18.83	32300	921.25	182.5	15	6154.96
Roof		32300	921.25	245.5	0	7929.65
					Total	44502.87

Area C

Level	Story Height (ft)	Area (ft^2)	Perimeter (ft)	Total Dead Load (PSF)	Exterior Wall Load (PSF)	Story Weight W (kips)
1st	13.67	14511	535.33	182.5	45	2977.57
Roof		14511	535.33	195	45	2829.65
					Total	5807.21
					Total Seismic Weight (kips)	50310.08

Shear Wall Stiffness

$f'c = 4000$ psi

Normal Weight Concrete

$t = 12''$ => all thickness are the same so $t = 1''$

$E_c = 57000\sqrt{f'c} = 57000\sqrt{4000} = 3605$ ksi

$G \approx 0.4E = 0.4(3605) = 1442$ ksi

$$K = \frac{E}{\left(\frac{h}{b}\right)^3 + 3\left(\frac{h}{b}\right)}$$

$$K_{sw1,3} = \frac{3605}{\left(\frac{13.67}{30}\right)^3 + 3\left(\frac{13.67}{30}\right)} = 2466.5 \text{ K/in}$$

$$K_{sw2} = \frac{3605}{\left(\frac{13.67}{13.17}\right)^3 + 3\left(\frac{13.67}{13.17}\right)} = 851.8 \text{ K/in}$$

$$K_{sw4,6} = \frac{3605}{\left(\frac{13.67}{21}\right)^3 + 3\left(\frac{13.67}{21}\right)} = 1617.5 \text{ K/in}$$

$$K_{sw5} = \frac{3605}{\left(\frac{13.67}{30}\right)^3 + 3\left(\frac{13.67}{30}\right)} = 2466.5 \text{ K/in}$$

$$K_{sw\,7,9} = \frac{3605}{\left(\frac{13.67}{12.5}\right)^3 + 3\left(\frac{13.67}{12.5}\right)} = 785.6 \text{ K/in}$$

$$K_{sw\,8} = \frac{3605}{\left(\frac{13.67}{30}\right)^3 + 3\left(\frac{13.67}{30}\right)} = 2466.5 \text{ K/in}$$

