Senior Thesis Final Report

Indiana Regional Medical Center

Indiana, PA

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The Pennsylvania State University

Architectural Engineering

Structural Option

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Thesis Abstract

Indiana Regional Medical Center

Indiana, Pennsylvania

Architecture

The Indiana Regional Medical Center is a seven story full-service hospital in the heart of western Pennsylvania. It was first introduced to Indiana, PA in November of 1914. It is a 130,000 square foot facility that stands approximately 90 feet in the air. Its orange brick façade and concrete lintels have lasted throughout its several renovations. Each floor's functionality ranges between patient care, laboratory procedures, and office work.

Structural System

The Indiana Regional Medical Center is constructed of a seven story moment steel frame. A typical bay for this steel structure is 26' by 16'. These bays are typically comprised of W16, W14, and W10 beams and girders at 8' on center. The flooring throughout the building is composed of Composite Steel Deck with 3 ½ " of lightweight 3000 psi concrete fill netting and a total thickness of 5 ½ ". The facility lays on a shallow foundation due to the placement of bedrock.

http://www.engr.psu.edu/ae/thesis/portfolios/2011/cas5180/index

Project Team

Owner: Not Released
Architect: Rea, Hayes, Large, & Suckling
Contractor: Not Selected
Engineer: Rea, Hayes, Large, & Suckling

Building Statistics

Size: 130,000 SF
Construction Dates: TBD
Delivery Method: CM at Risk

MEP Systems

- Rooftop cooling Towers
- Lab exhaust systems provided for specialized areas
- Three diesel powered emergency generators
- Heating, cooling, and ventilation primarily achieved using a variable air volume system
- 480V–208y/120Vpower distribution

Cody Scheller - Penn State Architectural Engineering, Structural Option

Executive Summary

This technical report is the final establishment for senior thesis and includes information on the Indiana Regional Medical Center. This building will be referred to as IRMC throughout the entirety of this document. The report includes a summary of the building's existing structural system, a concrete redesign of the lateral and gravity systems, a vibration analysis, a lighting breadth, and a construction management breadth.

The IRMC is a full-service health facility that rests in rural western Pennsylvania. It consists of six separate buildings with the main seven story building standing 97 feet high. This main central building is 208 feet by 96 feet and contains a concrete footing and pier foundation. A standard bay in the IRMC is 26 feet by 16 feet and is utilized throughout the building. Composite metal decking is used for the floor system and is placed upon a steel support structure. The rest of the gravity systems contain W14 steel columns that vary in weight throughout the building. The lateral system is comprised of steel moment frames and braced moment frames on the perimeter of the building.

Three alternative floor systems were taken into account when redesigning the gravity system of the building. A two-way flat plate system was chosen due to its low floor-to-floor heights and ease of construction. The new concrete system was designed to meet ACI minimum thickness standards and to resist punching shear. Column sizes and reinforcing were then determined using RAM Structural System and hand calculations. Concrete shear walls were utilized in the redesign of the lateral system. Relative stiffness and drift checks were completed with the assistance of an ETABS model.

A lighting breadth was completed that focused on a redesign of a lobby/waiting room area in the facility. New LED luminaires were selected and exchanged for the current fluorescent luminaires. The tasks of the space were redefined and used in the design.

The construction management breadth included a comparison of the existing structure and the new structure of the building in both cost estimate and scheduling aspects.

Building Overview

Figure 1 – Current Entrance to IRMC

Indiana Regional Medical Center (IRMC) is a 140,000 square foot hospital that resides in the heart of western Pennsylvania. It was first introduced to the public in November of 1914 and has seen many renovations and additions throughout its years. It is now the only full service health facility in its county. An elevation can be seen in Figure 1 and an aerial view in Figure 2. This building was designed and erected by Rea, Hayes, Large, & Suckling. This team is also responsible for all this building's renovations, including the most recent one in 1975. Future renovations are now starting to emerge.

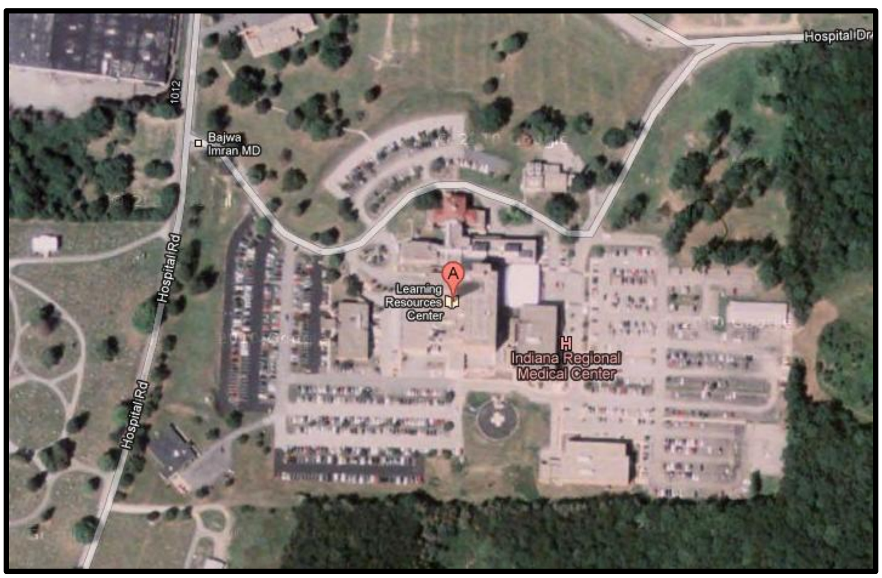


Figure 2 – Site of Indiana Regional Medical Center

This hospital is a combination of six separate buildings that have been made into one single-functioning health care facility. The central building has 7 stories that extend 97 feet in the air and is where most of the patients reside during their stay at the medical center. Even though it is only one of six buildings that make up the Indiana Regional Medical Center, it is where most of the functionalities of the business occur. As seen from Figure 2 above, the hospital resides in a more rural environment.

Existing Structural System

Foundation

The foundation for the IRMC incorporates a traditional foundation method. It is a T-Shaped foundation that consists of a standard sized 16 inch footing placed right below the frost line. And anchor bolts are used to connect the steel structural system to the concrete. A diagram of an anchor bolt used is show in Figure 3 above and an example of a concrete footing can be seen in Figure 4 to the right. These concrete footing characteristics can be found throughout the entire foundation except on the corners. Each corner of the IRMC's foundation has concrete piers in the ground that stand 32 inches high. An example of a concrete pier foundation can be seen in Figure 5.

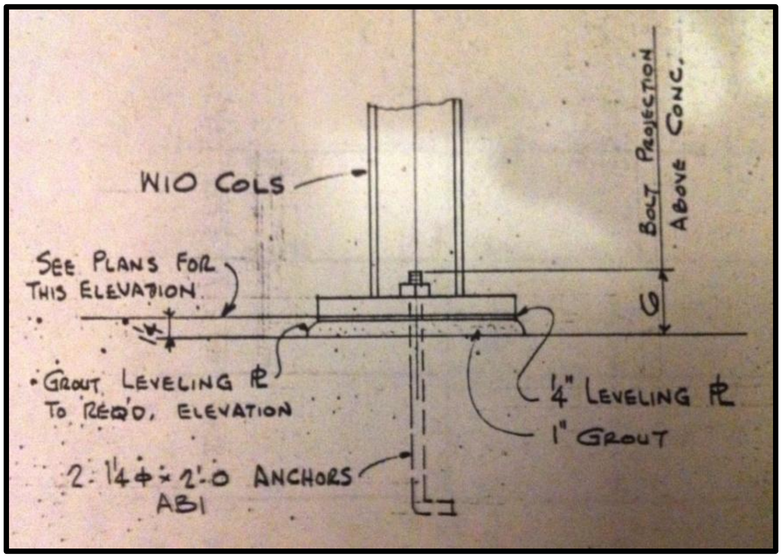


Figure 3 – Anchor Bolt

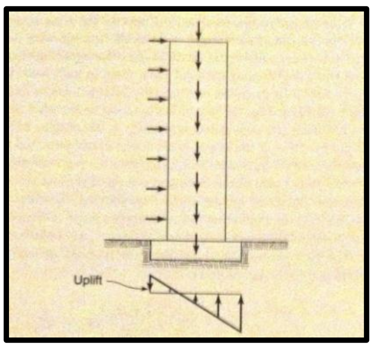


Figure 4 – Concrete Footing


Figure 5 – Concrete Pier

Floor System

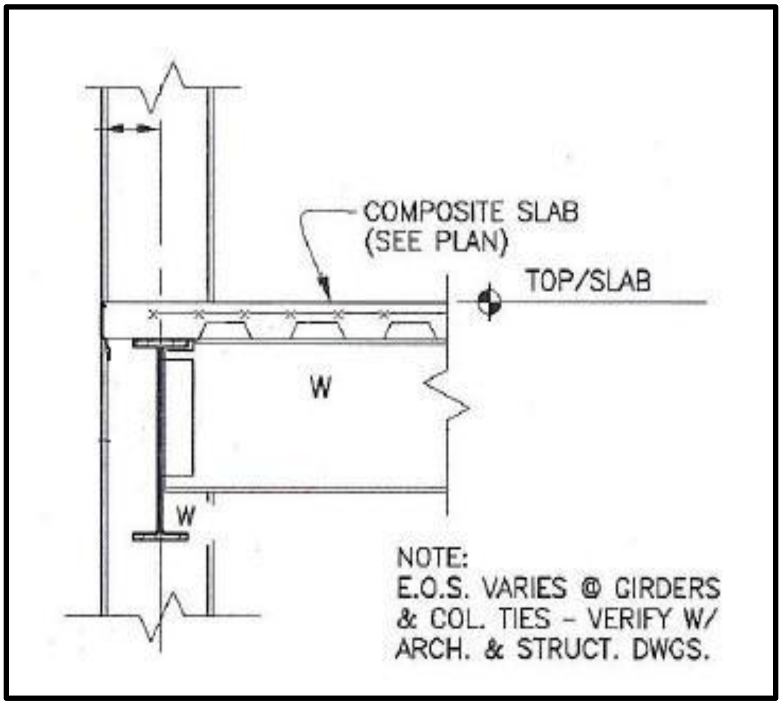


Figure 6 – Composite Steel & Concrete Floor System

The Indiana Regional Medical Center uses a composite steel and concrete floor system with a typical bay size of 26'-0'' x 16'-0''. The composite slab is a Composite Steel Deck with 3 ½'' of lightweight 3000 psi concrete fill netting with a total thickness of 5 ½''. This floor system is throughout the entire building as well as some fill beams to aid with the higher loads. Figure 6 shows a section of this floor system.

Columns

The Indiana Regional Medical Center uses a variety of steel columns to support the gravity load of the building. The sizes of these columns range from W14x38 all the way to a W14x111 throughout the entire structure. The column layout of the building is very symmetrical with its forty-eight 26'-0'' x 16'-0'' bays. The building is supported by these steel columns from the 97 ft height all the way to the ground level. The steel columns within this structural system are typically spliced together at a 24'-0'' distance.

Lateral System

The hospital consists of one large seven story building with five smaller buildings branching off from all sides. Each building is rectangular in shape with a brick façade and has a flat roof. The largest building stands 97 feet in the air and contains a braced steel frame along its perimeter and a steel moment frame. Along its North-South length, the hospital consists of 8 typical bays made up of W10, W14, and W16 steel. The moment frame portion of the building allows more flexibility with the floor plan and the braced frames result in a lower cost than the moment connections. Figure 7 shows a typical floor layout of the IRMC.

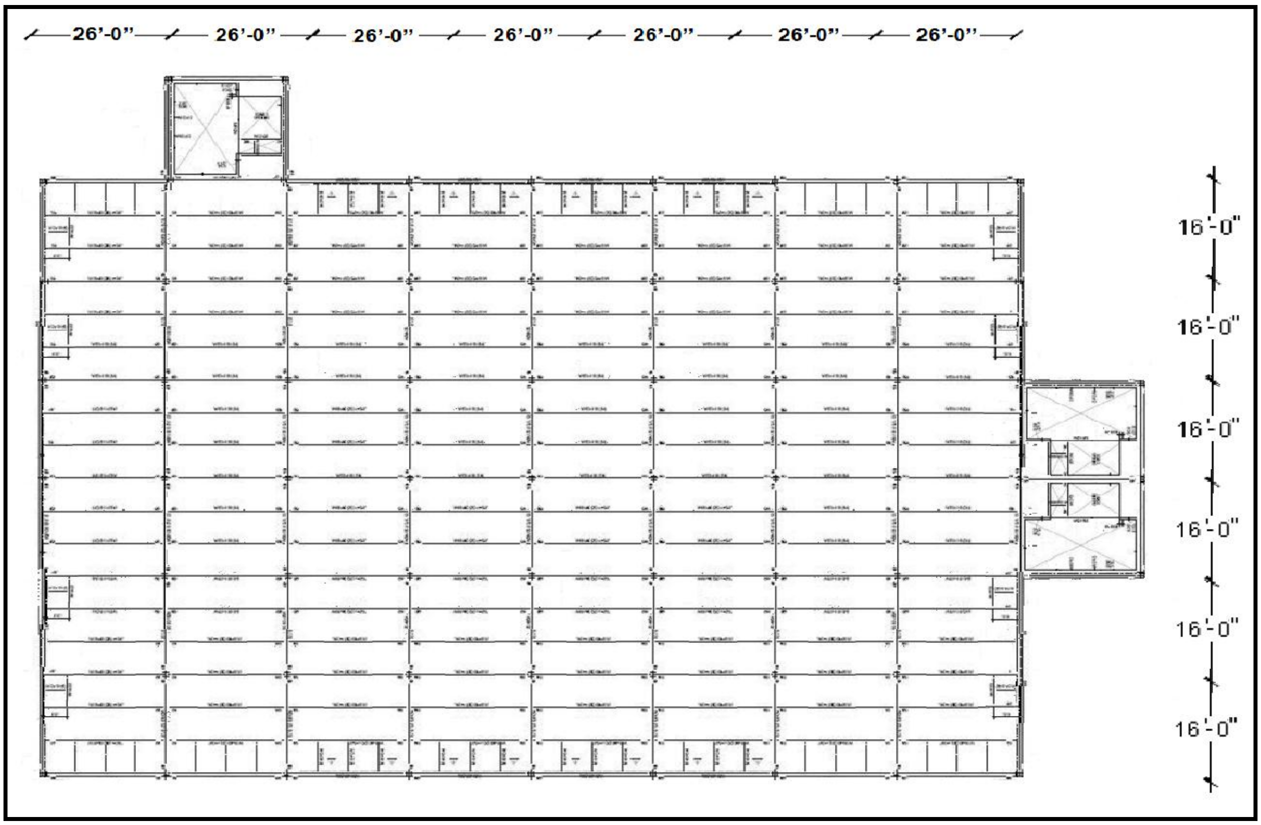


Figure 7 – IRMC Typical Floor Layout

Roof System

The roof structural system is similar to the floor structural system used throughout the Indiana Regional Medical Center. Even though the same composite deck and slab configuration is used, the fill beams are spaced closer together to handle the greater load from the mechanical equipment on the roof.

Codes & References

Design Codes

- ACI 318-05, Building Code Requirement for Structural Concrete
- AISC Steel Construction Manual, 13th Edition
- American Society of Civil Engineers, ASCE/SEI 7-10
- PCI Design Handbook, 6th Edition
- RSMeans Building Construction Cost Data

References

- Reinforced Concrete: Mechanics and Design

Deflection Criteria

- Allowable Story Drift: $0.010h_{sx}$ (Seismic)
- Allowable Building Drift: H/400 (Wind)

Materials

Original Design

Tables 1, 2, and 3 contain information used in the original design.

Structural Steel	
Type	**Standard**
Angles	ASTM A-36
Base Plates	ASTM A-572
Channels	ASTM A-36
Other Steel Members	ASTM A-36
Wide Flange Shapes	ASTM A-992

Table 1: Original Design Structural Steel

Reinforcing	
Type	**Standard**
Steel Fibers	ASTM A-706
Typical Bars	ASTM A-615
Welded Bars	ASTM A-615
Welded Wire Fabric	ASTM A-185

Table 2: Original Design Reinforcing

Concrete		
Type	**f'c (psi)**	**Unit Weight (pcf)**
Footings	4000	150
Foundation Walls	4000	150
Other Concrete	4000	150
Piers	4000	150

Table 3: Original Design Concrete

New Design

Tables 4 and 5 contain information used in the redesign.

Concrete		
Type	**f'c (psi)**	**Unit Weight (pcf)**
Columns	4000	150
Shear Walls	4000	150
Slabs	4000	150

Table 4: New Design Concrete

Reinforcing	
Type	**Standard**
Typical Bars	ASTM A-615

Table 5: New Design Reinforcing

Problem Statement

The Indiana Regional Medical Center has an existing braced frame and moment frame structure with a lightweight concrete composite decking system. From recent research it was found that this structural system is the most ideal for this specific building. However, it was also found that there are alternate systems that add comparable benefits to the building that the existing system does not. With this in mind, it is being proposed to change the current structural system of the Indiana Regional Medical Center to a system that utilizes concrete material. The difference of the concrete material could reduce total costs and result in low floor-to-floor heights. It will also be beneficial from an educational standpoint to look at concrete after spending an entire semester on steel.

Proposed Solution

Structural Depth

Information gathered from Technical Assignment #2 resulted in two potential alternatives for the proposed concrete structural system. The Two-Way Post-Tensioned system was the initial front runner because of its ability to span long distances and its small affects on the architecture of the building. It was later discovered that the Two-Way Flat Plate system is more convenient after a redesign was configured. The Two-Way Flat Plate system has a lower total weight and cost per square foot than the Two-Way Post-Tensioned system. The small impact on the architecture of the building is the only advantage the post-tensioned system has over the flat plate system. Easy formwork, simple bar placement, and low floor-to-floor heights are a few of the advantages gained with the Two-Way Flat Plate system. Lateral and gravity loads will have to be reexamined due to an increased weight of the building from the new structural system. Strengths and weaknesses of the current system and new system will be compared after the final design.

Lighting Breadth

A lighting breadth will be conducted that analyzes the current lighting design of a lobby/waiting room in the facility. A redesign of the space is expected because of the change in floor-to-floor heights and building materials due to the new structural system. New tasks of the area need to be redefined to ensure the lighting design is effective for the area. If a new lighting design is necessary, it will be a good opportunity to upgrade to more modernized luminaires.

Construction Breadth

The second breadth will study the alterations to the cost and schedule due to the new structural system. Updated cost estimates and construction schedules will be created for the new system. These updated costs and schedules will then be compared to cost estimates and construction schedules for the original structural system. This comparison is necessary to find which system is more efficient for the IRMC.

Building Loads

Live Loads

The values shown below in Table 6 were retrieved from the original design of the Indiana Regional Medical Center and compared to values that were determined from ASCE 7-10.

Location	Design (IBC 2003)	Thesis (ASCE 7-10)
Office	50 psf	50 psf
Restaurants	100 psf	100 psf
Retail	100 psf	100 psf
Mechanical Rooms	200 psf	-
Hospitals		
Operating rooms/Laboratories	60 psf	60 psf
Patient Rooms	40 psf	40 psf
Corridors Above First Floor	80 psf	80 psf
Roof	30 psf	20 psf
Stairs & Lobby	100 psf	100 psf
Corridors	80 psf	80 psf

Table 6: Live Load Summary

Dead Loads

The values shown below in Table 7 were used in the original design of the Indiana Regional Medical center. The dead load during the redesign consisted of the superimposed dead load and self-weight.

Dead Loads	
Roof Load	30 psf
Superimposed Dead Load	30 psf

Table 7: Dead Load Summary

Snow Loads

Chapter 7 in ASCE 7-10 was used for the snow load calculation for the IRMC. The exposure factor, importance factor, and thermal factor were obtained from the code and used to calculate the resulting load of 18 psf. Since there are other buildings surrounding the IRMC with lower roof heights, a basic drift calculation was done. The drift information is not important because the surrounding buildings are not being considered in this project. The hand calculations can be found in Appendix A.

Wind Loads

Chapter 26 and Chapter 27 of ASCE 7-10 was used when determining the wind load analysis for the Indiana Regional Medical Center. An analysis was done for both East/West and North/South directions.

To begin, it needed to be decided if the IRMC was calculated under a rigid structure or flexible structure. The calculations for this result are located in Appendix B and proved that this specific building should be calculated as a flexible structure. These calculations were computed using an occupancy category of III and a wind speed of 120 mph. Since the building is considered flexible, a gust effect factor was determined and these calculations can also be found in Appendix B.

