Technical Report #3

Lateral System Analysis

Indiana Regional Medical Center

Indiana, PA

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Executive Summary

The Indiana Regional Medical Center's existing conditions and structural system was analyzed for this technical report. Gravity loads and lateral loads were evaluated throughout the typical portions of the structure using design codes.

Indiana Regional Medical Center is a full service healthcare facility that resides in Indiana, Pennsylvania. It is made up of 6 separate buildings, but is mostly one seven story 98 ft high building that lies in the core of the other five. The entire structure has an orange brick façade and is used mostly as a hospital for the public. It is a constructed moment frame made mostly of steel with metal deck and lightweight concrete.

Gravity loads for calculations in this assignment were taken from ASCE 7-10. All calculations were compared to the actual loads on the plans used in the actual design of the building. Wind loads and seismic loads were also calculated using ASCE 7-10.

Spot checks were done on a typical bay within the building. A composite beam and girder were both analyzed and the results showed that they meet all design standards. Both an exterior and interior column were spot checked along the entire height of the building. These were then compared to the actual design forces given.

A lateral system analysis was completed throughout the building. The width of the building was checked for an overturning moment and was found to be sufficient for the applied loads. Direct shear and torsion were also analyzed in this technical report.

Alternative floor systems were also analyzed in this technical report. A Precast Hollow Core Plank System, Two Way Post-Tensioned System, and Two Way Flat Plate System were all designed and compared to the existing Composite Deck System. Even though positive results were not obtained from the plank system and flat plate system, the post-tensioned system should be looked into further.

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Introduction

Figure 1 – Current Entrance to IRMC

Indiana Regional Medical Center (IRMC) is a 130,000 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, & Sucking. 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.

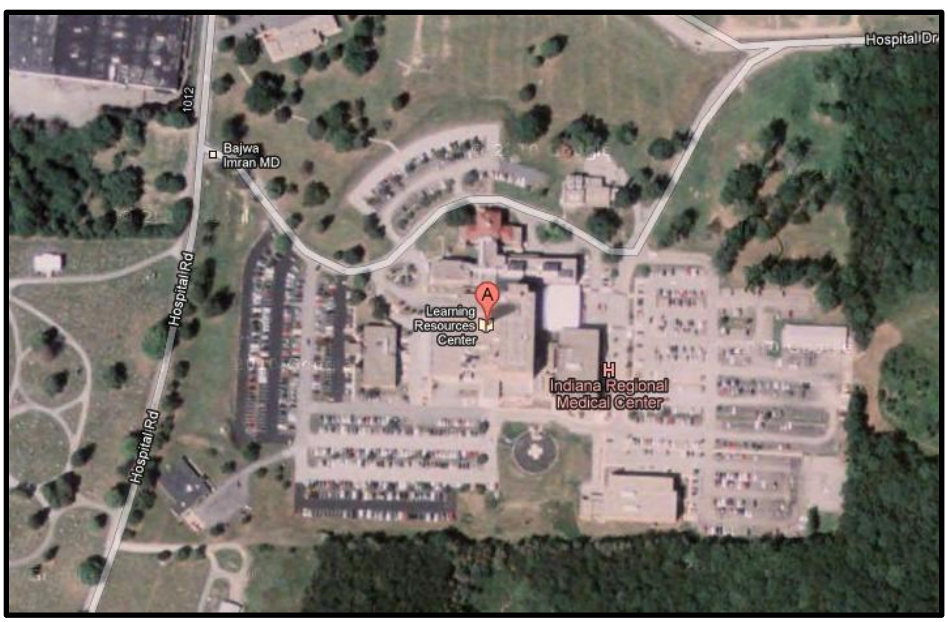


Figure 2 – Site of Indiana Regional Medical Center

This technical report collects and analyzes the existing structural conditions of the Indiana Regional Medical Center in Indiana, Pennsylvania. An analysis of gravity loads, lateral loads, and the overall structural system of this building has been included with this report along with visual aids to help with the understanding of each concept. The tallest building that makes up the hospital is the main building analyzed in this report.

Framing & Lateral Loading

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 98 feet in the air and has a rigid frame skeleton of steel. Along its North-South length, the hospital consists of 5 typical bays made up of W10, W14, and W16 steel. Moment frames allow more flexibility with the floor plan and awareness of moment connections throughout the structure. A simplified plan can be seen in Figure 3 below.

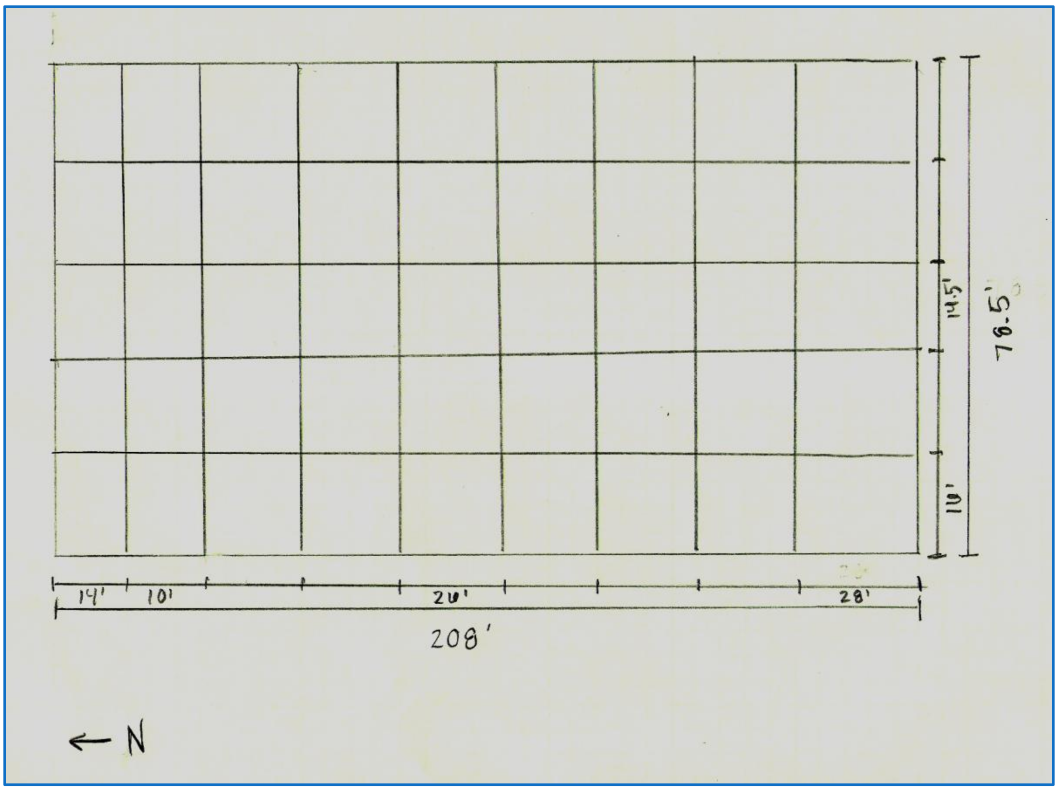


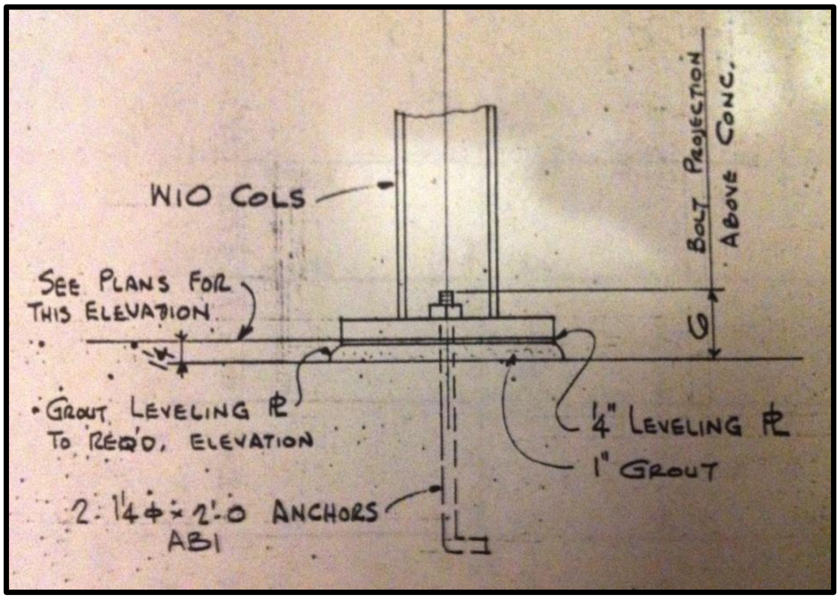
Figure 3 – Simplified Frame Layout

Other Structural Elements

Minor and secondary structural elements are not needed to be analyzed at this phase, but have to be recognized for their importance. Wind pressures and lateral soil pressures on existing walls do affect the overall loading on the building and should be taken into account. The fact that the building has had several renovations over the past 70 years should not be ignored and should always be involved when doing an analysis.

Foundation

Figure 4 – Anchor Bolt



IRMC rests on a shallow layer of bedrock so the foundation of the overall building is very shallow. The current level of grade is actually higher than initially since the foundation could not be placed deep into the ground. Concrete footings and columns make up the entire base of the building and our attached to the upper steel skeleton by anchor bolts as seen in Figure 4. Since the building rests on a shallow foundation it is very important to check load impact and load transfer. This foundation makes the building very vulnerable and could be easily affected by wind and seismic loadings. It may also be relevant to check the current foundation for any damages since this building has been renovated several times in the past.

General Structural Information

The following codes were used throughout the entire technical report for the identification of loads, wind load calculations, seismic load calculations, spot checking, and overall accuracy of research.

Design Codes

1. AISC Manual of Steel Construction Ninth Edition (ASD)
2. AISC Manual of Steel Construction Load and Resistance Factor Design Second Edition
3. ASCE 7-98 Minimum Design Loads for Buildings and Other Structures
4. International Building Code 2003
5. AISC Manual of Steel Construction Thirteenth Edition
6. AISC 7-10
7. International Building Code 2010

Determination of Loads

Live Loads

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

Dead Loads

Roof Dead Load = 20 psf

Floor Dead Loads	
Composite Decking	44 psf
Superimposed Dead Load	30 psf
Total	74 psf

Snow Loads

Snow load criteria were obtained from section 7.3 of ASCE 7-10. It was found that P_f would be 17.325 lb/ft^2. The shape of the building also results in a drift load calculation. The drift load was found to be 18.625 lb/ft^2. All respected calculations can be found in Appendix A. This is important because it will add extra weight to the surrounding buildings.

Wind Loads

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 rigid structure.

When considering the actual calculations for the wind loads, only the 98 ft tower of the hospital was taken into account. From Figure 7 shown below, it is evident that East-West direction produces the strongest wind forces of 841.3 K due to the larger surface area.

All wind load calculations were performed with the assumption that the geometry and façade of the building was regular with no protrusions. The summary of results can be found in Figure 5, Figure 6, Figure 7, and Figure 8 below. All other general information used for these calculations can be found in Appendix B.