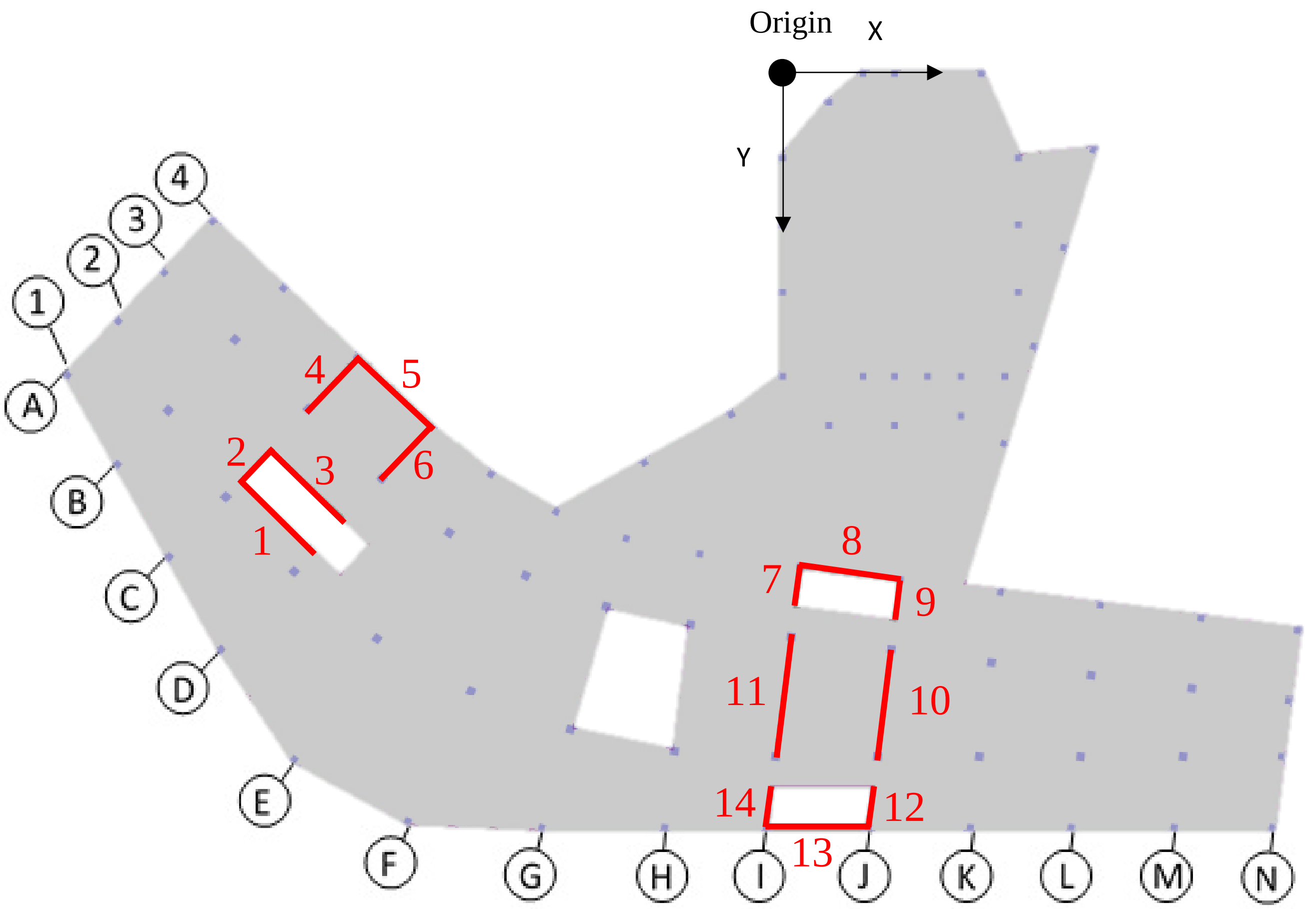
$$K_{sw\,10} = \frac{3605}{\left(\frac{13.67}{32}\right)^3 + 3\left(\frac{13.67}{32}\right)} = 2651.7 \text{ K/in}$$

$$K_{sw\,11} = \frac{3605}{\left(\frac{13.67}{35.83}\right)^3 + 3\left(\frac{13.67}{35.83}\right)} = 3003.9 \text{ K/in}$$

$$K_{sw\,12,14} = \frac{3605}{\left(\frac{13.67}{12.67}\right)^3 + 3\left(\frac{13.67}{12.67}\right)} = 802.4 \text{ K/in}$$

$$K_{sw\,13} = \frac{3605}{\left(\frac{13.67}{30.25}\right)^3 + 3\left(\frac{13.67}{30.25}\right)} = 2489.7 \text{ K/in}$$