Under ASCE 7-10, four cases need to be taken into account when applying wind loads to a building. All four cases can be seen in Figure 8 below. Microsoft Excel was used to compute the eccentricities, torsional moments, and wind pressures for each case and a summary of the results can be found in Appendix B.

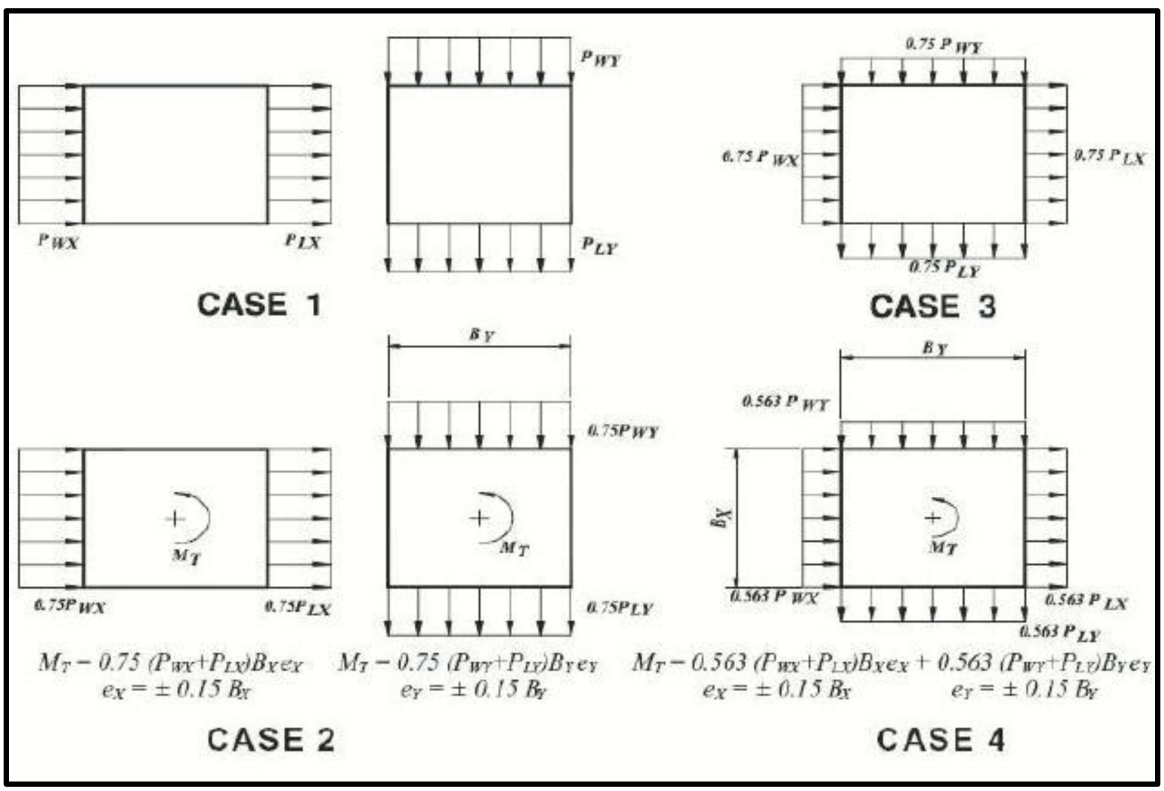


Figure 8: Wind Load Cases

Wind Loads

Four separate analyses were completed on an ETABS model to determine which wind load case controlled. Torsional moments and wind story forces were applied in each case and the deformed shapes of the shear walls were checked. Case #1 was determined to be the controlling wind case and a summary of the forces can be seen in Table 8 and Table 9 below. A summary of each case can be seen in Appendix B.

North-South Direction - Case #1										
Floor	Height (ft)	Story Height (ft)	K_z	q_z	Wind Pressures (psf)			Story Force (kips)	Story Shear (kips)	Overturning Moment (kips-ft)
					Windward N-S	Leeward N-S	Total N-S			
Roof	97	14	0.981	30.73	36.01	-24.58	60.60	81.44	0.00	0.00
7	83	14	0.939	29.42	34.71	-24.58	59.30	79.70	81.44	7899.88
6	69	14	0.886	27.76	33.07	-24.58	57.65	77.48	161.14	6614.73
5	55	14	0.830	26.00	31.32	-24.58	55.90	75.14	238.62	5346.28
4	41	14	0.765	23.97	29.31	-24.58	53.89	72.43	313.76	4132.47
3	27	14	0.676	21.18	26.54	-24.58	51.12	68.71	386.18	2969.60
2	13	14	0.570	17.86	23.25	-24.58	47.83	64.28	454.89	1855.16
1	0	13	0.000	0.00	0.00	0.00	0.00	0.00	519.18	835.68
								Total	**519.18**	**29653.79**

Table 8: Case #1 North/South Wind Load Summary

East-West Direction - Case #1										
Floor	Height (ft)	Story Height (ft)	K_z	q_z	Wind Pressures (psf)			Story Force (kips)	Story Shear (kips)	Overturning Moment (kips-ft)
					Windward E-W	Leeward E-W	Total E-W			
Roof	97	14	0.981	30.73	36.01	-16.96	52.98	154.27	0.00	0.00
7	83	14	0.939	29.42	34.71	-16.96	51.68	150.48	154.27	14963.73
6	69	14	0.886	27.76	33.07	-16.96	50.03	145.69	304.75	12489.93
5	55	14	0.830	26.00	31.32	-16.96	48.28	140.60	450.43	10052.32
4	41	14	0.765	23.97	29.31	-16.96	46.27	134.74	591.03	7733.09
3	27	14	0.676	21.18	26.54	-16.96	43.50	126.68	725.77	5524.24
2	13	14	0.570	17.86	23.25	-16.96	40.21	117.09	852.45	3420.31
1	0	13	0.000	0.00	0.00	0.00	0.00	0.00	969.54	1522.14
								Total	**969.54**	**55705.78**

Table 9: Case #1 East/West Wind Load Summary

Wind Loads

With Case #1 being the controlling wind load factor on IRMC, the story forces and story shear were determined for that case. As you can see in Table 8 and Table 9, the East/West direction has a larger base story shear of 969.54 kips. The North/South direction has a base story shear of 519.18 kips. The larger base story shear in the East/West direction was expected because of the larger surface area. Figure 9 and Figure 10 below show wind diagrams for both directions and summarize how the wind loads affect the building.

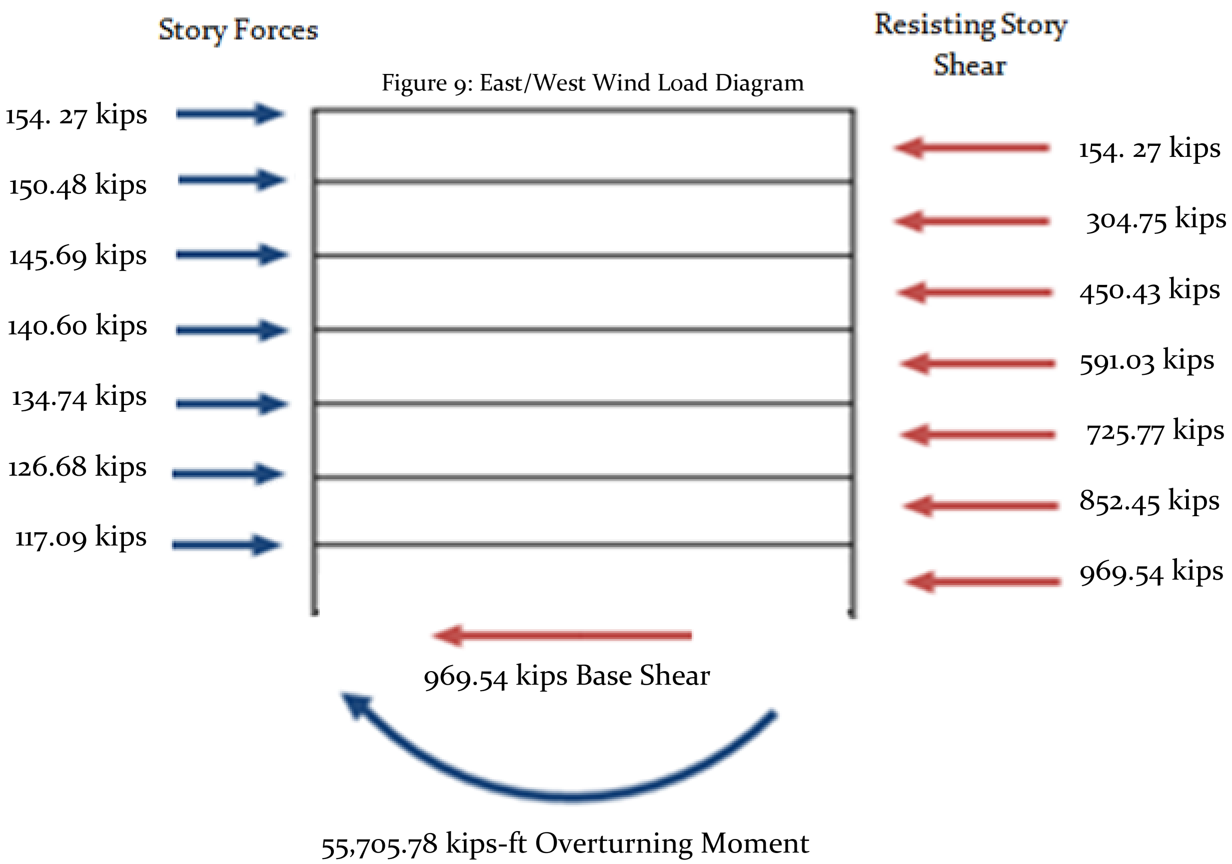


Figure 9: East/West Wind Load Diagram

Wind Loads

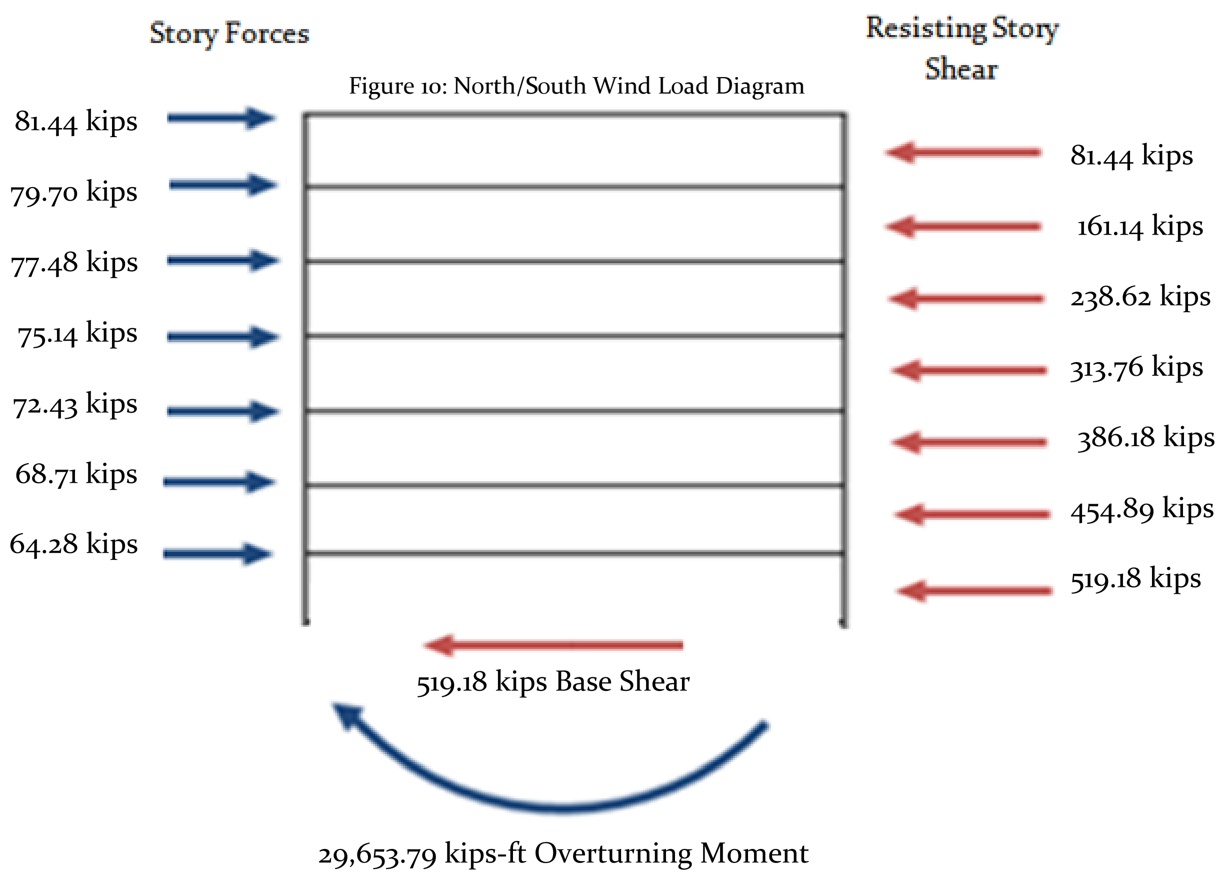


Figure 10: North/South Wind Load Diagram

Seismic Loads

ASCE 7-10 was referenced when calculating the seismic loading on the Indiana Regional Medical Center. The total weight of the building above grade first needed computed to do a seismic analysis. Excel spreadsheets were used to compile the total weights of slabs, columns, and walls throughout the building. These tables can be found in Appendix C. The estimated weight of the building was found to be about 26,000 kips. This value was much larger than the weight of the existing steel structure. The Equivalent Lateral Force Procedure was used to find the base shear which loads the IRMC. Table 10 shows all the design parameters used for the analysis and Table 11 is a summary of story and shear forces that were obtained from the analysis. Detailed hand calculations can be found in Appendix C.

Seismic Parameters			
Occupancy	III	Design Short Spectral Response	0.355
Site Class	D	Design Spectral Response	0.128
Seismic Design Category	C	Maximum Short Period Spectral Response	0.532
Effective Period	1.05	Maximum Spectral Response	0.192
Seismic Response Coefficient	0.025	Short Period Spectral Response	35% g
Response Modification Coefficient	6	Spectral Response	8 % g

Table 10: Seismic Parameters

Seismic Analysis: Base Shear and Overturning Moment Distribution									
Floor	Height h_x (ft)	Story Height (ft)	Story Weight w_x (lbs)	h_x^k	$w_x{*}h_x^k$	C_{vx}	Lateral Force F_x (kips)	Story Shear V_x (kips)	Moment M_x (kips-ft)
Roof	97	14	3342.40	341.30	1140755.01	0.29	187.46	0.00	0.00
7	83	14	3205.87	279.79	896954.63	0.23	147.40	187.46	18183.87
6	69	14	3205.87	221.07	708726.22	0.18	116.47	334.86	12234.06
5	55	14	3205.87	165.56	530772.41	0.13	87.22	451.33	8036.18
4	41	14	3279.37	113.84	373327.38	0.09	61.35	538.55	4797.26
3	27	14	3279.37	66.83	219169.05	0.06	36.02	599.90	2515.33
2	13	13	3279.37	26.32	86311.41	0.02	14.18	635.92	972.45
1	0	0	3205.58	0.00	0.00	0.00	0.00	650.10	184.39
$\sum w_x h_x^k$ = 3,956,016.09			$\sum F_x$ = Base Shear = 650.10 kips			Overturning Moment = 46,923.54 kip-ft			

Table 11: Seismic Load Summary

Table 11 shows that the resulting base shear from the seismic analysis is 650.10 kips and the overturning moment is 46,923.43 kip-ft.

Load Combinations

The lateral system analyzed in this report is governed by the load combinations found in ASCE 7-10 and can be seen in Figure 11 below. Both the north/south direction and east/west direction were considered when running this evaluation in ETABS. Load combination #4 resulted to be the controlling load type. The roof live load was used in the calculation because it is greater than both the rain and snow loads. East/west directional wind load was the controlling lateral load type because of its large base shear.

BASIC LOAD COMBINATIONS		
	APPLICABLE LOAD TYPES	LATERAL LOAD TYPES ONLY
1	1.4D	–
2	1.2D + 1.0L + 0.5(L_r or S or R)	–
3	1.2D + 1.0(L_r or S or R) + (L or 0.5W)	0.5W
4	1.2D + 1.0W + L + 0.5(L_r or S or R) **CONTROLS**	1.0W
5	1.2D + 1.0E + L + 0.2S	1.0E
6	0.9D + 1.0W	1.0W
7	0.9D + 1.0E	1.0E

D = DEAD LOAD, L = LIVE LOAD, R = RAIN LOAD, W = WIND LOAD
E = EARTHQUAKE LOAD, L_r = ROOF LIVE LOAD, S = SNOW LOAD

Figure 11 – Load Combinations

Design of Gravity System

Two-Way Flat Plate Design

When designing the two-way flat plate system for the Indiana Regional Medical Center, it was a goal to keep the current perfect symmetry of the building. The current layout of the bays for the building did not meet the requirements for the Direct Design Method, so the Equivalent Frame Method was used. All panels were assumed to be loaded by a full live load and all initial dimensions were used. A basic example of a two-way flat plate system is shown in Figure 12.

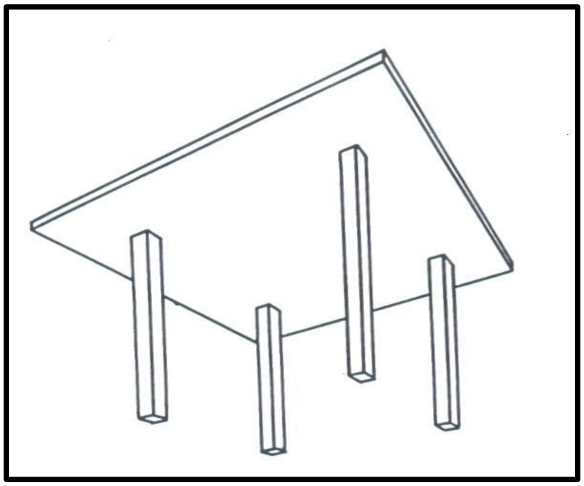


Figure 12 – Two-Way Flat Plate Example

Once the correct minimum thickness equations were obtained, the resultant thickness for the two-way flat plate system was 9''. This slab thickness is used throughout all seven stories of the building and used 4000 psi concrete. The large span is 26'-0'' in the North-South direction and 16'-0'' in the East-West direction.

Steel reinforcing is used in the newly designed two-way flat plate system. It utilizes both top and bottom #6 bars throughout the column and middles strips. Top bars are placed where negative moments occur and bottom bars are places where positive moments occur. There are as many as 12 bars in the strips and as few as 6 bars in others. The steel reinforcing spans both the North-South direction of the building as well as the East-West direction. Figure 13 represents how the column and middle strips were split up to design the steel reinforcement.

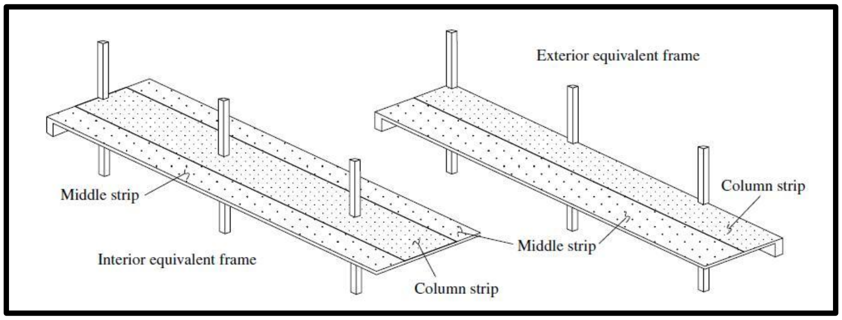


Figure 13 – Example of Column and Middle Strip Division

Two-Way Flat Plate Design

All hand calculations for the two-way flat plate system design and a summary of the calculated slab weights are located in Appendix D. A final summary of the reinforcement needed to support the slab can be seen in Table 12, Table 13, Table 14, and Table 15 below. The reinforcement for the interior and exterior bays are very similar due to the symmetry of the building.