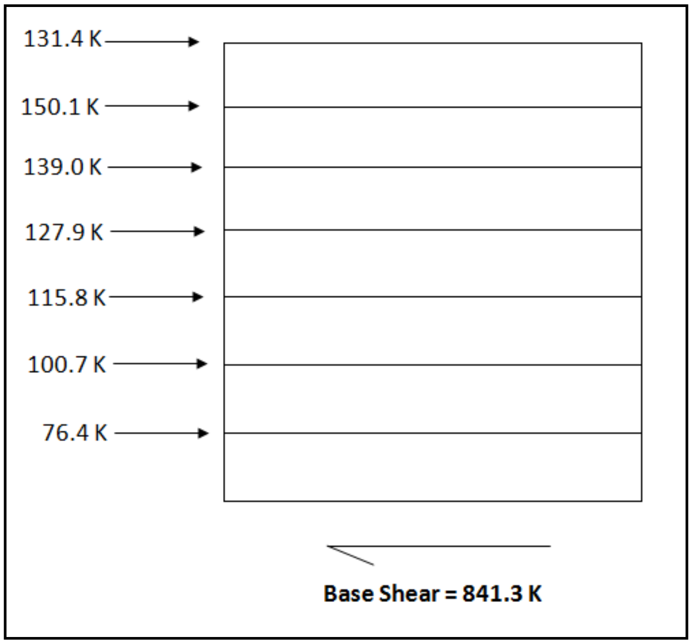


Figure 5 – East/West Wind Load Diagram

East/West Wind Loads								
Floor	Story Height (FT)	Height Above Ground (FT)	Controlling Wind Pressure (PSF)		Total Controlling Pressure (PSF)	Force of Windward Pressure (K)	Story Shear Windward (K)	Moment Windward (FT-K)
			Windward	Leeward				
Roof	14	98	19.42	-13.11	32.53	131.4	0.0	1903.16
7	14	84	17.91	-13.11	31.02	150.1	131.4	1504.44
6	14	70	16.66	-13.11	29.77	139.0	281.5	1166.20
5	14	56	15.15	-13.11	28.26	127.9	420.5	848.40
4	14	42	13.65	-13.11	26.76	115.8	548.4	573.30
3	14	28	11.39	-13.11	24.50	100.7	664.2	318.92
2	14	14	7.13	-13.11	20.24	76.4	764.9	99.82
1	14	0	0.00	0.00	0.00	0.0	841.3	0.00
							841.3 K	**1903.16 FT-K**

Figure 6 – East/West Wind Load Summary

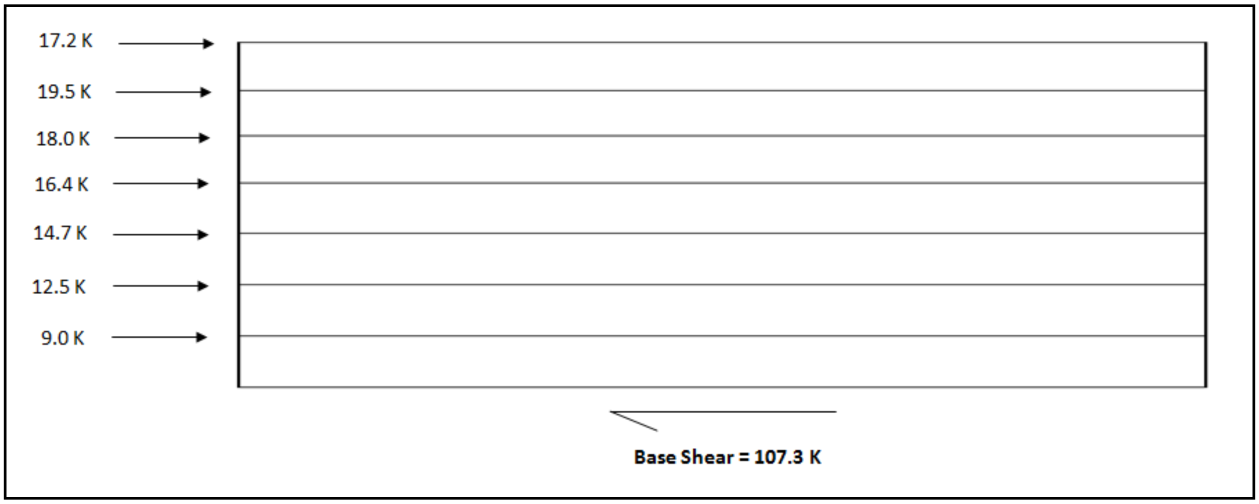


Figure 7 – North/South Wind Load Diagram

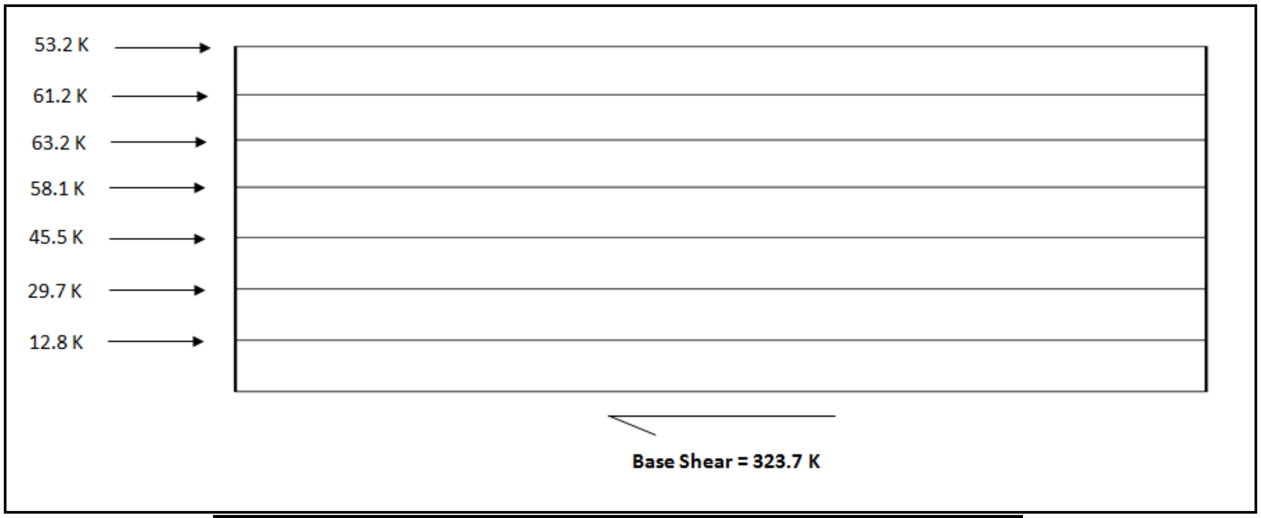
North/South Wind Loads								
Floor	Story Height (FT)	Height Above Ground (FT)	Controlling Wind Pressure (PSF)		Total Controlling Pressure (PSF)	Force of Windward Pressure (K)	Story Shear Windward (K)	Moment Windward (FT-K)
			Windward	Leeward				
Roof	14	98	14.30	-20.06	34.36	17.2	0.0	1401.40
7	14	84	13.10	-20.06	33.16	19.5	17.2	1100.40
6	14	70	12.10	-20.06	32.16	18.0	36.7	847.00
5	14	56	10.89	-20.06	30.95	16.4	54.7	609.84
4	14	42	9.69	-20.06	29.75	14.7	71.1	406.98
3	14	28	7.89	-20.06	27.95	12.5	85.8	220.92
2	14	14	4.48	-20.06	24.54	9.0	98.3	62.72
1	14	0	0.00	0.00	0.00	0.0	107.3	0.00
							107.3 K	**1401.40 FT-K**

Figure 8 – North/South Wind Load Summary

Seismic Loads

Seismic loads for the Indiana Regional Medical Center were found using the Equivalent Lateral Force Procedure of ASCE 7-10. Please refer to Appendix C to review all seismic load calculations. Calculations were completed by hand and various square footages were assumed and approximated. The analysis includes dead loads of floor slabs, superimposed dead loads, steel framing, and an allowance for mechanical equipment. A summary of the results can be found in Figure 9, Figure 10, Figure 11, and Figure 12 below.

Figure 9 – North/South Seismic Load Diagram



North/South Seismic Forces						
Floor	Weight w_x (K)	Height h_x (FT)	$w_x h_x^k$	C_{vx}	Lateral Force F_x (K)	Story Shear V_x (K)
Roof	6,456	98	2,621,016	0.30	53.2	175
7	5,762	84	1,911,521	0.22	61.2	276
6	5,762	70	1,505,399	0.17	63.2	362
5	5,869	56	1,144,696	0.13	58.1	438
4	5,869	42	785,273	0.09	45.5	500
3	5,950	28	468,051	0.05	29.7	548
2	5,950	14	188,775	0.02	12.8	583
Total	41,618		8,624,731		323.7	

Figure 10 – North/South Seismic Load Summary

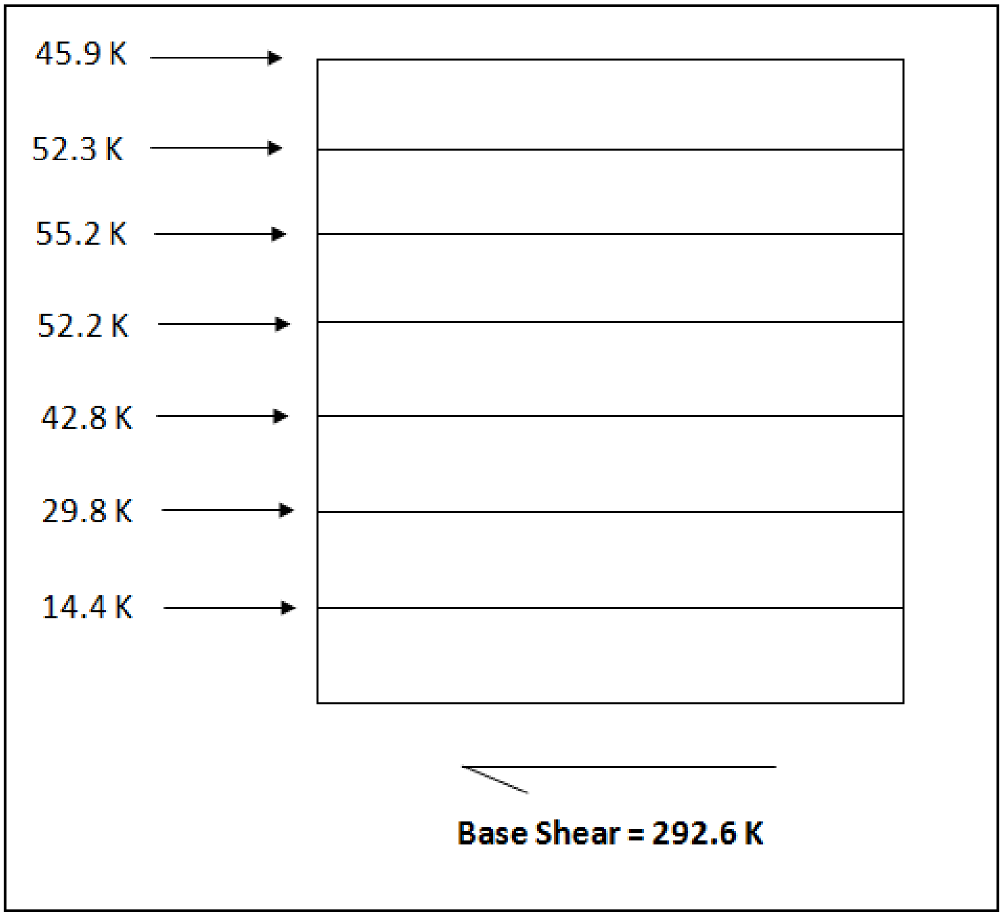


Figure 11 – East/West Seismic Load Diagram

East/West Seismic Forces						
Floor	Weight w_x (K)	Height h_x (FT)	$w_x h_x^k$	C_{vx}	Lateral Force F_x (K)	Story Shear V_x (K)
Roof	6,456	98	1,202,157	0.29	45.9	161
7	5,762	84	900,018	0.21	52.3	245
6	5,762	70	731,113	0.17	55.2	318
5	5,869	56	577,428	0.14	52.2	381
4	5,869	42	415,976	0.10	42.8	433
3	5,950	28	265,629	0.06	29.8	473
2	5,950	14	120,532	0.03	14.4	502
Total	41,618		4,212,853		292.6	

