The Commonwealth Medical College

Scranton, PA

Technical Report Two 2012

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Structural Option

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Executive Summary

This technical report discusses and compares three alternative floor systems to the current existing floor system of The Commonwealth Medical College. This is accomplished through hand calculations performed on a typical 26'x30' bay. A comparison in weight of the systems, depth of the systems, cost to construct each system, and several more criterions, were made. Through analysis, these criterions were used to determine whether or not each system would be a feasible alternative. The existing floor system is a 7.5" thick composite slab with W15x55 beams and W27x84 girders. The other systems designed in this report are, non-composite on joists and joists girders, one-way slab on concrete beams, and precast plank on wide flange girders.

It was found that the existing, composite system, is the second least expensive to construct, and also the second lightest. It has a depth of 34.4", a weight of 84 psf, and cost around $25.04 per square foot. The light weight and the ease of construction were believed to be the reasons that the composite system was chosen for the TCMC.

The non-composite with joists and joists girders system was found to be the best alternative since it has a smaller depth and weigh a lot less. However, it does cost $26.57 per square foot, $1.53 per square foot more than the composite system. It is also easy to construct since there is no shear studs involved. Overall, it was found to be an adequate alternative system.

The one-way slab on concrete beams was found to be an excellent alternative since it cost significantly less than the composite system. It does weight around 20% more, causing a need to increase the size of the foundations. A 6" thick slab with 13.5"x22.5" beams and 15"x25.5" girders resulted from this one-way concrete design.

The precast plank on wide flange girders is an expensive alternative, at $32.9 per square foot. This is the largest setback for this system. Nitterhouse Concrete Products was the selected manufacturer for the precast plank. Using their product information sheet, an 8" thick hollow core with a 2" topping and a2 hour fire rating was chosen. These are supported by W27x84 girders. This system has the largest structural depth, 34.7", and this system is the second heaviest. The extreme fabrication and construction difficulties in trying to reduce the structural depth make this system hard to construct. Out of the four systems, the precast plank on wide flange girders is the worst system to use.

Building Introduction

The Commonwealth Medical College (TCMC), also known as The Medical Sciences Building (MSB), is a medical school located in the heart of Scranton, PA. Costing over $120 million, this four story building, with an additional penthouse on the roof, was completed in April, 2011. The architecture was intended to complement the existing schools and hospitals in the surrounding area. Shown in Figures 1 is the building footprint of TCMC, highlighted in yellow, and the surrounding site.

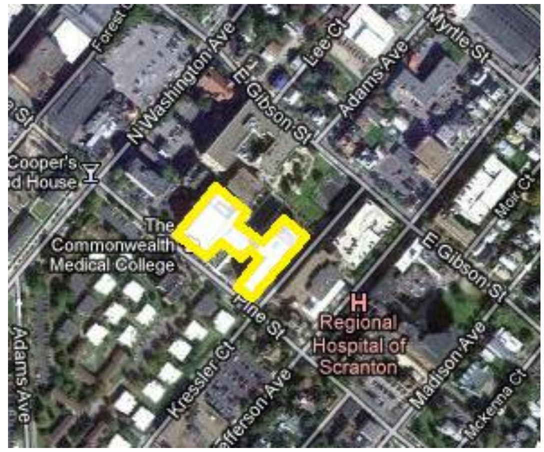


Figure 1 Aerial map from Google.com showing the location of the building site

TCMC is clad in brick, stone, and glass curtain wall. The building is separated into two individual wings, west wing and east wing. The link is the lobby area that connects the two wings and it is clad largely in insulated glass units to let natural sunlight in. An additional feature is the tower which is also clad largely in glass, as shown in Figure 2. The tower, located in the east wing, is considered the main focal point of the building. The interior space of the tower is mainly corridors and small meeting rooms so the students can enjoy the view.

Figure 2 Picture of the exterior showing the glass and brick facade on the TCMC. The Tower is shown, made will all glass walls. http://www.hok.com

TCMC is a multi-use building, using all modern technology. It has a library where students go for information, Clinical Skills and Simulation Center where students learn from beyond classrooms, lecture halls that can seat up to 160 students, classrooms with Wi-Fi connections, small group meeting rooms where a team of students can work together, and a luxurious student lounge for study or relaxation. Figure 3 shows the interior lobby of TCMC. TCMC also has a garden around the link that allows the occupants to enjoy the nice green views that the city cannot offer. The building is 93 feet tall, 185,000 square feet of space, and is a composite steel framed building that utilizes moment frames for its lateral system.

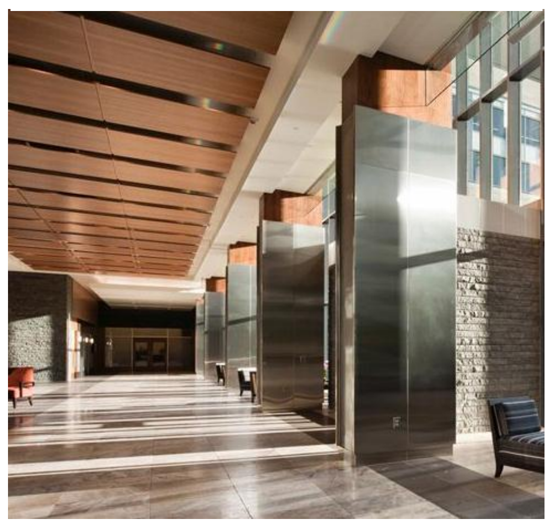


Figure 3 Interior picture of the TCMC lobby. http://www.hok.com

Structural Overview

Design Codes

According to Sheet LS100, the building was designed to comply with:

❖ Building Code	2006 International Building Code (IBC)
❖ Mechanical	2006 International Mechanical Code
❖ Electrical	2005 NFPA 70/ Nation Electrical Code
❖ Plumbing	2006 International Plumbing Code
	2006 International Fuel Gas Code
❖ Fire Protection	2006 International fire Code

All concrete work conforms to the requirements of the American Concrete Institute ACI-318-05.

Additional Code Reference from American Concrete Institute:

- ACI-211
- ACI-301
- ACI-302
- ACI-304
- ACI-305
- ACI-306
- ACI-315
- ACI-347

Regulatory Guidelines and Standards

❖ Accessibility	ICC/ANSI A117.1 1998

Material Properties

Concrete		
Usage	Weight	Strength (psi)
MAT Slab	Normal	4000psi
Columns	Normal	4000psi
Slab on Grade	Normal	3000psi
Caisson	Normal	4000psi
Wall	Normal	4000psi
Grade Beam	Normal	4000psi
Floor Slab	Normal	4000psi
Floor Slab	Lightweight	3500psi
Floor Slab	Normal	3500psi
Lean Concrete Fill	Normal	2000psi

Steel		
Type	Standard	Grade
Reinforcing Bars	ASTM A615	60
Composite Floor Deck	ASTM A992	20 gauge
Roof Deck	ASTM A992	B
Galvanized Plate	ASTM A992	50
W shape Steel	ASTM A992	50
Angles	ASTM A992	50
Bolts	ASTM A325	N/A
Anchor Rods	ASTM F1554	N/A
HSS	ASTM A992	50
Welded Wire Fabric	ASTM A185	70,000psi

Masonry		
Type	Standard	Strength (psi)
Grout	ASTM C476	5000psi
Concrete Masonry Units	ASTM C90	2100psi
Mortar	ASTM C270	N/A

Miscellaneous	
Type	Strength (psi)
Non-Shrink Grout	10,000psi

Figure 4 Tables showing materials that are used in the TCMC project

Foundations

The west wing of the TCMC is built with a mat slab foundation that is 4’-0” thick. The mat slab is designed for a soil bearing pressure of 3000psf. It is on top of a 2’-0” thick structural fill and a 4” mud slab. Figure 5 shows a typical section of the mat slab. After the mat slab, over 4’ of compacted AASHTO # 57 stone typical was placed in followed by a 5” slab on grade. Due to the confidentially of the geotechnical report, the actual bearing capacity of the soil and the recommended type of foundations were never released.

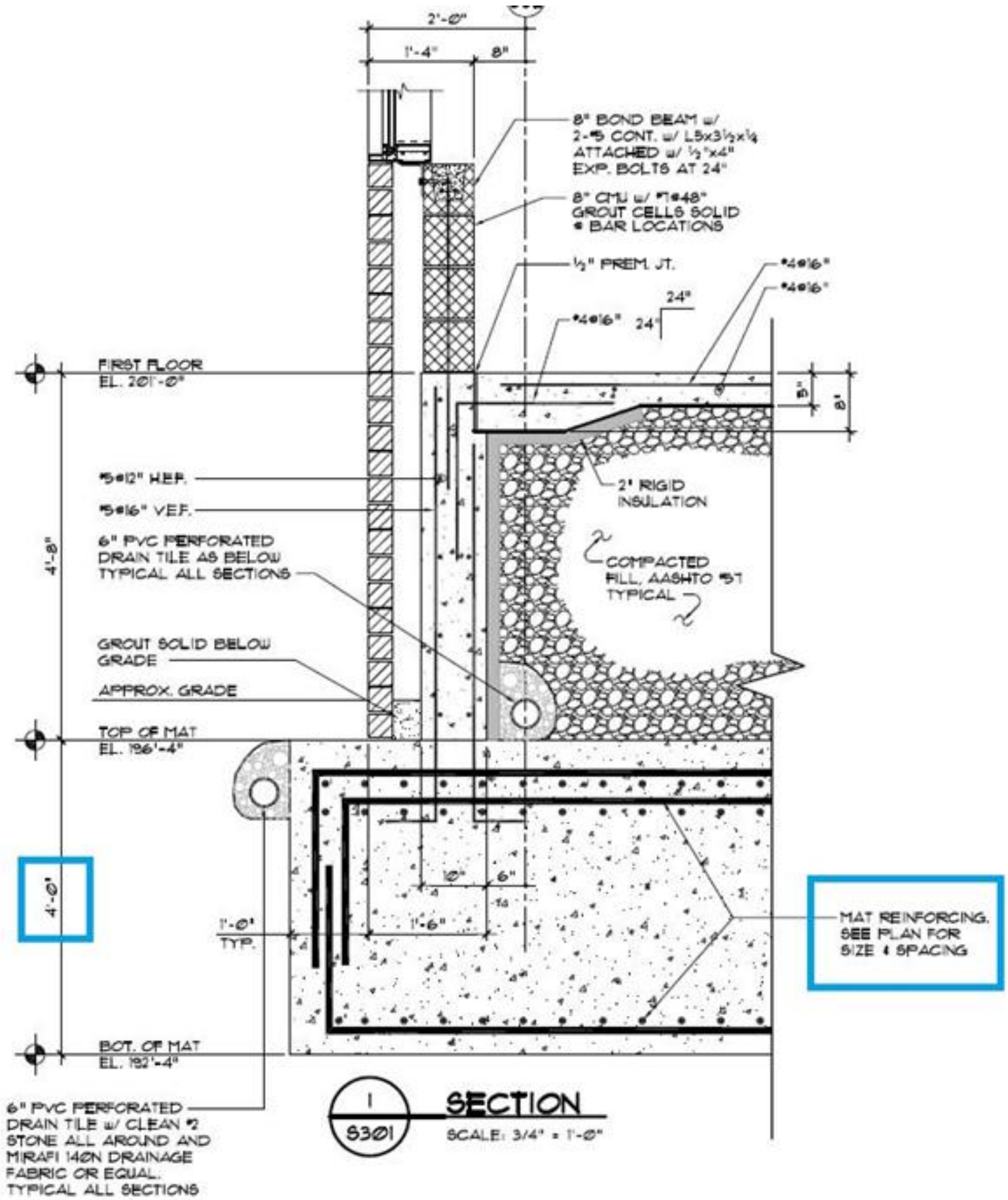


Figure 5 A typical Section cut showing the mat slab foundation. Courtesy of Highland Associates

The east wing of the TCMC has drilled caissons ranging from 36” to 60” in diameter and is used to carry loads from grade beams to bedrock below. The typical floor slab in the east wing is 7.5” and it’s also on top of compacted AASHTO material. This can all be visualized by looking at a typical section cut from figure 6 below.

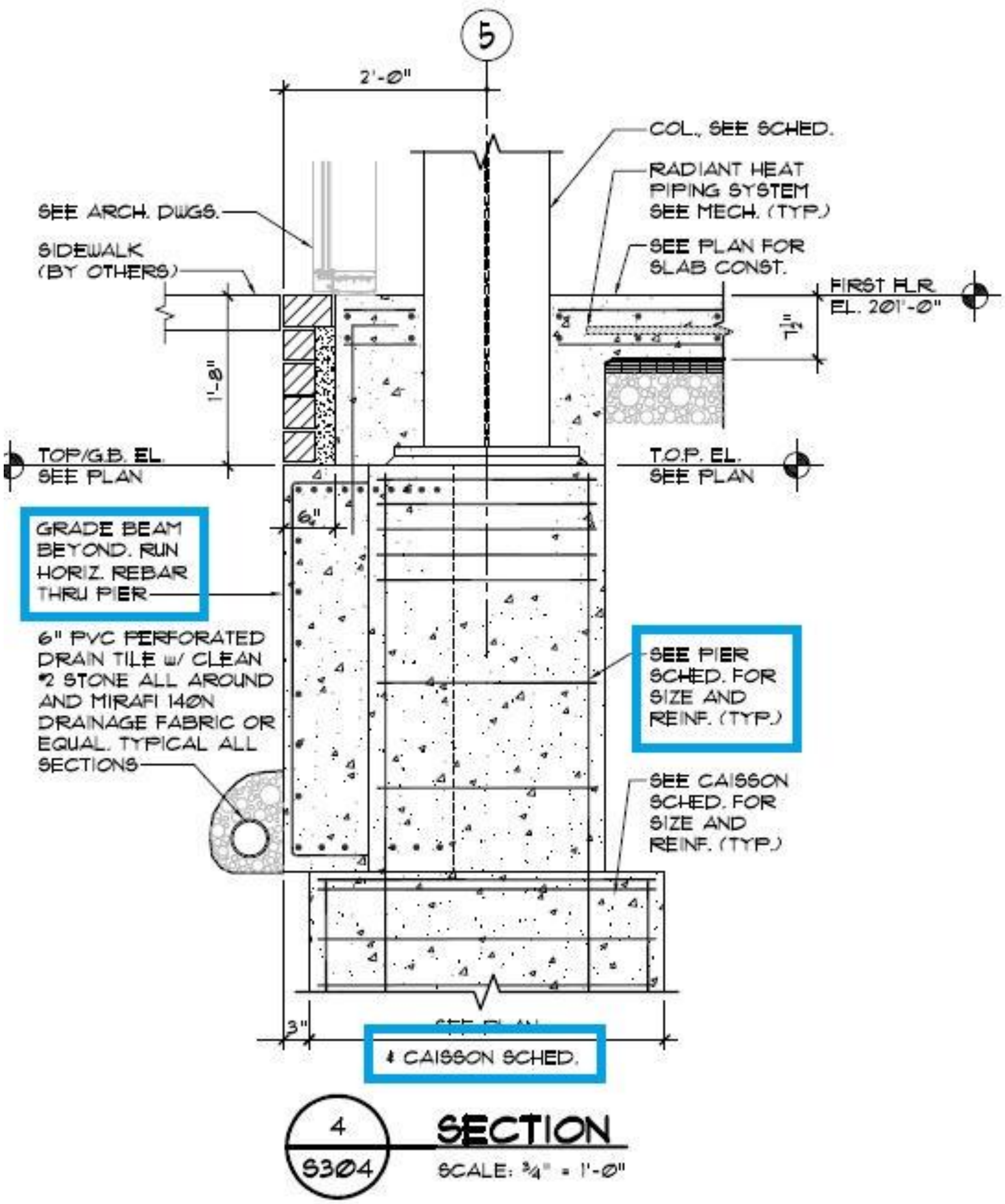


Figure 6 A section cut of a drilled caisson foundation. Courtesy of Highland Associates

Framing System

TCMC has a composite steel framed system. The sizes of the beams and columns ranged from W8x24, being the lightest, to W14x257, being the heaviest. The longest column is 44’-7” and it stopped between the third and fourth floor. An additional 48’-0” of lighter steel column is connected to this column, extending it all the way up to the penthouse.