Interior Bay Reinforcement - Long Direction		
Bar Position	**Bar Size**	**Number of Bars**
Column Strip		
Top Bars	#6	12
Bottom Bars	#6	6
Middle Strip		
Top Bars	#6	12
Bottom Bars	#6	12

Table 12: Interior Reinforcement Summary

Interior Bay Reinforcement - Short Direction		
Bar Position	**Bar Size**	**Number of Bars**
Column Strip		
Top Bars	#6	8
Bottom Bars	#6	6
Middle Strip		
Top Bars	#6	6
Bottom Bars	#6	6

Table 13: Interior Reinforcement Summary

Exterior Bay Reinforcement - Long Direction		
Bar Position	**Bar Size**	**Number of Bars**
Column Strip		
Top Bars	#6	13
Bottom Bars	#6	6
Middle Strip		
Top Bars	#6	10
Bottom Bars	#6	10

Table 14: Exterior Reinforcement Summary

Exterior Bay Reinforcement - Short Direction		
Bar Position	**Bar Size**	**Number of Bars**
Column Strip		
Top Bars	#6	8
Bottom Bars	#6	4
Middle Strip		
Top Bars	#6	6
Bottom Bars	#6	6

Table 15: Exterior Reinforcement Summary

After the slab thickness was determined and the needed reinforcement was configured, the slab was checked for shear and punching shear. Punching shear was checked for interior, exterior, and corner columns. It was found that punching shear would not be a problem and no drop panels would be needed for the slab design. The shear check also met all necessary standards.

Two-Way Flat Plate Design

The maximum total load deflections of the interior and exterior bay met the limits of L/240 for dead loads and L/360 for live loads. The largest deflection calculated was 0.652 and that is less than both limitations. An image of the deformed structure from gravity loads can be seen in Figure 13A below.

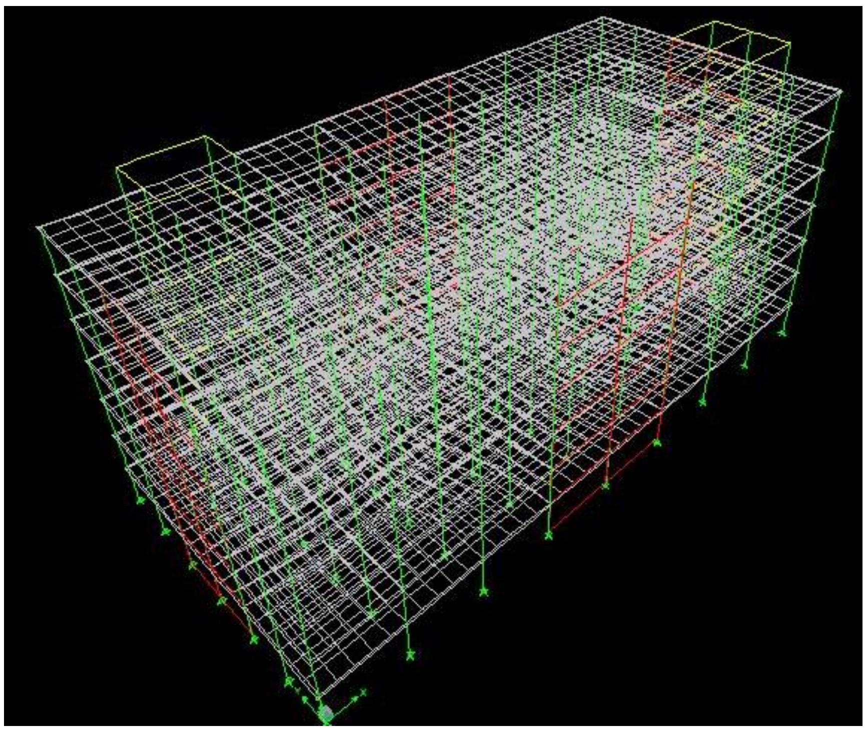


Figure 13A – Deformed Image of IRMC

Roof Layout

The roof layout of the building is almost identical to that of the floor system except that it has a slab thickness of 12'' to account for the mechanical room. It still has the 26'-0'' x 16'0'' spans just like every other floor throughout the building.

Column Design

One of the main goals for this project was to keep the symmetrical layout of the building as well as enhancing its potential to be renovated at anytime. The column layout that resulted consists of 26'-0'' x 16'-0'' bays completely throughout the entire building. As seen in Figure 14 below, the initial bay and column layout has been preserved. Columns range from 16'' x 16'' to 20'' x 20'' throughout the building and extend the entire 14'-0'' space between each floor. Each floor-to-floor height in the building is 14'-0'' except the ground floor which has a vertical distance of 13'-0''. Every column utilizes #9 bars running vertically and #3 ties every 16 inches.

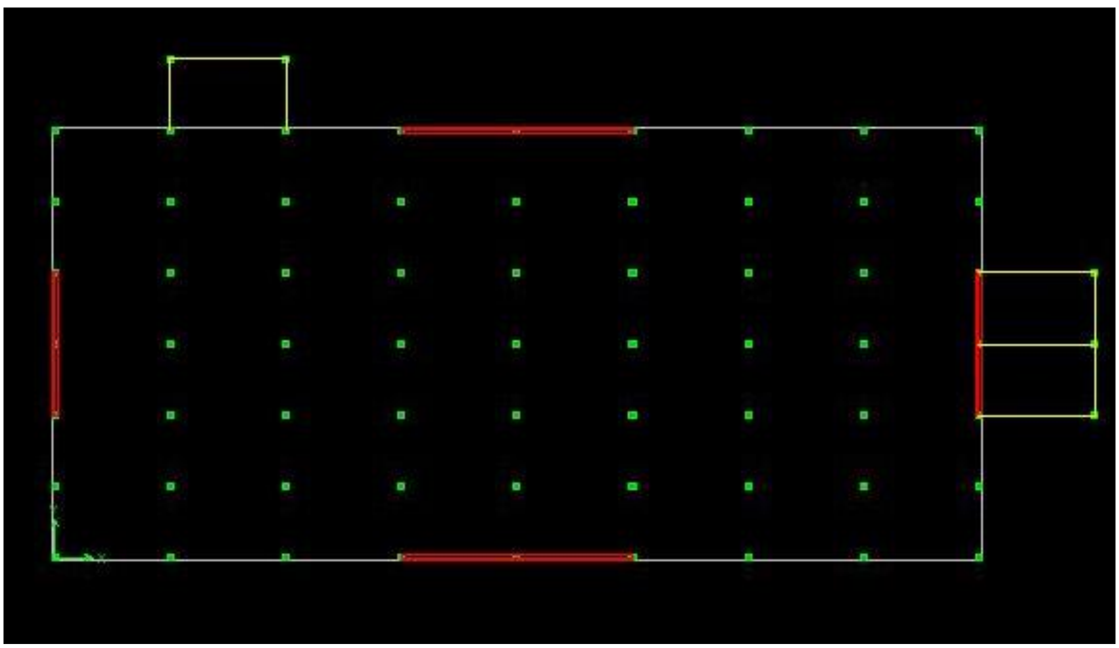


Figure 14 – Concrete Column Layout

Hand calculations were completed when designing the new column layout for the IRMC. These calculations can be found in Appendix E. RAM Structural system was also an aid during the design process to help approximate the gravity loads on the building. After base column sizes were determined from the preliminary analyses, Microsoft Excel was used to summarize the total axial loads on each column.

Column Design

A corner column, interior column, and exterior column on each floor were all taken into consideration because of the different tributary areas they obtain. A summary of the results for each of these column locations can be found in Table 16, Table, 17, and Table 18 below.

Corner Column										
Level	Height (ft)	Size (in x in)	A_T (ft^2)	Slab (in)	w_c (pcf)	SDL (psf)	Live (psf)	Factored (lb)	Slab Weight (lb)	P_u (kips)
Roof	14	16 x 16	118	12	150	30	20	8024	17700	25.72
7	14	16 x 16	118	9	150	30	80	19352	13275	58.35
6	14	16 x 16	118	9	150	30	80	19352	13275	90.98
5	14	16 x 16	118	9	150	30	80	19352	13275	123.61
4	14	16 x 16	118	9	150	30	80	19352	13275	156.23
3	14	16 x 16	118	9	150	30	80	19352	13275	188.86
2	13	16 x 16	118	9	150	30	80	19352	13275	221.49

Table 16: Corner Column Axial Load

Interior Column										
Level	Height (ft)	Size (in x in)	A_T (ft^2)	Slab (in)	w_c (pcf)	SDL (psf)	Live (psf)	Factored (lb)	Slab Weight (lb)	P_u (kips)
Roof	14	16 x 16	416	12	150	30	20	28288	62400	90.69
7	14	16 x 16	416	9	150	30	80	68224	46800	205.71
6	14	16 x 16	416	9	150	30	80	68224	46800	320.74
5	14	16 x 16	416	9	150	30	80	68224	46800	435.76
4	14	20 x 20	416	9	150	30	80	68224	46800	550.78
3	14	20 x 20	416	9	150	30	80	68224	46800	665.81
2	13	20 x 20	416	9	150	30	80	68224	46800	780.83

Table 17: Interior Column Axial Load

Column Design

Exterior Column											
Level	Height (ft)	Size (in x in)	A_T (ft^2)	Slab (in)	w_c (pcf)	SDL (psf)	Live (psf)	Factored (lb)	Slab Weight (lb)	SW (lb)	P_u (kips)
Roof	14	16 x 16	225.33	12	150	30	20	15322.4	33800	36400	85.52
7	14	16 x 16	225.33	9	150	30	80	36954.1	25350	36400	184.23
6	14	16 x 16	225.33	9	150	30	80	36954.1	25350	36400	282.93
5	14	16 x 16	225.33	9	150	30	80	36954.1	25350	36400	381.63
4	14	16 x 16	225.33	9	150	30	80	36954.1	25350	36400	480.34
3	14	16 x 16	225.33	9	150	30	80	36954.1	25350	36400	579.04
2	13	16 x 16	225.33	9	150	30	80	36954.1	25350	36400	677.74

Table 18: Exterior Column Axial Load

Once all the axial loads were calculated, sample columns were designed by hand. The column was found to be part of a non-sway frame and not a sway frame. The moment magnification factor did need to be calculated because the column was found to be a little over the slenderness standard, but it was discovered that the moment magnification would not influence the column. During the design process 4000 psi was used for concrete strength and 60000 psi was used for the strength of steel. The resultant column sizes were within the base sizes acquired from RAM. A summary of the column sizes and the reinforcement needed for each are located in Table 19, Table 20, and Table 21.

Column Design

Corner Column Design			
Level	Size (in x in)	A_s (in^2)	Long Reinforcing
Roof	16 x 16	8.00	8 #9
7	16 x 16	8.00	8 #9
6	16 x 16	8.00	8 #9
5	16 x 16	8.00	8 #9
4	16 x 16	12.00	8 #9
3	16 x 16	12.00	8 #9
2	16 x 16	12.00	8 #9
1	16 x 16	12.00	8 #9

Table 19: Corner Column Size & Reinforcing

Interior Column Design			
Level	Size (in x in)	A_s (in^2)	Long Reinforcing
Roof	16 x 16	8.00	8 #9
7	16 x 16	8.00	8 #9
6	16 x 16	8.00	8 #9
5	16 x 16	8.00	8 #9
4	20 x 20	12.00	12 #9
3	20 x 20	12.00	12 #9
2	20 x 20	12.00	12 #9
1	20 x 20	12.00	12 #9

Table 20: Interior Column Size & Reinforcing

Exterior Column Design			
Level	Size (in x in)	A_s (in^2)	Long Reinforcing
Roof	16 x 16	8.00	8 #9
7	16 x 16	8.00	8 #9
6	16 x 16	8.00	8 #9
5	16 x 16	8.00	8 #9
4	16 x 16	12.00	8 #9
3	16 x 16	12.00	8 #9
2	16 x 16	12.00	8 #9
1	16 x 16	12.00	8 #9

Table 21: Exterior Column Size & Reinforcing

Design of Lateral System

Shear Wall Design

Once the gravity system was redesigned to meet the proposed concrete structural solution of the project, the lateral system was taken into account. Since the existing steel structure has both braced frames and a partial steel moment frame, perimeter beams and shear walls were both considered. After much research, it was decided to replace the current braced frames with concrete shear walls. Creating a model in ETABS was the first step in this lateral redesign. A three dimensional model of the building can be seen in Figure 15.

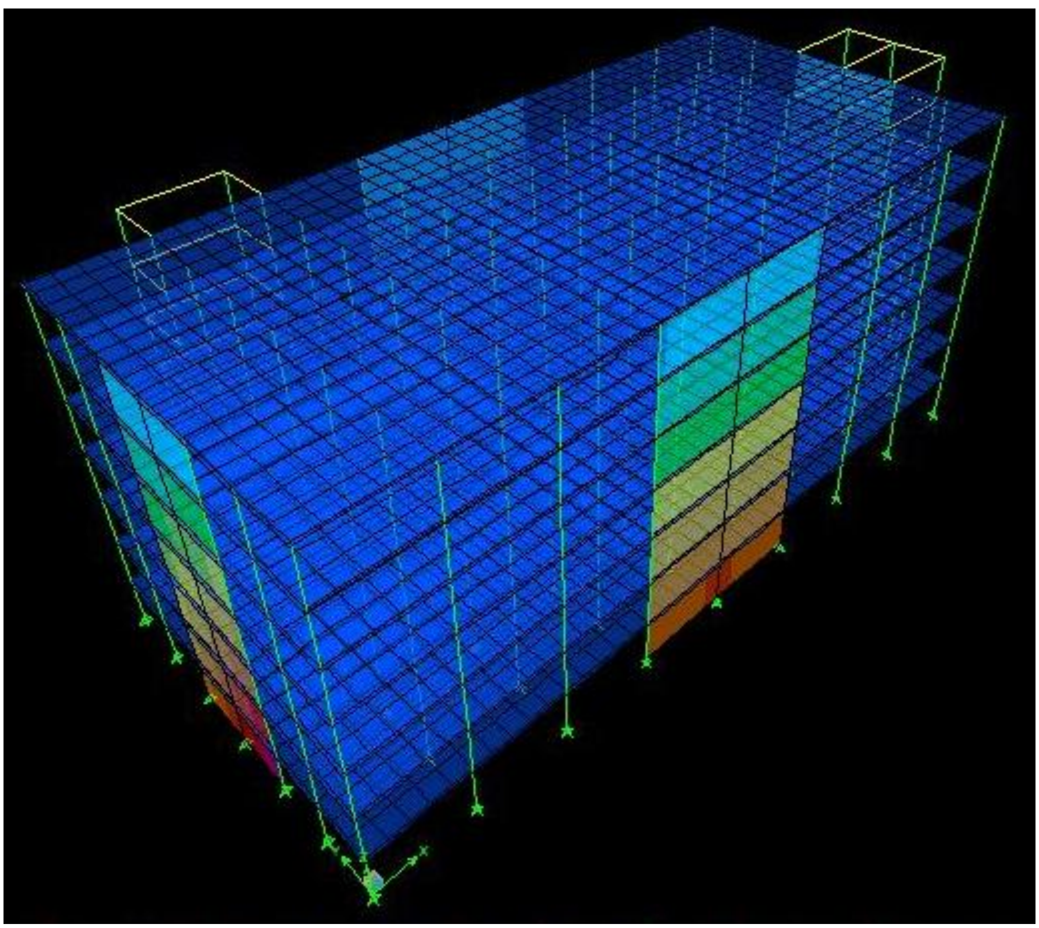


Figure 15 – Three Dimensional ETABS Model

Shear Wall Design

The 9'' two-way flat plate which was designed for the new gravity system was used in the ETABS model. It was also assumed that the slabs brace all the columns that were designed previously. The column sizes that were used for the lateral analysis were the resultant sizes from the concrete gravity system redesign. Placement of the concrete shear walls for analysis was based solely on the location of the current braced frames in the IRMC. The shear walls were placed in four separate locations on the exterior the building and they were assumed to be 16'' thick. They can be seen below in Figure 16. The red areas surrounding the structure of the building are where the shear walls are located.

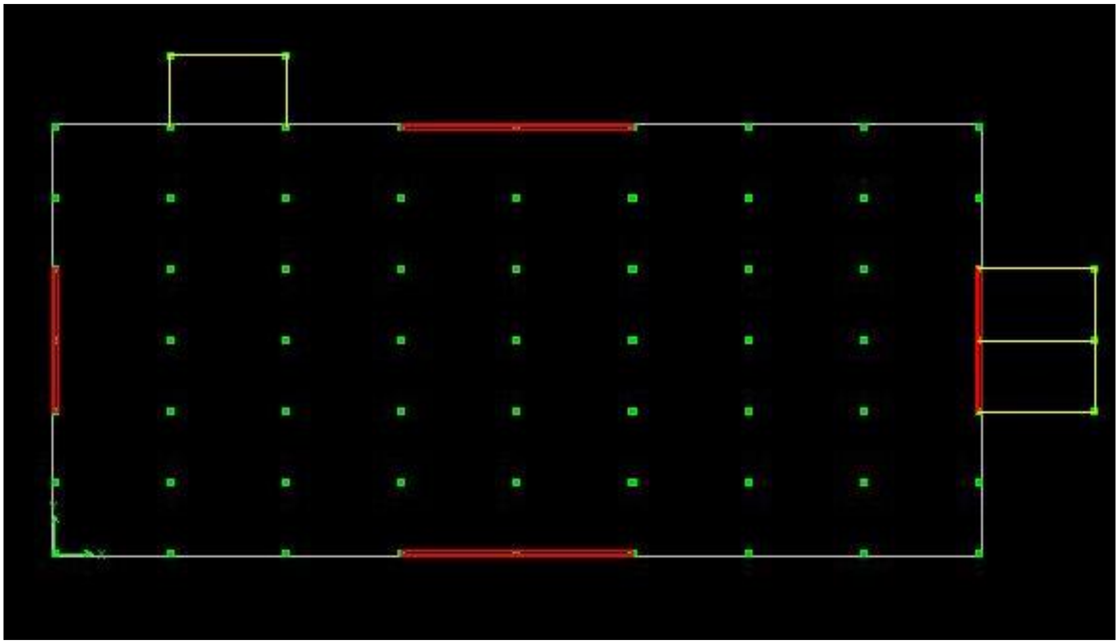


Figure 16 – Shear Wall Location

Overall, the theory on placing the shear walls was encouraged by the symmetry of the building. It was assumed that the evenly placed layout would allow each set of shear walls to get an equal portion of the shear that loaded the building. In the end this was not the case because of the open shaft areas that are along the southern and western sides of the building.

Shear Wall Design

Before the shear wall design even began, the greatest shear that these walls would experience had to be determined. The load combinations that were used previously were relied on again for this lateral design. ASCE 7-10 was referenced for this calculation and it requires that 30% of the seismic load be considered for one direction of the building and 100% for the other respected direction. Load combination #4 created the controlling shear in this analysis as well. The resulting controlling load was close to 500 kips, so that was the load used in the shear wall design.

The final designed shear walls have a thickness of 16'' and are located in four separate locations around the outside of the building. Walls on the North/South sides have a length of 16'-0'' and the walls on the East/West sides have a 26'-0'' length. It was determined that reinforcement was needed for the shear walls and a summary of the respected bars are located below in Table 22. Hand calculations for the shear wall design are located in Appendix F. A 3D wire frame model is also shown in Figure 17.

Shear Wall Reinforcement			
Type	Number of Bars	Bar Size	Spacing (in)
Horizontal	2	#4	10
Vertical	2	#4	10
Flexural	6	#9	2

Table 22: Shear Wall Reinforcement Summary

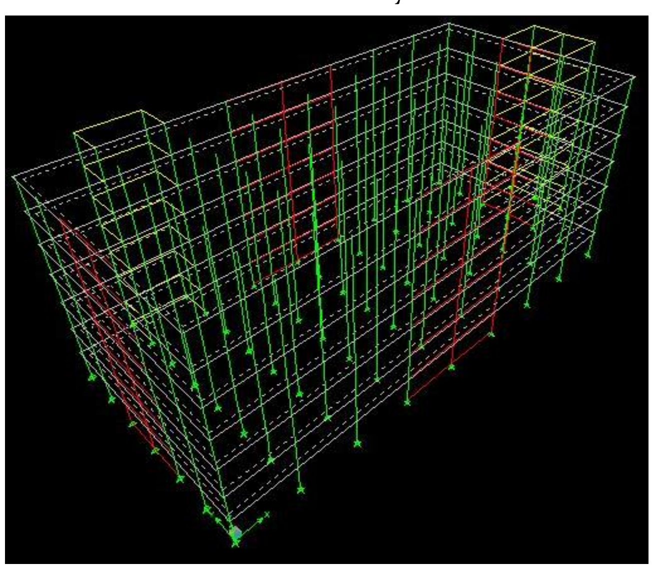


Figure 17: ETABS Wire Frame Model

Drift Analysis

Wind drift and seismic drift need to be determine for strength and serviceability issues. These factors are necessary to prevent any damage to a newly designed building, but also to keep the occupants of buildings safe and comfortable.

Story drift values were retrieved from the ETABS model used in the lateral design and compared to the allowable values stated by ASCE 7-10. Seismic loading was compared to the allowable story drift equation $0.010h_{sx}$ and wind loading was compared to the H/400 equation.