Figure 12 – East/West Seismic Load Summary

Evaluation of Systems

Floor System for Typical Bay

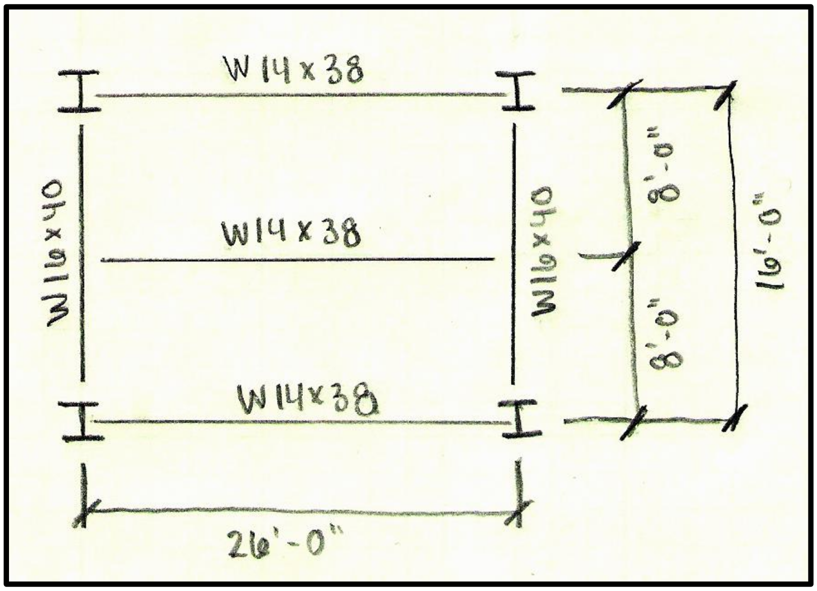


Figure 13 – Typical Bay

Spot checks were done to determine the result of gravity loads on the structure. A typical bay from the second floor of the building was used and can be seen in Figure 13. Detailed hand calculations for this bay are located in Appendix D. The 1st spot check was that of the composite slab. The slab used throughout the building is a Composite Steel Deck with 3 ½'' of lightweight 3000 psi concrete fill netting and a total thickness of 5 ½'' as seen in Appendix E. Vulcraft Decking Catalog was used to check the values of the decking. After all necessary calculations were completed; it was found that the composite decking used met all standard requirements.

Two more spot checks were done next. One was evaluating a W14x38 composite beam and the other was evaluating a W16x40 composite girder. The calculations in Appendix B show that the beam is more than adequate for the specific loads it needs to carry. When checking the shear stud requirements it was found that the calculated number was slightly less than what was used in the plans. This could come from conservative reasoning or manufacturer changes. The beam also met deflection checks for both live and wet concrete. Results from the composite girder checked yielded positive results as well. They were not as conservative as the beam's numbers were, but it was still adequate for the loading.

Typical Columns

The final two checks were that of an interior and an exterior column. Column F3 was selected to be spot checked as an interior column and Column F2 was selected for an exterior column as seen in Figure 14. Tributary Area calculations for these spot checks are located in Appendix F. The live load selected for each floor was 80 to be conservative and the dead load consisted of 44 psf for the slab and 30 psf for superimposed load. Self weight of each column was taken into account as well as the 24' splice length that is used throughout the building. Figure 15 and Figure 16 below show the resultant forces on these columns.

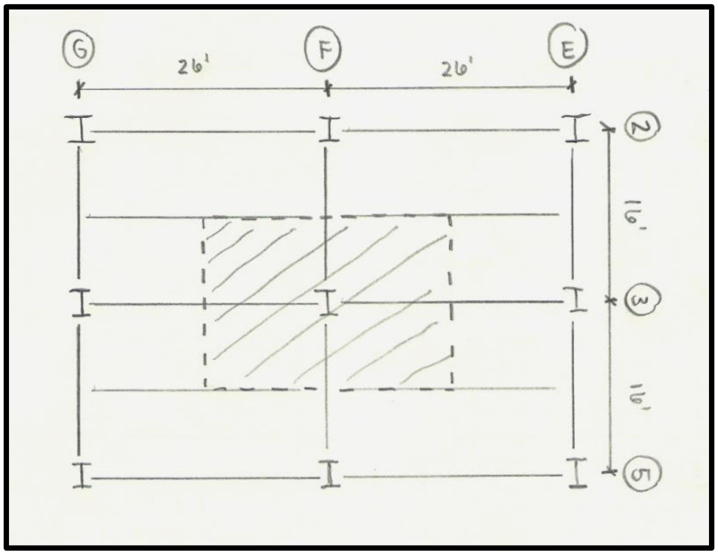


Figure 14 – Tributary Area for Interior Column

Figure 15 – Interior Column Check

Interior Column						
Column Check						
Floor	Area	DL	LL	Column Size	Splice	Pu
3	416	74	80	95	24	92468.8
4	416	74	80	95	24	92468.8
5	416	74	80	87	24	92276.8
6	416	74	80	87	24	92276.8
7	416	74	80	87	24	92276.8
Roof	416	74	20		24	50252.8
Total =						512.02 kips

Figure 16 – Exterior Column Check

Exterior Column						
Column Check						
Floor	**Area**	**DL**	**LL**	**Column Size**	**Splice**	**Pu**
3	208	74	80	95	24	47374.4
4	208	74	80	95	24	47374.4
5	208	74	80	87	24	47182.4
6	208	74	80	87	24	47182.4
7	208	74	80	87	24	47182.4
Roof	208	74	20		24	25126.4
Total =						261.42 kips

Lateral System Analysis

There are two lateral force resisting systems in the Indiana Regional Medical Center. One system is labeled "X-Frame" and has the lesser 78'-6'' depth of the building. The other system is labeled "Y-Frame" and has the greater 208'-0'' depth of the building. X-Frame contains mostly pinned connections while the Y-Frame continues mostly moment connections. The "X-Frame" and "Y-Frame" designations are used in the later calculations.

- **Load Combinations**

  The lateral systems analyzed in this report are governed by the load combinations found in ASCE 7-10 and can be seen in Figure 17 below. After analyzing wind and seismic loads it appears that Case 4 controls the East/West direction and Case 5 controls the North/South direction.

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)	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 E = EARTHQUAKE LOAD	L = LIVE LOAD L_r = ROOF LIVE LOAD	R = RAIN LOAD S = SNOW LOAD W = WIND LOAD

Figure 17 – Load Combinations

- Overturning Moment

Overturning Moments			
Floor	**Wind Force (K)**	**Elevation (IN)**	**Moment (IN-K)**
Roof	131.4	1176	154,526.4
7	150.1	1008	151,300.8
6	139.0	840	116,760.0
5	127.9	672	85,948.8
4	115.8	504	58,363.2
3	100.7	336	33,835.2
2	76.4	168	12,835.2
Total			613.569.6/12 = **51,130.8 FT-K**
M_n = 41,618 K x (78.5/2) =			**1,633,506.5 FT-K**

Figure 19 – Overturning Summary

The width of the Indiana Regional Medical Center is where the critical overturning moment results because it has the lowest depth. It is a 78'-6'' lateral resisting X-Frame and the wind load controls in its direction. The resisting moment is calculated by multiplying the weight of the building by half of the width of the building. For the building to withstand overturning, the resisting moments needs to be greater than the moment that the wind loads put on the building. Since the building is rather heavy, the moment created by the wind does not reach the magnitude created by the dead load of the Indiana Regional Medical Center. A summary of the results can be seen in Figure 19 above and the hand calculations can be seen in Appendix G.

Direct Shear

Direct shear is caused by lateral forces acting on a building and distributed to the lateral resisting system. The direct shears for each frame by story are located in Figure 20 and Figure 21 below. These results are achieved by multiplying the story force by the relative stiffness. This allows the engineer to know what force is being applied to what member throughout the building.

Direct Shear (K)						
X-Frames						
Floor	Force	Frame A	Frame B	Frame C	Frame D	Frame E
Roof	17.2	3.44	3.44	3.44	3.44	3.44
7	19.5	3.90	3.90	3.90	3.90	3.90
6	18.0	3.60	3.60	3.60	3.60	3.60
5	16.4	3.28	3.28	3.28	3.28	3.28
4	14.7	2.94	2.94	2.94	2.94	2.94
3	12.5	2.50	2.50	2.50	2.50	2.50
2	9.0	1.80	1.80	1.80	1.80	1.80
∑	107.3	21.46	21.46	21.46	21.46	21.46

Figure 20 – X-Frame Direct Shear

Direct Shear (K)						
Y-Frames						
Floor	Force	Frame 1	Frame 3	Frame 5	Frame 7	Frame 9
Roof	45.90	9.18	9.18	9.18	9.18	9.18
7	52.30	10.46	10.46	10.46	10.46	10.46
6	55.20	11.04	11.04	11.04	11.04	11.04
5	52.20	10.44	10.44	10.44	10.44	10.44
4	42.80	8.56	8.56	8.56	8.56	8.56
3	29.80	5.96	5.96	5.96	5.96	5.96
2	14.40	2.88	2.88	2.88	2.88	2.88
∑	292.6	58.52	58.52	58.52	58.52	58.52

Figure 21 – Y-Frame Direct Shear

- Torsion

Torsion will be induced by applied loads when the centers of rigidity or pressure are not located at the same point as the center of mass. Wind loads act on the center of pressure and seismic loads act on the center of rigidity. If either of those centers are not equal to the center of mass there will be a moment equal to the force multiplied by the eccentricity induced. The centers of mass, pressure, and rigidity are located in Figure 22 below.

Center of Mass, Pressure, & Rigidity (FT)				
	X	**Y**	**X Difference**	**Y Difference**
Center of Mass	104.00	39.25		
Center of Pressure	104.00	39.25	0.00	0.00
Center of Rigidity	102.80	37.80	1.20	1.45

Figure 22 – Center of Mass, Pressure, & Rigidity

The centers of pressure and rigidity are very similar if not exactly the same as the center of mass. Even though they are very similar, torsion still needs to be considered to make sure its effects on the building are minimal. For the calculations of the centers of mass, pressure, and rigidity please refer to Appendix H.