Lateral System

The main lateral system used in TCMC consists of multiple moment frames. They are present in the west wing, east wing, and also in the link, as shown in Figure 7.1. Most frames are near the exterior wall to maximize the lateral force it can resist. The moment frames span across the entire building, from north to south and from east to west. This provides lateral resistance in each direction. The frames in the link begin on the first floor and extend to the roof, the third floor. The frames in the two wings begin on the first floor and extend to the floor of the penthouse. Figure 7.2 shows the only four frames that extend to the roof of the penthouse.

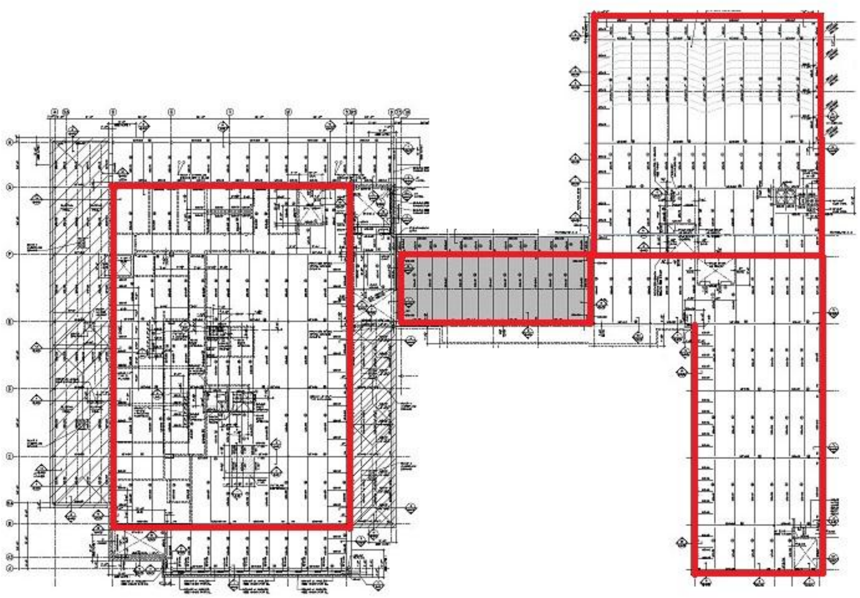


Figure 7.1 Locations of Moment Frames at TCMC. Courtesy of Highland Associates, edited by Xiao Zheng

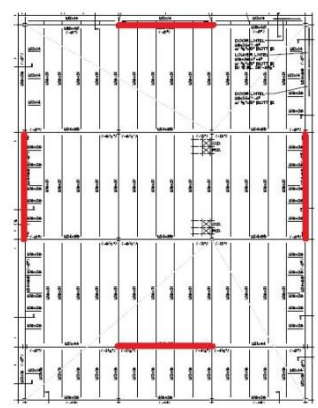


Figure 7.2 Locations of Moment Frames at the Penthouse of TCMC. Courtesy of Highland Associates, edited by Xiao Zheng

Roof Systems

TCMC has over 9 different roof heights, as shown in figure 8, with the ground referenced at 0’-0”. The link between two wings has an average roof height of 36’. The west wing goes up to 92’. The Tower, shaded in red, in the east wing goes up to 89’-4”. The rest of the east wing goes up to 81’-4” while the east wing penthouse goes up to 102’.

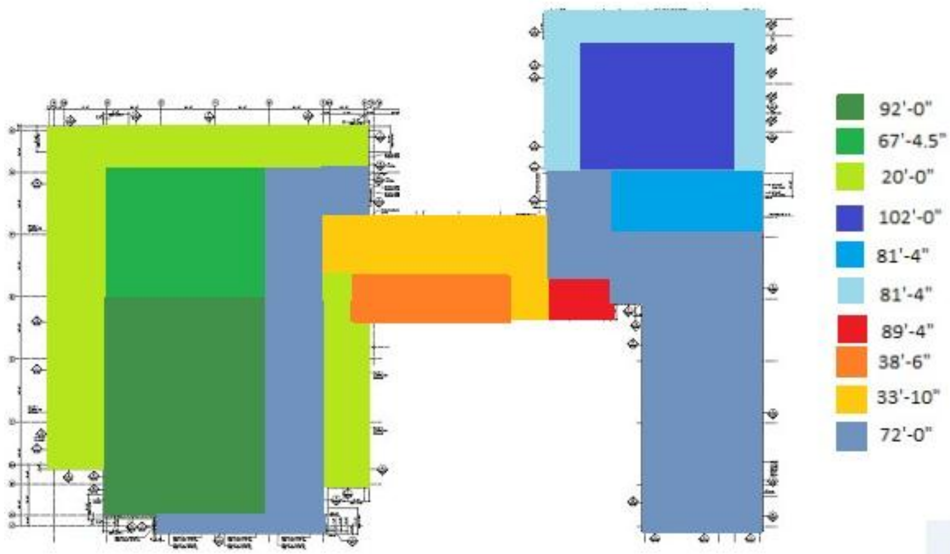


Figure 8 Plan showing the different roof heights; the darker, the higher.

The main roof is constructed of 1.5” type B wide rib, 22 gauge, painted roof deck supported by W-shape framing. A typical roof section cut is shown on figure 9. The typical roofing system has two layers of 2” rigid roof insulation. The walls around the roof extend 4’ higher than the steel deck so that it can be used as railings.

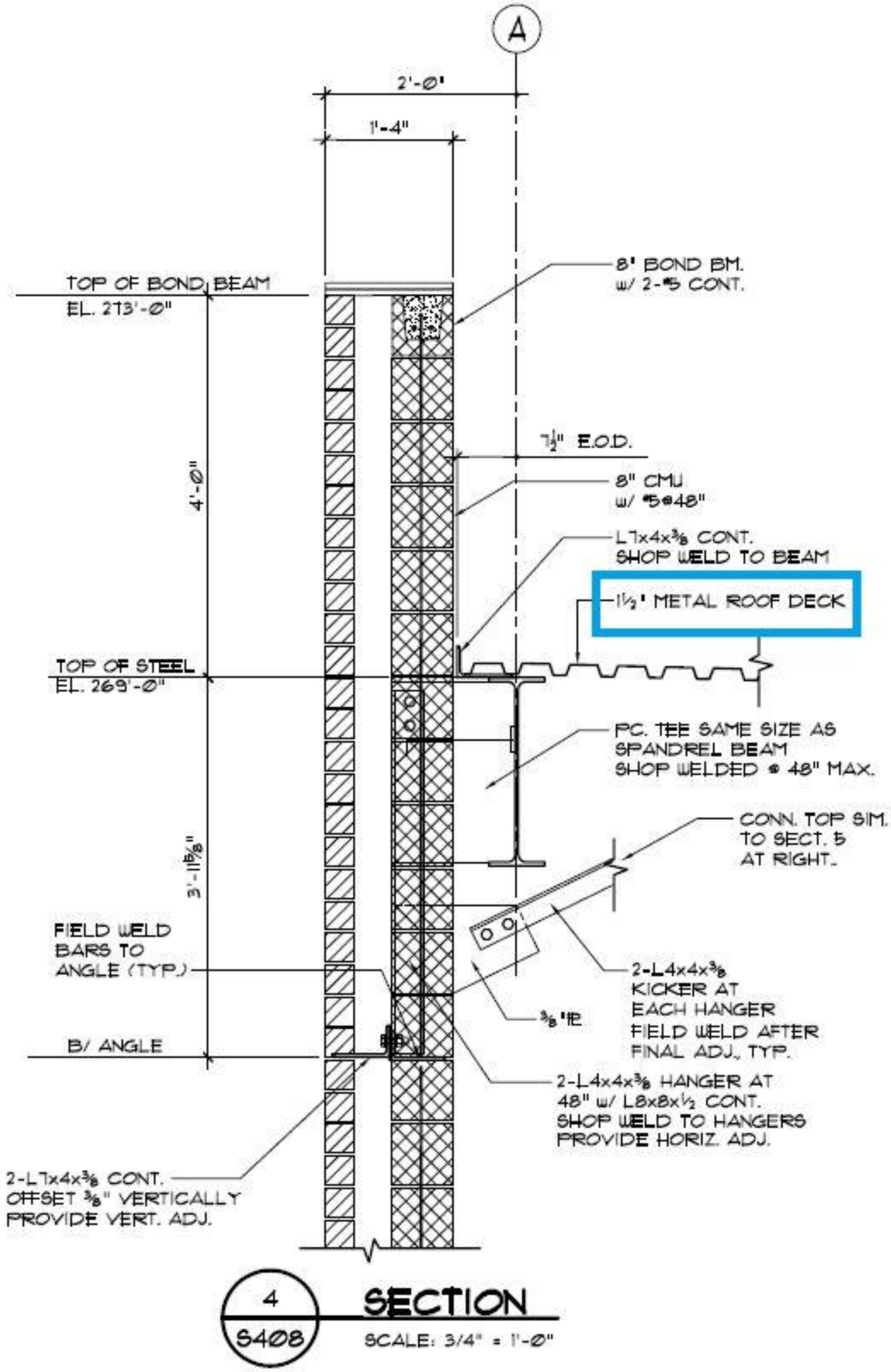


Figure 9 Typical roof section cut showing the roof deck. Courtesy of Highland Associates

Gravity Loads

The dead, live, and snow loads were calculated under this section for TCMC using IBC 2006, ASCE 7-05, and estimation.

Dead and Live Loads

For the dead load calculations, the materials that have the most impact on the dead weight of the building were found and then calculated. The west wing primarily uses composite 3" steel deck with concrete slab that weighs 75 psf according to Vulcraft Steel Deck catalog. The east wing and the hallway use 2" steel deck, lightweight concrete, so it only weights 42 psf. Then W-shape Steel Beams and Columns are assumed as 15 psf that covers that whole entire building. The heaviest exterior wall is chosen and is assumed throughout the building at 1000plf. Then these weights are multiplied by the area or the length that they occupied in to get the weight in pounds. A sample of this calculation is shown for the 2nd floor of the TCMC in Figure 10 below. Doing this for every level, a weight in psf and lbs are both obtained. Then the total dead weight is found to be around 22,378 kips and will be used later in seismic calculations. A breakdown of the weight per Level is shown in Figure 11.

Weight for 2nd Floor			
Material	Weight (psf)	Area or Length	Total Weight (lb)
Normal Weight Conc Slab with Deck	75 (psf)	20408 sf	1,530,600
Light Weight Conc Slab with Deck	42 (psf)	24952 sf	1,047,984
W-Shape Steel	15 (psf)	45360 sf	680,400
Exterior Walls	1000 (plf)	1418 lf	1,418,000
Total Weight			4,676,984
Total Weight per sf (close to design average dead load of 93 psf)			103.11

Figure 10 Total Weight per square foot of TCMC

Weight Per Level			
Level	Area (ft^2)	Weight (psf)	Weight (k)
1st	51,348.00	99.3	5099
2nd	45,360.00	103.1	4677
3rd	40,425.00	106.0	4286
4th	40,422.00	106.0	4286
Penthouse	10,337.00	209.2	2163
Roof (all level)	40,455.00	46.0	1867
Total	228,347.00		22378

Figure 11 Total Weights per Level of TCMC

The design live load for the TCMC can be found in the drawings on sheet S201A and S201B. A comparison of it to the minimum live load requirement from ASCE 7-05 can be seen on Figure 12. Notice that most design load are the same as the minimum required live load. However, some are design live loads for several locations are higher because more live loads are expected.

Design Live Loads for West Wing			
Location	Design Live Load (psf)	ASCE 7-05 Live Load (psf)	Notes
Offices	50	50	
Lobbies/ Corridors	100	100	
Corridors above 1st	80	80	
Stairs	100	100	
Classrooms	40	40	
Laboratories	100	60	Larger equipment needed in TCMC Labs
Storage Rooms	125	125	Light warehouse
Restrooms	60	N/A	
Mechanical Room	150	N/A	
Mechanical Roof	30	N/A	
Roof	20	20	ordinary flat
Partitions	15	15	

Design Live Loads for Rest of Building			
Location	Design Live Load (psf)	ASCE 7-05 Live Load (psf)	Notes
Offices above 1st	65	50	Partitions and some heavier office equipment
Lobbies/ Corridors	100	100	
Corridors above 1st	80	80	
Stairs	100	100	
Classrooms	50	40	
Sorage above 1st	125	125	
Restrooms above 1st	75	N/A	
Auditorium	100	100	if seats are fixed, then only 60psf
Bookstore	150	N/A	
Lecture Halls	60	N/A	
Mechanical Room	150	N/A	
Library	75	N/A	
1st floor offices	65	50	
1st floor restrooms	75	N/A	
Roof	30	20	
Mechanical Roof	30	N/A	
1st floor storage	125	100	

Figure 12 Design live load is compared to ASCE 7-05, required live load

Snow Loads

The variables needed for snow load calculations are found on sheet S201B of the drawings. Figure 13 shows all the loads and variables that are from Sheet S201B of the structural drawing. Also, because of the many different roof heights, snow drifts can happen in over 10 different areas of the building. One of these areas is calculated and shown under Appendix A, snow load calculations. The result of that area is that the snow acuminated in the corner reached over 73 psf, more than double the amount compared to the regular flat roof amount of 30 psf. Snow drift is an important factor when designing TCMC.

Flat Roof Snow Load Calculations	
Variable	Value
Ground Snow Load (P_G)	35 psf
Flat Roof Snow Load (P_F)	30 psf
Snow Exposure Factor (C_E)	1.0
Importance Factor (I_S)	1.1
Thermal Factor (C_T)	1.0

Figure 13 Variable for snow load obtained from S201B

Floor Systems

The existing floor system of the TCMC is held up by W-shaped steel columns and composite steel beams. Figure 14 shows the floor plan with different bay sizes in different colors. Bay sizes are shown along with the figure, with the span required for the slab first and the span required for the girder next, match with their colors. Small bays sizes are not shown in Figure 14.