All respected drifts were found to meet ASCE 7-10 standards and a summary of the results can be seen below in Table 23, Table 24, Table 25, and Table 26.

Controlling Seismic Drift: North/South Direction				
Level	Height (ft)	Story Drift (in)	Allowable Story Drift (in)	Acceptable or Not Acceptable?
Roof	97	0.10991	0.14	Acceptable
7	83	0.10215	0.14	Acceptable
6	69	0.08341	0.14	Acceptable
5	55	0.04112	0.14	Acceptable
4	41	0.02045	0.14	Acceptable
3	27	0.00643	0.14	Acceptable
2	13	0.00276	0.13	Acceptable
1	0	0.00000	0.00	Acceptable

Table 23: North/South Seismic Drift Summary

Controlling Seismic Drift: East/West Direction				
Level	Height (ft)	Story Drift (in)	Allowable Story Drift (in)	Acceptable or Not Acceptable?
Roof	97	0.11347	0.14	Acceptable
7	83	0.10953	0.14	Acceptable
6	69	0.08837	0.14	Acceptable
5	55	0.06321	0.14	Acceptable
4	41	0.02751	0.14	Acceptable
3	27	0.00699	0.14	Acceptable
2	13	0.00325	0.13	Acceptable
1	0	0.00000	0.00	Acceptable

Table 24: East/West Seismic Drift Summary

Drift Analysis

Wind Deflection: North/South Direction				
Level	Height (ft)	Story Drift (in)	Allowable Story Drift (in)	Acceptable or Not Acceptable?
Roof	97	1.2622	2.91	Acceptable
7	83	1.1007	2.49	Acceptable
6	69	0.8523	2.07	Acceptable
5	55	0.7716	1.65	Acceptable
4	41	0.5792	1.23	Acceptable
3	27	0.3735	0.81	Acceptable
2	13	0.2257	0.39	Acceptable
1	0	0.0000	0.00	Acceptable

Table 25: North/South Wind Deflection Summary

Wind Deflection: East/West Direction				
Level	Height (ft)	Story Drift (in)	Allowable Story Drift (in)	Acceptable or Not Acceptable?
Roof	97	1.1062	2.91	Acceptable
7	83	1.0075	2.49	Acceptable
6	69	0.9363	2.07	Acceptable
5	55	0.6735	1.65	Acceptable
4	41	0.5481	1.23	Acceptable
3	27	0.3766	0.81	Acceptable
2	13	0.1733	0.39	Acceptable
1	0	0.0000	0.00	Acceptable

Table 26: East/West Wind Deflection Summary

Relative Stiffness Check

Relative stiffness was checked in the shear walls in both the North/South direction and the East/West direction. This calculation is necessary to evaluate the distribution of the lateral load throughout the building. The equation of relative stiffness and stiffness were utilized in this analysis. The equations are as follows:

$$k_i = \frac{P}{\delta} \qquad\qquad Relative\ Stiffness = \frac{k_i}{\sum k_i}$$

An arbitrary force, P of 100 kips was used as well as a lateral displacement recorded from the ETABS model. Table 27 and Table 28 below have the results from this calculation.

Relative Stiffness: North/South Direction					
Frame	Height (ft)	Load (kips)	Displacement (in)	Stiffness (k/in)	Relative Stiffness
1	97	100	0.6982	143.23	0.50
2	97	100	0.6982	143.23	0.50
Total				286.45	1.00

Table 27: North/South Relative Stiffness Summary

Relative Stiffness: East/West Direction					
Frame	Height (ft)	Load (kips)	Displacement (in)	Stiffness (k/in)	Relative Stiffness
1	97	100	0.7165	139.57	0.50
2	97	100	0.7165	139.57	0.50
Total				279.13	1.00

Table 28: East/West Relative Stiffness Summary

Overturning

After some lateral analysis it was found that Seismic Loads control in the North-South Direction and Wind Loads control in the E-W Direction. Overturning moments caused by the Seismic Loads were counteracted by the dead load of the building. The Seismic load is on 2.5% of the dead load. Overturning moments from Wind Loads need to be less than the resisting moment to avoid any risks. The resisting moment is the weight of the building multiplied by half the width of the building. In this case, the overturning moment created from the wind load is much smaller than the resisting moment. The overturning hand calculations can be found in Appendix G.

Lighting Breadth

Current Design

The Indiana Regional Medical Center has a 20'-0'' x 30'-0'' lobby and waiting room on the first floor of their facility. The new two-way flat plate system does not affect the space at a drastic level, but it does affect the lighting design. The change of structural systems has increased the height of the room cavity ratio from 11 ft to 12 ft. The different use of materials will also change the initial reflectances used for the respected surfaces. This is also a good opportunity to upgrade the current luminaries from fluorescent based lamps to LED luminaries.

Design Criteria

It is always important to outline the most important tasks for the space that is being lit. The target illuminance for a lobby is usually between 10 and 15 foot-candles. Since the area is a medical center and has a majority of elderly patients the target illuminance can be doubled to account for the older population. The following is a list of guidelines that are important when designing this specific space.

- Target illuminance level: 15 fc
- Tasks: walking, gathering, and talking
- CCT: neutral/warm, utilize natural daylight
- CRI: at least 70, higher desirable
- Control: two switches utilized for daytime and nighttime lighting for energy savings
- Aesthetic/Style issues: Use luminaires to guide people to where they need to be.
- Luminance ratio: 10:1 for far and 3:1 for near
- Light Distribution: direct

Luminaire Selection

The new luminaire that was selected is a 6'' LED downlight from Gotham Architectural Downlighting. It is a one 31 Watt fixture that is installed into the celing cavity. Figure 18 is a picture of the actual luminaire. The specifications for this lamp are located in Appendix H. This specific fixture is designed strictly for aesthetics. Its modern feel and use of LED technology makes it a wise choice for a lobby in any building.


Figure 18 – 6" LED Downlight

The Lumen Method

The Lumen Method calculations were completed and can be found in Appendix H. The resultant factor allows this space to receive twelve 6'' LED Downlights throughout the entirety of the room. The target illuminance for the room was 15 footcandles and the actual calculated illuminance is 14.2 footcandles after light loss factors. A simple layout of the lighting fixtures can be seen in Figure 19. The Photometric Viewer files can be found in Appendix H.

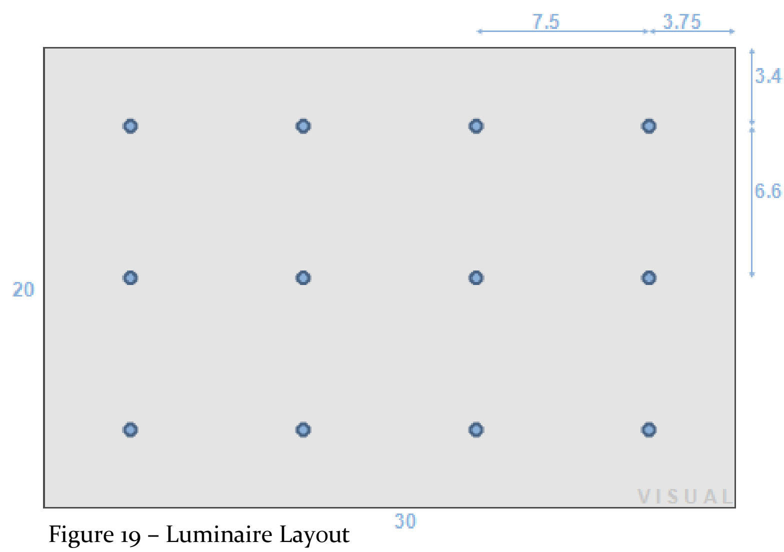


Figure 19 – Luminaire Layout

Lighting Breadth Conclusion

The lighting for this space was designed based on convenience, functionality, and aesthetics. The luminaires selected do not make the room too gaudy and omit the perfect amount of light for the area. Figure 20 shows these specific lamps in an environment that is very similar to the one that was designed for.

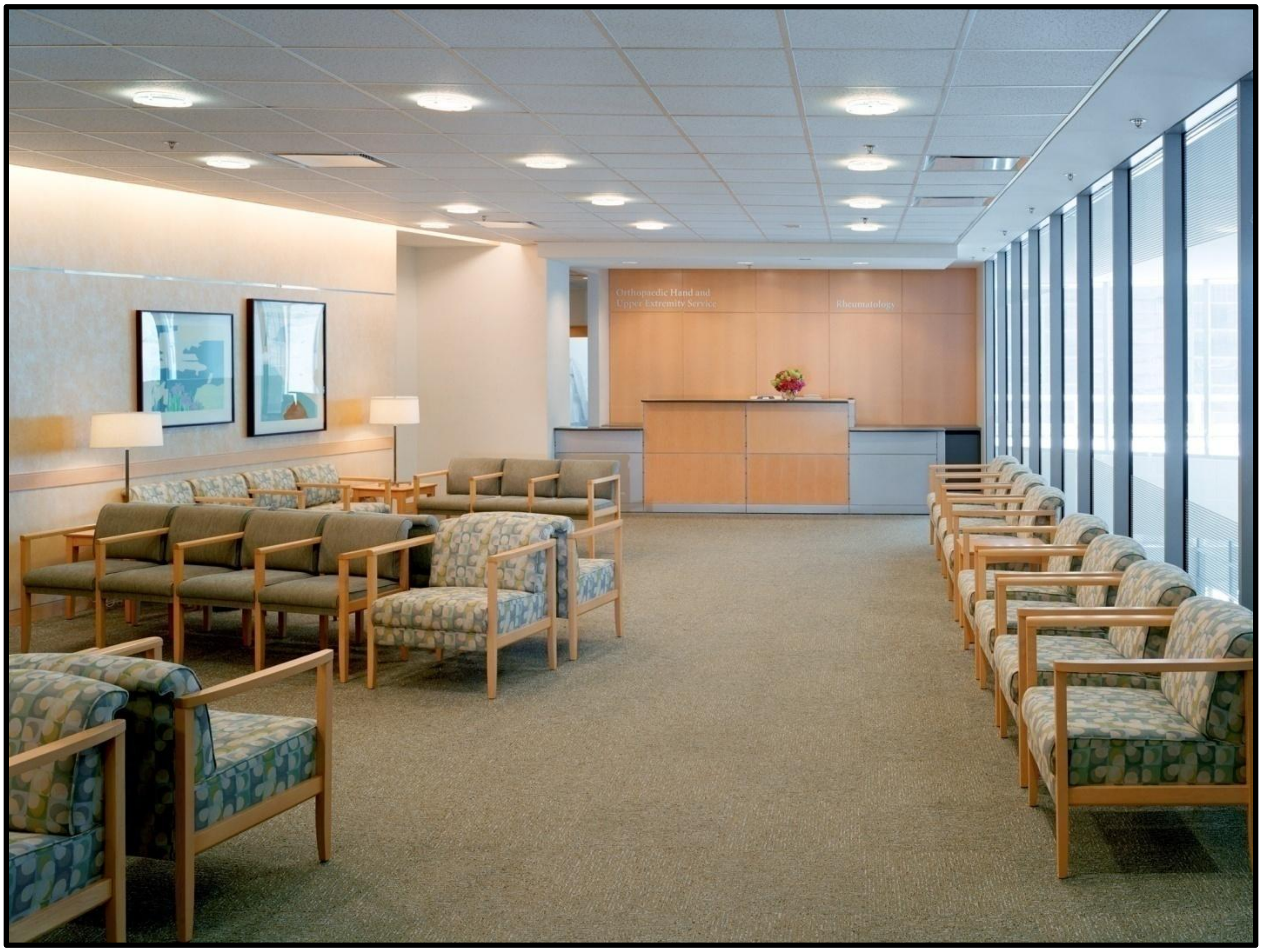


Figure 20 – Reality example of new luminaires

Construction Management Breadth

The construction management breadth's true purpose is to compare the actual structural system's cost and sequencing with the new structural systems. Neither schedule was available to compare so they both were created respectively. This is also a good chance to see if the new structural system is affordable.

Schedule Comparison

A random day was chosen for both projects to start when designing both the schedules. They both started on the same day because it was interesting to discover that the concrete system was erected at a faster pace than the steel system. The concrete system's construction was split into 3 different sections while the steel system was one continuous job. The steel structure might take a few more weeks to construct, but the construction of the concrete system needs constant attention. Both schedules designed can be found in Appendix I. Overall, the concrete system would be completed well before the steel system.

Cost Comparison

When a new structure is being designed, the owner usually wants to get the best price possible. That is where cost comparison comes in handy, especially for firms who are in the bid-build business. The cost for each system was run through a pricing application from data that was retrieved from a pricing website. Figure 24 has the comparison in a chart of both systems.

Cost Comparison			
	O&P Costs	Bare Costs	**Total**
Concrete	$3,970,430	$10,243,456	**$14,213,886**
Steel	$1,546,452	$8,190,760	**$9,737,212**

Figure 29 – Cost Comparisons

The fact that the O&P costs were more for the concrete system was not a surprise since it is the more tedious system to construct. The closeness in the values overall was not expected in the end.

Construction Management Conclusion

The newly designed two-way flat plate system has shown some advantages in the cost comparison and schedule analysis. Its speed of construction is definitely an asset, but its steep prices and constant labor might not outlast the appeal of the steel structure.

Conclusion

From the outcome of the research and analyses done on the Indiana Regional Medical Center, it is safe to say that the two-way flat plate system would be an effective structural alternative for the building. It is not the most beneficial in every aspect, but it has achieved some goals.

One main goal of this project was to keep, if not enhance, the layout of the building. The owner appreciates the very simple and symmetric layout of the hospital because it is easy to renovate or adjust as needed. The fact that the two-way flat plate system could maintain the symmetrical bay sizes and even incorporate the same size columns throughout the building was an achievement.

This concrete system even proved to be slightly promising with its fast construction, easy formwork, and improved floor-to-floor heights. The braced frame that is currently the structure of the building stilled proved to be cheaper, less labor intense and the clear choice in the long-run.

Overall, being able to explore the structural world of concrete after spending so much time on steel was a nice change of pace. The outcome was a lot better than expected and there is still much to be learned.

Appendix

Appendix A: Snow Load Analysis

Cody Scheller | Senior Thesis | Snow Analysis | 1

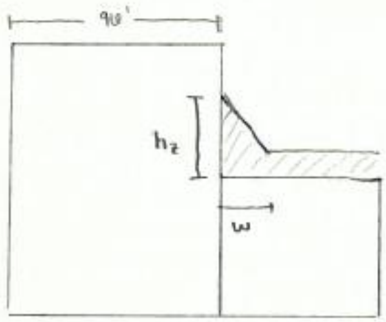
7.3 → Flat Roof Snow Load

$P_f = 0.7\, C_e C_t I_s P_g$

→ where
$C_e = 0.9$
$C_t = 0.1$
$I_s = 1.1$
$P_g = 25$

$P_f = (0.7)(0.9)(1.0)(1.1)(25) = \boxed{17.325\ \text{lb/ft}^2}$

→ Snow Drift



$h_z = 0.43\sqrt[3]{l_u}\,\sqrt[4]{P_g+10} - 1.5$
$= 0.43\sqrt[3]{96}\,\sqrt[4]{25+10} - 1.5$
$= 3.29'$

$w = 4h_z = 13.2'$

$P_z = h_z \gamma$

$\gamma = 0.13 P_g + 3$

$\gamma = 0.13(25) + 3 = 6.25$

Drift $= P_d = 3.29\,(6.25)$
$= \boxed{20.56\ \text{lb/ft}^2}$

Appendix B: Wind Load Analysis

Wind Design

→ Use ASCE 7-10 – MWFRS (Directional Procedure)

27.2.1

→ Basic Wind Speed (26.5)

→ Occupancy Category III (Table 1.5-1)

↳ wind speed = V = 120 mph

→ Wind Directionality Factor (K_d) = 0.85

→ Exposure Factor → B

→ Topographic Factor (K_{zt}) → 1.0

→ Gust-Effect Factor (26.9) → Rigid?

→ Approximate Natural Frequency Limitations

1.) Building Height = 97' < 300' ∴ ok ✓

2.) Building Height = 97' < 4 L_{eff}

→ Check N-S Direction

$$L_{eff} = \frac{\sum_{i=1}^{n} h_i L_i}{\sum_{i=1}^{n} h_i} = 208' \rightarrow 4(208) > 97' \text{ ok} ✓$$

→ Check E-W Direction

$$L_{eff} = \frac{\sum_{i=1}^{n} h_i L_i}{\sum_{i=1}^{n} h_i} = 97' \rightarrow 4(97) > 97' \text{ ok} ✓$$

∴ Can Approximate

Appendix B: Wind Load Analysis

Cody Scheller | Senior Thesis | Wind Analysis | 2

Wind Design

26.9.3

→ Concrete Moment-resisting Frame Building

$$n_a = \frac{385 c_w^{0.5}}{h} = \frac{385 (0.006)^{0.5}}{97} = 0.31 < 1.0$$

∴ Flexible

26.9.5

→ Flexible Building

$$G_f = 0.925 \left[\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}} \right] = 1.24$$

$$I_{\bar{z}} = c \left(\frac{33}{\bar{z}} \right)^{1/6} = 0.30 \left(\frac{33}{58.2} \right)^{1/6} = 0.27$$

$C = 0.30$

$$\bar{z} = \max \begin{cases} 0.6h = 0.6(97) = 58.2 \\ z_{min} = 14 \end{cases}$$

$g_Q = g_v = 3.4$

$$g_R = \sqrt{2 \ln(3600 n_1)} + \frac{0.577}{\sqrt{2 \ln(3600 n_1)}} = 3.79$$

$n_1 = n_a = 0.31$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)} = 1.21$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(0.984)}{[1 + 10.3(0.984)]^{5/3}} = 0.132$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{V_{\bar{z}}} = \frac{0.31(251.30)}{79.2} = 0.984$$

Appendix B: Wind Load Analysis

Cody Scheller | Senior Thesis | Wind Analysis | 3

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 208\left(\frac{58.2}{33}\right)^{1/3} = 251.30$$

$$V_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)V = 0.45\left(\frac{58.2}{33}\right)^{1/4}\left(\frac{88}{60}\right)120 = 79.2$$

R_h:

$$\eta = \frac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \frac{4.6(0.31)(97)}{79.2} = 1.75$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) = 0.413$$

R_B:

$$\eta = \frac{4.6 n_1 B}{V_{\bar{z}}} = \frac{4.6(0.3)(96)}{79.2} = 1.67$$

$$R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) = 0.423$$

R_L:

$$\eta = \frac{4.6 n_1 L}{V_{\bar{z}}} = \frac{4.6(0.31)(208)}{79.2} = 3.75$$

$$R_L = \frac{1}{\eta} - \frac{1}{2\eta_2}\left(1 - e^{-2\eta}\right) = 0.231$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = 0.81$$

Assume $\beta = 0.01 \rightarrow$ 1% damping for concrete structure

Appendix B: Wind Load Analysis

→ Enclosure Classification (26.10)

↳ ENCLOSED

→ Internal pressure Coefficient = ±0.18

→ Wall Pressure Coefficients, C_p

→ Wind in N/S Direction

→ Windward Wall: $C_p = 0.8$

→ Leeward Wall: $(L/B) = 0.46$ $C_p = -0.5$

→ Wind in E/W Direction

→ Windward Wall: $C_p = 0.8$

→ Leeward wall: $(L/B) = 2.17$ → $C_p = -0.3$

→ Vertical Pressure

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

$$q_z = (0.00256)(K_z)(1.0)(0.85)(120)^2$$

$$q_z = 31.33 K_z$$

→ Design Pressures

→ Windward (N/S & E/W)

→ $p = G_F C_p q_z = (1.24)(0.8) q_z - q_h(\pm 0.18)$

→ Leeward (N/S)

→ $p = G_F C_p q_h = (1.24)(-0.5) q_h - q_h(\pm 0.18)$

→ Leeward (E/W)

→ $p = G_F C_p q_h = (1.24)(-0.3) q_h - (q_h)(\pm 0.18)$

Appendix B: Wind Load Analysis

North-South Direction - Case #1										
Floor	Height (ft)	Story Height (ft)	K_z	q_z	Wind Pressures (psf) Windward N-S	Leeward N-S	Total N-S	Story Force (kips)	Story Shear (kips)	Overturning Moment (kips-ft)
Roof	97	14	0.981	30.73	36.01	-24.58	60.60	81.44	0.00	0.00
7	83	14	0.939	29.42	34.71	-24.58	59.30	79.70	81.44	7899.88
6	69	14	0.886	27.76	33.07	-24.58	57.65	77.48	161.14	6614.73
5	55	14	0.830	26.00	31.32	-24.58	55.90	75.14	238.62	5346.28
4	41	14	0.765	23.97	29.31	-24.58	53.89	72.43	313.76	4132.47
3	27	14	0.676	21.18	26.54	-24.58	51.12	68.71	386.18	2969.60
2	13	14	0.570	17.86	23.25	-24.58	47.83	64.28	454.89	1855.16
1	0	13	0.000	0.00	0.00	0.00	0.00	0.00	519.18	835.68
								Total	**519.18**	**29653.79**