The stiffness of the two lateral resisting frames can be calculated by applying a unit load on each frame and recording the resultant displacement of each floor. The stiffness, k, can be calculated by the equation:

$$k = \frac{P}{\delta}$$

The relative stiffness can be calculated using the equation: $Relative\ Stiffness = \frac{R}{\Sigma R}$

Torsional shear can be calculated by using the equation:

$$T = \frac{V_{tot} \cdot e \cdot d_i \cdot R_i}{J}$$

where V_{tot} = Story Shear

e = distance from the center of mass to the center of rigidity

d_i = distance from frame to the center of rigidity

R_i = relative stiffness of the frame

J = torsional moment of inertia $[\sum(R_i d_i^{\,2})]$

- **Lateral Movement**

Drift is a serviceability consideration that needs to be taken into account during building design. Drift is inversely proportionate to rigidity. The lateral displacement in this report has been limited to 1/400th of the building height for wind and 1/50th of the building height for seismic considerations. The maximum wind and seismic drifts are both acceptable.

- **Strength Check**

Lateral bracing members and columns were checked for strength when wind and seismic loads were applied. The members are more than sufficient for the given loads.

Alternate System Pro-Con Study

- **Existing Floor System: Composite Deck**

The existing floor system for the Indiana Regional Medical Center consists Composite Steel Deck with 3 ½'' of lightweight 3000 psi concrete fill netting and a total thickness of 5 ½''. A description of this system is located on Page 15 of this technical assignment along with detailed calculations in Appendix D.

Pro-Con Analysis

Composite deck systems are very appealing because they keep the weight of the building extremely low compared to other systems. It allows for shallower depth of members as well as giving the overall building a low profile. Some serviceability considerations include deflections and vibrations. Vibrations were not calculated in this report, but deflections met the allowable limits. A comparison between advantages and disadvantages can be seen in Figure 23.

Conclusion	
Advantages	**Disadvantages**
Two Hour Fire Rating	Steel Requires Spray-on Fire Proofing
Smaller Beam Sizes	
Low Building Weight Impact	
Quick Constructability	

Figure 23 – Composite Deck Advantages & Disadvantages

The composite deck floor system was an excellent choice for the Indiana Regional Medical Center. It leaves a lot of flexibility with floor plans and allows the ability to span long distances that other systems cannot achieve. That does not mean that other systems are not reliable and these will be looked at later in the report.

- Precast Hollow Core Planks

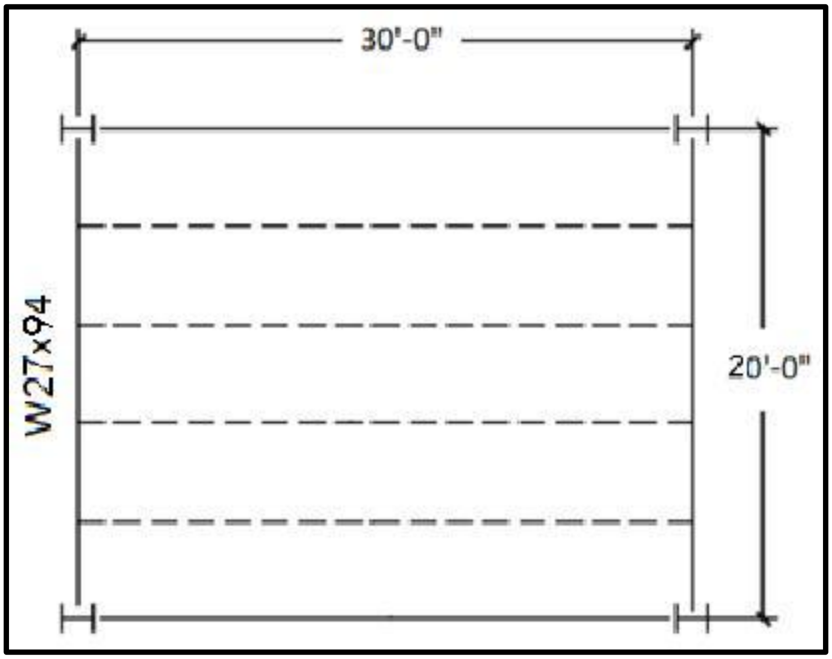


Figure 24 – Plank Layout

The first alternative floor system analyzed for Indiana Regional Medical Center was precast hollow core planks. PCI load tables received from the handbook were used in the design of this system. It was found that 4'-10'' Normal Weight Concrete Hollow Core Plank would be used according to the safe superimposed service loads from PCI. Girders were then calculated after the plank size was decided. They were determined to be W27x94 girders. The layout can be found in Figure 24. The typical 26'-0'' x 16'-0'' bay was changed to a 30'-0'' x 20'-0'' for this particular system. See Appendix I for detailed calculations.

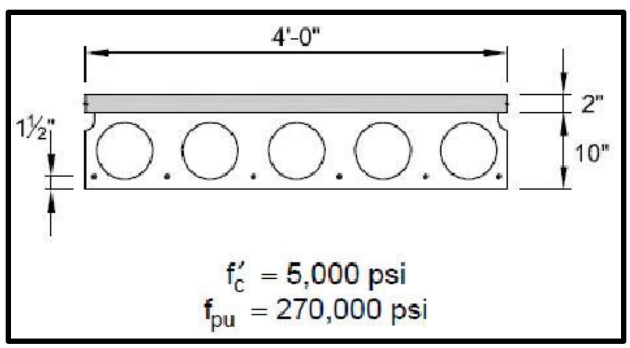


Figure 25 – Plank Detail

Pro-Con Analysis

Overall, the hollow core planks do not provide a reduction of total weight when compared to the existing composite system. A detailed image of the planks can be seen in Figure 25. This difference in weight is mainly a result of the normal weight concrete used in the planks versus the lightweight concrete used in the composite system. Larger girders are even needed to carry the exerted loads from the planks. This system would not produce any savings from a building weight perspective.

From an architectural stand point, the modular 4' sizes would lead to many changes in the bay dimensions. This would create changes in the overall floor plan of the building, which would disrupt the functionality of most of the building due to its current layout. A comparison between advantages and disadvantages can be seen in Figure 26.

Conclusion	
Advantages	**Disadvantages**
Two Hour Fire Rating	Column Grid Changes
Pre-manufactured	Leveling For Planks
Ease of Constructability	Lead Time Requirement
Low Noise Transmission	

Figure 26– Plank Advantages & Disadvantages

The Precast Hollow Core Planks are easy to rule out because of the affect it would have on the architecture of the building. It has some advantages, but changing the actual architecture of the building is not something that can be considered.

- Two Way Post-Tensioned

The second alternative floor system analyzed for Indiana Regional Medical Center was the Two Way Post-Tensioning Slab. The existing bay dimensions of 16'-0'' by 26'-0'' were able to be used with this specific system. Portland Cement Association and ACI 318-08 were used to design the system. An example of a Two Way Post-Tensioned System can be seen in Figure 27. It was found that a two way slab was needed because the existing bay dimensions satisfied the equation L2/L1 < 2. The post-tensioning comes from the ½'', 7-wire tendons used throughout the design and the overall slab thickness used was 10.5''. The tendons in the 26' side of the bay will be laid out uniformly and the tendons on the 16' side of the bay will be banded together over the column strip. Detailed calculations can be seen in Appendix J.

Figure 27 – Example of a Post-Tensioned System

Pro-Con Analysis

The Two Way Post-Tensioned System is successful because it does not alter the layout of the existing bays. This system would not affect the current floor plan of the building if it was implemented. This is a huge advantage because the other alternative floor systems chosen would change the current dimensions of the typical bay. This specific design would also create greater floor to floor heights compared to the existing composite system. The slab also provides the required two hour fire ratings from its clear cover.

A disadvantage of the system includes the complexity involved with the construction of the system. A specialized contractor would be needed during the erection. The lateral systems and foundation would also need to be reevaluated due to the increase in weight that would be present. A comparison between advantages and disadvantages can be seen in Figure 28.

Conclusion	
Advantages	**Disadvantages**
Two Hour Fire Rating	Specialized Construction
Floor Depths	Formwork
Maintaining Existing Dimensions	
Long Spans	

Figure 28– Post-Tensioned Advantages & Disadvantages

The Two Way Post-Tensioned System seems to be a valid alternative floor system for the Indiana Regional Medical Center and should have further investigation.

- Two Way Flat Plate System

The third alternative floor system analyzed for Indiana Regional Medical Center was the Two Way Flat Plate System. It consists of a two way reinforced concrete slab to transfer loads to columns. ACI 318-08 for structural steel was used in the design of this system. An existing typical bay of the building has dimensions of 16'-0'' by 26'-0'', but these spans were found to be too large for this specific system. Each span was cut in half to make a new dimensioned bay of 8'-0'' by 13'-0''. This bay was assumed to meet all design reinforcement requirements while allowing for a consistent slab thickness. A slab thickness of 5'' was found and used throughout the calculations. An example of a Two Way Flat Plate is shown in Figure 28 and a set of detailed calculations can be found in Appendix K.

Pro-Con Analysis

The Two Way Flat Plate System's advantages do not out-weigh its disadvantages. Its slab thickness would allow for much greater floor to floor heights, which could result in a higher building overall. Besides that, the system's weight is greater than any other alternative system analyzed in this report and new spaces for MEP systems would need provided.

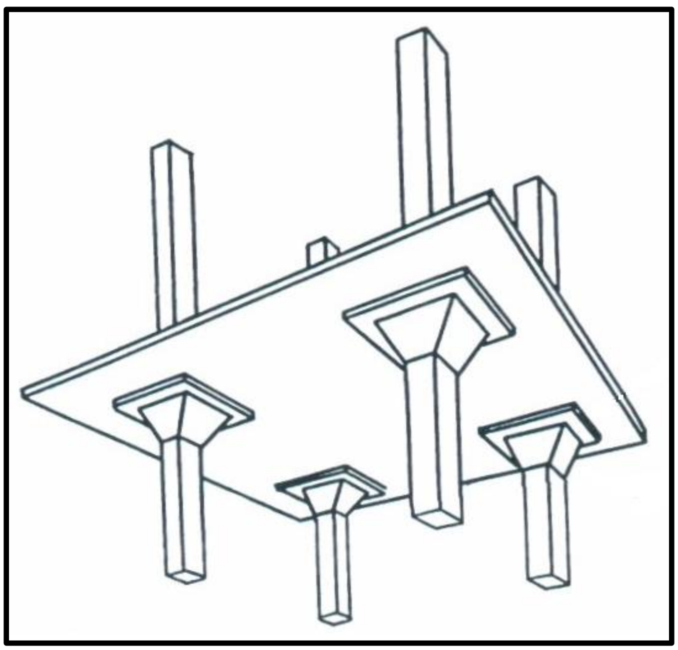


Figure 28 – Two Way Flat Plate Example

From an architectural aspect, the system is very large and would double the amount of columns that are already placed throughout the facility. This would put a strain on the layout of the building and on the lateral system. Both the lateral system and foundation would need reevaluated if this system was implemented. All open spaces within the building would also be invaded with columns.