The floor is composite steel deck with concrete topping. The typical floor plan in the west wing is shown in Figure 15 along with two section cuts, Figures 16 and 17. It is a 4.5” normal weight concrete topping on a 3” lok-floor 20 gauge galvanized composite floor deck, giving it a total slab construction of 7.5”. The east wing, and the link, has different slab thickness than the west wing. They are 3.25” lightweight concrete topping on U.S.D. 2” lok-floor 20 gauge galvanized composite floor deck, making the total thickness of 5.25”.

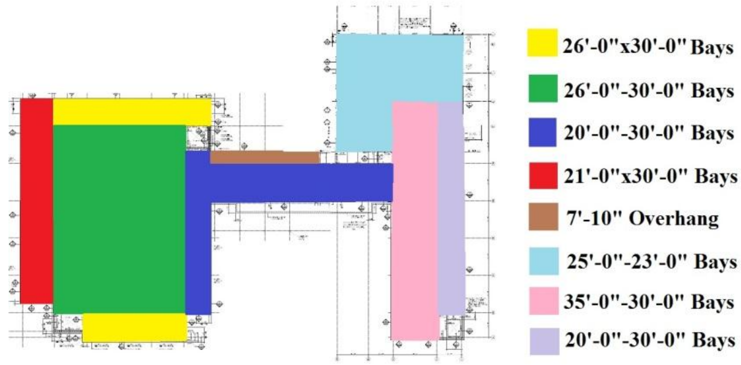


Figure 14 Different Bay sizes respective to their color

The main focus in this technical report was to analyze the existing floor system, and then design three other alternative floor systems. All analysis and design was conducted on a 26’x30’ typical bay in the west wing. All four systems were then compared to see which systems is the best for TCMC.

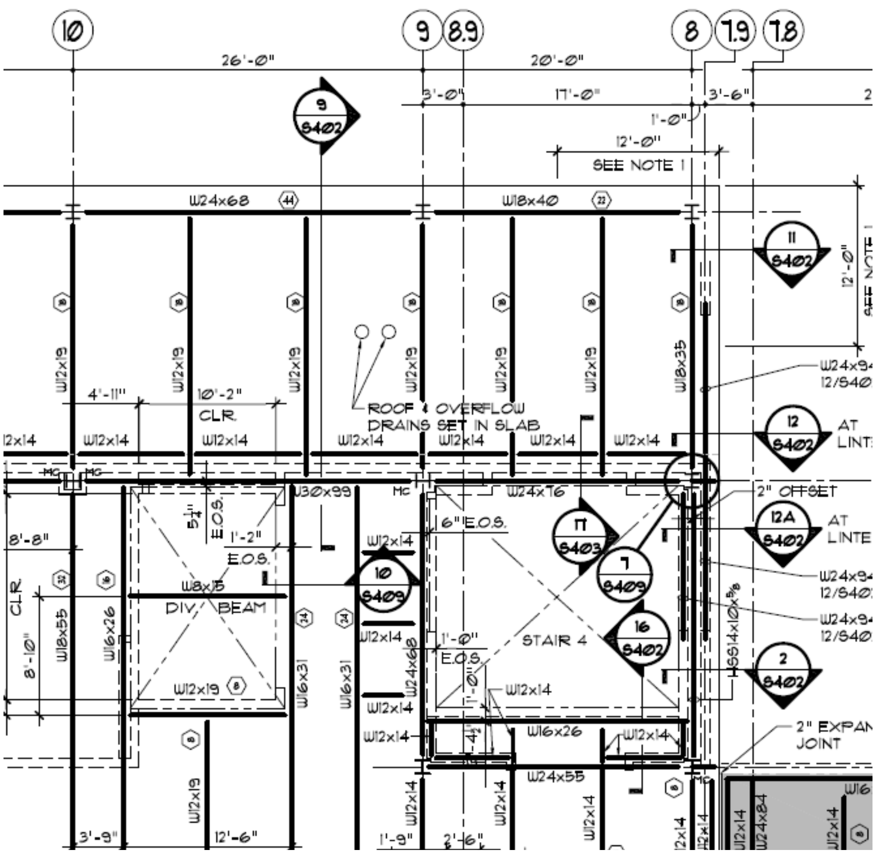


Figure 15 Partial plan showing the second floor, northeast corner of the west wing

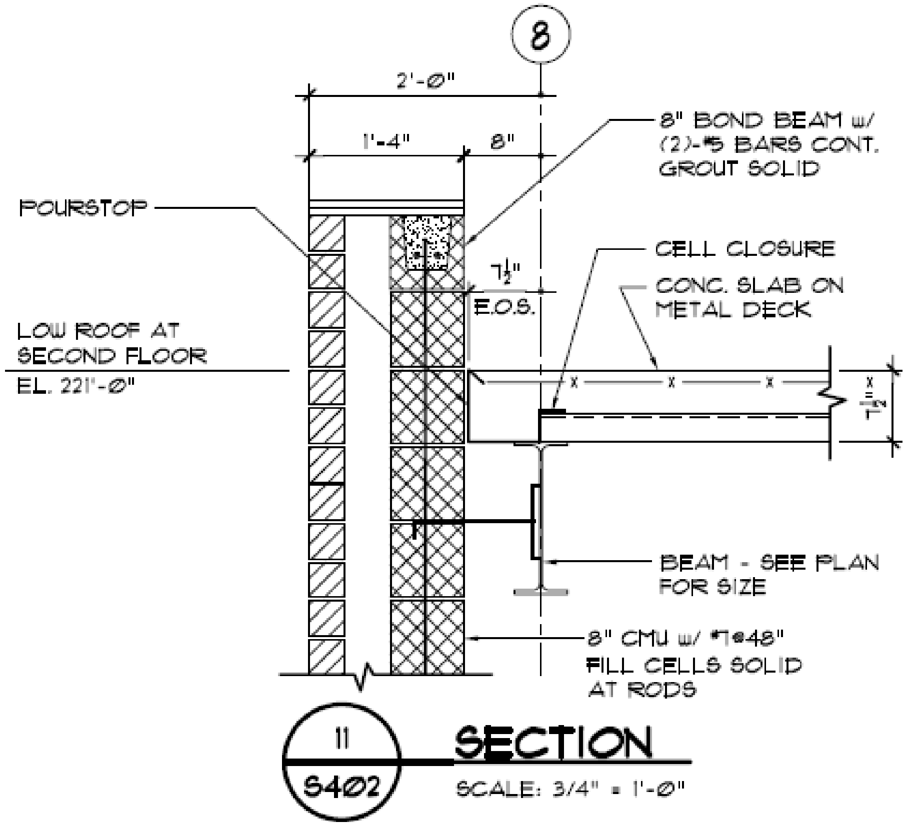


Figure 16 Section cut 11 from Figure 15

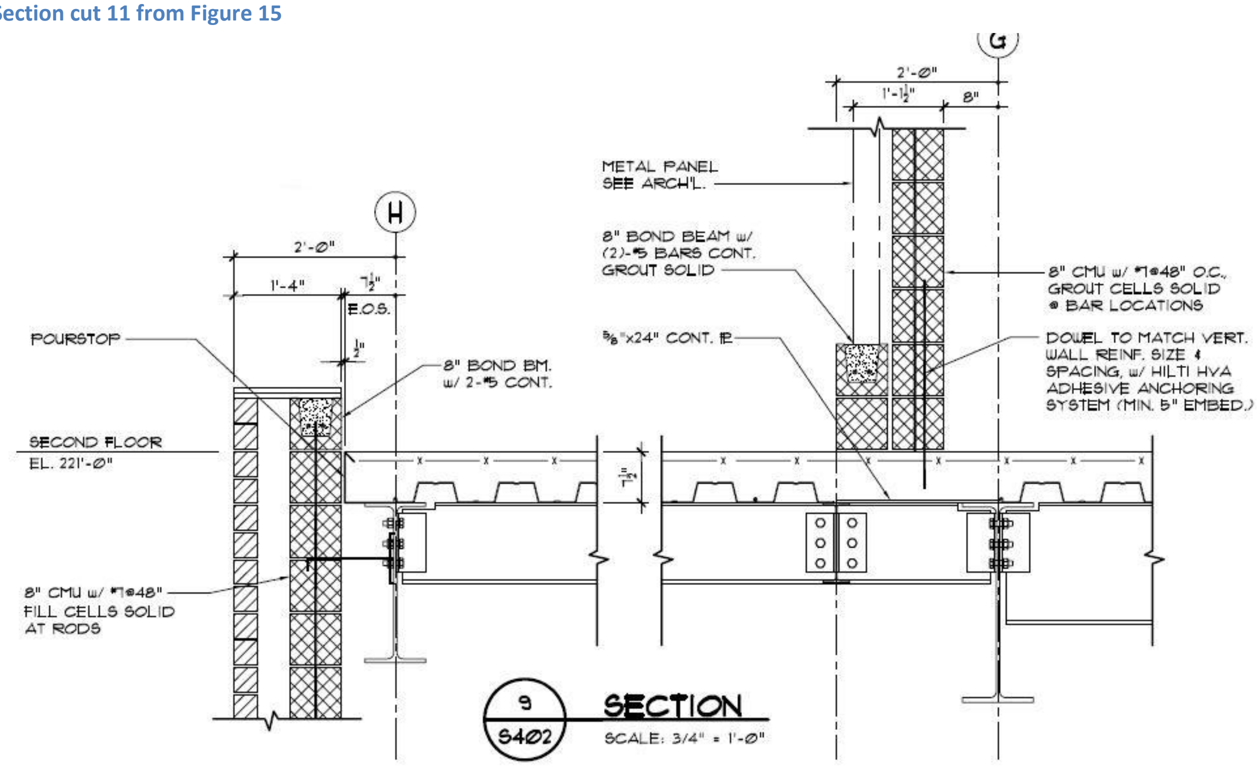


Figure 17 Section cut 9 from Figure 15

Composite Slab System

The existing floor system of TCMC consists of composite slab and decking with composite steel beam and girders. Through a series of spot checks on the typical bay, the slab, beam, and girders were found to be adequate to carry the loads. Figure 18 shows the existing floor system on the typical bay. The design was spot-checked by hand calculations, which can be found in Appendix B.

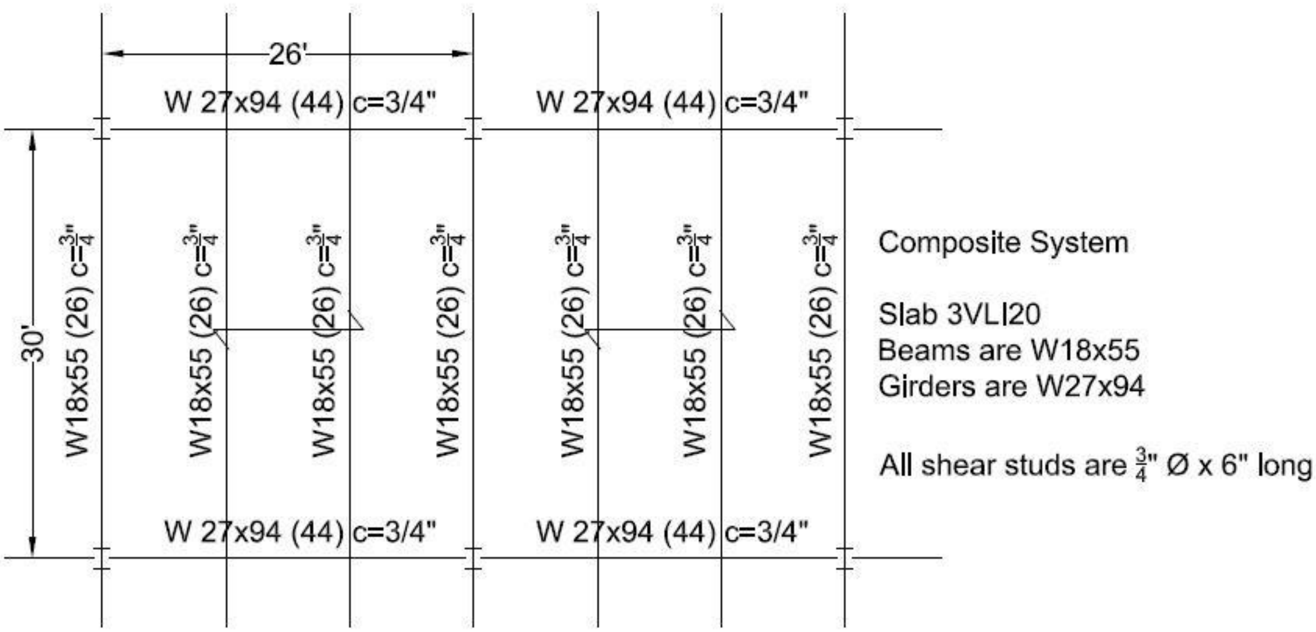


Figure 18 Composite System

Advantages

A composite system is relatively light compared to a concrete system or even a non-composite system. This makes the building lighter in design, which reduces the need of large foundations. The concrete slab resists compression and the steel beam resists tension, maximizing the efficiency of the system. A composite system also helps minimize deflections when the beam is chambered; 1.12in total system deflection in this case. Additionally, it is easy to construct, which is preferred when a schedule is tight.

Disadvantages

Although a composite system is easy to construct, it does require a large amount of labor. The welding of shear studs to the beams required a lot of work. Also, fireproofing is required, compared to a concrete system which usually doesn't. This also increases the cost for the system.

Analysis

The composite system used in the TCMC had a weight of 84 psf, and a depth of 34.4”. This fit right in the middle of the other three systems. Spray fireproofing was added to the beams and girders to achieve the 2-hour fire rating required. Using s steel frame system allows the building to use moment frames as a lateral system, which does not add additional weight to the building. This system cost about $25.04 per square feet. All cost figures are found in 2013, RSMeans Assemblies.

Model

For the steel model, it will just be a quick check to see the moment that was created by the loads on the typical bays. Figure 18.1 shows the typical bays used in the model. The maximum moment caused on the beam is 243.5 kip-feet, shown in Figure 18.2, which is compared to 256.5 kip-feet in the hand calculations. The model was more accurate because hand calculations tend to be more conservative. The concrete model will be more complex than this, showing more results.