East-West Direction - Case #1										
Floor	Height (ft)	Story Height (ft)	K_z	q_z	Wind Pressures (psf) Windward E-W	Leeward E-W	Total E-W	Story Force (kips)	Story Shear (kips)	Overturning Moment (kips-ft)
Roof	97	14	0.981	30.73	36.01	-16.96	52.98	154.27	0.00	0.00
7	83	14	0.939	29.42	34.71	-16.96	51.68	150.48	154.27	14963.73
6	69	14	0.886	27.76	33.07	-16.96	50.03	145.69	304.75	12489.93
5	55	14	0.830	26.00	31.32	-16.96	48.28	140.60	450.43	10052.32
4	41	14	0.765	23.97	29.31	-16.96	46.27	134.74	591.03	7733.09
3	27	14	0.676	21.18	26.54	-16.96	43.50	126.68	725.77	5524.24
2	13	14	0.570	17.86	23.25	-16.96	40.21	117.09	852.45	3420.31
1	0	13	0.000	0.00	0.00	0.00	0.00	0.00	969.54	1522.14
								Total	**969.54**	**55705.78**

Appendix B: Wind Load Analysis

North-South Direction - Case #2										
Floor	Height (ft)	Story Height (ft)	K_z	q_z	Wind Pressures (psf)			Story Force (kips)	Story Shear (kips)	Overturning Moment (kips-ft)
					Windward N-S	Leeward N-S	Total N-S			
Roof	97	14	0.981	30.73	36.01	-24.58	60.60	61.08	0.00	879.57
7	83	14	0.939	29.42	34.71	-24.58	59.30	59.77	61.08	860.71
6	69	14	0.886	27.76	33.07	-24.58	57.65	58.11	120.85	836.81
5	55	14	0.830	26.00	31.32	-24.58	55.90	56.35	178.96	811.47
4	41	14	0.765	23.97	29.31	-24.58	53.89	54.32	235.32	782.24
3	27	14	0.676	21.18	26.54	-24.58	51.12	51.53	289.64	742.06
2	13	14	0.570	17.86	23.25	-24.58	47.83	48.21	341.17	694.26
1	0	13	0.000	0.00	0.00	0.00	0.00	0.00	389.38	0.00
								Total	**389.38**	**4727.54**

East-West Direction - Case #2										
Floor	Height (ft)	Story Height (ft)	K_z	q_z	Wind Pressures (psf)			Story Force (kips)	Story Shear (kips)	Overturning Moment (kips-ft)
					Windward E-W	Leeward E-W	Total E-W			
Roof	97	14	0.981	30.73	36.01	-16.96	52.98	115.70	0.00	2406.54
7	83	14	0.939	29.42	34.71	-16.96	51.68	112.86	115.70	2347.51
6	69	14	0.886	27.76	33.07	-16.96	50.03	109.26	228.56	2272.70
5	55	14	0.830	26.00	31.32	-16.96	48.28	105.45	337.82	2193.39
4	41	14	0.765	23.97	29.31	-16.96	46.27	101.05	443.28	2101.91
3	27	14	0.676	21.18	26.54	-16.96	43.50	95.01	544.33	1976.18
2	13	14	0.570	17.86	23.25	-16.96	40.21	87.82	639.34	1826.57
1	0	13	0.000	0.00	0.00	0.00	0.00	0.00	727.15	1141.60
								Total	**727.15**	**13859.85**

Appendix B: Wind Load Analysis

North-South Direction - Case #3										
Floor	Height (ft)	Story Height (ft)	K_z	q_z	Wind Pressures (psf)			Story Force (kips)	Story Shear (kips)	Overturning Moment (kips-ft)
					Windward N-S	Leeward N-S	Total N-S			
Roof	97	14	0.981	30.73	36.01	-24.58	60.60	61.08	0.00	0.00
7	83	14	0.939	29.42	34.71	-24.58	59.30	59.77	61.08	5924.91
6	69	14	0.886	27.76	33.07	-24.58	57.65	58.11	120.85	4961.04
5	55	14	0.830	26.00	31.32	-24.58	55.90	56.35	178.96	4009.71
4	41	14	0.765	23.97	29.31	-24.58	53.89	54.32	235.32	3099.35
3	27	14	0.676	21.18	26.54	-24.58	51.12	51.53	289.64	2227.20
2	13	14	0.570	17.86	23.25	-24.58	47.83	48.21	341.17	1391.37
1	0	13	0.000	0.00	0.00	0.00	0.00	0.00	389.38	626.76
								Total	**389.38**	**22240.34**

East-West Direction - Case #3										
Floor	Height (ft)	Story Height (ft)	K_z	q_z	Wind Pressures (psf)			Story Force (kips)	Story Shear (kips)	Overturning Moment (kips-ft)
					Windward E-W	Leeward E-W	Total E-W			
Roof	97	14	0.981	30.73	36.01	-16.96	52.98	115.70	0.00	0.00
7	83	14	0.939	29.42	34.71	-16.96	51.68	112.86	115.70	11222.80
6	69	14	0.886	27.76	33.07	-16.96	50.03	109.26	228.56	9367.45
5	55	14	0.830	26.00	31.32	-16.96	48.28	105.45	337.82	7539.24
4	41	14	0.765	23.97	29.31	-16.96	46.27	101.05	443.28	5799.82
3	27	14	0.676	21.18	26.54	-16.96	43.50	95.01	544.33	4143.18
2	13	14	0.570	17.86	23.25	-16.96	40.21	87.82	639.34	2565.23
1	0	13	0.000	0.00	0.00	0.00	0.00	0.00	727.15	1141.60
								Total	**727.15**	**41779.33**

Appendix B: Wind Load Analysis

North-South Direction - Case #4										
Floor	Height (ft)	Story Height (ft)	K_z	q_z	Wind Pressures (psf)			Story Force (kips)	Story Shear (kips)	Overturning Moment (kips-ft)
					Windward N-S	Leeward N-S	Total N-S			
Roof	97	14	0.981	30.73	36.01	-24.58	60.60	45.85	0.00	660.27
7	83	14	0.939	29.42	34.71	-24.58	59.30	44.87	45.85	646.11
6	69	14	0.886	27.76	33.07	-24.58	57.65	43.62	90.72	628.16
5	55	14	0.830	26.00	31.32	-24.58	55.90	42.30	134.34	609.14
4	41	14	0.765	23.97	29.31	-24.58	53.89	40.78	176.64	587.20
3	27	14	0.676	21.18	26.54	-24.58	51.12	38.68	217.42	557.04
2	13	14	0.570	17.86	23.25	-24.58	47.83	36.19	256.11	521.16
1	0	13	0.000	0.00	0.00	0.00	0.00	0.00	292.30	0.00
								Total	**389.38**	**3548.81**

East-West Direction - Case #4										
Floor	Height (ft)	Story Height (ft)	K_z	q_z	Wind Pressures (psf)			Story Force (kips)	Story Shear (kips)	Overturning Moment (kips-ft)
					Windward E-W	Leeward E-W	Total E-W			
Roof	97	14	0.981	30.73	36.01	-16.96	52.98	86.85	0.00	1806.51
7	83	14	0.939	29.42	34.71	-16.96	51.68	84.72	86.85	1762.19
6	69	14	0.886	27.76	33.07	-16.96	50.03	82.02	171.57	1706.04
5	55	14	0.830	26.00	31.32	-16.96	48.28	79.16	253.59	1646.50
4	41	14	0.765	23.97	29.31	-16.96	46.27	75.86	332.75	1577.83
3	27	14	0.676	21.18	26.54	-16.96	43.50	71.32	408.61	1483.45
2	13	14	0.570	17.86	23.25	-16.96	40.21	65.92	479.93	1371.14
1	0	13	0.000	0.00	0.00	0.00	0.00	0.00	545.85	856.96
								Total	**727.15**	**10404.13**

Appendix C: Seismic Load Analysis

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ASCE/SEI 7-10

11.4.2 → Site Class

D → As per geotechnical report

11.4.3 → Spectral Response Acceleration

$S_S = 0.35$

$S_1 = 0.08$

Fa:	$S_S \le 0.25$	0.35	$S_S = 0.5$
D	1.6	$\boxed{1.52}$	1.4

Fv: $S_1 \le 0.1$ then $\boxed{F_v = 2.4}$

$S_{MS} = F_a S_S = 1.52(0.35) = 0.532$

$S_{M1} = F_v S_1 = (2.4)(0.08) = 0.192$

11.4.4 → Design Spectral Response Acceleration

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.532) = 0.355$

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.192) = 0.128$

→ Seismic Design Category C

→ Occupancy III

12.8 → Equivalent Lateral Force Procedure

12.8.1 → Seismic Base Shear

$V = C_S W$

$W = 26{,}003.68^k$

Appendix C: Seismic Load Analysis

Cody Scheller | Senior Thesis | Seismic Analysis | 2

12.8.1.1 → Seismic Response Coefficient

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$$

$R = 6.0$ → Table 12.2-1

$I_e = 1.25$ → Table 1.5-2, III

$$C_s = \frac{0.355}{\left(\frac{6}{1.25}\right)} \rightarrow C_s = 0.0739$$

→ C_s shall not exceed:

T = Fundamental Period → 12.8.2

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} \quad \text{For } T \leq T_L$$

$$C_s = \frac{S_{D1} T_L}{T^2\left(\frac{R}{I_e}\right)} \quad \text{For } T > T_L$$

T_L = Long-Period transition Period → 11.4.5

$T_L = 6$

12.8.2 → Period Determination

$$T = \min \begin{cases} C_u T_a \\ T_b \rightarrow \text{determined later from model} \end{cases}$$

$C_u = 1.7$

$C_t = 0.02 \quad x = 0.75$

$h_n = 97$ ft

$$T_a = C_t h_n^x$$

$$T_a = 0.02(97)^{0.75}$$

$$T_a = 0.618$$

$$T = C_u T_a = 1.7(0.618) = 1.05\text{ s}$$

$$C_s = \min \begin{cases} \frac{S_{DS}}{R/I_e} = \frac{0.355}{6/1.25} = 0.0739 \\ \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.128}{(1.05)(6/1.25)} = \boxed{0.025} \rightarrow \text{controls.} \\ \frac{S_{D1} T_L}{T^2\left(R/I_e\right)} = 0.145 \end{cases}$$

Appendix C: Seismic Load Analysis

$V = C_s W = (0.025)(26{,}003.68k) = \boxed{650.1k}$

12.8.3 → Vertical Distribution of Seismic Forces

$F_x = C_{vx} V$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

	x_1	x	x_2
T	0.5	1.05	2.5
K	1	$\boxed{1.275}$	2

$k = 1.275$

Appendix C: Seismic Load Analysis

Seismic Parameters			
Occupancy	III	Design Short Spectral Response	0.355
Site Class	D	Design Spectral Response	0.128
Seismic Design Category	C	Maximum Short Period Spectral Response	0.532
Effective Period	1.05	Maximum Spectral Response	0.192
Seismic Response Coefficient	0.025	Short Period Spectral Response	35% g
Response Modification Coefficient	6	Spectral Response	8 % g

Seismic Analysis: Base Shear and Overturning Moment Distribution

Floor	Height h_x (ft)	Story Height (ft)	Story Weight w_x (lbs)	h_x^k	$w_x * h_x^k$	C_{vx}	Lateral Force F_x (kips)	Story Shear V_x (kips)	Moment M_x (kips-ft)
Roof	97	14	3342.40	341.30	1140755.01	0.29	187.46	0.00	0.00
7	83	14	3205.87	279.79	896954.63	0.23	147.40	187.46	18183.87
6	69	14	3205.87	221.07	708726.22	0.18	116.47	334.86	12234.06
5	55	14	3205.87	165.56	530772.41	0.13	87.22	451.33	8036.18
4	41	14	3279.37	113.84	373327.38	0.09	61.35	538.55	4797.26
3	27	14	3279.37	66.83	219169.05	0.06	36.02	599.90	2515.33
2	13	13	3279.37	26.32	86311.41	0.02	14.18	635.92	972.45
1	0	0	3205.58	0.00	0.00	0.00	0.00	650.10	184.39
$\sum w_x h_x^k$ = 3,956,016.09			$\sum F_x$ = Base Shear = 650.10 kips			Overturning Moment = 46,923.54 kip-ft			

Appendix C: Seismic Load Analysis

Slab Weights				
Floor	Area (ft^2)	Perimeter (ft)	Weight of Concrete Slab (psf)	Total Slab Weight (kips)
1	19968	692	112.5	2246.4
2	19968	692	112.5	2246.4
3	19968	692	112.5	2246.4
4	19968	692	112.5	2246.4
5	19968	692	112.5	2246.4
6	19968	692	112.5	2246.4
7	19968	692	112.5	2246.4
Roof	19968	692	150	2995.2
		Total Slab Weight =		18720

Column Weights				
Column	Number of Columns	Length (ft)	w_c (pcf)	Total Weight (kips)
16" x 16"	33	13	150.00	114.40
16" x 16"	309	14	150.00	1153.60
20" x 20"	35	13	150.00	189.58
20" x 20"	105	14	150.00	612.50
Total	482	Total Column Weight =		2070.08

Shear Wall Weights					
Length (ft)	Number of Shear Walls	Height (ft)	Thickness (in)	w_c (pcf)	Total Weight (kips)
16	6	13	16	150	249.60
16	40	14	16	150	1792.00
26	6	13	16	150	405.60
26	38	14	16	150	2766.40
Total	90	Total Shear Wall Weight =			5213.60

Appendix D: Two-Way Flat Plate Design

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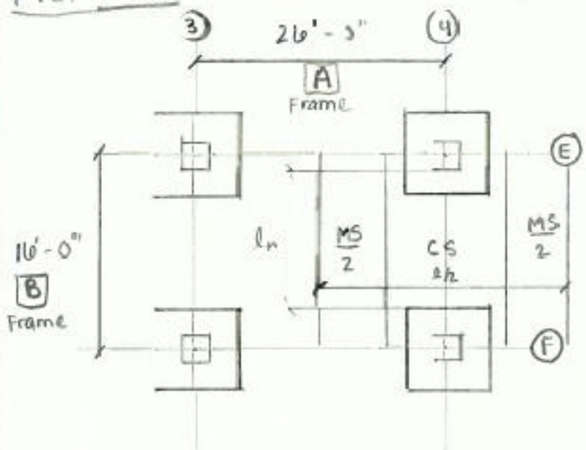
Flat Plate



→ Utilize 26'-0" X16'-0" Bays

Assume 16"x16" columns

Short Span → Frame B

$l_n = 16' - 2\left(\frac{16}{2\cdot 12}\right) = 14.7'$

Long Span → Frame A

$l_n = 26' - 2\left(\frac{16}{2\cdot 12}\right) = 24.7'$

Determine Slab Thickness

→ Slab without interior beams

ACI 9.5.3.2:

$$t = \begin{cases} 4\text{ in} \\ \text{Int.} = \frac{l_n}{33} = \frac{24.7'(12\text{in}/\text{ft})}{33} = 8.9\text{in} \\ \text{Ext.} = \frac{l_n}{33} = \frac{24.7'(12\text{in}/\text{ft})}{33} = 8.9\text{in} \end{cases}$$

→ use 9" slab

→ Use Equivalent Frame Method

Good for columns as low as 16" x 16"

Appendix D: Two-Way Flat Plate Design

Cody Scheller | Senior Thesis | Flat Slab 2

Factored Design Loads

→ Slab Dead Load = $\frac{9''}{12}(150) = 112.5$ psf

→ $1.2w_d = 1.2(112.5+30) = 171$ psf

→ $1.6w_\ell = 1.6(80) = 128$ psf

→ $w_u = 1.2w_d + 1.6w_\ell = 299$ psf

Total Factored Static Moment

→ Frame A (16" x 16" Columns)

$M_o = \frac{1}{8} w_u \ell_2 \ell_n^2$ $\ell_n = 24.7'$

$M_o = \frac{1}{8}(299)(16')(24.7')^2/1000 = 364.83^{1\text{-}k}$

↳ Longitudinal Distribution of M_o

$M^- = 0.65 M_o = -237.14^{1\text{-}k}$

$M^+ = 0.35 M_o = +127.69^{1\text{-}k}$

→ Frame B (16" x 16" columns)

$M_o = \frac{1}{8} w_u \ell_2 \ell_n^2$ $\ell_n = 14.7'$

$M_o = \frac{1}{8}(299)(26')(14.7')^2/1000 = 210.0^{1\text{-}k}$

↳ Longitudinal Distribution of M_o

$M^- = 0.65 M_o = -136.5^{1\text{-}k}$

$M^+ = 0.35 M_o = +73.5^{1\text{-}k}$

Appendix D: Two-Way Flat Plate Design

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Interior Tranverse Distribution %'s

Item/Description	Frame A	Frame B
1.) Total Tranverse Width	312"	192"
2.) Column Strip Width	96"	96"
3.) Middle Strip Width	216"	96"
4.) Torsional Constant ↳ no beams → $C = 0$	0	0
5.) $I_s = \ell t^3/12 = (192")(9)^3/12$	11,664	11,664
6.) $\beta_t = C/2I_s = 0$	0	0
7.) $\alpha_1 = I_b/I_s$	0	0
8.) ℓ_2/ℓ_1 = aspect ratio	$\frac{312}{192} = 1.625$	$\frac{192}{312} = 0.62$
9.) $\alpha_1 \ell_2 \ell_1 = 0$	0	0
10.) M^+ % to C.S.	60%	60%
11.) M^- % to C.S.	75%	75%

Appendix D: Two-Way Flat Plate Design

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Frame A

	M^- (1-k)	M^+ (1-k)
Total Moment	-237.14 1-k	+127.69 1-k
% to C.S.	75%	60%
Moment in C.S.	-177.86	+76.61
Moment in M.S.	-59.29	+51.08

Frame B

	M^- (1-k)	M^+ (1-k)
Total Moment	-136.5 1-k	+73.5 1-k
% to C.S.	75%	60%
Moment in C.S.	-102.38	+44.10
Moment in M.S.	-34.13	+29.4

Calculate Effective Depth

→ Moments in long direction are larger than in the short direction

↳ ∴ Need to put reinforcement lower in the long direction

$d_{long} = 9.00'' - 0.75'' - \frac{1}{2}(0.75'') = 7.875''$

$d_{short} = d_{long} - (\#6 \text{ bar diameter}) = 7.125''$

Appendix D: Two-Way Flat Plate Design

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$\boxed{\text{Frame A}}$ → Interior Bays

Negative Moments

column strip = $(-177.86^{1\text{-k}})/8\text{ ft} = -22.23\text{ k/ft}$

middle strip = $(-59.29^{1\text{-k}})/18\text{ ft} = -3.29\text{ k/ft}$

Positive Moments

column strip = $(+76.61^{1\text{-k}})/8\text{ ft} = +9.58\text{ k/ft}$

middle strip = $(+51.08^{1\text{k}})/18\text{ ft} = +2.84\text{ k/ft}$

$\boxed{\text{Frame B}}$ → Interior Bays

Negative Moments

column strip = $(-102.38^{1\text{-k}})/8\text{ ft} = -12.79\text{ k/ft}$

middle strip = $(-34.13^{1\text{-k}})/8\text{ ft} = -1.89\text{ k/ft}$

Positive Moments

column strip = $(+44.10^{1\text{-k}})/8\text{ ft} = +5.51\text{ k/ft}$

middle strip = $(+29.10^{1\text{-k}})/8\text{ ft} = +3.64\text{ k/ft}$

Appendix D: Two-Way Flat Plate Design

Cody Scheller | Senior Thesis | Flat Slab 6

Design of Reinforcement → Column Strip → A

	M^- (1-k)	M^+ (1-k)
1.) M_u (1-k)	-177.86	+76.61
2.) C.S. width = b	96"	96"
3.) d_{long}	7.875"	7.875"
4.) $M_n = M_u/\phi = 0.9$	-197.62	+85.12
5.) $R = M_n/bd^2 \times 12000$	398.33	154.42
6.) ρ_{req} Tables $R = \rho f_y \left(1 - 0.59\rho \frac{f_y}{f'_c}\right)$	0.00708	0.00264
7.) $A_{s,required} = \rho bd$	$\boxed{5.35 in^2}$	$\boxed{1.99 in^2}$
8.) $A_{s,min} = (0.002)bt$	$1.728 in^2$	$1.728 in^2$
9.) $N = \frac{\text{Larger of 7 \& 8}}{\text{\#6 Bar Area}}$	$\frac{5.35}{0.4481} = 11.9$	$\frac{1.99}{0.4481} = 4.44$
10.) $N_{min} = \frac{\text{Strip Width}}{2t}$	$\frac{96}{2(9)} = 5.33$	$\frac{96}{2(9)} = 5.33$
11.) Larger of 9 & 10	$\boxed{N = 12 \text{ \#6 TOP BARS}}$	$\boxed{N = 6 \text{ \#6 BOT. BARS}}$