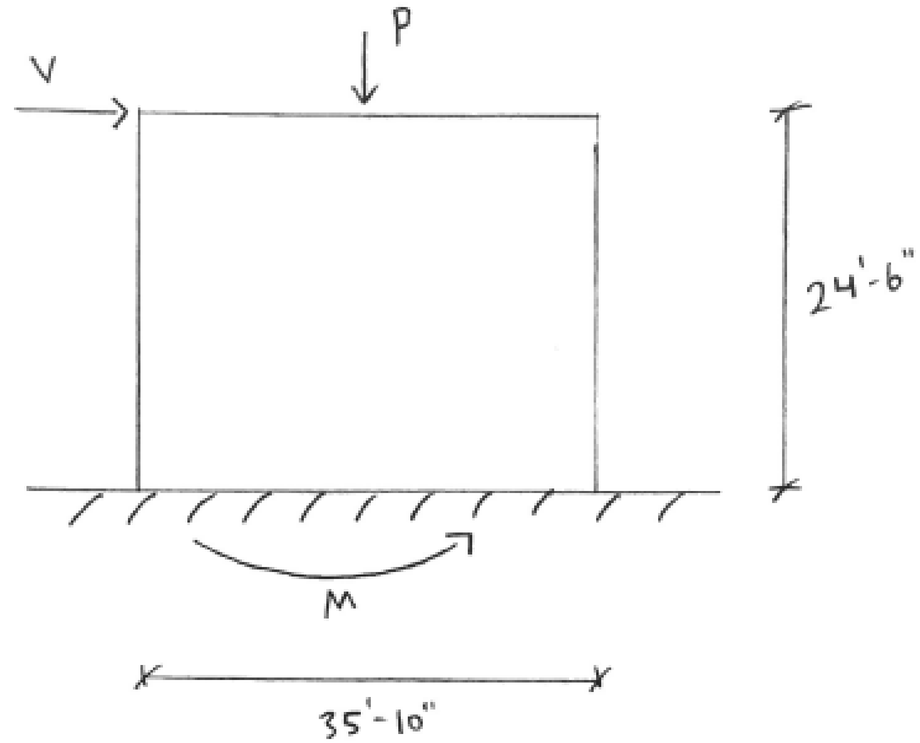
Center of Rigidity Calculation



		Distance from Datum					
Shear Wall Number	Element Direction	x	y	Rx	Ry	Rx*Y	Ry*X
1	x		146.67	1803.88		264575.81	
	y	-149.33			1682.15		-251195.13
2	x		134.5	580.93		78134.63	
	y	-158.67			622.97		-98846.12
3	x		138.25	1803.88		249387.10	
	y	-141			1682.15		-237182.84
4	x		107	1103.13		118035.25	
	y	-132.33			1182.96		-156541.61
5	x		110.5	1803.88		199329.29	
	y	-113			1682.15		-190082.70
6	x		127.75	1103.13		140925.26	
	y	-110.25			1182.96		-130421.76
7	x		167.83	95.74		16068.30	
	y	4.67			779.74		3641.41
8	x		163.5	2448.12		400266.82	
	y	20.5			300.59		6162.10
9	x		171.75	95.74		16443.60	
	y	34.5			779.74		26901.17
10	x		202.17	323.16		65334.10	
	y	31			2631.93		81589.96
11	x		200.25	366.09		73308.92	
	y	1			2981.51		2981.51
12	x		232.75	97.79		22760.37	
	y	26.83			796.42		21367.92
13	x		239	2489.70		595038.30	
14	x		232.83	97.79		22768.20	
	y	-3.5			796.42		-2787.47
			Σ	14212.98	17101.70	2262375.95	-924413.56

Center of Rigidity	x	-54.05
	y	159.18

Note: The origin for hand calculations and for RAM are different. Therefore the coordinates for the center of rigidity are different. The coordinates for the center of rigidity in the lateral depth of the report have been converted to RAM's coordinates to accurately compare hand calculations with RAM.



$f'_c = 4000$ psi

$f_y = 60{,}000$ psi

Minimum Shear Wall Thickness:

$$h = \max \begin{cases} 4'' \\ 1/25 \times (24.5 \times 12) = 12'' \Leftarrow \text{governs} \end{cases}$$

Design Forces

P = 1127.96 Kips
V = 555.3 Kips
m = 30,961.4 ft-Kips

} From RAM * See Appendix

Controlling Axial/Flexural Load Combo: 0.9D - 1.4E

Controlling Shear Load Combo: $1.2\,D + 0.5\,L_R + 1.6\,W$

$$\text{Critical Section} = \begin{cases} l_w/2 = 35.83/2 = 17.92' \\ h/2 = 24.5/2 = 12.25' \leftarrow \text{governs} \end{cases}$$

Shear Wall Check

Maximum Shear Strength needed:

$\phi V_n = \phi 10\sqrt{f'_c} h d$

$d = 0.8 l_w = 0.8(35.83 \times 12) = 343.97''$

$\phi V_n = 0.75(10)\sqrt{4000}(12)(343.97) = 1958^K > V_u = 555.3^K$

$\therefore$ OK

$$V_c = \min \begin{cases} 3.3\lambda\sqrt{f'_c} h d + \dfrac{N_u d}{4 l_w} \\ \left[0.6\lambda\sqrt{f'_c} + \dfrac{l_w\left(1.25\lambda\sqrt{f'_c} + \dfrac{0.2 N_u}{l_w h}\right)}{\dfrac{M_u}{V_u} - \dfrac{l_w}{2}}\right] h d \end{cases}$$

$$= \min \begin{cases} 3.3\sqrt{4000}(12)(343.97) + \dfrac{1127960(343.97)}{4(430)} = 1087.1^k \\ \left[0.6\sqrt{4000} + \dfrac{430\left(1.25\sqrt{4000} + \dfrac{0.2(1127960)}{430(12)}\right)}{\dfrac{371536800}{555300} - \dfrac{430}{2}}\right](12)(343.97) = \boxed{636.5^k} \end{cases}$$

$\therefore$ governs

$V_u = 555.3^K > \dfrac{\phi V_c}{2} = \dfrac{0.75(636.5)}{2} = 238.7^K$

$\therefore$ need Shear RFT

<u>Horizontal Shear Reinforcement</u>

$V_u \le \phi V_n$

$\le \phi(V_c + V_s)$

$\le \phi V_c + \dfrac{\phi A_v f_y d}{s}$

Shear Wall Check

$$\frac{A_v}{s} = \frac{V_u - \phi V_c}{\phi f_y d} = \frac{555.3 - 477.4}{0.75(60)(343.97)} = 0.005 \text{ in}^2/\text{in}$$

$\Rightarrow$ use 2 #4 @ 12"

$$\rho_t = \frac{2 \times 0.20}{12'' \times 12''} = 0.0028 > 0.0025 \quad \therefore OK$$

$$\text{maximum spacing} = \min \left| \begin{array}{l} l_w/5 = 430/5 = 86'' \\ 3h = 3(12) = 36'' \\ 18'' \Leftarrow \text{governs} \quad \therefore OK \end{array} \right.$$

<u>Vertical Shear Reinforcement</u>

$$\rho_l = 0.0025 + 0.5\left(2.5 - \frac{h_w}{l_w}\right)(\rho_t - 0.0025) \geq 0.0025$$

$$= 0.0025 + 0.5\left(2.5 - \frac{24.5}{35.83}\right)(0.0028 - 0.0025)$$

$$= 0.0028$$

$$\text{maximum spacing} = \min \left| \begin{array}{l} l_w/3 = 430/3 = 143'' \\ 3h = 36'' \\ 18'' \Leftarrow \text{governs} \end{array} \right.$$

$\Rightarrow$ use 2 #4 @ 12" ($\rho_l = 0.0028$)

<u>Flexural Reinforcement</u>

$$R_n = \frac{M_u}{\phi b d^2} = \frac{30,961.4 \times 12000}{0.9(12)(343.97)^2} = 290.8 \text{ psi}$$

$$\rho = \frac{0.85 f'_c}{f_y}\left[1 - \sqrt{1 - \frac{2R_n}{0.85 f'_c}}\right]$$

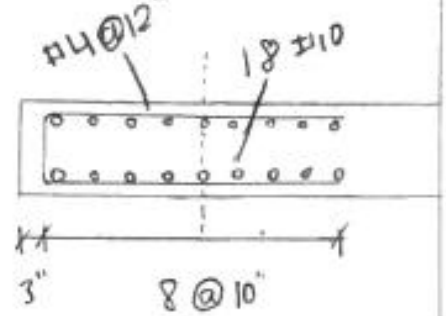
$$= \frac{0.85(4)}{60}\left[1 - \sqrt{1 - \frac{2(290.8)}{0.85(4000)}}\right] = 0.005$$

Shear Wall Check

$A_s = \rho b d = 0.005(12)(343.97) = 20.64\ in^2$

$\Rightarrow$ Use 18 #10 @ 10"

Force in Boundary Element:



$T = A_s f_y = 10 \times 1.27 \times 60 = 762^k$

$d = l_w - (3 + 4(10))$

$= 430 - (3 + 4(10))$

$= 387"$

$$a = \frac{T + N_u}{0.85 f'_c b} = \frac{762 + 1127.96}{0.85(4)(12)} = 46.3"$$

$$c = \frac{a}{\beta_1} = \frac{46.3}{0.85} = 54.5" \qquad \frac{c}{d} = \frac{54.5}{387} = 0.14 < 0.375$$

$\therefore$ tension controlled

$\phi = 0.9$

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

$$= 0.9\left[762\left(387 - \frac{46.3}{2}\right) + 1127.96\left(\frac{430 - 46.3}{2}\right)\right]$$

$= 37{,}024.0$ ft-kips $> 30{,}961.4$ ft-kips $\therefore$ OK

Shear Check

$$V_u\ (\text{cap based}) = \frac{M}{0.5 h_w} = \frac{41{,}137.8\ \text{ft-k}}{0.5(24.5')} = 3358.2\ \text{kips}$$

$A_{cv} = h l_w = 12(430) = 5160\ in^2$

$$V_n = A_{cv}(\alpha_c \lambda \sqrt{f'_c} + \rho_t f_y) \qquad \frac{h_w}{l_w} = \frac{24.5}{35.83} = 0.68 < 1.5$$

$\therefore \alpha_c = 3$

$= 5160(3\sqrt{4000} + 0.0028(60000))$

$= 1845.9$ kips

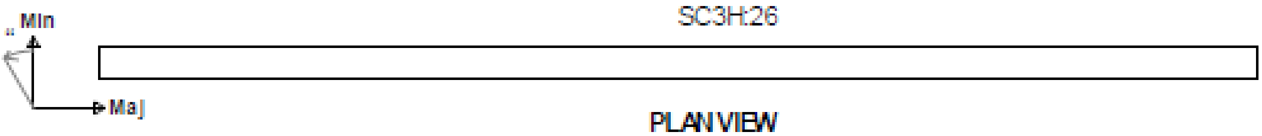
$V_n \le 8 A_{cv}\sqrt{f'_c} = 8(5160)\sqrt{4000} = 2610.8$ kips > 1845.9 kips $\therefore$ OK

$\phi V_n = 0.75(1845.9) = 1384.4$ kips > 555.3 kips $\therefore$ OK

Section Cut Design Summary

RAM Concrete Shearwall 15.04.00.000
Database: Brendan Iribe CCSI Voided Slab_Current 03/27/17 13:36:47
Design Code: ACI 318-11

Section Cut ID: SC3H:26 (Horizontal) Shear Wall 5
Story: 1st Floor
Ag = 5163 in2 Imaj = 79648137 in4 Imin = 61957 in4
Major Axis Orientation: 82.68 degrees (CCW from global X-axis)
Wall Design Group: 3
Design Status: PASS



PLAN VIEW

Axial/Flexural Results:
Interaction: 0.508 OK
Pu = 1127.96 kips phiPn = 2220.64 kips
Mu = 30961.4 kip-ft at Beta = -0.0 deg CCW from Major axis
Controlling Load Combo: 0.900 D - 1.400 E7 (LC 97)
Code Ref: 10.3.7

Shear Results:
Segment SC3H:26:
Length = 35.85 ft Thick = 12.00 in f'c = 4000 psi fy = 60 ksi
Vert Bar Pat: #4@12" Horiz Bar Pat: #4@12"
Vu = 555.3 kip phiVn = 898.7 kip OK
Controlling Load Combo: 1.200 D + 0.500 Lp + 1.600 W14 (LC 4)
Code Ref: 14.2.3 & 11.9.5

Reinforcement Checks:
Min Vert Reinf Ratio: Limit: 0.250% Actual: 0.670% (11.9.9.4) OK
Segment SC3H:26:
Max Vert Bar Spacing Limit: 18.00 in Actual: 12.00 in (11.9.9.5) OK
Min Vert Bar Spacing Limit: 1.00 in Actual: 11.50 in (7.6.1) OK
Min Number of Reinf Curtains: 2 Actual: 2 (14.3.4) OK