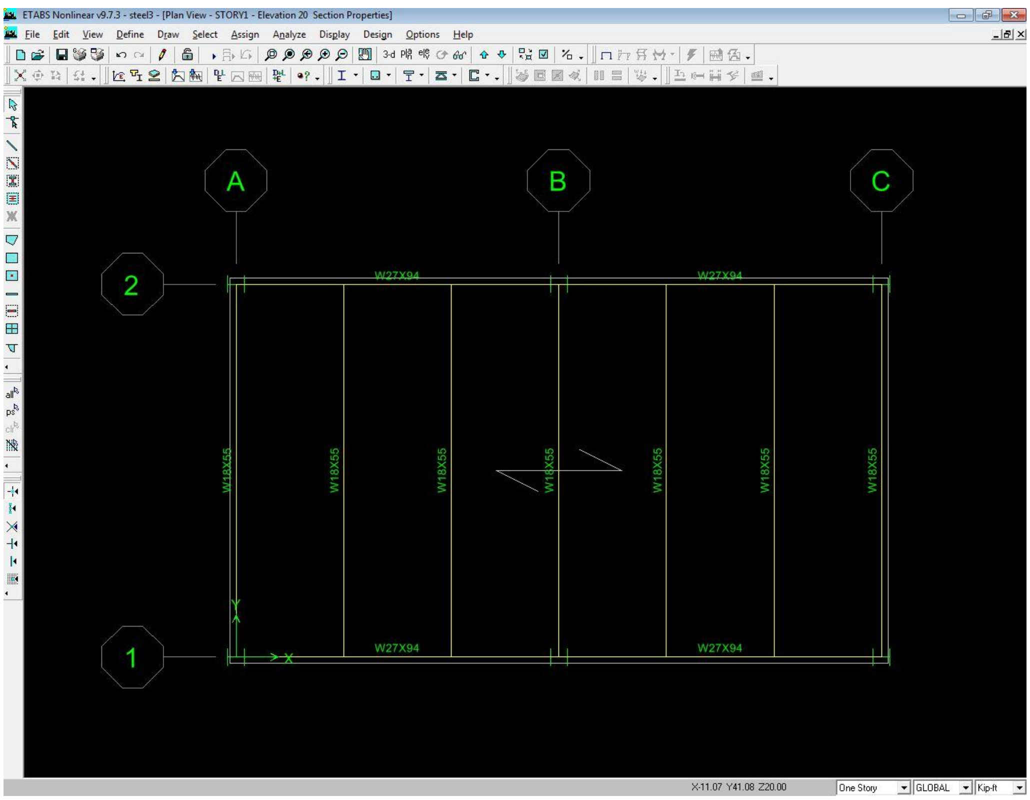


Figure 18.1 ETABS model of the Composite System

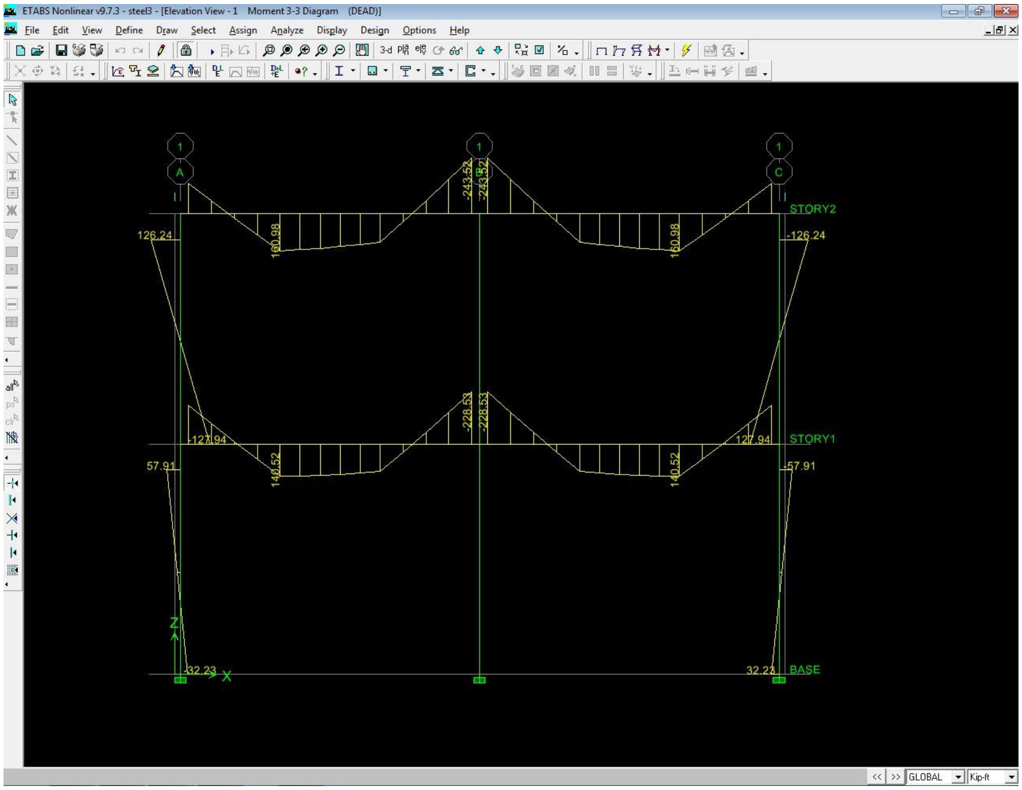


Figure 18.2 ETABS model of the Composite System showing the maximum moment on the beam.

Non-Composite Slab with Joist and Joist Girders

The first alternative floor system that was investigated was a non-composite slab with joist and joist girders. Keeping the original 26'x30' bay size, it was found that a 3C18 non-composite deck with 4.5" concrete topping is required to carry the load. The joists required for this system were 26K9 at 2'-10.66" on center, and the joist girders required were 28G9N19.4F at 30' in length. Figure 19, on the next page, shows the typical bay used for this system for TCMC. The design was performed by hand calculations, which can be found in Appendix C.

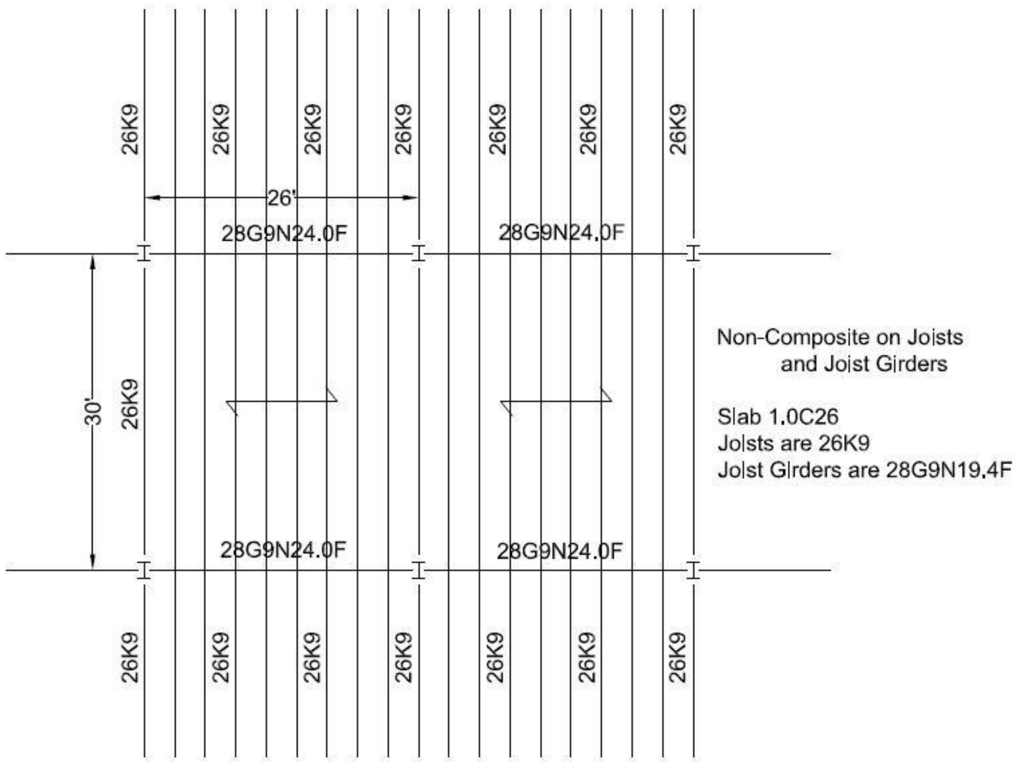


Figure 19 Non-composite on joists and joist girders

Advantages

A non-composite deck with joist and joist girders is a very economical choice for several reasons. It is the lightest of all four systems, by more than half the weight per square foot. Joists are very light and can span greater distances than a concrete beam. This system is easy to construct and quick to erect. This is the best system that allows a large, open floor plan, which is preferred for offices and classrooms. The depth is 3.3" smaller also, so the ceiling can be higher or the building can be shorter, which will a little extra cost.

Disadvantages

This system has a total deflection of 1.45” if used in TCMC, which is more than 30 percent than the existing system. Although, it is still within the deflection limit, it may not be what the owners want. Because many joists are used, this system cost almost $2 per square feet more than the existing structure. That is close to half a million more on the project. There is also a longer lead time for this steel system, which will add stress to the construction schedule. Lastly, vibrations would be expected to be the greatest in this system compared to the other three. This can be one primary reason why this system was not chosen for TCMC.

Analysis

The weight of the non-composite, joist and joist girder system, was determined to be 34.8 psf, which makes it the lightest system among the four being compared. Because of the light weight, the size of the foundation system can be greatly reduced. Because more joists are used to support the slab, it will not span as far, therefore, it will not be as thick. Through analysis using Vulcraft Steel Deck catalog, a 3” total slab thickness is adequate to carry the load.

The non-composite slab with joists and joist girders was found to cost around $26.57per square foot using 2013, RSMeans Assemblies. This includes the price of additional fire proofing for the slab and steel joists.

One-Way Slab on Concrete Beams

The one-way slab on concrete beams was chosen as the second alternative to the existing system. The same typical bay size of 26'x30' was chosen for this analysis. The beams span ing the 30' direction, the girders spanning the 26' direction, and the slab spanning over 13'. Through analysis of this system, a 6" thickness would be required for the slab, a 13.5"x22.5" beam would be required to span over 30', and a 15"x25.5" girder would be required to span over 26'. Figure 20 shows the typical bay used for this one-way slab design. The design was performed by hand calculations, which can be found in Appendix D.

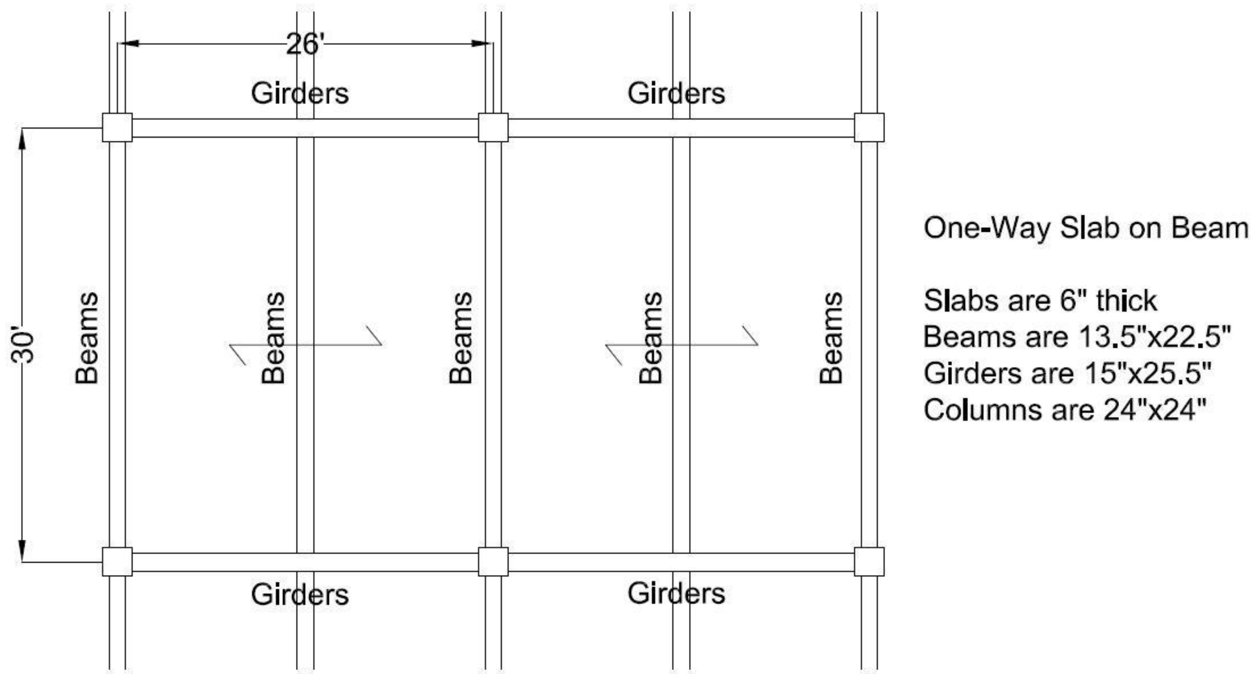


Figure 20 One-way slab

Advantages

There are many reasons why a one-way slab is economical. It has a high compressive strength, and the concrete floor system is fire-rated without any extra fire protection. Its large mass provides an excellent vibration control. Concrete is widely available, cheap, and easy to construct. In the city of Scranton, concrete more preferred in construction than steel. That is because it is cheaper, and buildings are not as high.

Disadvantages

A one-way slab floor system has a larger system depth and weight a lot more than a steel deck and beam system. Concrete is very poor in tension so steel reinforcement must be added to help carry the flexural loads. Although concrete is cheap, formwork can be costly. Additionally, shrinkage and creep are also problems that a concrete system must face, later in the life of the structure.

Analysis

The-one way slab system has a weight of 103.7 psf, 20 psf more than the current system. Because it's heavier, the foundations need to be increased.

The estimated cost of this system is around $19.09 per square foot. That is around $6 per square foot less than the current system. Compared to the other three systems, this system cost the least. This will be a huge saving in cost, which is a very good thing for the owner.

The one-way slab floor system has a total system depth of 25.5", making this system the shortest depth among the other three. It is 9" shorter in depth compared to the current system, but this does not mean the building height can be decreased. The building height might still need to be increased for a mechanical system. Because there are no height restrictions in Scranton, this height increase will not be a big problem. However, it is preferably not to increase the building height because that would increase the weight as well as the surface area of the building and hence, would increase both seismic and wind forces.

Through this investigation, a one-way slab would be a viable system. Although it is the heaviest compared to the other three systems, it is the cheapest to construct, and the most capable of handling vibrations, which makes it appealing to the owner. Foundations do need to be increased and shear walls need to be added for lateral resistance. And because it is a popular material in Scranton, it makes it an attractive alternative.

Model

The model of the one-way slab is designed on spSlap. The output of deflection, moment, and shear was used to compare with the hand calculations. According to the model, the maximum deflection is 0.284" while the hand calculation resulted in 0.524". This could also be that hand calculations are more conservative. The moment and shear outputs came out to be close to the hand calculations. The model has 170.52 kip-feet for the moment and 55.2 kip-feet for the shear. In hand calculations, the moment is 259 kip-feet 73.8 kips.