Even if this system was more convenient, it failed to meet many design reinforcement requirements. The calculations in Appendix L show both dimensions being cut in half, but other calculations were also done with only reducing one length of the bay. All calculations failed to meet the required reinforcement for design. A comparison between advantages and disadvantages can be seen in Figure 29.

Conclusion	
Advantages	**Disadvantages**
Two Hour Fire Rating	Span Length
Ease of Constructability	Addition of Columns
Floor to Floor Height	System Weight
	Construction Time
	Failure to meet Design Requirements

Figure 29– Post-Tensioned Advantages & Disadvantages

A Two Way Flat Plate system does not seem to be appropriate for the Indiana Regional Medical Center and will not be evaluated any further. The negative changes it would create to the building as a whole well not be beneficial.

Conclusion

From the analysis of the Indiana Regional Medical Center, it is safe to conclude that it can withstand all applied loads that were calculated. All typical layouts of the structural system were spot checked for any failures: including a composite slab, composite girder, interior column, and exterior column. All beams and girders have also met deflection standards

The lateral forces due to wind and seismic were also analyzed throughout the report. It was shown that the East/West direction had the strongest wind pressures due to large surface area. Seismic Loadings were also calculated for the building. The North/South direction resulted in a larger pressure due to the length of the building

An analysis of the lateral system was completed. Overturning moment was checked on the short depth of the building and was found to not result in a problem. Direct shear and torsion were considered due to the slight differences in center of mass and center of rigidity. Strength checks proved all members are sufficient for the applied loadings.

Three alternative floor systems were evaluated and compared to each other to the existing floor system. Out of the three alternative systems evaluated, the Two Way Post-Tensioned System is the one that needs to be looked into further. Figure 30 below compares all four systems that are present in the report.

Floor System	Weight	Architectural Impact	Fireproofing	Fire rating	Cost	Constructability	Future Investigation
Composite Deck	44psf	No	Spray on	2 hr	33.20/sqft	Easy	**Yes**
Two-Way Flat Slab	125psf	Yes	Built In	2 hr	16.85/sqft	Medium	No
Hollow Core Planks	93psf	Yes	Built In	2 hr	23.48/sqft	Easy	No
Two-Way PT	144psf	No	Built In	2 hr	17.18/sqft	Difficult	**Yes**

Figure 30– Floor System Comparisons

The loads used in the actual design of the building were not significantly different than the loads discovered in ASCE 7-10. In comparing the results, it is easy to see that some characteristics seemed to be over designed, but this error could be related to documents that were not included on the actual floor and structural plans.

Appendix

Appendix A: Snow Load Calculations

Tech #1 Calc. | Cody Scheller

Snow Load

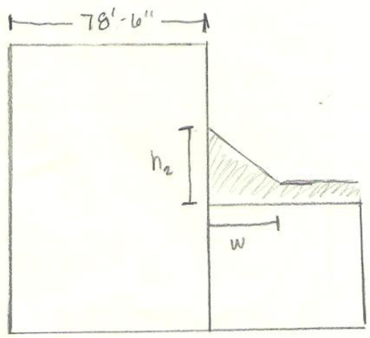
#7.3 Flat Roof Snow Load

$P_f = 0.7 C_e C_t I_s P_g$

where
$C_e = 0.9$
$C_t = 1.0$
$I_s = 1.1$
$P_g = 25$

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

→ Snow Drift



$h_2 = 0.43 \sqrt[3]{l_u} \sqrt[4]{P_g + 10} - 1.5$
$= 0.43 \sqrt[3]{78.5} \sqrt[4]{25+10} - 1.5$
$= 2.98'$

$w = 4h_2 = 11.9'$

$P_2 = h_2 \gamma$ $\quad \gamma = 0.13 P_g + 3$
$\gamma = 0.13(25) + 3$
$= 6.25$

Drift: $P_2 = 2.98(6.25) = 18.625 \text{ lb/ft}^2$

Appendix B: Wind Load Calculations

Tech. #1 Calc.

Wind Loads

Risk Category: III

Directionality Factor: $K_d = 0.85$

Exposure Category: B

Topographic Factor: $K_{zt} = 1.0$

Wind Speed: 120 mph

Section 26.9.5

$$g_Q = 3.4 \qquad g_v = 3.4$$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}}, \text{ where } n_1 = \frac{1}{T_a}$$

T_a = natural period

Section 12.8.2.1

$T_a = C_t h_n^x \rightarrow C_t = 0.02$ & $x = 0.75$

$T_a = (0.02)(146)^{0.75}$ $\qquad h_n$ = main roof height = 146'

$T_a = 0.84$

$$n_1 = \frac{1}{0.84} = 1.19$$

↳ greater than one ∴ rigid structure.

Tech. #1 Calc.

Wind Loads

Section 27

$$g_R = \sqrt{2\ln[(3600)(1.19)]} + \frac{0.577}{\sqrt{2\ln[(3600)(1.19)]}} =$$

$$g_R = 4.0897 + 0.1411 = \underline{4.231}$$

$$I_{\bar{z}} = C\left(\frac{33}{\bar{z}}\right)^{1/6} \qquad \bar{z} = 0.6(\text{height}) = 0.6(146')$$

$$\bar{z} = 87.6'$$

$$C = 0.30$$

$$I_{\bar{z}} = (0.30)\left(\frac{33}{87.6}\right)^{1/6} = \underline{0.255}$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} \qquad \ell = 320 \quad \bar{\epsilon} = 1/3$$

$$L_{\bar{z}} = (320)\left(\frac{87.6}{33}\right)^{1/3} = \underline{443.08}$$

$$Q = \frac{1}{\sqrt{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \frac{1}{\sqrt{1+0.63\left(\frac{226' + 146'}{443.08}\right)^{0.63}}}$$

$$\underline{Q = 0.7996}$$

Tech. #1 Calc.

Wind Loads

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

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} \quad , \quad N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} \quad , \quad \bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right) \cdot V$$

→ Solve $\bar{V}_{\bar{z}}$: $\bar{\alpha} = 1/4 \quad \bar{b} = 0.45$

$$\bar{V}_{\bar{z}} = 0.45\left(\frac{87.6}{33}\right)^{1/4}\left(\frac{88}{60}\right)(120) = 91.9$$

→ Solve $N_1 = \dfrac{(1.19)(443.08)}{91.9} = 5.737$

→ Solve $R_n = \dfrac{(7.47)(5.737)}{[1 + 10.3(5.737)]^{5/3}} = \dfrac{42.855}{921.8995} = 0.04649$

R_L: $\eta = 15.4 n_1 L / \bar{V}_{\bar{z}} = 15.4(1.19)(92)/(91.9) = 18.35$

$$R_L = \frac{1}{18.35} - \frac{1}{2(18.35)^2}\left(1 - e^{-2(18.35)}\right) = \underline{0.05301}$$

R_B: $\eta = 4.6 n_1 B / \bar{V}_{\bar{z}} = 4.6(1.19)(226)/(91.9) = 13.46$

$$R_B = \frac{1}{13.46} - \frac{1}{2(13.46)^2}\left(1 - e^{-2(13.46)}\right) = \underline{0.0715}$$

R_h: $\eta = 4.6 n_1 h / \bar{V}_{\bar{z}} = 4.6(1.19)(146)/(91.9) = 8.696$

$$R_h = \frac{1}{8.696} - \frac{1}{2(8.696)^2}\left(1 - e^{-2(8.696)}\right) = \underline{0.1084}$$

→ $\beta = 1.5\%$ for steel and concrete buildings

$$R = \sqrt{\frac{1}{0.015}(0.04649)(0.1084)(0.0715)[0.53 + 0.47(0.05301)]}$$

$$R = \underline{0.1155}$$

Tech. #1 Calc.

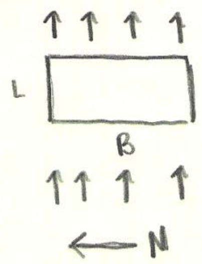
Wind Loads

$$G_F = 0.925\left(\frac{1+1.7I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1+1.7g_v I_{\bar{z}}}\right) =$$

$$G_F = (0.925)\left(\frac{2.197419}{2.4739}\right) = 0.822$$

→ Enclosure Classification → Enclosed

↳ $GC_{pi} = \pm 0.18$



$L/B = 92/226 = 0.407$

Windward Wall → 0.8

Leeward Wall → −0.5

$p = qG_F C_p - q_i(GC_{pi})$ $K_z = 1.3728$

$q = 0.00256(1.3728)(1.0)(0.85)(120)^2 = 43.0159$

windward: $p = 28.78 - 28.78(-0.18) = 33.96$ psf

leeward: $p = -14.39 - 28.78(0.18) = -19.57$ psf

→ Wind Loads calculated for the parts of the building over two stories.

Appendix C: Seismic Load Calculations

Tech. #3 Calc. | Cody Scheller

Seismic Loads

Use ASCE 7-10 Seismic Analysis for Building Structures.

Site Class: B (Table 20.3-1)

Occupancy Category: II (Table 1-1)

$F_a = 1.0$ (Table 11.4-1)

$F_v = 1.0$ (Table 11.4-2)

$S_{ms} = F_a S_s = 1.0(0.356) = 0.356$ (EQ 11.4-1)

$S_{m1} = F_v S_1 = 1.0(0.070) = 0.070$ (EQ 11.4-2)

Importance Factor: $I = 1.00$ (Table 1.5-2)

Seismic Design Category: B (Table 11.6-1)

A (Table 11.6-2)

Mapped Long-Period Transition Period: $T_L = 6s$

Values of Approximate Period Parameters:

System Resisting is N/S Direction
= Steel Concentricity Braced Frames
$C_t = 0.02$ $x = 0.75$ (Table 12.8-2)

System Resisting in E/W Direction
= Steel moment resisting frames
$C_t = 0.028$ $x = 0.8$ (Table 12.8-2)

Tech. #3 Calc. Cody Scheller

Seismic Loads

	N/S Direction	E/W Direction
$T_a = C_t h_n^x$	$T_a = 0.028(90)^{0.8}$	$T_a = 0.02(90)^{0.75}$
	$T_a = 1.097s$	$T_a = 0.623s$
$T = C_u T_a$	$T = 1.7(1.097)$	$T = 1.7(0.623)$
	$T = 1.86s$	$T = 1.06s$
$T = \min \begin{cases} C_u T_a \\ T_b \end{cases}$	$T_b = 1.11s$	$T_b = 0.776s$

T_b obtained from ETABS model

R-value	$R = 3.5$	$R = 6$

Equivalent Lateral Force Procedure

		N/S	E/W
$C_s =$	$\frac{S_{D1}}{T(R/I_e)}$	$= 0.012$	$= 0.01$
	$\frac{S_{DS}}{(R/I_e)}$	$= 0.068$	$= 0.04$
	$0.044 S_{D1} I_e \geq 0.01$	$= 0.01$	$= 0.01$

Use $C_s = 0.01$

Seismic Base Shear

$V = C_s W$ (EQ 12.8-1)

W = Seismic weight → calculated in spreadsheet

$V = 0.01(41668\,k) = 416^k$

Tech. #3 Calc. | | Cody Scheller

Seismic Loads

Vertical Distribution of Seismic Forces

$F_x = C_{vx} V$ (EQ. 12.8-11)

$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k}$ (EQ 12.8-12)

$k = 1$ for 0.5s & $k = 2$ for $T = 2.5$s

Interpolate →

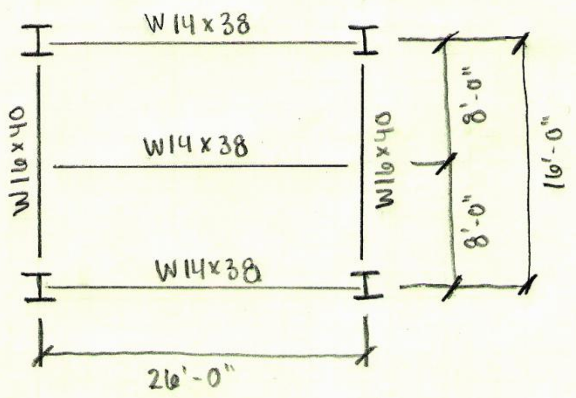
	N/S Dir.	E/W Dir.
New values →	$k = 1.31$	$k = 1.14$

→ See spreadsheets for forces & Shears

Appendix D: Floor System Calculations

Tech. #1 Calc.