Appendix D: Two-Way Flat Plate Design

Cody Scheller | Senior Thesis | Flat Slab | 7

Design of Reinforcement → Middle Strip - A

	M^- (1-K)	M^+ (1-K)
1.) Mu (1-K)	-59.29	+51.08
2.) M.S. width = b	216"	216"
3.) d_{long}	7.875"	7.875"
4.) $M_n = M_u/\phi$; $\phi = 0.9$	-65.88	+56.76
5.) $R = M_n/bd^2 \times 12000$	59.02	50.85
6.) ρ_{req} = Tables	0.000992	0.000854
7.) $A_{s,required} = \rho bd$	1.69 in^2	1.45 in^2
8.) $A_{s,min} = (0.002)bt$	$\boxed{3.89\ in^2}$	$\boxed{3.89\ in^2}$
9.) $N = \frac{\text{Larger of 7 \& 8}}{\text{\#6 Bar Area}}$	$\frac{3.89}{0.4481} = 8.68$	8.68
10.) $N_{min} = \frac{\text{Strip Width}}{2t}$	$\frac{216}{2(9)} = 12$	12
11.) Larger of 9 & 10	$\boxed{N = 12\ \#6\ \text{TOP BARS}}$	$\boxed{N = 12\ \#6\ \text{BOT BARS}}$

Appendix D: Two-Way Flat Plate Design

Cody Scheller | Senior Thesis | Flat Slab | 8

Design of Reinforcement → Column Strip → B

	$M^-(1-k)$	$M^+(1-k)$
1.) $M_u(1-k)$	-102.38	+44.10
2.) C.S. width = b	96"	96"
3.) d_{short}	7.125"	7.125"
4.) $M_n = M_u/\phi\, 0.9$	-113.76	+49.0
5.) $R = M_n/bd^2 \times 12000$	280.11	120.65
6.) ρ_{req} = Tables	0.00488	0.00205
7.) $A_{s,required} = \rho bd$	$\boxed{3.34\ in^2}$	$1.402 in^2$
8.) $A_{s,min} = (0.002)bt$	$1.73 in^2$	$\boxed{1.73\ in^2}$
9.) $N = \frac{\text{Larger of 7 \& 8}}{\#6 \text{ Bar Area}}$	$\frac{3.34}{0.4481} = 7.45$	$\frac{1.73}{0.4481} = 3.86$
10.) $N_{min} = \frac{\text{Strip Width}}{2t}$	$\frac{96}{2(9)} = 5.33$	5.33
11.) Larger of 9 & 10	N = 8 #6 TOP BARS	N = 6 #6 BOT BARS

Appendix D: Two-Way Flat Plate Design

Cody Scheller | Senior Thesis | Flat Slab | 9

Design of Reinforcement → Middle Strip - B

	$M^-(1-k)$	$M^+(1-k)$
1.) $M_u(1-k)$	-34.13	+29.4
2.) M.S width = b	96"	96"
3.) d_{short}	7.125"	7.125"
4.) $M_n = M_u/\phi$: 0.9	-37.92	+32.67
5.) $R = M_n/bd^2 \times 12000$	93.37	72.39
6.) ρ_{req} = Tables	0.00158	0.00122
7.) $A_{s,required} = \rho bd$	1.08 in²	0.834 in²
8.) $A_{s,min} = (0.002)bt$	$\boxed{1.73 in^2}$	$\boxed{1.73 in^2}$
9.) $N = \frac{\text{Larger of 7 \& 8}}{\text{\#6 Bar Area}}$	$\frac{1.73}{0.4481} \approx 3.86$	3.86
10.) $N_{min} = \frac{\text{Strip Width}}{2t}$	5.33	5.33
11.) Larger of 9 & 10	$\boxed{N = 6 \text{ \#6 TOP BARS}}$	$\boxed{N = 6 \text{ \#6 BOT. BARS}}$

Appendix D: Two-Way Flat Plate Design

Exterior Tranverse Distribution %'s

Item/Description	Frame A	Frame B
1.) Total Tranverse Width	164"	104"
2.) Column Strip Width	56"	56"
3.) Middle Strip Width	108"	48"
4.) Torsional Constant ↳ no beams → C = 0	0	0
5.) $I_s = \ell t^3/12$	6,318	6,318
6.) B_t	0	0
7.) α_1	0	0
8.) ℓ_2/ℓ_1	1.56	0.63
9.) $\alpha_1 \ell_2 \ell_1$	0	0
10.) M^+ % to C.S.	60%	60%
11.) M^- % to C.S.	75%	75%

Appendix D: Two-Way Flat Plate Design

Cody Scheller | Senior Thesis | Flat Slab | 11

$\boxed{\text{Frame A}}$ → Exterior Bays

Negative Moments

column strip = $(-177.86^{1\text{-}k})/4.67' = -38.09\ ^{k}/ft$

middle strip = $(-59.29^{1\text{-}k})/9' = -6.59\ ^{k}/ft$

Positive Moments

column strip = $(+76.61^{1\text{-}k})/4.67' = +16.40\ ^{k}/ft$

middle strip = $(+51.08^{1k})/9' = +5.68\ ^{k}/ft$

$\boxed{\text{Frame B}}$ → Exterior Bays

Negative Moments

column strip = $(-102.38^{1\text{-}k})/4.67' = -21.92\ ^{k}/ft$

middle strip = $(-34.13^{1\text{-}k})/4' = -8.53\ ^{k}/ft$

Positive Moments

column strip = $(+44.10^{1\text{-}k})/4.67' = +9.44\ ^{k}/ft$

middle strip = $(+29.10^{1\text{-}k})/4' = +7.28\ ^{k}/ft$

Appendix D: Two-Way Flat Plate Design

Cody Scheller | Senior Thesis | Flat Slab 12

Design of Reinforcement → Column Strip → A

	$M^-(1-k)$	$M^+(1-k)$
1.) $M_u(1-k)$	-177.86	$+76.61$
2.) c.s. width = b	56"	56"
3.) d_{long}	7.875"	7.875"
4.) $M_n = M_u/\phi$	-197.62	$+85.12$
5.) $R = M_n/bd^2 \times 12000$	682.85	294.12
6.) ρ_{req}	0.0128	0.00514
7.) As, required	$\boxed{5.64 in^2}$	$\boxed{2.27 in^2}$
8.) $A_{s,min}$	$1.008 in^2$	$1.008 in^2$
9.) $N = \frac{\text{Larger of 7 \& 8}}{\text{\#6 Bar Area}}$	$\frac{5.64}{0.4481} = 12.9$	$\frac{2.27}{0.4481} = 5.07$
10.) $N_{min} = \frac{\text{Strip width}}{2t}$	3.11	3.11
11.) Larger of 9 & 10	N = 13 #6 TOP BARS	N = 6 #6 BOTTOM BARS

Appendix D: Two-Way Flat Plate Design

Cody Scheller | Senior Thesis | Flat Slab 13

Design of Reinforcement → Middle Strip → A

	$M^-(1-k)$	$M^+(1-k)$
1.) $M_u(1-k)$	-59.29	+51.08
2.) M.S. width = b	164"	164"
3.) d_{long}	7.875"	7.875"
4.) M_n	-65.88	+56.76
5.) R	77.73	66.97
6.) ρ_{req}	0.00131	0.00113
7.) $A_{s,req}$	1.69 in^2	1.46 in^2
8.) $A_{s,min}$	$\boxed{2.95\ in^2}$	$\boxed{2.95\ in^2}$
9.) N	6.58	6.58
10.) N_{min}	9.11	9.11
11.) Larger of 9 & 10	$\boxed{N = 10\ \#6\ \text{TOP BARS}}$	$\boxed{N = 10\ \#6\ \text{BOT. BARS}}$

Appendix D: Two-Way Flat Plate Design

Cody Scheller | Senior Thesis | Flat Slab 14

Design of Reinforcement → column Strip → B

	$M^-(1-k)$	$M^+(1-k)$
1.) M_u (1-k)	-102.38	+44.10
2.) C.S. width = b	56"	56"
3.) d_{short}	7.125"	7.125"
4.) M_n	-113.76	+49.0
5.) R	480.19	206.83
6.) ρ_{req}	0.00867	0.00356
7.) $A_{s,req}$	$\boxed{3.46 in^2}$	$\boxed{1.42 in^2}$
8.) $A_{s,min}$	$1.008 in^2$	$1.008 in^2$
9.) N	7.72	3.17
10.) N_{min}	3.11	3.11
11.) Larger of 9 & 10	N = 8 #6 TOP BARS	N = 4 #6 BOT. BARS

Appendix D: Two-Way Flat Plate Design

Cody Scheller | Senior Thesis | Flat Slab | 15

Design of Reinforcement → Middle Strip → B

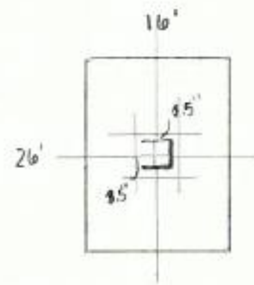
	$M^-(1-k)$	$M^+(1-k)$
1.) M_u (1-k)	-34.13	+29.4
2.) M.S width = b	104"	104"
3.) d_{short}	7.125"	7.125"
4.) M_n	-37.92	+32.67
5.) R	86.19	74.26
6.) ρ_{req}	0.00146	0.00125
7.) $A_{s,req.}$	1.08 in^2	0.926 in^2
8.) $A_{s,min}$	$\boxed{1.872 in^2}$	$\boxed{1.872 in^2}$
9.) N	4.18	4.18
10.) N_{min}	5.78	5.78
11.) Larger of 9 & 10	$\boxed{N = 6 \#6 \text{ TOP BARS}}$	$\boxed{N = 6 \#6 \text{ BOT BARS}}$

Appendix D: Two-Way Flat Plate Design

Cody Scheller | Senior Thesis | Flat Slab | 16

Design for Shear

$d = 10 - \underset{\text{cover}}{3/4} - 0.75 = 8.5''$



$\boxed{\text{Frame A}}$

$V_u = 0.299\text{ ksf}\left(\frac{26}{2} - \frac{24''/2 + 8.5''}{12\text{ in/ft}}\right)(16')$

$V_u = 54.02\text{ k}$

$\phi V_n = \phi V_c = 0.75\left[2\lambda\sqrt{f'c}\,bd\right]$

$\phi V_c = 0.75(2)(1)\sqrt{5000}(16')(12\text{ in/ft})(8.5) = 173.1\text{ k}$

$173.1\text{ k} > 54.02\text{ k} \quad \therefore \text{ ok} \checkmark$

$\boxed{\text{Frame B}}$

$V_u = 0.299\text{ ksf}\left(\frac{16}{2} - \frac{24''/2 + 8.5''}{12\text{ in/ft}}\right)(26')$

$V_u = 48.9\text{ k}$

$\phi V_n = \phi V_c = 0.75\left[2\lambda\sqrt{f'c}\,bd\right]$

$\phi V_c = 0.75(2)(1)\sqrt{5000}(26')(12\text{ in/ft})(8.5) = 281.3\text{ k}$

$281.3\text{ k} > 48.9\text{ k} \quad \therefore \text{ ok} \checkmark$

Appendix D: Two-Way Flat Plate Design

Cody Scheller | Senior Thesis | Flat Slab 17

Two-Way Punching Shear → Interior

$$V_n = 0.299\ ksf\left[(26')(16') - \left[\frac{24+2(8.5/2)}{12 in/ft}\right]\left[\frac{24+2(8.5/2)}{12 in/ft}\right]\right] =$$

$$V_n = 122.2\ K$$

$$b_o = 4[24+2(8.5/2)] = 130\ in.$$

$$V_c = \min\begin{cases} 4\lambda\sqrt{f'c}\,b_o d = 4(1.0)(\sqrt{5000})(130)(8.5) = \boxed{312.54\ K} \\ \left(2+\frac{4}{\beta}\right)\lambda\sqrt{f'c}\,b_o d = \left(2+\frac{4}{1}\right)(1.0)\sqrt{5000}(130)(8.5) = 468.8^K \\ \left(\frac{\alpha_s d}{b_o}+2\right)\lambda\sqrt{f'c}\,b_o d = \left[\frac{40(8.5)}{130}+2\right](1.0)\sqrt{5000}(130)(8.5) = 360.6^K \end{cases}$$

$\beta = \frac{24}{24} = 1$; $\alpha_s = 40$ for interior column

$\phi V_c = 0.75(312.54k) = 234.41 > V_n = 122.2K$ ∴ OK

Two-Way Punching Shear → Edge Column

$$V_n = 0.299\ ksf\left[(26')(9.25') - \left(\frac{24+2(8.5/2)}{12 in/ft}\right)\left(\frac{24+8.5/2}{12 in/ft}\right)\right] =$$

$$V_n = 70.0\ K$$

$$b_o = 2\left[24+2(8.5/2)\right] + 2\left[24+8.5/2\right] = 121.5 in$$

$\alpha_s = 30$ for edge columns

$$V_c = \min\begin{cases} 4\lambda\sqrt{f'c}\,b_o d = 4(1)\sqrt{5000}(121.5)(8.5) = \boxed{292.1\ K} \\ \left(2+\frac{4}{\beta}\right)\lambda\sqrt{f'c}\,b_o d = (2+4)(1)(\sqrt{5000})(121.5)(8.5) = 438.19^K \\ \left(\frac{\alpha_s d}{b_o}+2\right)\lambda\sqrt{f'c}\,b_o d = \left(\frac{30(8.5)}{121.5}+2\right)(1)\sqrt{5000}(121.5)(8.5) = 249.3^k \end{cases}$$

$\phi V_c = 0.75(292.1K) = 219.1 > V_n = 70.0^k$ ∴ OK

Appendix D: Two-Way Flat Plate Design

Cody Scheller | Senior Thesis | Flat Slab 18

Two-Way Punching Shear → Corner Columns

$$V_n = 0.299\,ksf\left[(14.25')(9.25') - \left[\frac{24+\frac{8.5}{2}}{12}\right]\left[\frac{24+\frac{8.5}{2}}{12}\right]\right] =$$

$$V_n = 37.75\,k$$

$$b_o = 4\left[24+\frac{8.5}{2}\right] = 113\,in$$

$$V_c = \min \begin{cases} 4\lambda\sqrt{f'_c}\,b_o d = 4(1.0)\sqrt{5000}\,(113)(8.5) = 271.7\,k \\ \left(2+\frac{4}{\beta}\right)\lambda\sqrt{f'_c}\,b_o d = (2+4)(1)(\sqrt{5000})(113)(8.5) = 407.5\,k \\ \left(\frac{\alpha_s d}{b_o}+2\right)\lambda\sqrt{f'_c}\,b_o d = \left(\frac{20(8.5)}{113}+2\right)(1)\sqrt{5000}\,(113)(8.5) = \boxed{238\,k} \end{cases}$$

$\alpha_s = 20$ for corner columns

$$\phi V_c = 0.75(238^k) = 178.5\,k > V_n = 37.75^k$$

$\therefore$ ok

→ No drop panels needed

Appendix D: Two-Way Flat Plate Design

Slab Weights				
Floor	Area (ft^2)	Perimeter (ft)	Weight of Concrete Slab (psf)	Total Slab Weight (kips)
1	19968	692	112.5	2246.4
2	19968	692	112.5	2246.4
3	19968	692	112.5	2246.4
4	19968	692	112.5	2246.4
5	19968	692	112.5	2246.4
6	19968	692	112.5	2246.4
7	19968	692	112.5	2246.4
Roof	19968	692	150	2995.2
		Total Slab Weight =		18720

Interior Bay Reinforcement - Long Direction		
Bar Position	Bar Size	Number of Bars
Column Strip		
Top Bars	#6	12
Bottom Bars	#6	6
Middle Strip		
Top Bars	#6	12
Bottom Bars	#6	12

Interior Bay Reinforcement - Short Direction		
Bar Position	Bar Size	Number of Bars
Column Strip		
Top Bars	#6	8
Bottom Bars	#6	6
Middle Strip		
Top Bars	#6	6
Bottom Bars	#6	6

Exterior Bay Reinforcement - Long Direction		
Bar Position	Bar Size	Number of Bars
Column Strip		
Top Bars	#6	13
Bottom Bars	#6	6
Middle Strip		
Top Bars	#6	10
Bottom Bars	#6	10

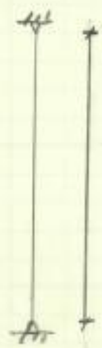
Exterior Bay Reinforcement - Short Direction		
Bar Position	Bar Size	Number of Bars
Column Strip		
Top Bars	#6	8
Bottom Bars	#6	4
Middle Strip		
Top Bars	#6	6
Bottom Bars	#6	6

Appendix E: Column Design

Cody Scheller | Senior Thesis | Column Design | 1

Check Column on Bottom Floor → Corner Column

→ (16"x16") Column



$13' = 156'' = \ell_c$

use load case $1.2D + 1.0L + 1.6W$

$\Sigma P_u = 865.24$ kips

$V_{us} = 969.54$

$\Delta_o = 0.13$

→ Is it a sway frame? (ACI 10.10.5.2)

$$Q = \frac{\Sigma P_u \Delta_o}{V_{us} \ell_c} \leq 0.05$$

$$Q = \frac{865.24(0.13)}{969.54(156)} = 0.00074 < 0.05 \checkmark$$

∴ Nonsway Frame

→ Check Slenderness

$$\frac{k \ell_u}{r} \leq 22$$

$k = 1.0$ (for nonsway)

$\ell_n = 168''$

$r = 0.3h = 0.3(16) = 4.8$

$$\frac{1.0(156)}{4.8} = 32.5 > 22$$ → Column is slender

Appendix E: Column Design

Cody Scheller | Senior Thesis | Column Design | 2

→ Moment Magnification Factor

$M_e = \delta_{ns} M_2$

$P_u = 502\text{ k}$

$M_2 = 135\text{ in-k}$

$M_1 = 0$

$C_m = 0.6 + 0.4\left(\cancel{M_1}^{\,0}/M_2\right) = 0.6$

$P_c = \dfrac{\pi^2 EI}{(k\ell_u)^2} = 40406.59\text{ k}$

$EI_{eff} = \dfrac{0.4 E_c I_g}{1+\beta_{dns}} = \dfrac{0.4(3600)(16^4/12)}{1+1.0} = 3.93\times10^6\text{ k-in}^2$

$\delta_{ns} = \dfrac{C_m}{1 - \dfrac{P_u}{0.75P_c}} = 0.6 < 1.0$ → Moment magnification does not influence column behavior.

Appendix E: Column Design

Cody Scheller | Senior Thesis | Column Design 3

Interior Column → First Floor

→ column size: 20"x20" → First Four Floors column size

→ $P_u = 780.83$ kips

→ assume $\rho_g = 0.03$

Required Area of the column

$$A_g = \frac{P_u}{0.8\phi[0.85 f'_c(1-\rho_g) + f_y \rho_g]}$$

$$A_g = \frac{780.83\,k}{0.8(0.65)[0.85(4)(1-0.03)+60(0.03)]}$$

$$A_g = 294.55\ in^2$$

$294.55\ in^2 < (20in \times 20in) = 400in^2$ ∴ ok

→ still use 20" x 20" column on first floor to be conservative.