Figure 20.1 spSlap Model: Loads on the system

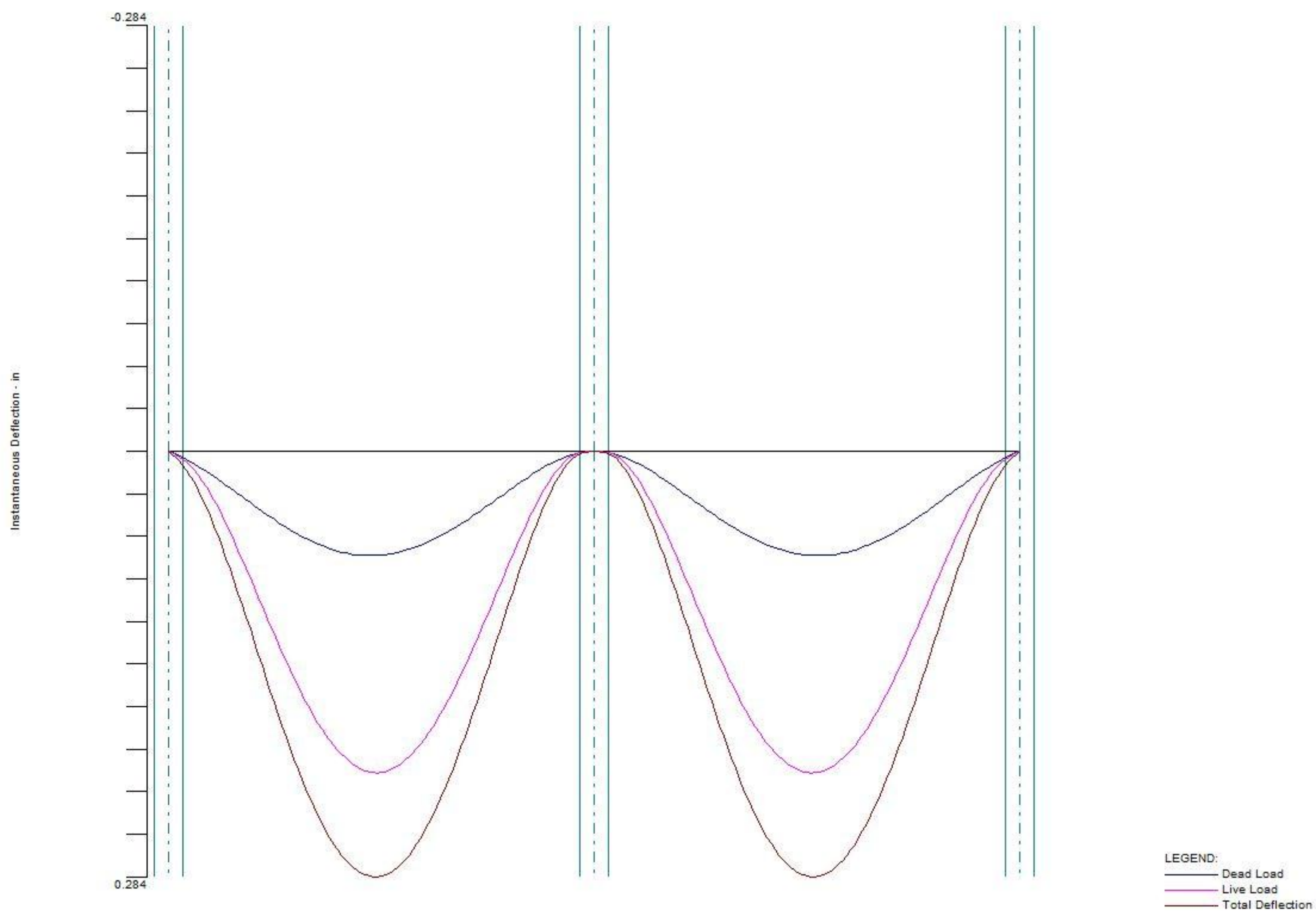


Figure 20.2 spSlap Model: Deflections on the system

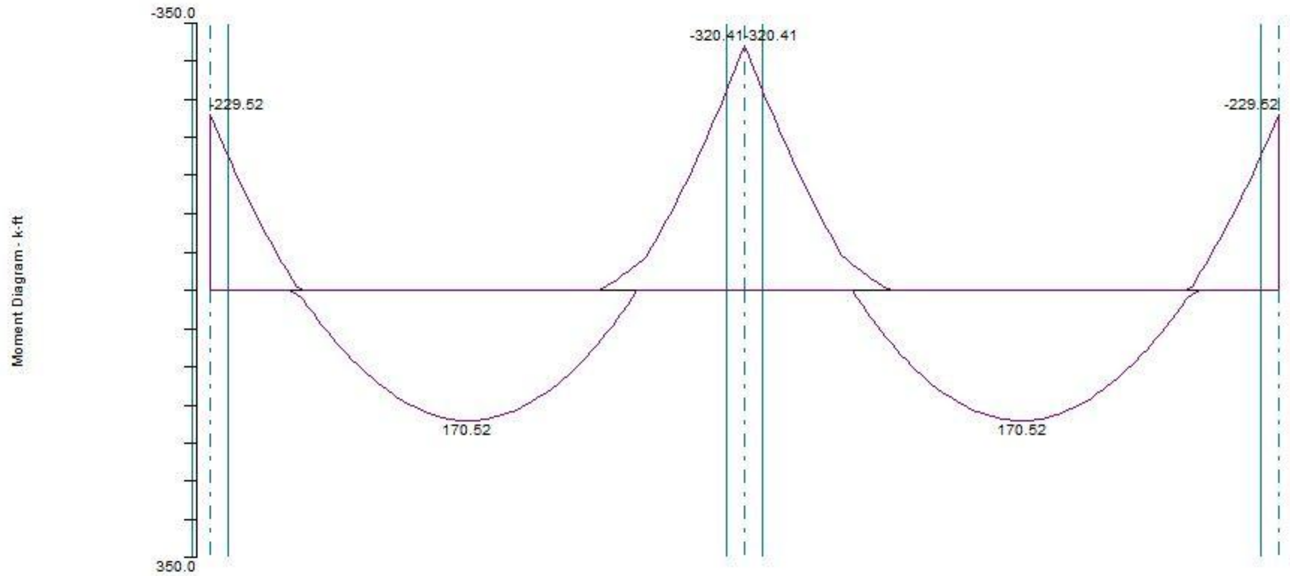


Figure 20.3 spSlap Model: Moment

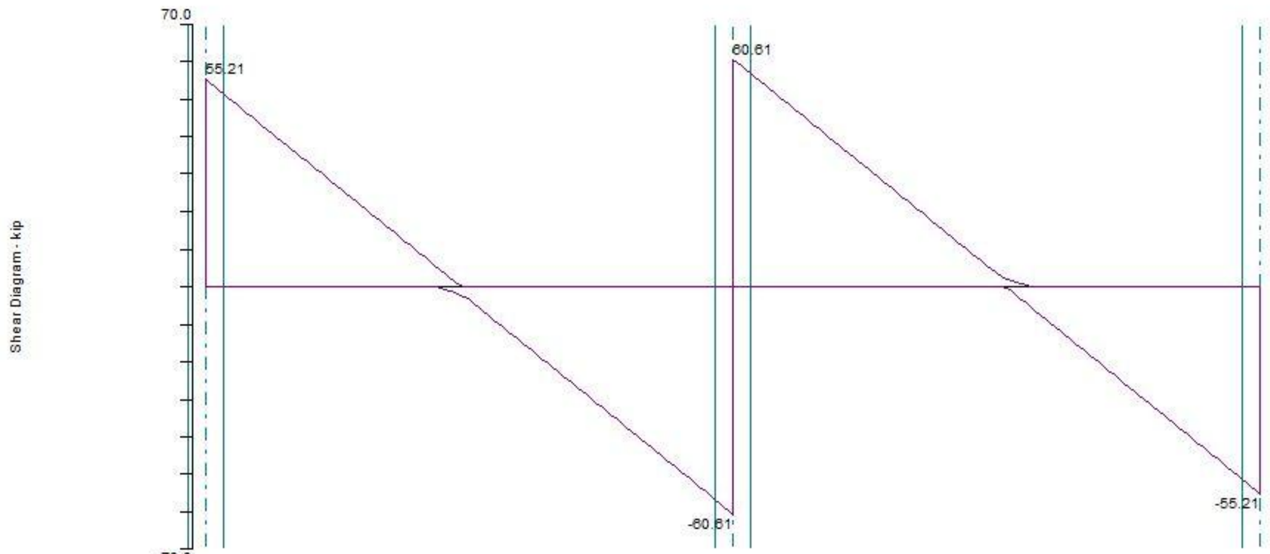


Figure 20.4 spSlap Model: Shear

Precast Plank with Wide Flange Girders

The third alternative floor system that was investigated was a precast plank with wide flange girders. The same typical bay size of 26'x30' was chosen for this analysis. It was found that an 8"x4'-0" hollow core plank, from Nitterhouse Concrete Products, with a 2" normal weight concrete topping is required to carry the load. The hollow cores were chosen to span on the shorter direction, 26', because it requires a much larger hollow core plank to span on the longer direction, 30'. The plank rest on W27x84 steel girders, which span 30'. The design was performed by hand calculations, which can be found in Appendix E The design sheet from Nitterhouse Concrete Products, for the hollow core plank, was also in Appendix E. Figure 21 shows the typical bay used for this system for TCMC.

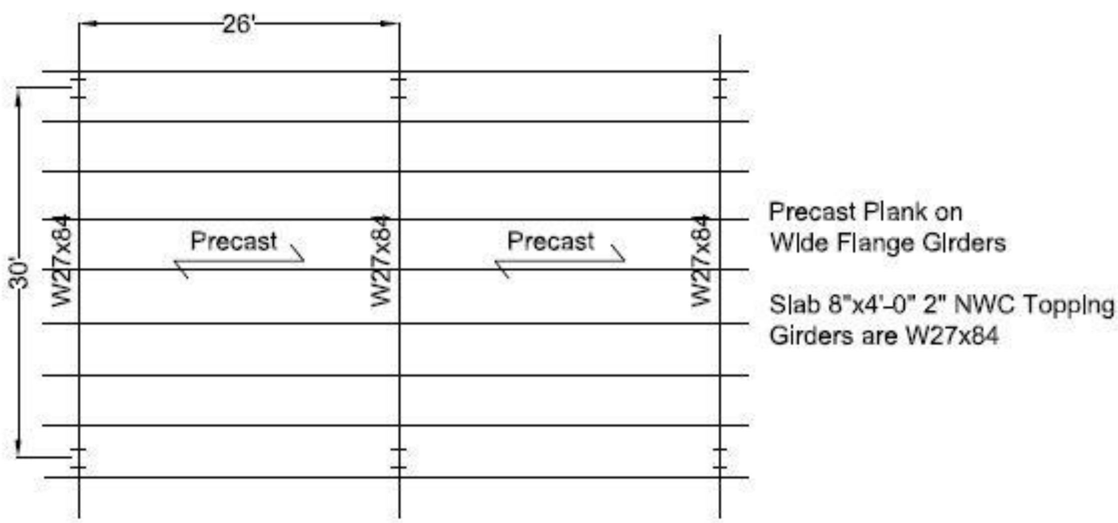


Figure 21 Precast Plank on wide flange girders

Advantages

Not many advantages can be found from this system. Its weight is similar to the two steel systems so the foundations can be kept the same. However, it does have a short lead time, reducing the stress for the construction schedule.

Disadvantages

This system has a very high cost. The construction of this system is very difficult. The performance of this system in vibration is unknown. Because the hollow core planks are pre-stressed, it is very difficult to drill through the slab when needed, and TCMC may need to drill through the slab in the near future.

Analysis

The weight of this system, at 89.5 pounds per square foot, falls in the middle for the four systems in this report. However, it costs the most, at $32.9 per square foot. This cost includes the precast production, transportation, installation, steel girders, erection, concrete topping, and fireproofing for the steel. The precast portion of the slab achieves the required 2 hour rating for fire protection by its design.

This system has the largest depth, at 34.7”. This does not create major changes to the original design because the difference is relatively small. It also does not handle well in deflection compared to the one-way slab system. One possible reason is that because the span was over 26’ while the one-way slab system span, 13ft. The lateral system does not need to be changed since steel girders and columns are still in use. Overall this system is not preferred because it is the most expensive with very little to no benefits compared to the other three systems.

Comparison of Systems

Comparison of Systems:				
Criterion	Composite System	Non-Composite	One-Way Slab	Precast Plank
Weight of System	84 psf	34.8 psf	103.7 psf	89.5 psf
Depth of Slab	7.5"	3.0"	6"	10"
Depth of System	34.4"	31"	25.5"	34.7"
Cost ($/SF)*	25.04	26.57	19.09	32.9
Deflection	1.12"	1.45"	.524"	1.32"
Architectural Impact	No change in bay size	No change in bay size	No change in bay size	No change in bay size
Fire Rating	2 hr	2 hr	2 hr	2 hr
Fire Protection	Unprotected Deck and spray on for beams	Spray-on for deck and joists	None	Spray-on for beams
Foundation Impact	N/A	May reduce required foundations	Needs to be increased	Needs to be increased slightly
Vibration	Moderate	Moderate High	Minimal	Unknown
Lateral System	No Change	No Change	Shear Walls	No Change
Constructability	Easy	Easy	Moderate	Hard
Lead Time	Long	Long	Short	Short
Viable System	N/A	Yes	Yes	Yes

* All costs are calculated using 2013, RS Means Assemblies Costs, which carries an approximate error of ±15%. Included in the cost are materials, installation, fire proofing, transportation, and labor.

Conclusions

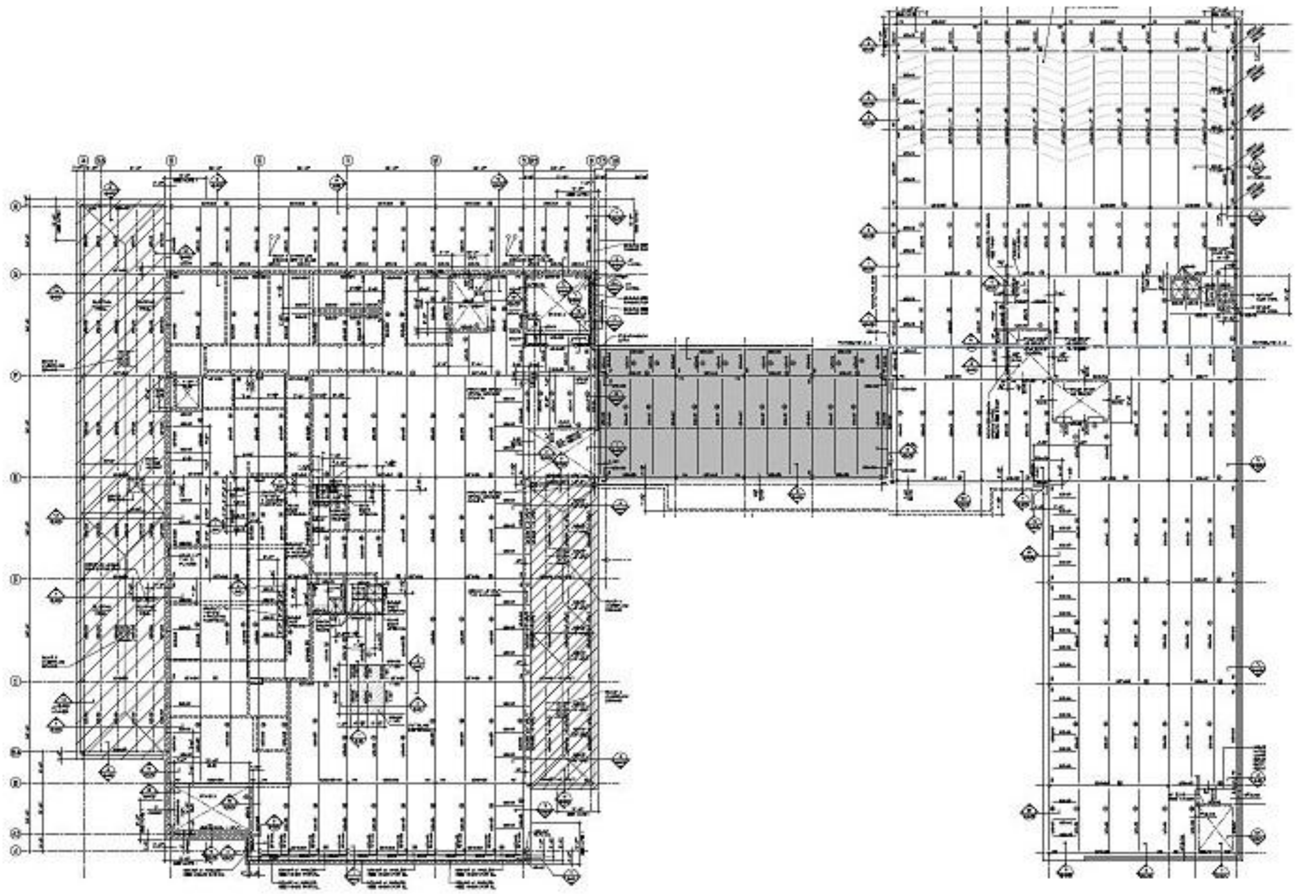
Technical Report Two was prepared with the intention of providing three other alternative floor systems that could be used in the construction of The Commonwealth Medical College. The composite system was compared with a non-composite deck on joist and joist girders, a one-way slab on concrete beams, and a precast plank on wide flange girders. The comparison made included, system weight, system cost per square foot, system depth, deflections, impact on foundations, impact on lateral system, impact on architecture, susceptibility to vibration, and fire protection.