Typical Bay with Typical Floor



Composite Slab

2" x 18 G.A. Steel Deck → Use Vulcraft Decking Catalog
3½" Lightweight Concrete
3000 psi Fill

↳ use 2VLI18 → Max. unshored clear span
↳ 2 span = 10'-4"

Actual Bay Clear Span = 8'0" < 10'4" ∴ ok ✓

Superimposed Live Load = 8'-0" span = 306 psf

Loading ASCE 7-10: Operating Rooms = 60 psf
Patient Rooms = 40 psf
Corridors: 80 psf
S.I. Dead Load: 30 psf
Total = 210 psf

210 psf < 306 psf → ∴ ok ✓

↳ The decking designed is ok. It is over designed for this loading, but it is not the controlling load factor.

Tech. #1 Calc.

Composite Beam → W14x38

$A_g = 11.2$ LL = 80 - not reduced for conservation

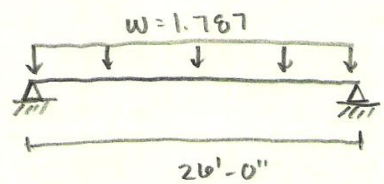
$I_x = 428$ DL = 44psf SW = 44plf

$F_y = 50$ ksi SDL = 30 psf

$W_U = 1.2D + 1.6L$ Dead = (44+30)(8') + 44 = 636plf

Live = 80(8) = 640 plf

$W_U = 1.2(636) + 1.6(640) = 1787.2$ plf = 1.787 K/ft



$V_U = 1.787(26)(\frac{1}{2}) = 23.23^K$

$M_U = 1.787(26)^2(\frac{1}{8}) = 151^{1K}$

$\phi V_n = 131$ K > 23.23^K ∴ ok ✓

$b_{eff} = \min$ | span/4 = 6.5 → controls; Spacing = 8'

PNA = 7 $\Sigma Q_n = 140$ $a = \frac{\Sigma Q_n}{0.85(f'c)(b_{eff})} = \frac{140}{0.85(3)(6.5 \times 12)}$

$a = 0.704 < 1$

→ treat a = 1

$Y_2 = \text{thickness}_{slab} - \frac{a}{2} = 5.5 - \frac{1}{2} = 5$

$\phi M_n = 231^K$ → greater than 151^{1K} ∴ ok

$Q_n = \frac{140}{17.2} = 8.14$ → 9 studs required

Deflection

$\Delta_{LL} = \frac{l}{360} = \frac{26(12)}{360} = 0.867$ in → max deflection

$\Delta_{LL} = \frac{5 w_{LL} l^4}{384EI} = \frac{5(0.64)(26)^4}{384(29000)(428)}(1728) = 0.53$ in

0.53in < 0.867in ∴ ok

Tech. #1 Calc.

Deflection of wet concrete:

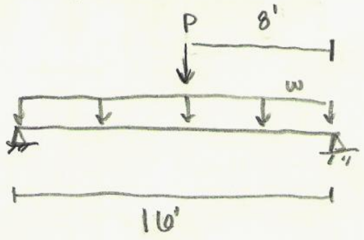
$$\Delta_{max} = \frac{l}{240} = \frac{26(12)}{240} = 1.3\text{ in}$$

$$I_{req} = \frac{5wl^4}{384\,\Delta_{max}E} \qquad w = \frac{(44\times 8)+44}{1000} = 0.396$$

$$= \frac{5(0.396)(26)^4}{384(1.3)(29000)}(1728) = 108\text{ in}^4$$

$$108\text{ in}^4 < 385\text{ in}^4 \quad \therefore \text{ ok } \checkmark$$

Composite Girder = W16x40



→ assume pin supports

→ assume load: $P = 38^{k}$

$W_u = \frac{80}{1000} = 0.08\,klf$

$$V_u = \frac{38}{2} + \frac{0.08(16)}{2} = 19.64^{k}$$

$$M_u = \frac{0.08(16)^2}{8} + 38(8) = 306.56^{'k}$$

PNA = 7 $\quad \Sigma Q_n = 147$

$$b_{eff} = \left| \begin{array}{l} 16/4 = 4 \rightarrow \text{controls.} \\ \text{spacing} = 8 \end{array} \right.$$

$$a = \frac{147}{0.85(3)(4)(12)} \cong 1.2$$

↳ use $a = 1.2$

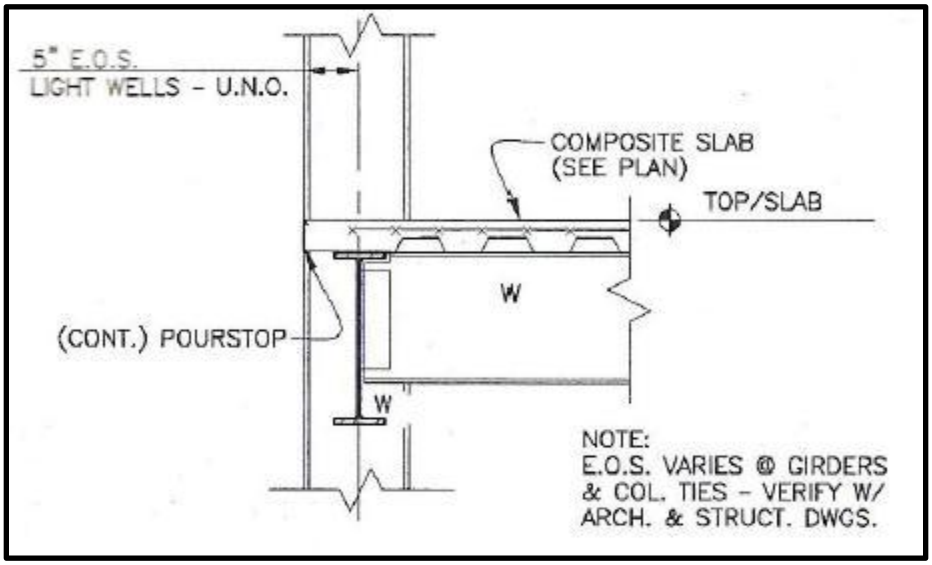
$Y_2 = 5.5 - 0.5 = 5$

$\phi M_n = 309^{'k} > 306.56^{'k}$

$\phi V_n = 146^{k} > 19.64^{k}$

$> \therefore$ ok.

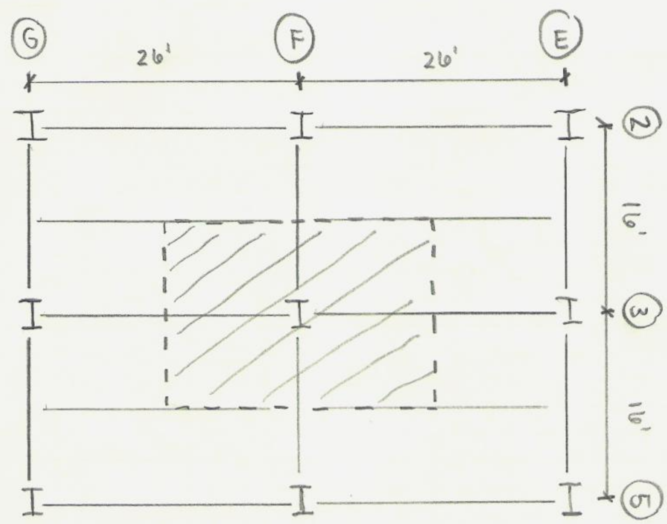
Appendix E: Floor System



Appendix F: Column Check

Tech. #1 Calc.

Columns



→ Looking at column F3

Tributary Area

$(16')(26') = 416\ ft^2$

LL: Floors = 80psf

Appendix G: Overturning Calculations

Tech. #3 Calc. | Cody Scheller

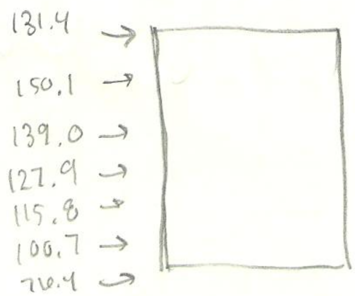
Overturning Moment

Resisting Moment (M_R) = Building Weight × Moment Arm

use width of building

$M_R = 41618\,k \times \frac{78.5}{2} = 1633506.5\,'k$

Controlling Load Case → E/S Wind



Overturning Moment (M_O) =

$98(131.4) + 84(150.1) + 70(139) + 56(127.9)$
$+ 42(115.8) + 28(100.7) + (14)(76.4) =$
$51,130.8\,'k \ll M_R \therefore ok.$

Appendix H: Center of Mass, Pressure, & Rigidity

Center of Mass Calculations					
Floor	**m_i**	**x_i**	**y_i**	**$\sum m_i x_i$**	**$\sum m_i y_i$**
Roof	6,456	104	39.25	671,424	253,398
7	5,762	104	39.25	599,248	226,159
6	5,762	104	39.25	599,248	226,159
5	5,869	104	39.25	610,376	610,376
4	5,869	104	39.25	610,376	610,376
3	5,950	104	39.25	618,800	618,800
2	5,950	104	39.25	618,800	618,800
Total	41,618			4,328,272	3,164,067
		$\sum mx/m$	**$\sum my/m$**		
		x = 104 FT	**y = 39.25 FT**		

Center of Pressure Calculations				
Floor	**Story Height**	**Total Height**	**X**	**Y**
Roof	14	98	104	39.25
7	14	84	104	39.25
6	14	70	104	39.25
5	14	56	104	39.25
4	14	42	104	39.25
3	14	28	104	39.25
2	14	14	104	39.25
1	14	0	104	39.25
			X = 104 FT	**Y = 39.25 FT**

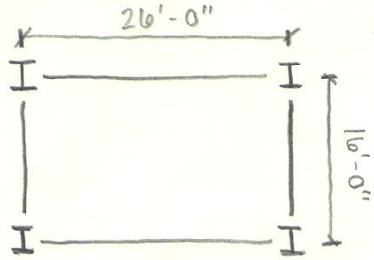
Center of Rigidity Calculations					
Y-Frames					
Frame	**Load Applied**	**Distribution**	**Percentage**	**Distance to Origin**	**% Distance**
1	1,000	200.00	20.0%	0.0	0.0
3	1,000	200.00	20.0%	50.0	10.0
5	1,000	200.00	20.0%	102.0	20.4
7	1,000	200.00	20.0%	154.0	30.8
9	1,000	200.00	20.0%	208.0	41.6
					x = 102.8 FT

Center of Rigidity Calculations					
X-Frames					
Frame	Load Applied	Distribution	Percentage	Distance to Origin	% Distance
A	1,000	200.00	20.0%	0.0	0.0
B	1,000	200.00	20.0%	16.0	3.2
C	1,000	200.00	20.0%	32.0	6.4
D	1,000	200.00	20.0%	62.5	12.5
E	1,000	200.00	20.0%	78.5	15.7
					y = 37.8 FT

Appendix I: Precast Hollow Core Planks

Tech. #2 Calc.