Required Amount of Steel

$$A_{st} = \left|\frac{P_u - 0.8\phi(0.85 f'_c A_g)}{0.8\phi(f_y - 0.85 f'_c)}\right| \text{ or } \boxed{A_{st} = \rho bd = (0.03)(20)(20) = 12in^2}$$

controls

$$= \frac{780.83 - (0.8)(0.65)(0.85 \times 4 \times 400)}{(0.8)(0.65)(60 - (0.85)(4))} = 2.5\ in^2$$

∴ use 12 #9 bars → $12(1.00) = 12in^2$

→ $12in^2 = 12in^2$ ∴ ok

Ties → #3 ties

$S_{max} = \min[16d_B, 48d_t, b_{min}] = \min[18, 18, 20] = 18$ in

use #3 ties @ 18in

Appendix E: Column Design

Cody Scheller | Senior Thesis | Column Design | 4

Corner Column → First Floor

→ column size: 16"x 16" → All Floors

→ $P_u = 221.49$ kips

→ assume $\rho_g = 0.03$

Required Area of the column

$$A_g = \frac{P_u}{0.8\phi\left[0.85 f'_c (1-\rho_g) + f_y \rho_g\right]}$$

$$A_g = \frac{221.49}{0.8(0.65)\left[(0.85)(4)(1-0.03)+60(0.03)\right]}$$

$A_g = 84.6\ in^2 < 256\ in^2$ → 16"x 16" columns ok

Required Amount of Steel

$$A_{st} = \left|\frac{P_u - 0.8\phi\left[0.85 f'_c A_g\right]}{0.8\phi\,(f_y - 0.85 f'_c)}\right| \quad \text{or} \quad A_{st} = \rho bd = (0.03)(16)(16) = 7.68\ in^2$$

controls.

$$= \left|\frac{221.49 - 0.8(0.65)\left[0.85\times 4\times 256\right]}{0.8(0.65)\left[60 - 0.85(4)\right]}\right| = \boxed{7.85\ in^2}$$

∴ use 8 #9 bars → $8(1.00) = 8\ in^2$

→ $8\ in^2 > 7.85\ in^2$ ∴ ok

Ties → #3 ties

$S_{max} = \min\left[16d_b, 48d_b, b_{min}\right] = \min[18, 18, 16] = 16\ in$

use #3 ties @ 16 in

Appendix E: Column Design

Cody Scheller | Senior Thesis | Column Design 5

Exterior Column Design → First Floor

→ column size : 16" x 16" → All Floors

→ $P_u = 677.74$ kips

→ assume $\rho_g = 0.03$

Required Area of the Column

$$A_g = \frac{P_u}{0.8\,\Phi\left[0.85 f'c\,(1-\rho_g) + f_y \rho_g\right]}$$

$$A_g = \frac{677.74\text{ k}}{0.8(0.65)\left[0.85(4)(1-0.03)+60(0.03)\right]}$$

$A_g = 255.66\text{ in}^2 < 256\text{ in}^2$ → 16" x 16" columns ok ✓

Required Amount of Steel

$$A_{st} = \left|\frac{P_u - 0.8\phi(0.85 f'c A_g)}{0.8\phi(f_y - 0.85 f'c)}\right| \text{ or } A_{st} = \rho bd = (0.03)(16)(16) = 7.68\text{ in}^2$$

↕ same

$$= \frac{677.74-(0.8)(0.65)(0.85\times4\times256)}{(0.8)(0.65)\left[60-0.85(4)\right]} = 7.65\text{ in}^2$$

∴ use 8 #9 bars → $8(1.00) = 8\text{ in}^2$

→ $8\text{ in}^2 > 7.65\text{ in}^2$ ∴ ok

Ties → #3 ties

$S_{max} = \min\left[16d_b, 48d_t, b_{min}\right] = \min\left[18, 18, 16\right] = 16\text{ in}$

use #3 ties @ 16in.

Appendix E: Column Design

Cody Scheller | Senior Thesis | Column Design 6

Interior Column → Fifth Floor

→ column size: 16" x 16"

→ $P_u = 435.76$ kips

→ assume $\rho_g = 0.03$

Required Area of the Column

$$A_g = \frac{P_u}{0.8\,\phi\,[0.85 f'_c(1-\rho_g) + f_y \rho_g]}$$

$$A_g = \frac{435.76 \text{ kips}}{0.8(0.65)[0.85(4)(1-0.03) + 60(0.03)]}$$

$A_g = 164.38 \text{ in}^2 < 256 \text{ in}^2$ → ∴ use 16" x 16" columns

Required Amount of Steel

$$A_{st} = \frac{P_u - 0.8\phi(0.85 f'_c A_g)}{0.8\phi(f_y - 0.85 f'_c)} \quad \text{or} \quad \boxed{A_{st} = \rho bd = (0.03)(16)(16) = 7.68 \text{ in}^2}$$

controls.

$$= \frac{435.76 - (0.8)(0.65)[0.85 \times 4 \times 256]}{(0.8)(0.65)[60 - 0.85(4)]} = 0.57 \text{ in}^2$$

∴ use 8 #9 bars → $8(1.00) = 8 \text{ in}^2$

$8 \text{ in}^2 > 7.68 \text{ in}^2$ → ∴ ok

Ties → #3 ties

$S_{max} = \min[16 d_b, 48 d_t, b_{min}] = \min[18, 18, 16] = 16$ in.

use #3 ties @ 16in.

Appendix E: Column Design

Cody Scheller | Senior Thesis | Column Design | 7

→ Check P_o for each

$P_o = 0.85 f'_c (A_g - A_{st}) + f_y A_{st}$

$\phi P_n = \phi r P_o > P_u$

→ Interior Column → First Floor

$P_o = (0.85)(4)(400-12) + (60)(12) = 2039.2$

$\phi P_n = \phi r P_o = (0.65)(0.8)(2039.2) = 1060.38\ k$

$1060.38 > 783.83$ ✓ ok

→ Corner Column → First Floor

$P_o = (0.85)(4)(256-7.85) + (60)(7.85) = 1314.71\ k$

$1314.71\ k > 221.49\ k$ ✓ ok

→ Exterior Column → First Floor

$P_o = (0.85)(4)(256-7.68) + (60)(7.68) = 1305.09\ k$

$1305.09\ k > 677.74$ kips

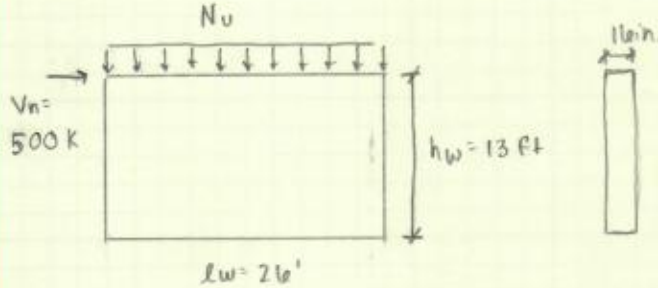
→ Exterior Column → Fifth Floor

$P_o = (0.85)(4)(256-7.68) + 60(7.68) = 1305.09\ k$

$1305.09\ k > 435.76\ k$

Appendix F: Shear Wall Design

Cody Scheller | Senior Thesis | Shear Walls | 1



Nu = self weight of shear walls above (84')

$N_u = (84')(16in/12in/ft)(26')(150pcf) = 436.8\,k$

$\phi V_n \geq V_n = V_c + V_s$

$\underline{V_c}$

Simplified Method:

$V_c \leq 2\sqrt{f'c}\,hd$ $\qquad d = 0.8\,l_n$

$\leq 2\sqrt{4000}(13)(249.6)/1000$ $\qquad = 0.8(26 \times 12) = 249.6\,in$

$= 410.44\,k$ ← USE THIS

Other: (min of 1 or 2)

① $V_c \leq 3.3\sqrt{f'c}\,hd + \frac{N_u d}{4 l_w}$

$\leq (3.3)\sqrt{4000}(16)(249.6)/1000 + \frac{436.8(249.6)}{4(26 \times 12)}$

$V_c \leq 920.9\,k$

② $V_c \leq \left[0.6\sqrt{f'c} + \frac{l_w(1.25\sqrt{f'c} + 0.2N_u/l_w h)}{M_u/V_u - l_w/2}\right] hd$

$M_u = 500(13) = 6500'^k = 78\,000\,in\text{-}k$

$V_c \leq \left[0.6\sqrt{4000} + \frac{(26 \times 12)(1.25\sqrt{4000} + 0.2(249.6)/(26 \times 12 \times 13))}{\frac{78000}{500}}\right](13)(249.6)$

$V_c \leq 636.2\,k$

Appendix F: Shear Wall Design

Cody Scheller | Senior Thesis | Shear Walls 2

→ Horizontal Reinforcement

$\frac{1}{2} \phi V_c = \frac{1}{2}(0.75)(410.44) = 153.9k < 500k$

∴ Needs Reinforce.

$V_u \leq \phi V_n = \phi (V_c + V_s)$

$500 = 0.75(410.44 + V_s)$ → $V_s = 256.23$

→ Try 2-#4 @ 10"

$V_s = \frac{A_v f_y d}{s} = \frac{2(0.2)(60)(249.6)}{10} = 599.04 > 256.23$

∴ ok ✓

$P_t = \frac{A_v}{sh} = \frac{2(0.2)}{10(13)} = 0.003 \geq 0.0025$ ✓ ok.

$$S \leq \begin{cases} l_w/5 = 26(12)/5 = 62.4\text{in} \\ 3h = 3(13) = 39\text{in} \\ 18\text{in}. \leftarrow \text{controls} \end{cases}$$

18in > 10in ✓ ∴ ok

$\boxed{\therefore \text{use 2-\#4 @ 10" o.c. for horizontal reinforcement}}$

→ Vertical Reinforcement

$P_l = \frac{A_v}{sh} \geq 0.0025 + 0.5\left(2.5 - \frac{h_w}{l_w}\right)(P_t - 0.0025)$

$P_l = 0.003 \geq 0.0022$

$\boxed{\therefore \text{use 2-\#4 @ 10" o.c. for vertical reinforcement}}$

Appendix F: Shear Wall Design

Cody Scheller | Senior Thesis | Shear Walls | 3

Flexural Design

$C = T$

$0.85 f'c\, b a = A_s f_y$

$M_u = A_s f_y \left(d - \frac{a}{2}\right)$ or $M_u = A_s f_y (jd) \rightarrow jd = 0.9d$

$\rightarrow d = 249.6\text{in}$

$\rightarrow jd = 0.9(249.6\text{in}) = 224.64$.

$\rightarrow M_u = 500(13) = 6500'^{k}$

$\rightarrow M_u = \phi M_n = \phi A_s f_y jd$

$= 6500(12\text{in/ft}) = 0.9 A_s (60)(224.64)$

$A_s = 6.4\text{in}^2$

$C = T$

$a = \dfrac{A_s f_y}{0.85 f'c\, b} = \dfrac{6.4\text{in}^2 (60000)}{0.85(4000)(16)} = 7.06\text{in}$

$jd = d - \frac{a}{2} = 249.6 - \frac{7.06}{2} = 246.07$

$A_s = \dfrac{6500(12)}{0.9(60)(246.07)} = 5.87\text{in}^2$

Try 6 - #9s, $A_s = 6.0\text{in}^2$

Appendix F: Shear Wall Design

Cody Scheller | Senior Thesis | Shear Walls | 4

→ Tension Controlled Section

$d_t = 26.0'(12in/ft) - 3'' = 309in$

$C = T$

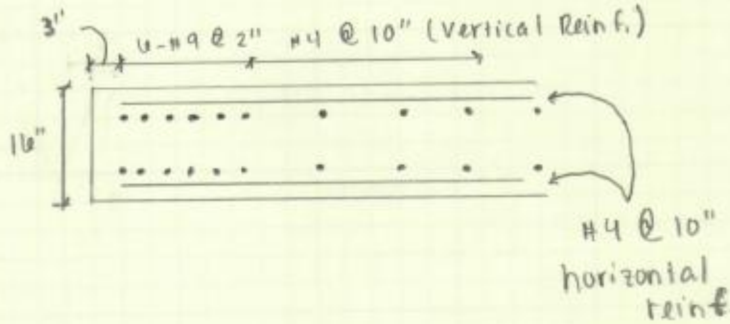
$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{6(60000)}{0.85(4000)(16)} = 6.62in$$

$$c = \frac{a}{B_1} = \frac{6.62in}{0.85} = 7.79in$$

$$\varepsilon_t = \varepsilon_u\left(\frac{d_t - c}{c}\right) = 0.003\left(\frac{309 - 7.79}{7.79}\right) =$$

$\rightarrow = 0.116 > 0.005$ ∴ ok ✓

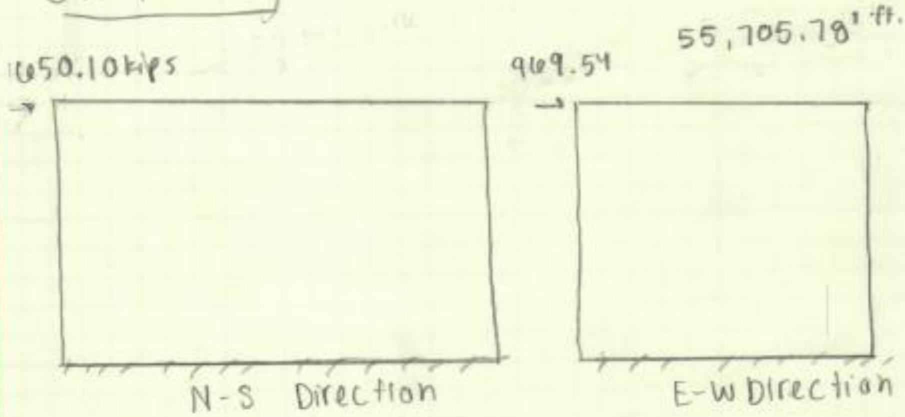
∴ Tension controls.



Appendix G: Overturning Calculation

Cody Scheller | Senior Thesis 1

Overturning



→ Seismic Loads control in N-S Direction

→ Wind Loads control in E-W Direction.

→ Overturning moments caused from lateral forces will be counteracted by the Dead Loads. → for Seismic

foundation Area = 21216
Building weight = 26000 k

Stress due to dead load

$= \frac{\text{Building Weight}}{\text{foundation Area}} = \frac{26000}{21216} = 1.23 \times 1000\text{ lb} =$ 1225.5 psf

Stress due to Seismic

$= \frac{1650.1(1000)}{21216} = 30.64$ $\qquad \frac{30.64}{1225.5} = 2.5\%$ of dead load.

∴ ok

Appendix G: Overturning Calculation

Cody Scheller | Senior Thesis | 2

Overturning

→ For the wind load, the resisting moment needs to be greater than the moment the wind puts on the building.

$$M_r = \text{weight of building} \times \frac{\text{width}}{2}$$

$$= 26000^{k} \times \frac{96'}{2} = 1248000^{'k}$$

$$1{,}248{,}000^{'k} > 55{,}705.78^{'k}$$

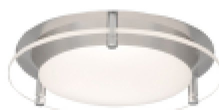
$\therefore$ ok

Appendix H: Lighting Breadth Material

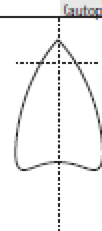
gotham®

Luminaire Type:
Catalog Number (autopopulated):

Ring



Disk



Gotham Architectural Downlighting
Decorative LED Downlights

6" Evo®

A-Series LED, Drop Luminous Ring/Disk

Solid-State Lighting

FEATURES

OPTICAL SYSTEM
- Self-flanged semi-specular, matte-diffuse or specular lower reflector
- Patented Bounding Ray™ optical design (U.S. Patent No. 5,800,050)
- 45° cutoff to source and source image
- Decorative element: 3/16"-thick low-iron glass with polished edges and sandblasted finish
- Precision-machined aluminum hardware with threaded spacers

MECHANICAL SYSTEM
- 16-gauge galvanized steel construction; maximum 1-1/4" ceiling thickness
- Telescopic mounting bars maximum of 32" and minimum of 15", preinstalled, 4" vertical adjustment
- Toolless adjustments post installation
- Junction box capacity: 8 (4 in, 4 out) 12AWG rated for 90°C
- Light engine and driver accessible through aperture

ELECTRICAL SYSTEM
- Fully serviceable and upgradeable LED light engine
- 70% lumen maintenance at 50,000 hours based on IESNA LM-79-2008
- 120-277VAC, 50/60hz power supply with 0-10V dimming (10-100%); rated for 50,000-hour life
- Overload and short circuit protected

LISTINGS
- Fixtures are CSA certified to meet US and Canadian standards; wet location, covered ceiling

WARRANTY
- 5-year limited warranty. Complete warranty terms located at: www.acuitybrands.com/CustomerResources/Terms_and_conditions.aspx

ORDERING INFORMATION

EXAMPLE: ALED 35/10 6AR DLR 120

Series	Color temperature		Nominal lumen values		Aperture/Trim color		Decorative element		Finish		Voltage
ALED	**27/**	2700 K	**10**	1000 lumens	**6AR**	Clear	**DLR**	Drop luminous ring	(blank)	Semi-specular	**MVOLT**
	30/	3000 K	**14**	1400 lumens	**6PR**	Pewter	**DLD**	Drop luminous disk	**LD**	Matte diffuse	**120**
	35/	3500 K	**18**	1800 lumens	**6WTR**	Wheat			**LS**	Specular	**277**
	41/	4100 K			**6GR**	Gold					**347**
					6WR[1]	White					

Driver		Options			
(blank)[2]	0-10V dimming driver. Minimum dimming level 10%	**SF**	Single fuse	**TRBL**	Black painted flange
ECOS3[2,3]	Lutron Hi-Lume® dimming driver. Minimum dimming level 1%	**LRC**	Lithonia Reloc® system	**ELR**[5]	Emergency battery pack with remote test switch
		NSD[4]	Sensor Switch nLight™ dimming relay	**CP**	Chicago plenum
		TRW[1]	White painted flange		

ACCESSORIES order as separate catalog numbers (shipped separately)	
ISD BC	0-10V wallbox dimmer. Refer to ISD-BC.

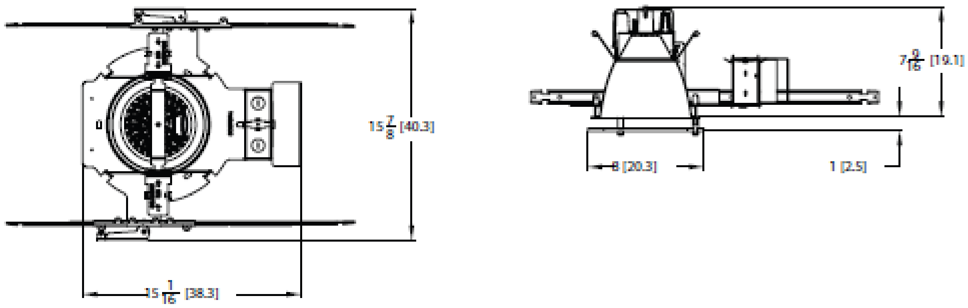
Appendix H: Lighting Breadth Material

6" EVO
A-Series LED, Drop Luminous Ring/Disk
Solid-State Lighting

gotham

DIMENSIONAL DATA

All dimensions are inches (centimeters) unless otherwise noted.