It is found that the precast plank system would be the least economical and least efficient alternative floor system. The one-way slab would be the most economical system to use, found in this analysis.

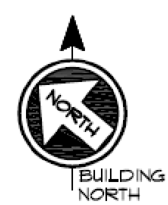
The one-way slab system cost the least to build, comparing just the price per square foot of the floor systems, but would result in significant increase in the foundations, therefore an increase in cost there. Additionally, the lateral system will be changed to shear walls.

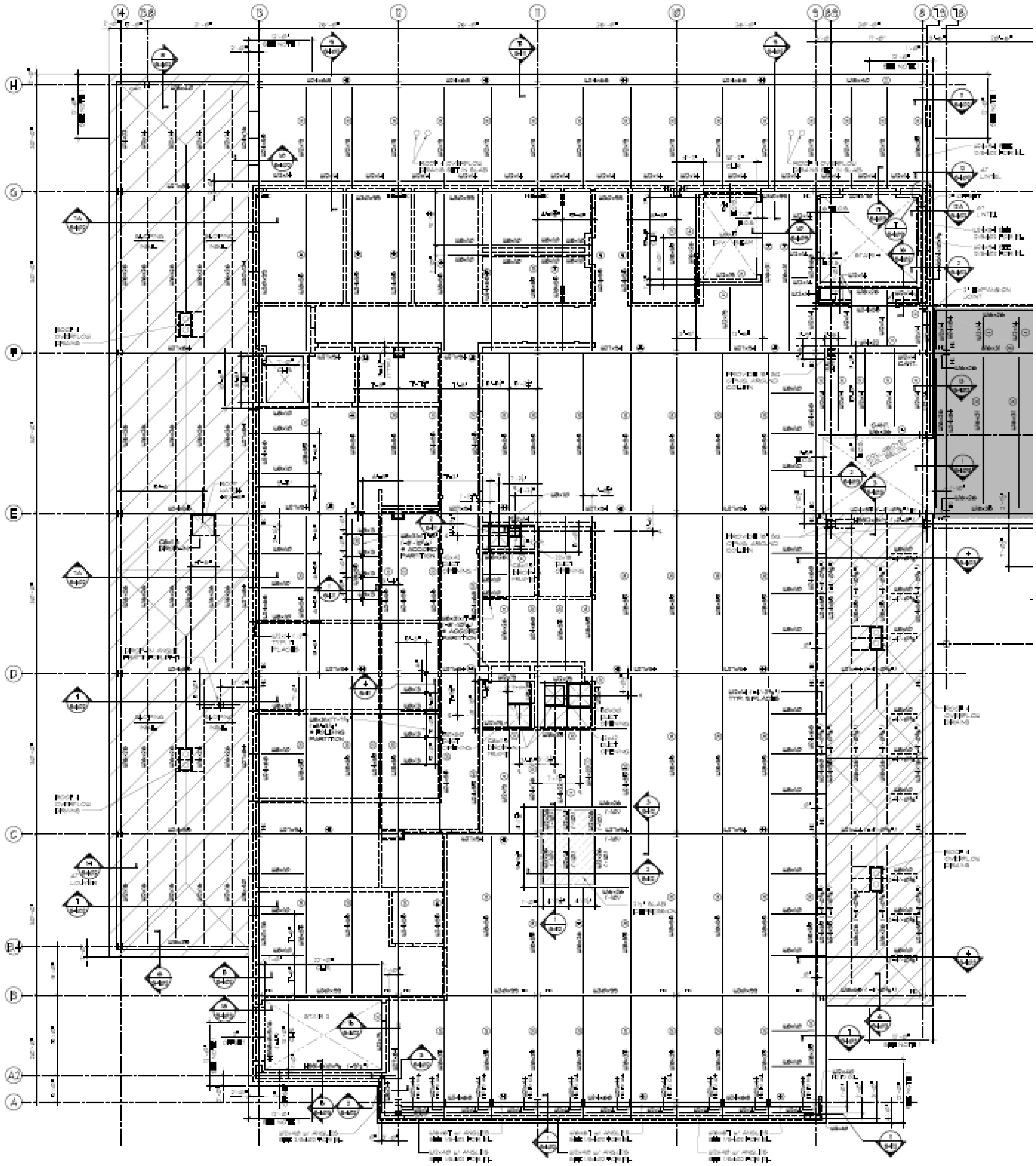
The existing system cost came in between the other systems. It was most likely chosen because if performs fairly well in deflection, average in cost, average in weight, easy to construct, and moderate sense of vibration control. The one-way slab would have been a more economical choice in this analysis but maybe the weight of the structure is what drove the owner or designer away. Although, the non-composite system has many advantages, it does cost more and performs poorly in deflection and vibration. Handling vibration is one of the most important factors for TCMC because of the medical usage of the building.

Appendix A

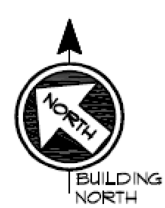


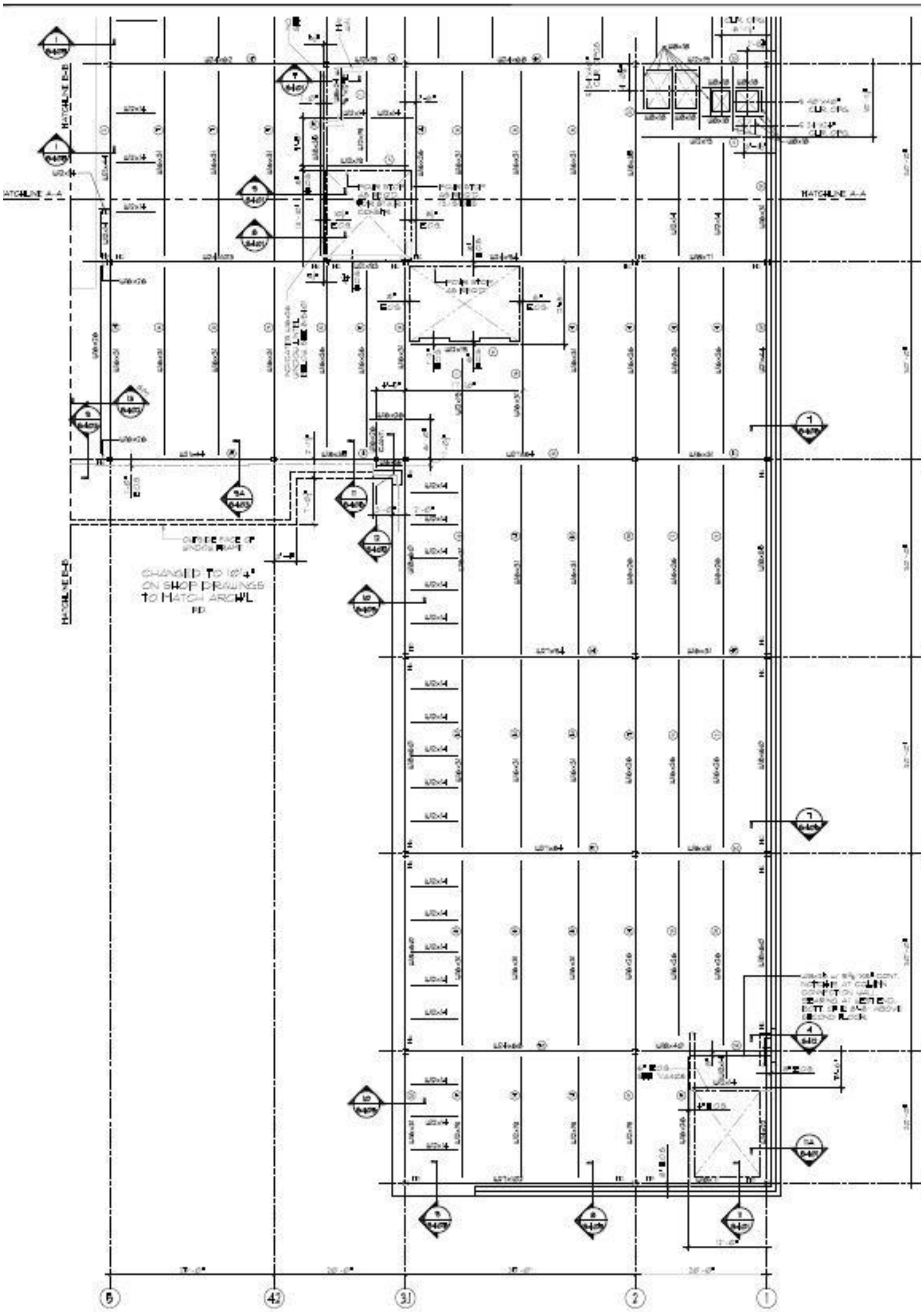
Framing Plan of the 2nd Floor, Courtesy of Highland Associates



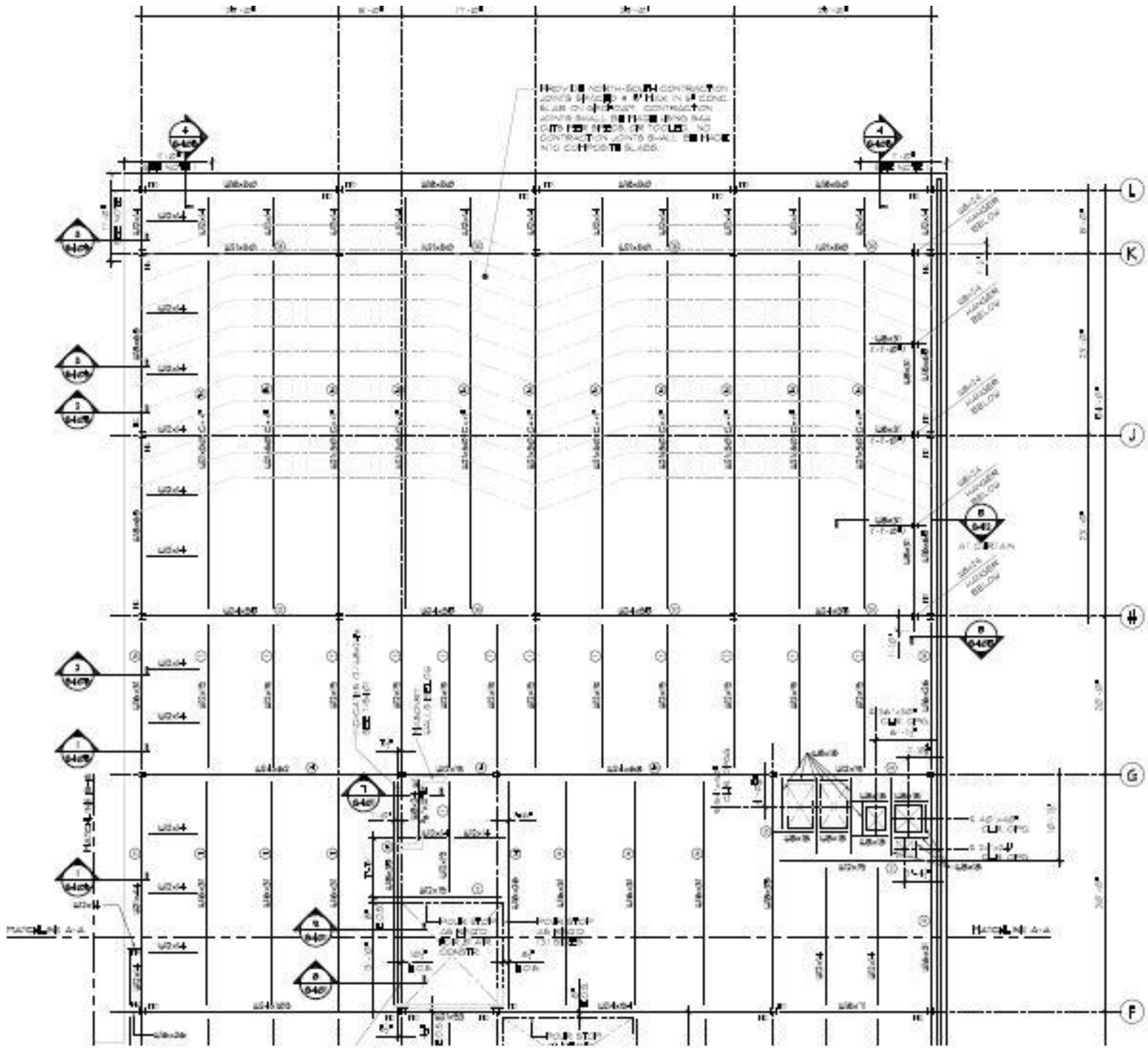


2nd Story frame, west wing, Courtesy of Highland Associates

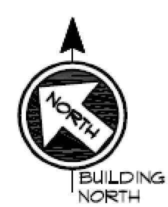




2nd Story frame, east wing (south side), Courtesy of Highland Associates

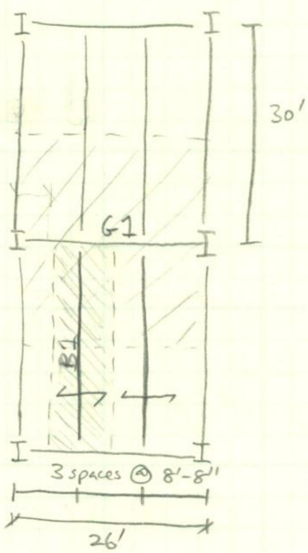


2nd Story frame, east wing (north side), Courtesy of Highland Associates



Appendix B

Typical Bay



Plan View

Current System – Composite System

3VLI20, 4½" NWC topping

B1 = W18x55 span 30' (26)

G1 = W27x94 span 26' (44)

Current system has 2hr fire rating.