Precast Hollow Core Planks



Typical Bay

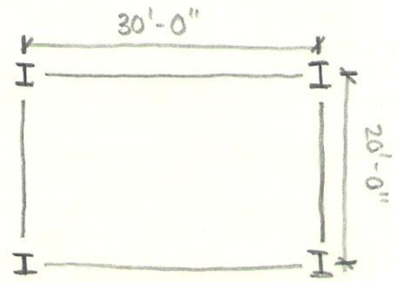
PCI Design Handbook → 4'x10" NWC

SDL = 25

LL = 100 100 + 25 = 125

PCI uses unfactored loads

→ Would change bay dimensions to 30'-0" x 20'x0" to allow for easy fitting of planks.



New Typical Bay

→ Span of 30'

→ 2" topping for 2 hour fire rating

58-5 → DL = 93psf with 2" topping

93 + 25 = 118 psf , LL = 100

1.2 (118) + 1.6 (100) = 301.6 psf

Tributary Width → 30'(301.6) = 9048 = 9.05 k/ft

$M_U = \frac{w_U l^2}{8} = \frac{(9.05)(30)^2}{8} = 1018.12^k$ → Try

@ 30' → $\Delta_{LL_{max}} = \frac{l}{360} = 1.0"$

↳ $\frac{5(100 \cdot 20)(30)^4(1728)}{384(29000)(I)(1000)}$ $I \geq 1257$

Tech. #2 Calc.

Precast Hollow Core Planks

@30' → $\Delta_{D+L} = \frac{l}{240} = 1.5'' = \frac{5(118+100)(20)(30)^4(1728)}{384(29000)(I)(1000)}$

↳ $I \geq 1827$

W27x94 → $\emptyset M_n = 1040 \geq 1018.12^k$ ∴ ok

$I = 3270 > 1827$ ∴ ok

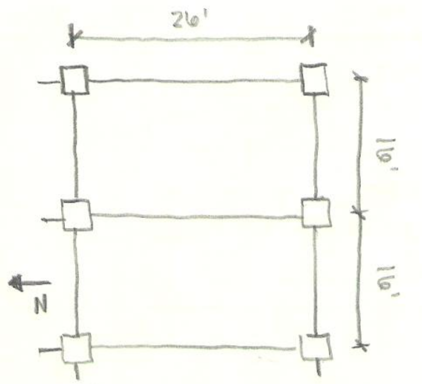
→ use W27x94 girders with hollow core plank

↳ 4'-0" x 10" w/ 2" topping, 58-5

Appendix J: Two Way Post-Tension System

Tech. #2 Calc.

Two-Way Post Tension



Loads: selfweight
SDL = 25
LL = 100

2 Hour Fire Rating

Rectangular Spans:
→ Bands in short direction over columns
→ Uniform in long direction

Rebar: $F_y = 60{,}000$ psi

Normal Weight Concrete: 150 pcf
→ $f'_c = 5000$ psi
→ $f'_{ci} = 3000$ psi

PT: unbonded tendons → $\frac{1}{2}$" Ø, 7 wire strands, ($A = 0.153 \text{ in}^2$)

$f_{pu} = 270$ ksi

Prestress losses ≈ 15 ksi

$f_{ce} = 0.7(270) - 15 = 174$ ksi

$P_{eff} = 0.153(174) = 26.6^{k}$/tendon

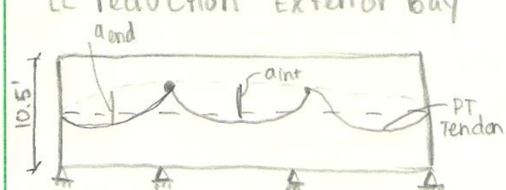
Preliminary Slab Thickness: $L/n = 30$ $h = \frac{26(12)}{30} = 10.4$
↳ ≈ 10.5"

$DL = \frac{10.5}{12}(150) = 131.25$ psf SDL = 25 psf

LL reduction Exterior Bay $A_T = 16(26) = 416$

$K_{LL} = 1$

$LL = 0.67(100) = 67$



$a_{int} = 9.5 - 1 = 8.5"$ $a_{end} = \left(\frac{5.25 + 8.5}{2}\right) - 1.25 = 5.625"$

Tech. #2 Calc.

Two-Way Post Tension

→ E-W Direction

→ Prestress force to balance 60% of SW DL

$w_D = 0.6(131.25)(26) = 2.048\ k/ft$

$P = \dfrac{2.048(16)^2}{8\left(\frac{5.625}{12}\right)} = 139.81$ $\qquad \dfrac{139.81}{26.6} = \boxed{6 \text{ tendons}}$

$P_{act} = 6(26.6) = 159.6$ $\qquad$ Balance Load $= \dfrac{159.6}{139.81}(2.048)$

↳ $2.34\ k/ft$

Actual Precompression Stress $= \dfrac{159.6}{(10.5 \cdot 26 \cdot 12)} = 48.7$ psi

→ N-S Direction

→ Prestress force to balance 95% of SW DL

$w_b = 0.95(131.25)(13) = 1.62\ k/ft$

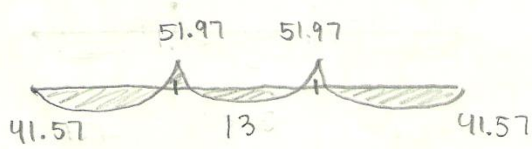
$P = \dfrac{(16)^2}{8\left(\frac{5.625}{12}\right)} = 68.27^k$ $\qquad \dfrac{68.27}{26.6} = \boxed{3 \text{ tendons}}$

$P_{act} = 3(26.6) = 79.8$ $\qquad$ Balance Load $= \dfrac{79.8}{68.27} = 1.17\ k/ft$

Actual Precompression Stress $= \dfrac{79.8}{(10.5 \cdot 13 \cdot 12)} = 48.7$ psi

→ DL Moments N-S

$W_{DL} = (131.25 + 25)(13)/1000 = 2.03$

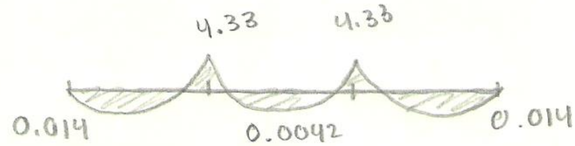


Tech. #2 Calc.

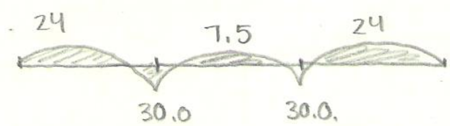
Two-Way Post Tension

→ LL Moments N-S

$w_{LL} = 13(13)/1000 = 0.169$

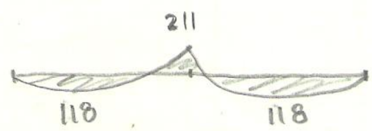


→ Balance Moments → 1.17

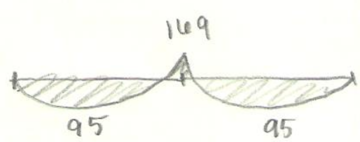


E-W

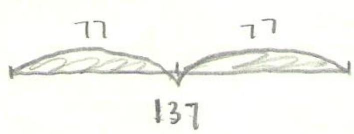
→ DL Moments → $w_{DL} = 2.5$



→ LL Moments E-W → $w_{LL} = 2$



→ Balance Moment → $w_b = 1.62$



Tech. #2 Calc.

Two-Way Post Tension

$S = \frac{(13)(10.5)^2(12)}{6} = 2866.5$

N-S → Stage 1:

→ Int. Span

$F_{top} = [(-13+7.5)(12)(1000)]/2866.5 - 48.7$
$= \boxed{-71.7 \text{ psi} \rightarrow \text{ok}}$

$F_{BOT} = [(13-7.5)(12)(1000)]/2866.5 - 48.7$
$\boxed{= -25.7 \text{ psi} \rightarrow \text{ok}}$

→ End Span

$F_{top} = [(-41.57+24)(12000)]/2866.5 - 48.7$
$= \boxed{-122.3 \text{ psi} \rightarrow \text{ok}}$

$f_{BOT}\ [(41.57-24)(12000)]/2866.5 - 48.7$
$= \boxed{24.9 \text{ psi} \rightarrow \text{ok}}$

→ Support stress

$F_{top} = [(51.97-30)(12000)]/2866.5 - 48.7$
$= \boxed{43.3 \text{ psi} \rightarrow \text{ok}}$

$F_{BOT} = [(-51.97+30)(12000)]/2866.5 - 48.7$
$= \boxed{-140.7 \text{ psi} \rightarrow \text{ok}}$

N-S → Stage 2: Stress at Service (DL+LL+PT)

→ Int. Span

$F_{TOP} = [(-13-0.0042+7.5)(12000)]/2866.5 - 48.7$
$= \boxed{-71.7 \text{ psi} \rightarrow \text{ok}}$

$F_{BOT} = [(13+0.0042-7.5)(12000)]/2866.5 - 48.7$
$= \boxed{-25.7 \text{ psi} \rightarrow \text{ok}}$

Tech. #2 Calc.

Two-Way Post Tension

→ End Span

$F_{top} = [(-41.57 - 0.014 + 24)(12000)] / 28666.5 - 48.7$

$= \boxed{-122.31 \text{ psi} \rightarrow \text{ok}}$

$F_{BOT} = [(41.57 + 0.014 - 24)(12000)] / 28666.5 - 48.7$

$= \boxed{24.9 \text{ psi} \rightarrow \text{ok}}$

→ Support Stress

$F_{top} = [(51.97 + 4.33 - 30)(12000)] / 28666.5 - 48.7$

$= \boxed{61.4 \text{ psi} \rightarrow \text{ok}}$

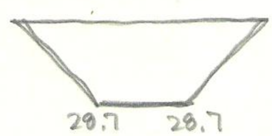
$F_{BOT} = [(-51.97 - 4.33 + 30)(12000)] / 28666.5 - 48.7$

$= \boxed{-158.8 \text{ psi} \rightarrow \text{ok}}$

Ultimate Strength:

$M_1 = P * e = (79.8)(3'')/12 = 20.0$

$M_{sec} = 48.7 - 20.0 = 28.7$ at int. supports.