Aperture: 6-1/4 (15.9)
Ceiling Opening: 7-1/8 (18.1)
Overlap Trim: 7-3/4 (19.6)

Appendix H: Lighting Breadth Material

gotham

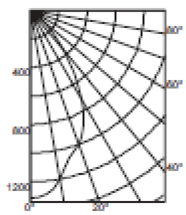
6" EVO
A-Series LED, Drop Luminous Ring/Disk
Solid-State Lighting

PHOTOMETRY

Distribution Curve | Distribution Data | Output Data | Coefficient of Utilization | Illuminance: Single Luminaire 30" Above Floor

ALED 35/18 6AR DLD

INPUT WATTS: 37, DELIVERED LUMENS: 1505, LM/W=40.7, 0.9 S/MH, TEST NO. LTL19773



Ave	Lumens	
0	1318	
5	1293	120
15	1069	304
25	888	402
35	534	334
45	220	175
55	84	77
65	52	52
75	30	31
85	9	11
90	0	

Zone	Lumens	% Lamp
0° - 30°	826.0	54.9
0° - 40°	1159.8	77.0
0° - 60°	1411.7	93.8
0° - 90°	1505.3	100.0
90° - 180°	0.0	0.0
0° - 180°	1505.3	*100.0

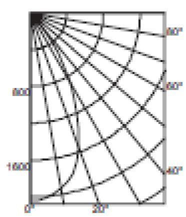
*Efficiency

pf	20%								
pc	80%			70%			50%		
pw	50%	30%	10%	50%	30%	10%	50%	30%	10%
0	119	119	119	116	116	116	111	111	111
1	109	106	103	106	104	101	102	100	98
2	99	94	90	97	93	89	94	90	87
3	91	85	80	89	84	79	87	82	78
4	83	77	72	82	76	71	80	75	70
5	77	70	65	76	70	65	74	69	64
6	71	64	59	71	64	59	69	63	59
7	66	59	55	66	59	54	64	58	54
8	62	55	50	61	55	50	60	54	50
9	58	51	47	57	51	47	56	51	46
10	54	48	43	54	48	43	53	47	43

Mounting Height	Initial FC Center Beam	50% beam - 50.1° Diameter	FC	10% beam - 83.5° Diameter	FC
8.0	43.6	5.1	21.8	9.8	4.4
10.0	23.4	7.0	11.7	13.4	2.3
12.0	14.6	8.9	7.3	17.0	1.5
14.0	10.0	10.7	5.0	20.5	1.0
16.0	7.2	12.6	3.6	24.1	0.7

ALED 35/18 6AR DLR

INPUT WATTS: 37, DELIVERED LUMENS: 1696, LM/W=45.8, 0.9 S/MH, TEST NO. LTL19774



Ave	Lumens	
0	2062	
5	2015	190
15	1803	498
25	1215	552
35	520	320
45	81	72
55	32	30
65	19	19
75	11	11
85	4	5
90	0	

Zone	Lumens	% Lamp
0° - 30°	1239.4	73.1
0° - 40°	1559.8	91.9
0° - 60°	1682.2	99.0
0° - 90°	1696.6	100.0
90° - 180°	0.0	0.0
0° - 180°	1696.6	*100.0

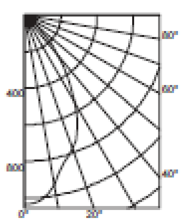
*Efficiency

pf	20%								
pc	80%			70%			50%		
pw	50%	30%	10%	50%	30%	10%	50%	30%	10%
0	119	119	119	116	116	116	111	111	111
1	110	108	106	108	106	104	104	102	101
2	103	99	95	101	97	94	98	95	92
3	96	91	87	94	90	86	92	88	85
4	90	86	80	88	83	79	86	82	78
5	84	78	74	83	78	74	81	76	73
6	79	73	69	78	73	68	77	72	68
7	74	68	64	74	68	64	72	67	64
8	70	66	60	70	64	60	69	63	60
9	66	61	57	66	60	56	65	60	56
10	63	57	53	62	57	53	62	57	53

Mounting Height	Initial FC Center Beam	50% beam - 46.9° Diameter	FC	10% beam - 73.3° Diameter	FC
8.0	67.5	4.8	33.7	8.2	6.7
10.0	36.3	6.5	18.1	11.2	3.6
12.0	22.6	8.2	11.3	14.1	2.3
14.0	15.4	10.0	7.7	17.1	1.5
16.0	11.2	11.7	5.6	20.1	1.1

ALED 35/14 6AR DLD

INPUT WATTS: 30, DELIVERED LUMENS: 1167, LM/W=38.9, 0.9 S/MH, TEST NO. LTL19772



Ave	Lumens	
0	1096	
5	1095	94
15	835	237
25	691	313
35	412	258
45	168	134
55	65	60
65	40	40
75	23	24
85	7	8
90	0	

Zone	Lumens	% Lamp
0° - 30°	644.3	55.2
0° - 40°	901.9	77.3
0° - 60°	1095.0	93.8
0° - 90°	1167.4	100.0
90° - 180°	0.0	0.0
0° - 180°	1167.4	*100.0

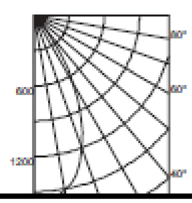
*Efficiency

pf	20%								
pc	80%			70%			50%		
pw	50%	30%	10%	50%	30%	10%	50%	30%	10%
0	119	119	119	116	116	116	111	111	111
1	109	106	103	106	104	101	102	100	98
2	99	94	90	97	93	89	94	90	87
3	91	85	80	89	84	79	87	82	78
4	86	77	72	82	76	71	80	75	71
5	77	70	65	76	70	65	74	69	64
6	72	65	60	71	64	59	69	63	59
7	67	60	55	66	59	55	64	59	54
8	62	55	51	61	55	50	60	54	50
9	58	51	47	58	51	47	57	51	47
10	55	48	44	54	48	44	53	47	43

Mounting Height	Initial FC Center Beam	50% beam - 49.7° Diameter	FC	10% beam - 83.2° Diameter	FC
8.0	34.2	5.1	17.1	9.8	3.4
10.0	18.4	6.9	9.2	13.3	1.8
12.0	11.5	8.8	5.7	16.9	1.1
14.0	7.8	10.7	3.9	20.4	0.8
16.0	5.7	12.5	2.8	24.0	0.6

ALED 35/14 6AR DLR

INPUT WATTS: 31, DELIVERED LUMENS: 1315, LM/W=42.4, 0.9 S/MH, TEST NO. LTL19771



Ave	Lumens	
0	1601	
5	1578	148
15	1404	388
25	945	429
35	405	247
45	62	55
55	24	23
65	14	14
75	8	8
85	3	3
90	0	

Zone	Lumens	% Lamp
0° - 30°	966.6	73.3
0° - 40°	1211.4	92.1
0° - 60°	1289.6	98.0
0° - 90°	1315.9	100.0
90° - 180°	0.0	0.0
0° - 180°	1315.9	*100.0

*Efficiency

pf	20%								
pc	80%			70%			50%		
pw	50%	30%	10%	50%	30%	10%	50%	30%	10%
0	119	119	119	116	116	116	111	111	111
1	111	108	106	108	106	104	104	102	101
2	103	99	95	101	97	94	98	95	92
3	96	91	87	94	90	86	92	88	85
4	90	84	80	89	83	79	86	82	78
5	84	78	74	83	78	74	81	77	73
6	79	73	69	78	73	69	77	72	68
7	74	69	64	74	68	64	73	67	64
8	70	64	60	70	64	60	69	64	60
9	67	61	57	66	60	57	65	60	56
10	63	57	54	63	57	53	62	57	53

Mounting Height	Initial FC Center Beam	50% beam - 46.6° Diameter	FC	10% beam - 73.1° Diameter	FC
8.0	52.9	4.7	26.5	8.2	5.3
10.0	28.5	6.5	14.2	11.1	2.8
12.0	17.7	8.2	8.9	14.1	1.8
14.0	12.1	9.9	6.1	17.1	1.2
16.0	8.8	11.6	4.4	20.0	0.9

Appendix H: Lighting Breadth Material

Lumen Method Calculations

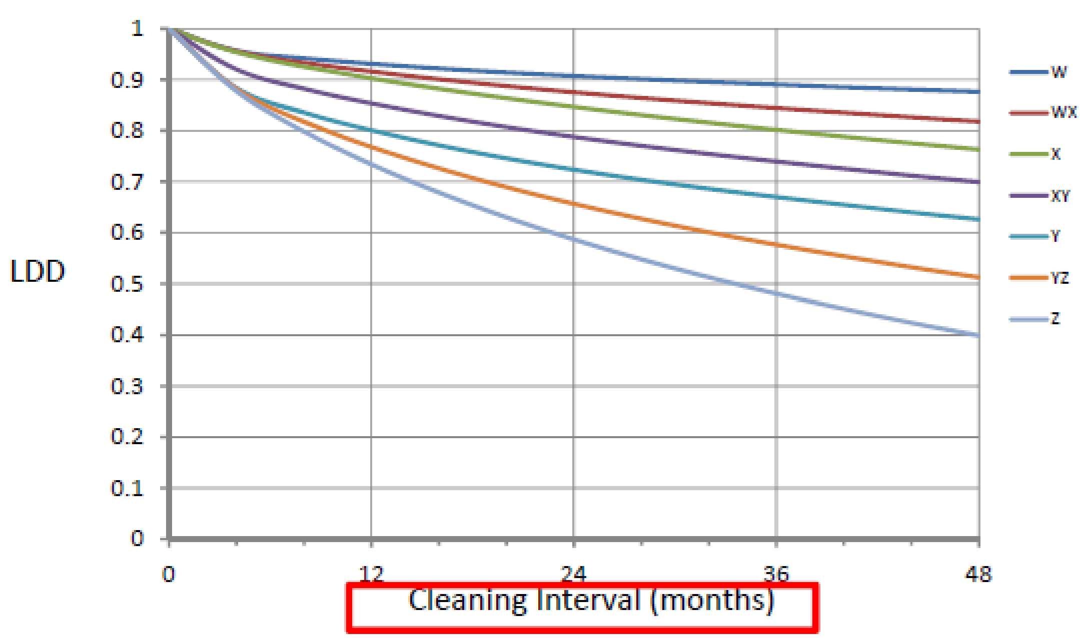
Lobby Luminaire:

Light Loss Factors(LLF)

- Lamp Lumen Depreciation(LLD) = LED Life = 0.7
- Luminaire Dirt Depreciation(LDD) = Clean, Direct, Open/Unvented

 = W + 24 Month Cleaning Interval = 0.91 = From Tables

Environment	Luminaire	DIRECT	SEMI-DIRECT	DIRECT-INDIRECT	SEMI-INDIRECT	INDIRECT
CLEAN	Open/Unvented	W	W	W	X	X
	Other	W	W	W	X	X
MODERATE	Open/Unvented	XY	XY	XY	Y	Y
	Other	X	X	X	Y	Y
DIRTY	Open/Unvented	Z	Z	Z	Z	Z
	Other	Y	Y	Y	Z	Z



Appendix H: Lighting Breadth Material

- Ballast Factor(BF) = 1.00
- Room Surface Dirt Depreciation = Direct, 10%, RCR of 5 = 0.96 = From Tables
- LLF = 0.7 x 0.91 x 1.00 x 0.96 = 0.61152

Room Cavity Ratio(RCR)

- $$\text{RCR} = \frac{\text{5 x Cavity Height x (Cavity Length + Cavity Width)}}{\text{(Cavity Length x Cavity Width)}}$$

- $$\text{RCR} = \frac{\text{5 x (12ft) x (20ft + 30ft)}}{\text{(20ft x 30ft)}} = 5$$

Coefficient of Utilization(CU)

- CU = 0.88 = Interpolated = From Tables in Photometric Viewer
- Reflectance's = 80/60/20

COEFFICIENTS OF UTILIZATION - ZONAL CAVITY METHOD

EFFECTIVE FLOOR CAVITY REFLECTANCE: 20%

RCC %:	80				70				50			30			10			0
RW %:	70	50	30	0	70	50	30	0	50	30	20	50	30	20	50	30	20	0
RCR: 0	1.19	1.19	1.19	1.19	1.16	1.16	1.16	1.00	1.11	1.11	1.11	1.06	1.06	1.06	1.02	1.02	1.02	1.00
1	1.13	1.11	1.08	1.06	1.11	1.08	1.06	.94	1.04	1.02	1.01	1.01	.99	.98	.97	.96	.95	.93
2	1.08	1.03	.99	.95	1.05	1.01	.97	.87	.98	.95	.92	.95	.92	.90	.92	.90	.88	.87
3	1.02	.96	.91	.87	1.00	.94	.90	.82	.92	.88	.85	.89	.86	.83	.87	.84	.82	.81
4	.97	.90	.84	.80	.95	.89	.83	.76	.86	.82	.78	.84	.81	.78	.82	.79	.77	.75
5	.92	.84	.78	.74	.91	.83	.78	.71	.81	.77	.73	.80	.76	.72	.78	.75	.72	.70
6	.88	.79	.73	.69	.86	.78	.73	.67	.77	.72	.68	.75	.71	.68	.74	.70	.67	.66
7	.84	.74	.69	.64	.82	.74	.68	.63	.73	.67	.64	.71	.67	.63	.70	.66	.63	.62
8	.80	.70	.64	.60	.79	.70	.64	.59	.69	.64	.60	.68	.63	.60	.67	.63	.59	.58
9	.76	.67	.61	.57	.75	.66	.60	.56	.65	.60	.56	.64	.60	.56	.63	.59	.56	.55
10	.73	.63	.57	.54	.72	.63	.57	.53	.62	.57	.53	.61	.56	.53	.60	.56	.53	.52

Lumen Method

- $$E_{avg} = \frac{\text{(luminaires) x (lamps/luminaire) x (lumens/lamp) x (CU) x (LLF)}}{\text{Room Area}}$$
- $$E_{avg} = \frac{\text{(12) x (1) x (1315 lumens) x (0.88) x (0.612)}}{(600\text{ft}^2)} = \text{14.2 footcandles}$$
- Use 12 Gotham 6'' LED Downlights

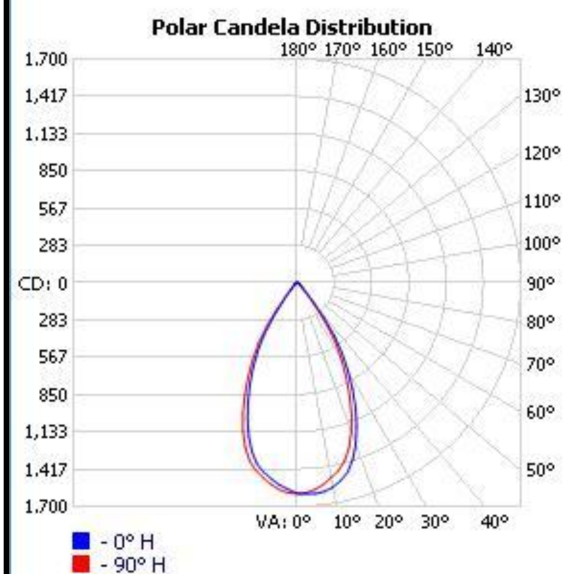
Appendix H: Lighting Breadth Material

INDOOR PHOTOMETRIC REPORT
CATALOG: ALED 35/14 6AR DLR

TEST #:	LTL19771
CATALOG #:	ALED 35/14 6AR DLR
LUMINAIRE:	6" LED DOWNLIGHT 3500K/1400 LUMENS WITH DECORATIVE GLASS RING
LAMP:	ONE 31-WATT LED, DOWNLIGHT POS.
LAMP OUTPUT:	1 LAMP, TOTAL LUMINAIRE LUMENS: 1315.8 TEST BASED ON ABSOLUTE PHOTOMETRY *
BALLAST:	XITANIUM LEDINTA1000C60DB
INPUT WATTAGE:	31.1
LUMINOUS OPENING:	CIRCULAR (DIA: 6")
CIE CLASS:	DIRECT
MAX CD:	1,609.0 AT HORIZONTAL: 0°, VERTICAL: 5°
SPACING CRITERION:	@ 0 = 0.92 @ 90 = 0.86

AcuityBrands. gotham®

PRODUCT LINKS
IES LDT PDF



Appendix H: Lighting Breadth Material

INDOOR PHOTOMETRIC REPORT
CATALOG: ALED 35/14 6AR DLR

AcuityBrands.

ZONAL LUMEN SUMMARY

ZONE	LUMENS	% LUMINAIRE
0-30	964.7	73.3%
0-40	1,211.4	92.1%
0-60	1,289.6	98%
60-90	26.2	2%
70-100	11.9	0.9%
90-120	0	0%
0-90	1,315.8	100%
90-180	0	0%
0-180	1,315.8	100%

LUMENS PER ZONE

ZONE	LUMENS	% TOTAL	ZONE	LUMENS	% TOTAL
0-10	148.3	11.3%	90-100	0	0%
10-20	387.7	29.5%	100-110	0	0%
20-30	428.7	32.6%	110-120	0	0%
30-40	246.7	18.7%	120-130	0	0%
40-50	55.4	4.2%	130-140	0	0%
50-60	22.8	1.7%	140-150	0	0%
60-70	14.3	1.1%	150-160	0	0%
70-80	8.5	0.6%	160-170	0	0%
80-90	3.4	0.3%	170-180	0	0%

AVERAGE LUMINANCE (CD/M2)

	0	22.5	45	67.5	90	112.5	135	157.5	180
0	87767	87767	87767	87767	87767	87767	87767	87767	87767
45	5815	5272	4962	4729	4652	4652	4652	4497	4186
55	2676	2485	2389	2294	2294	2294	2294	2198	2103
65	2075	1946	1946	1816	1816	1816	1816	1816	1686
75	1906	1694	1906	1694	1694	1694	1694	1694	1483
85	2516	1887	2516	1887	2516	1887	1887	1887	1887

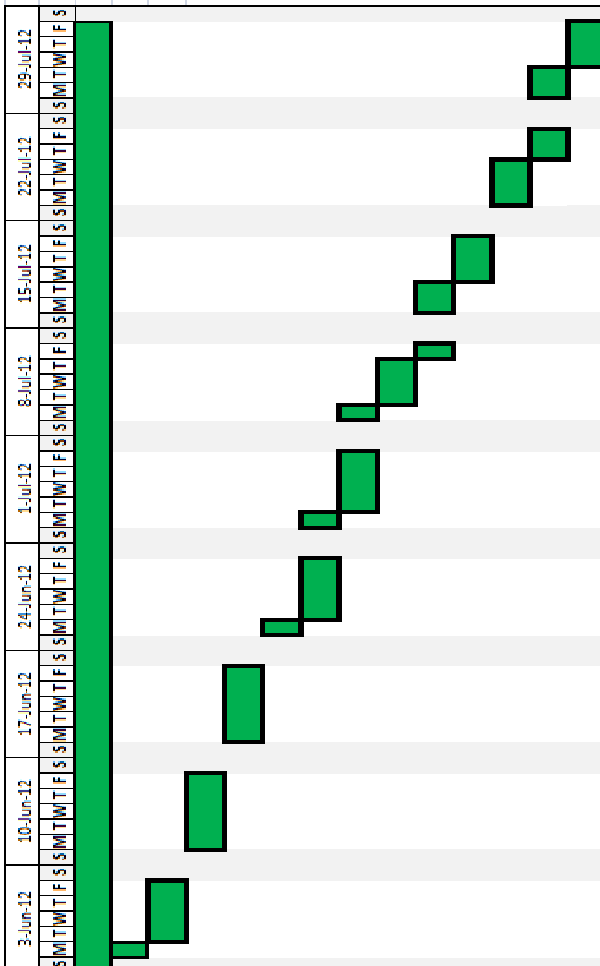
COEFFICIENTS OF UTILIZATION - ZONAL CAVITY METHOD

EFFECTIVE FLOOR CAVITY REFLECTANCE: 20%

RCC %:	80				70				50			30			10			0
RW %:	70	50	30	0	70	50	30	0	50	30	20	50	30	20	50	30	20	0
RCR: 0	1.19	1.19	1.19	1.19	1.16	1.16	1.16	1.00	1.11	1.11	1.11	1.06	1.06	1.06	1.02	1.02	1.02	1.00
1	1.13	1.11	1.08	1.06	1.11	1.08	1.06	.94	1.04	1.02	1.01	1.01	.99	.98	.97	.96	.95	.93
2	1.08	1.03	.99	.95	1.05	1.01	.97	.87	.98	.95	.92	.95	.92	.90	.92	.90	.88	.87
3	1.02	.96	.91	.87	1.00	.94	.90	.82	.92	.88	.85	.89	.86	.83	.87	.84	.82	.81
4	.97	.90	.84	.80	.95	.89	.83	.76	.86	.82	.78	.84	.81	.78	.82	.79	.77	.75
5	.92	.84	.78	.74	.91	.83	.78	.71	.81	.77	.73	.80	.76	.72	.78	.75	.72	.70
6	.88	.79	.73	.69	.86	.78	.73	.67	.77	.72	.68	.75	.71	.68	.74	.70	.67	.66
7	.84	.74	.69	.64	.82	.74	.68	.63	.73	.67	.64	.71	.67	.63	.70	.66	.63	.62
8	.80	.70	.64	.60	.79	.70	.64	.59	.69	.64	.60	.68	.63	.60	.67	.63	.59	.58
9	.76	.67	.61	.57	.75	.66	.60	.56	.65	.60	.56	.64	.60	.56	.63	.59	.56	.55
10	.73	.63	.57	.54	.72	.63	.57	.53	.62	.57	.53	.61	.56	.53	.60	.56	.53	.52

Appendix I: Construction Management Material

0	Task Name	Duration	Start	Finish
1	Steel Structure (Ground - 2nd Floor)	45 days	Mon 6/4/12	Fri 8/3/12
2	Shakeout	1 day	Mon 6/4/12	Mon 6/4/12
3	Erect Columns (Ground - 2nd Floor)	4 days	Tues 6/5/12	Fri 6/8/12
4	Erect Beams (1st Floor)	5 days	Mon 6/11/12	Fri 6/15/12
5	Erect Beams (2nd Floor)	5 days	Mon 6/18/12	Fri 6/22/12
6	Surveyors plumb up structure	1 day	Mon 6/25/12	Mon 6/25/12
7	Metal Decking (1st Floor)	5 days	Tues 6/26/12	Mon 7/2/12
8	Metal Decking (2nd Floor)	5 days	Tues 7/3/12	Mon 7/9/12
9	Detailing - Shear Studs	3 days	Tues 7/10/12	Thur 7/12/12
10	Detailing - Shear Studs	3 days	Thur 7/12/12	Mon 7/16/12
11	Edge Forms	3 days	Tues 7/17/12	Thur 7/19/12
12	WWF	3 days	Fri 7/20/12	Tues 7/24/12
13	Place Concrete	4 days	Wed 7/25/12	Mon 7/30/12
14	Finish Concrete	3 days	Tues 7/31/12	Thur 8/2/12



Appendix I: Construction Management Material