¾" ⌀ x 6" long shear studs
equally spaced

– Slab Design (From 2008 Vulcraft Decking Catalog)
Needs to span over 8'-8"

Loads DL = selfweight
LL = 100psf
SDL = 18psf ⇒ 10 partitions
5 M.E.P.
3 Finishings

Superimposed loads = 100 + 18 = 118 psf

3VLI20 3span, clear span = 11'-0" > 8'-8" ✓

Use 9'-0" (round 8'-8" up to be conservative)

Carry superimposed load of 231 psf > 118 psf ✓

Fire resistance, Unprotected Deck of 4½" NWC minimum ⇒ controls.

3VLI20 has 2 hr fire rating.

3VLI20 meets all requirements

Composite B1

$A_T = 8.66''(30') = 260\ ft^2$

$K_{LL} = 2$

$LL = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) = 100\left(.25 + \frac{15}{\sqrt{520}}\right) = 90.8\ psf$

7400 ✓

$w_u = 1.2(75+18) + 1.6(90.8)\ psf$
$= 256.9\ psf$
$\Rightarrow 256.9(8.66') = 2225\ plf$

$w_{LL} = 8.66(90.8) = 786\ plf$

$w_{TL} = 786 + (75+18)(8.66) = 1592\ plf$

$M_u = \frac{(2225)(30)^2}{8} = 251\ k\cdot ft$

$R_u = \frac{2225(30)}{2} = 33.4\ k$

Table 3-21

Deck Perpendicular, $R_p = 0.60$, 1 stud per rib

NWC, 3/4" Φ studs, $f'_c = 4\ ksi$

$\Rightarrow Q_n = 17.2\ k$

beff:

$b_1' = \min \begin{cases} \text{span}/8 \\ \text{½ dist to next beam} \end{cases} = \min \begin{cases} 30(12)/8 = 45'' \Rightarrow \text{controls.} \\ 8.66(12)/2 = 52'' \end{cases}$

$b_2' = b_1' = 45''$

$b_{eff} = 2(45'') = 90''$

Let $a = 1.0$ $Y2 = 7.5 - 2.0/2 = 7.0''$

$Q_n = \min \begin{cases} 17.2k \\ R_gR_p(28.7) = 1.0(0.6)(28.7) = 17.2\ k \end{cases}$

$A_{sc}F_u = \pi\left[\frac{(3/4)}{2}\right]^2(65)$
$= 28.7k$

Table 3-19

$I_{LB} = 1240\ in^4$ $I_x = 612\ in^4$ Try W18x40 $M_u = 446\ k\cdot ft$ $\Sigma Q_n = 148k$

$= 40\ lb/ft(30') + 10\ lb/stud(20)$
$= 1400\ lb$

$148/17.2 = 8.56 \Rightarrow$ 10 studs (2) = 20 studs

$I_{LB} = 1630\ in^4$ $I_x = 843\ in^4$ Try W21x44 $M_u = 542\ k\cdot ft$ $\Sigma Q_n = 163k$

$= 44(30) + (10)(20)$
$= 1520\ lb$

$163/17.2 = 9.47 \Rightarrow$ 10 studs (2) = 20 studs

$I_{LB} = 1730\ in^4$ $I_x = 890\ in^4$ Try W18x55 $M_u = 623\ k\cdot ft$ $\Sigma Q_n = 203\ k$

$= 55(30) + 10(24)$
$= 1890\ lb$

$203/17.2 = 11.8 \Rightarrow$ 12 studs (2) = 24 studs.

Check a: For W18x40 since its the most economical

$$a = \frac{\Sigma Q_n}{0.85 f'c\, b_{eff}} = \frac{148}{0.85(4.0)(90)} = 0.484 < 1'' \text{ ok.}$$

$I_{LB} = 1210 in^4$ $I_x = 612 in^4$

Check for unshored strength

$\phi M_p = 294$ k.ft

$w_{u,dl} = 1.4(75+18)(8.66'') + 1.4(40) = .966$ k/ft

$w_{u,const} = 1.2(75+18)(8.66'') + 1.2(40) + 1.6(90.8)(8.66) = 2.28$ k/ft

$$M_u = \frac{(2.28)(30)^2}{8} = 256.5 < 294 \text{ k.ft } \checkmark$$

Check for Deflections

$w_{LL} = 90.8 psf(8.66) = 787$ lb/ft

$I_{LB} = 1210 in^4$ $Y2 = 7''$ $\Sigma Q_n = 148k$

$$\Delta_L = \frac{5(.787)(30)^4(1728)}{384(29,000)(1210)} = 0.41''$$

$$\Delta_{allow} = \frac{L}{360} = 1'' > 0.41'' \checkmark$$

$w_{TL} = 787 + (93)(8.66) = 1.6$ k/ft

$$\Delta_{TL} = \frac{5(1.6)(30)^4(1728)}{384(29,000)(1210)} = 0.85''$$

$$\Delta_{allow} = \frac{L}{240} = 1.5'' > 0.85'' \checkmark$$

wet Concrete Deflection

$w_{wc} = 75psf(8.66) + 40 = 689.5$ plf

$$\Delta_{wc} = \frac{5(.6895)(30)^4(1728)}{384(29000)(1210)} = .358 in$$

$$\Delta_{wc,max} = \frac{l}{240} = \frac{30(12)}{240} = 1.5'' > .358 in \checkmark$$

Check a: For 18x55 used in TCMC

$$a = \frac{203}{0.85(4.0)(90)} = .664 < 1'' \text{ ok}$$

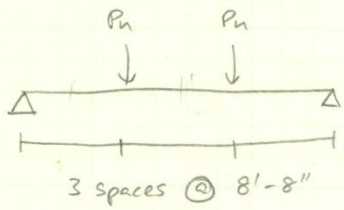
Check for deflections

$$\Delta_{LL} = \frac{5(.787)(30)^4(1728)}{384(29,000)(1730)} = 0.286'' < 1'' \checkmark$$

$$\Delta_{TL} = \frac{5(1.6)(30)^4(1728)}{384(29,000)(1730)} = 0.582'' < 1.5'' \checkmark$$

W18x40 met all criteria and is the most economical to use however, TCMC uses W18x55 which may be for vibrational purposes. So use $\boxed{W18 \times 55}$

Composite Girder (interior)



$E_c = w_c^{1.5}\sqrt{f'_c}$
$= 150^{1.5}\sqrt{4}$
$= 3674.23$

W27x94

$Q_n = 0.5A_{sc}\sqrt{f'_c E_c}$
$= 0.5\pi(5/8)^2\sqrt{4(3674)}$
$= 74.4 \text{ k}$

$\Sigma Q_n / Q_n = \#\text{ studs}$

$\Sigma Q_n / 26.77 = 22 \text{ studs}$

$\Sigma Q_n = 588.9$ K $\Rightarrow$ use $\Sigma Q_n = 490$ to be on the conservative side

$b'_1 = \min \begin{cases} \text{span}/8 & = 26(12)/8 = 39'' \Rightarrow \text{controls} \\ 1/2 \text{ dist to next bm} & = 30(12)/2 = 180'' \end{cases}$

$b'_2 = b'_1 = 39''$ for both interior/same span

$b_{eff} = 2(39) = 78''$

$a = \dfrac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \dfrac{490}{.85(4)(78)} = 1.85''$

$Y2 = 7.5 - 1.85/2 = 6.6''$

$\phi M_n = 1590 \text{ kft} @ Y2 = 6.5$
$= 1610 \text{ k·ft} @ Y2 = 7.0$

Interpolate $\dfrac{1610-1590}{7.0-6.5} = \dfrac{1610-x}{7.0-6.6} \Rightarrow x = 1594 \text{ k·ft}$

$\phi M_n = 1594 \text{ k·ft}$

$W_u = 1.2[(75+18)(8.66)] + 1.6(90.8)(8.66) = 1.91 \text{ klf}$

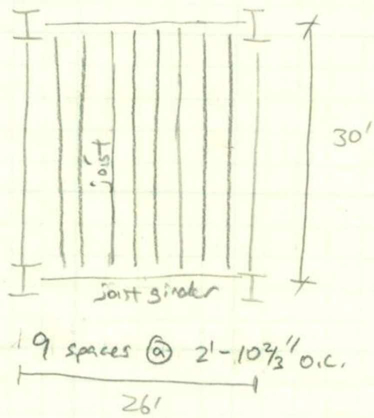
$P_u = \dfrac{1.91 \text{ klf}(30')}{2}(2) = 57.3\text{k} = V_u$

$M_u = P_u \cdot a = 57.3(8.66) = 500 \text{ k·ft} < 1594 \text{ k·ft}$ ✓

Appendix C

Non Composite Deck with Joist Girders | Tech 2 | page 1 of 3

Noncomposite Floor System



The following floor decks have a 2-hr fire resistance rating.

- Choose deck that meets both load and span criteria.

DL = self weight
SDL = 18 psf
LL = 100 psf (given S201A)

10 partitions
5 M.E.P.
3 Finishings
18 SDL Assumed

- Superimposed uniform load
  100 + 18 = 118 psf
- Span over 8'-8"
  use span 9'-0" to be conservative in the Vulcraft catalog.

NW Concrete (3-span)

2.0"	0.6C28	2'-11" ✓
3.0"	0.6C22	4'-7" ✓
3.0"	1.0C26	4'-7" ✓
3.5"	1.0C22	6'-10" ✓

> 2'-11"

- Starting with 2.0", 0.6C28 spanning 3'-0" (conservative cal)
  For total load: $F_b = 36{,}000$ ksi
  28 gage carries 119 psf @ 3'-0" span

Total Load = LL = 100
SDL = 18
DL = 23 Deck + CC
141 psf > 119 psf so deck needs to increase.
Use stronger deck.

Find deck with strength over 141 psf + additional dead weight.

- 3.0" 1.0C26 spanning 3'-0"
  For total: $F_b = 36{,}000$
  26 gage carries 232 psf @ 3' span > 141 psf ✓

Total Load = LL = 100
SDL = 18
DL = 31 Deck + CC
149 psf < 232 psf so works.

Non Composite Deck with Joist Girders	Tech 2	page 2 of 3

For live load $\ell/240$ 1.0C26

26 gage carries 188psf @ 3' span > 100 psf req'd

Summary: $\boxed{\text{Use 1.0C26 with 2.0'' NW Concrete topping (3.0'' total)}}$

because it satisfy all below.

Required < Allowed by slab

2'-10¾" < 13'-4" ✓

149 psf < 232 psi ✓

100 psi < 188 psi ✓

Joist

Unfactored Loads

SIO = 18 psf

LL = 100 psf

DL = 31 psf slab only

The joists in this bay span 30 feet long.

$w_{utl} = [1.2(31+18) + 1.6(100)](2.89') = 633$ plf

$w_{tl} = (31+18+100)(2.89) = 431$ plf

Both needs to include joist dead weight later.

$\Delta_{tl\,max} = L/240$ for floor

From K-series economy table: SJI ps C-4

26K9 : $w_{utl} = 825$ plf $> 633 + 1.2(12.2) = 648$ plf ✓

Check deflection for 26K9

w for $\ell/360$ = 459 given on table

w for $\ell/240 = 459(1.5) = 688.5$ plf > 431 plf ✓

within deflection limits.

$\boxed{\text{So use joist 26K9}}$

I=

Non Composite Deck with Joist Girders | Tech 2 | page 3 of 3

- Joist Girder

$W_{utl} = [1.2(31+18) + 1.6(100)](2.89') + 1.2(12.2) = 647.0$ plf

$P_u = \frac{647.0(30')}{1000} = 19.4^k$

Let's use 28" because it economical from joist girder table.

$\boxed{28G9N19.4F}$ girder will be used.

This girder weights about 82 plf.

- Deflection Check for joists

$26K9 \Rightarrow I_j = 26.767 W_{LL} L^3 (10^{-6})$

$W_{LL} = 459$ psf $\quad L = (30' - 0.33)$

$I_j = 26.767(459)(30'-0.33')^3(10^{-6})$

$= 321 in^4$

Formulas from {Standard Specifications K-series SJI Catalog 42nd Edition

$\Delta_{LL} = \frac{5 w_{LL} L^4}{384 E I_x} = \frac{5(.289)(30)^4(12)^3}{384(29,000)(321)}$

$W_{LL} = 2.89(100) = 289$ plf

$W_{TL} = 2.89(100) + (49)(2.89) = 431$ plf

$= .566 in$

$L/360 = \frac{30(12)}{360} = 1 in > .566 in$ ✓

$\Delta_{TL} = \frac{5(.431)(30)^4(12)^3}{384(29,000)(321)} = .843$

$L/240 = \frac{30(12)}{240} = 1.5 in > .843$ ✓ satisfies deflection

- Deflection Check for Joist Girders

$28G9N24.0F \Rightarrow I_j = 0.018 NPLd$

$= 0.018(9)(19.4)(26)(28) = 2288 in^4$

$\Delta_{LL} = \frac{5(2.6)(26)^4(12)^3}{384(29000)(2288 in)^4}$

$W_{LL} = 26(100) = 2600$ plf

$W_{TL} = 26(149) = 3874$ plf

Can be assume distributed since over 5 point loads.

$= .403'' < L/360 = 1 in$ ✓

$\Delta_{TL} = \frac{5(3.88)(26)^4(12)^3}{384(29,000)(2288)^4} = .601 in < L/240 = 1.5$ ✓

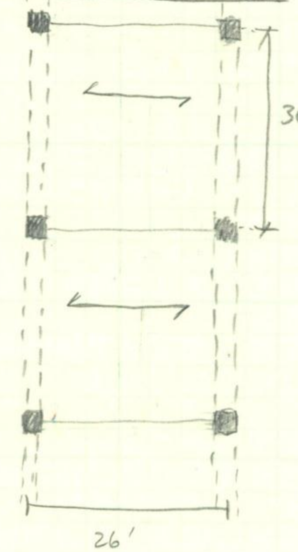
satisfies deflection

Appendix D

Two Layout will be done. The more efficient one will be used in report.

2Hr fire Rating, min 5in Slab thickness.

Layout One



Plan View

SDL = 18psf

LL = 100psf, Given S201A

DL = selfweight

10 psf partitions
5 psf M.E.P.
3 psf Finishings
18psf SDL Assumed

*Layout Two Starts on pg 6

Use ACI 318-11

Use all NW concrete
Grade 60 reinforcement
$f'_c = 4000$ psi

Try ⇒ #5 rebar for Slab
$d_b = .625$ in
$A = .31$ in^2
$w = 1.043$ lb/ft

#9 rebar for beam
$d_b = 1.128$ in
$A = 1.00$ in^2
$w = 3.40$ lb/ft

Try ⇒ #6 rebar for Slab
$d_b = .75$ in
$A = .44$ in^2
$W = 1.502$ lb/ft

#10 rebar for beam
$d_b = 1.270$
$A = 1.27$ in^2
$w = 4.303$ lb/ft

* Use 24×24 columns,
$f'_c = 4000$ psi
$f_y = 60{,}000$ psi

Slab Design

$h_{min} = \ell/28$ for interior, table 9.5(a)

$= \frac{26(12)}{28} = 11.14'' \approx 11.50'' > 5''$ min for 2hr fire rating.