M_u @ midspan $= 1.2(41.57) + 1.6(0.014) + 1.0\left(\frac{28.7}{2}\right) = 64.3^{\text{1k}}$

M_u @ support $= 1.2(51.97) + 1.6(4.33) + 1.0\left(\frac{28.7}{2}\right) = -55^{\text{1k}}$

Tech. #2 Calc.

Two-Way Post Tension

→ Min. Bonded Reinforcement

Positive Moment: Interior Span $f_t = - \leq 2\sqrt{f'c} = 141$ psi
→ none required

End Span $f_t = 69$ psi ≤ 141 psi
→ none required

Negative Moment:

Int. Supports → $A_{s,min} = 0.00075 A_{CF}$ $A_{CF} = (10.5)(\frac{16}{2})(12)$ ↳ 1008

$A_{s,min} = 0.756 \text{ in}^2$

↳ 4 - #4 bars top (0.80 in^2)

Ext. Supports: $A_{s,min} = 1.5 \text{ in}^2$ 4 - #4 bars top (0.8 in^2)

Check Min. Reinforcement

$M_n = (A_s f_y + A_{ps} f_{ps})(d - a/2)$ $d = 9.5$

$A_{ps} = 0.153(25) = 3.83$

$f_{ps} = 174000 + 10000 + [(5000)(26)(12)(9.5)] / [300 \cdot 3.83]$ ↳ 196,898.

$a = [(1.6)(60) + (3.83)(196)] / (.85 \cdot 5 \cdot 26 \cdot 12) = 0.64$

$\phi M_n = 0.9([(1.6)(60) + (3.83)(196)][9.5 - \frac{.64}{2}]) / 12$

$\phi M_n = 582.9$

N-S Summary

@ Neg. Moment 8 - #4 top bars

(25) ½" ϕ, 7-wire strands along col. strips

Tech. #2 Calc.

Two-Way Post Tension

E-W → Stage 1: (DL+PT) $w_D = 1.62$ $w_s = 2.5$ $w_L = 2$

End Span

$S = \frac{(16)(12)(10.5)^2}{6} = 3528$

$F_{TOP} = [(-118+77)(12000)]/3528 - 48.7$

$= -188.2$ psi tension → not ok → ignore for preliminary design

$F_{BOT} = [(118-77)(12000)]/3528 - 48.7$

$= 90.8$ psi → Comp. → ok.

Support Stress

$F_{TOP} = [(211-137)](12000)]/3528 - 48.7$

$= 203$ psi → C → ok

$F_{BOT} = [(-211+137)](12000)/3528 - 48.7$

$= -300$ psi → T → not ok but ignore

E-W → Stage 2: (DL+LL+PT)

End Span

$F_{TOP} = [(-118-95+77)(12000)]/3528 - 48.7 = -150.1$ → not ok, T

$F_{BOT} = [(11.8+95-77)(12000)]/3528 - 48.7 = 52.7$ → C → ok.

Support Stress

$F_{TOP} = [(211+169-137)(12000)]/3528 - 48.7 = 777.8$ psi - C - ok

$F_{BOT} = [(-211-169+137)(12000)]/3528 - 48.7 = -875$ → T - not ok.

→ Some tension values are off but decided they are ok for preliminary design.

→ Model as 3 span in future.

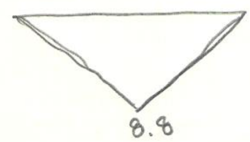
Tech. #2 Calc.

Two-Way Post Tension

Ultimate Strength

$M = P^* e = 159.6(3)/12 = 39.9$

$M_{sec} = 48.7 - 39.9 = 8.8$



Mu @ midspan $= 1.2(118) + 1.6(95) + 1.0\left(\frac{8.8}{2}\right) = 298^{1k}$

Mu @ support $= 1.2(-211) + 1.6(-169) + 8.8 = -514.8^{1k}$

→ Neglecting interior span for preliminary design.

Neg. Moment Region

$A_{cf} = 10.5(26)/2(12) = 1638$

$A_{smin} = 1.23\ in^2$

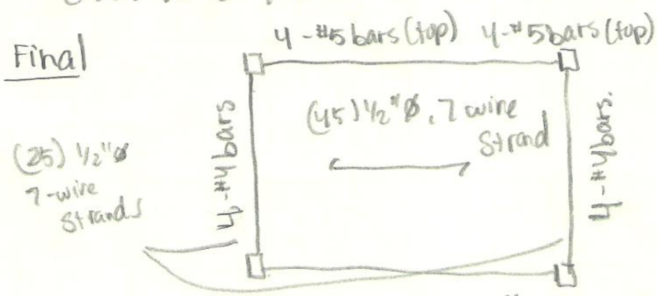
Use 4 - #5 bars top $(1.24 in^2)$

Summary E-W

4 - #5 bars top

(45) 1/2"Ø 7-wire strands.

Final



Slab: 10.5" thick.

Appendix K: Two Way Flat Plate System

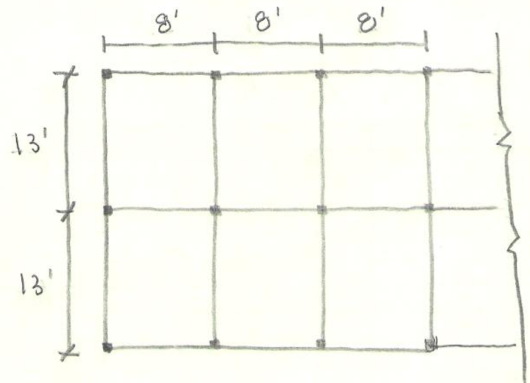
Tech. #2 Calc.

Two-Way Flat Plate System

→ Designed using ACI 318.08

→ Use typical bay floor plan: See Below ↓ change

↳ assume 18" x 18" columns



→ Critical bay size would be 8' x 26'

(Table 9.5c)

Min. Slab Thickness: $t = \frac{\ell_n}{33} = \frac{13 - \frac{13}{12}}{33} = 4.7''$ → use 5" slab

$\therefore DL = 150\left(\frac{5}{12}\right) = 62.5 + 25 = 87.5$ psf

$1.2(D) + 1.6(L) \Rightarrow 1.2(87.5) + 1.6(100) = 265 \Rightarrow 0.27$ ksf

$V_u = (0.265)\left[(13 \times 8) - (1.5)^2\right] = 26.69^k$

Punching Shear → Assume #5 bars

→ $d = 5'' - 0.75 - 0.625 = 3.625''$

→ $b_o = (18 + 3.625)4 = 86.5''$

$V_c =$ min. of:

$4\sqrt{f'c}\, b_o d = 4\sqrt{5000}(86.5)(3.625) = 89^k$

$\left(2 + \frac{4}{\beta_c}\right)\sqrt{f'c}\, b_o d = \left(2 + \frac{4}{1.63}\right)\sqrt{5000}(86.5)(3.625) = 99^k$

$\left(\frac{\alpha_s \times d}{b_o} + 2\right)\sqrt{f'c}\, b_o d = \left(\frac{40 \times 3.625}{86.5} + 2\right)(86.5)(3.625)\sqrt{5000} = 82^k$

$\beta_c = \frac{13}{8} = 1.63$

$V_c = 82^k$

Tech. #2 Calc.

Two-Way Flat Plate System

$\phi V_c = (0.75)(82) = 61.5^{k}$ ∴ OK

→ No Drop Panel needed

Direct Design Method

$M_0 = \frac{q_u}{8} l_2 l_n^2 \qquad q_u = 1.2\left(\frac{5}{12}87.5+25\right) + 1.6(100) = 234$

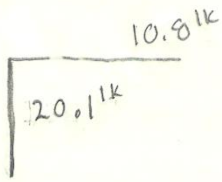
→ Frame A: $\frac{0.234}{8}(8)(13-1.5)^2 = 30.95^{1k}$

→ Frame B: $\frac{0.234}{8}(13)(8-1.5)^2 = 16.1^{1k}$

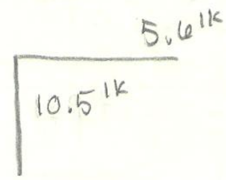
Distribution of Mu:

$-M = 0.65M_0 \qquad +M = 0.35M_0 \qquad (13.6.3.2)$

Frame A:



Frame B:



Distribution to Column Strip:

Frame A:

$-M_{nt} = 0.75M \Rightarrow 15.1^{1k}$ to C.S.
5^{1k} to M.S.

$+M_{nt} = 0.60M \Rightarrow 6.5^{1k}$ to C.S.
4.3^{1k} to M.S.

Frame B:

$-M_{nt}: 7.9^{1k}$ to C.S.
2.6^{1k} to M.S.

$+M_{nt}: 3.4^{1k}$ to C.S.
2.2^{1k} to M.S.

Tech. #2 Calc.

Two-Way Flat Plate System

→ Summary of Moments:

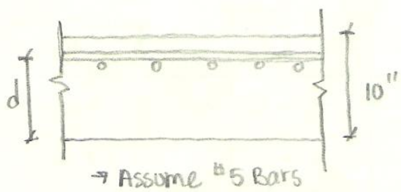
Frame A:

Total M	20.1	10.8
CS M	15.1	6.5
MS M	5	4.3

Frame B:

10.5	5.6
7.9	3.4
2.6	2.2

→ Assume d



Frame A: $d = 5 - 0.75 - \frac{0.625}{2} = 3.94''$

Frame B: $d = 3.94'' - 0.625 = 3.31''$

Design Reinforcement

Frame A:

Description	$-M_{cs}$	$-M_{ms}$	$+M_{cs}$	$+M_{ms}$
Moment	−15.1	−5	6.5	4.3
width b	48"	48"	48"	48"
Eff d	3.94"	3.94"	3.94"	3.94"
$M_n = M_u/\phi$	−16.8	−5.6	7.2	4.8
$R = M_n/d^2b \times 12000$	271	90	116	77
ρ	0.0047	0.0015	0.00197	0.00129
$A_s = \rho bd$	0.89	0.28	0.373	0.243
$A_{smin} = bt(.002)$	0.48	0.48 x	0.48 x	0.48 x
$N = A_s/A_{sbar}$	3	1	2	1
$N_{min} = w/2t$	5 x	5 x	5 x	5 x

→ #5 bars → $A_s = 0.31$

Tech. #2 Calc.

Two-Way Flat Plate System

Frame B:

Description	$-M_{CS}$	$-M_{MS}$	$+M_{CS}$	$+M_{MS}$
Moment	-7.9	-2.6	3.4	2.2
width, b	78"	78"	78"	78"
Eff, d	3.31"	3.31"	3.31"	3.31"
$M_n = M_u/\phi$	-8.8	-2.9	3.8	2.4
$R = M_n/d^2b \times 12000$	124	41	53	34
ρ	0.0021	0.00049	0.00089	0.00057
$A_s = \rho bd$	0.542	0.18	0.229	0.147
$A_{s\,min} = bt(.002)$	0.78 x	0.78x	0.78x	0.78x
$N = A_s/A_{s\,bar}$	1	1	1	1
$N_{min} = w/2t$	8 x	8 x	8x	8x

→ Results do not satisfy requirements.

→ Two-Way Flat Plate is not acceptable for IRMC.