SW of Slab $= (150\,pcf)\left(\frac{11.50}{12}\right) = 143.75$ psf

Minimum Concrete Cover $= 3/4''$

$d = h - cc - d_b/2 = 11.5'' - 3/4'' - .625''/2 = 10.44''$

$A_{s\,min} = .002bh$, use a 1' wide section

$= .002(12'')(11.5'') = .276$ in^2/ft of slab required

$S_{max} = 15\left(\frac{40{,}000}{40{,}000}\right) - 2.5(3/4) \le 12$

$f_s = \frac{2}{3}f_y = \frac{2}{3}(60{,}000) = 40{,}000$ *assumed

$= 13.1'' \le 12''$ ⇒ max 12'' spacing.

One-way Slab | Tech 2 | page 2 of 11

Try #5 rebars spaced at 10" O.C.

$A_s = \frac{.31(12)}{10} = .372 \text{ in}^2/\text{ft} > .276 \text{ in}^2/\text{ft}$ req'd ✓

$\rho = \frac{A_s}{bd} = \frac{.372}{(12)(10.44)} = .00297$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(.372)(60{,}000)}{.85(4000)(12)} = .547$

$\phi M_n = \phi A_s f_y (d - a/2)$

$= 0.9(.372)(60)(10.44 - .547/2)$

$= 204.2 \text{ k}\cdot\text{in}$

$w_u = 1.2(18 + 143.75) + 1.6(100) = 354.1 \text{ psf}$

$354.1(1') = 354.1 \text{ plf}$ per 1 ft section

$M_u = \frac{w_u l^2}{8} = \frac{.3541(26')^2}{8} = 29.93 \text{ k}\cdot\text{ft} = 359.1 \text{ k}\cdot\text{in}$

$359.1 \text{ k}\cdot\text{in} > 204.2 \text{ k}\cdot\text{in}$ so N.G. redesign.

Try #6 rebars spaced at 7" O.C.

$A_s = \frac{.44(12)}{7} = .754 \text{ in}^2/\text{ft} > .276 \text{ in}^2/\text{ft}$ req'd ✓

$d = 11.5" - 3/4" - .75/2 = 10.38"$

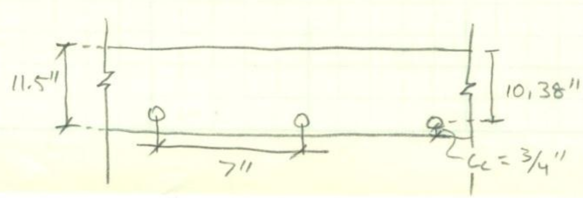
$\rho = \frac{A_s}{bd} = \frac{.754}{12(10.38)} = .00605 \Rightarrow \phi = 0.9$

$a = \frac{.754(60{,}000)}{.85(4000)(12)} = 1.109$

$\phi M_n = 0.9(.754)(60)(10.38 - 1.109/2)$

$= 400.0 \text{ k}\cdot\text{in} > 359.1 \text{ k}\cdot\text{in}$ ✓ good

Use a 11.5" slab w #6 rebars spaced at 7" O.C.



One-way Slab | Tech 2 | page 3 of 11

Beam Design

$DL = 18psf + 143.75psf + \text{Beam SW}$

$LL = 100psf$

$W_u = 1.2(18 + 143.75) + 1.6(100)$

$= 354.1 \text{ psf}$

$\Rightarrow 354.1 \text{ psf } (26') = 9207 \text{ plf}$

$= 9.207 \text{ klf}$

estimate of beam self weight

$M_u = \frac{9.207(30' - 24/12)^2}{8} \times (1.1) = 993 \text{ k·ft}$

* Use 24x24 Columns.

Estimate beam size

$bd^2 = 20 M_u$: Try $b = 2/3 d$

$d^3 = \frac{3}{2}(20)(993)$

$d = 31''$ $\quad b \Rightarrow 21''$

$h = d + 2.5'' \Rightarrow 33.5''$

$bd^2 = 21(31)^2 = 20181 \text{ in}^3$

Self Weight

$W_{sw} = \frac{(21)(33.5)(150pcf)}{144} = 733 \text{ plf}$

$W_u = 9.207 + .733 = 9.94 \text{ klf}$

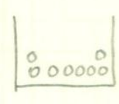
$M_u = \frac{9.94 (28')^2}{8} = 975 \text{ k·ft} < 993 \text{ k·ft}$ assumed o.k.

Required Steel

$A_s = \frac{M_u}{4d} = \frac{969}{4(31)} = 7.82 \text{ in}^2$

Try (8) # 9 rebars $= 8(1) = 8in^2 > 7.82in^2$ ✓

– Place 6 on the bottom and 2 on top of them



$d = 31'' - 1.128'' = 29.9''$

One-way slab | Tech 2 | page 4 of 11

Nominal Moment

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{8(60)}{0.85(4)(21)} = 6.73$$

$$c = a/\beta_1 = 6.73/.85 = 7.92$$

$$\varepsilon_s = \varepsilon_u\left(\frac{d-c}{c}\right) = .003\left(\frac{29.9 - 7.92}{7.92}\right) = .0083 > .005 \text{ yields}$$

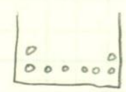
$$\Rightarrow \phi = 0.9$$

$$\phi M_n = \phi A_s f_y (d - a/2)$$
$$= .9(8)(60)(29.9 - 6.73/2)$$
$$= 955 \text{ k·ft}$$

$\phi M_n = 955$ k·ft $< M_u = 975$ k·ft N.G.

Try (8) #10 rebars $8(1.27) = 10.16 \text{ in}^2 > 7.82 \text{ in}^2$ ✓

- Place 6 on the bottom and 2 on top of them



$$d = 31'' - 1.27 = 29.8''$$

$$a = \frac{10.16(60)}{.85(4)(21)} = 8.54$$

$$c = 8.54/.85 = 10.04$$

$$\varepsilon_s = .003\left(\frac{29.8 - 10.04}{10.04}\right) = .0059 > .005 \text{ yields}$$

$$\Rightarrow \phi = 0.9$$

$$\phi M_n = .9(10.16)(60)(29.8 - 8.54/2)$$
$$= 1167.2 \text{ k·ft}$$

$\phi M_n = 1167$ k·ft $> M_u = 975$ k·ft ✓ good

Min Steel Area

$$A_{s,min} = \frac{200}{f_y} bd = \frac{200}{60{,}000}(21)(31) = 2.17 \text{ in}^2 < 10.16 \text{ in}^2 \checkmark$$

Max reinforcement

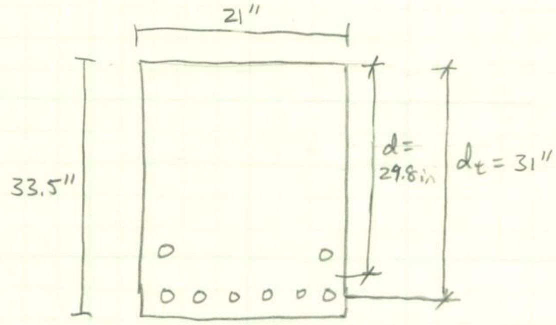
$$\rho_{max} = 0.85\beta_1 \frac{f'c}{f_y}\left(\frac{\varepsilon_u}{\varepsilon_u + .005}\right)$$

$$= .85(.85)\left(\frac{4}{60}\right)\left(\frac{.003}{.008}\right)$$

$$= .0181$$

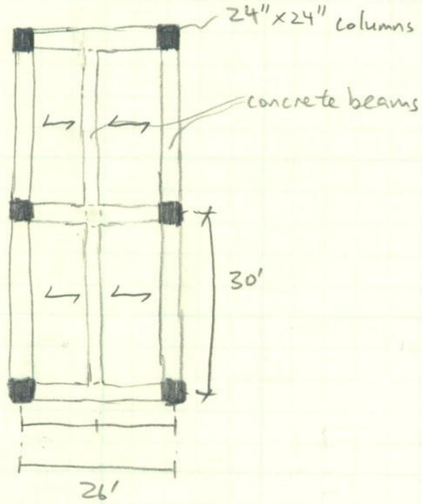
$$\rho = \frac{A_s}{bd} = \frac{10.16}{(21)(29.8)} = .0162 < .0181 \checkmark$$

Use 21" x 31" beam with (8) #10 rebars



One-way Slab | Tech 2 | Page 6 of 11

Layout Two



Same materials as layout one unless listed below.

Try #4 rebar for slab
$d_b = .5in$
$A_b = .2in$
$w = .668\ lb/ft$

Slab Design

$h_{min} = l/28$ for interior, table 9.5(a)

$= 13/28 = 5.57'' \approx 6''$ slab $> 5''$ min for fire rating

SW of slab $= (150pcf)(6/12) = 75psf$

minimum concrete cover $= 3/4''$

$d = h - cc - d_b/2 = 6'' - 3/4'' - \frac{.5}{2} = 5.0''$

$A_{s,min} = .0026h$, use a 1" wide section

$= .002(12'')(6) = .144\ in^2/ft$ of slab required

$S_{max} = 15\left(\frac{40,000}{40,000}\right) - 2.5(3/4) \leq 12$

$= 13.1'' \leq 12'' \Rightarrow 12''$ max spacing.

Try #4 rebars spaced at 12" O.C.

$A_s = \frac{.20(12)}{12} = .20 in^2 > .144\ in^2/ft$ required

$\rho = \frac{(.20)}{(12)(5.0)} = .00337$

$a = \frac{(.20)(60,000)}{.85(4000)(12)} = .294$

One Way Slab | Tech 2 | Page 7 of 11

$\phi M_n = 0.9(.20)(60)(5.0 - .294/2) = 51.76 \text{ k·in}$

$w_u = 1.2(18+75) + 1.6(100) = 271.6 \text{ psf}$

$271.6(1') = 271.6 \text{ plf}$ per 1 ft section

$M_u = \frac{w_u l^2}{8} = \frac{.2716(13)^2}{8} = 5.74^{\text{k·ft}} = 68.85 \text{ k·in}$

$68.85^{\text{k·in}} > 51.76^{\text{k·in}}$ so N.G. $\Rightarrow$ redesign

Try #6 rebars spaced at 11" o.c.

$A_s = \frac{.44(12)}{11} = .480 \text{ in}^2/\text{ft} > .276 \text{ in}^2/\text{ft}$

$d = 6'' - 3/4'' - .75/2 = 4.875''$

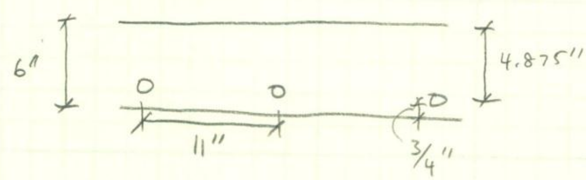
$\rho = \frac{.480}{(12)(4.875)} = .0082$

$a = \frac{.48(60{,}000)}{.85(4000)(12)} = .705$

$\phi M_n = 0.9(.480)(60)(4.875 - .705/2) = 117.2 \text{ k·in}$

$117.2^{\text{k·in}} > M_u = 68.85^{\text{k·in}}$ ✓ good

$\boxed{\text{Use a 6" slab with \#6 rebars @ 11" o.c.}}$



One - Way Slab | Tech 2 | Page 8 of 11

Beam Design

$DL = 18 + 75 + \text{Beam SW}$

$LL = 100 \text{ psf}$

$W_u = 1.2(18+75) + 1.6(100) = 178.6 \text{ psf}$

$\Rightarrow 178.6(13') = 2321.8 \text{ plf}$

$= 2.322 \text{ klf}$

estimate of beam SW

$$M_u = \frac{2.33(30' - 24/12)^2}{8}(1.2) = 275 \text{ k·ft}$$

Estimate beam size

$bd^2 = 20M_u$: Try $b = 2/3\, d$

$d^3 = 3/2(20)(275)$

$d = 19.6 \approx 20'' \Rightarrow b = 13.5''$

$h = d + 2.5'' = 22.5''$

$bd^2 = (13.5)(20)^2 = 5400 \text{ in}^3$

Self weight

$$W_{sw} = \frac{(22.5)(13.5)(150)}{144} = 316.4 \text{ plf}$$

$w_u = 2.322 + .3164 = 2.638$

$$M_u = \frac{2.638(28')^2}{8} = 259 \text{ k·ft} < 275 \text{ k·ft assumed. ✓}$$

Required steel

$$A_s = \frac{M_u}{4d} = \frac{275}{4(20)} = 3.44 \text{ in}^2$$

Try (5) #9 rebars $= (4)(1) = 4 \text{ in}^2 > 3.44 \text{ in}^2$ ✓

$d = 20''$

Nominal Moment

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(60)(4)}{0.85(4)(13.5)} = 5.23$$

$$c = a/\beta_1 = 5.23/.85 = 6.15 \qquad \varepsilon_s = .003\left(\frac{20 - 6.15}{6.15}\right) = .00676 > .005$$

$\Phi = .9$

$$\Phi M_n = .9(4)(60)(20 - 5.23/2) = 319.0 \text{ k·ft} > M_u = 259 \text{ k·ft ✓ good}$$

Min Steel Area

$$A_{s,min} = \frac{200}{f_y}bd = \frac{200}{60{,}000}(13.5)(20) = .9 \text{ in}^2 < 4 \text{ in}^2 \quad ✓$$

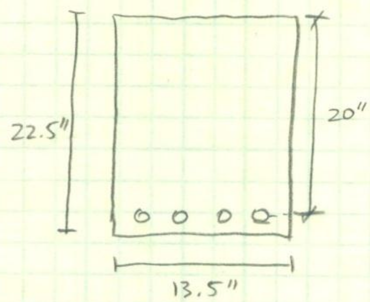
One - Way Slab | Tech 2 | Page 9 of 11

Max reinforcement

$$\rho_{max} = 0.85(.85)\left(\frac{4}{60}\right)\left(\frac{.003}{.008}\right) = .0181$$

$$\rho = \frac{4}{(13.5)(20)} = .0148 < .0181 \text{ in}^2 \checkmark$$

Use 13.5" x 22.5" beam with (4) #9 rebars



Girder Design

$$W_u = 1.2\left[(75+18)(13')\right] + 1.6(100)(13) + 316.4 = 3847.2 \text{ plf}$$

$$P_u = \frac{3.85^{klf}(30')}{2}(2) = 115.5 \text{ k} = V_u$$

$$M_u = \frac{(24)(115.5)}{8}(1.1) = 381.15$$

Estimate girder size.

$bd^2 = 20 M_u$ ∴ Try $b = 2/3\, d$

$d^3 = 3/2\,(20)(381.2)$

$d = 22.6 \approx 23" \Rightarrow b = 15"$

$h = 23" + 2.5" = 25.5"$

$bd^2 = (15)(23)^2 = 7935 \text{ in}^3 > 7623 \text{ in}^3 \checkmark$

Self weight

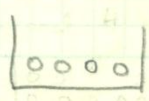
$$W_{sw} = \frac{(25.5)(15)(150)}{144} = 399 \text{ plf}$$

$$M_u = \frac{(24)(115.5)}{8} + \frac{(399)(24)^2}{8} = 375.3 \text{ k·ft}$$

Required Steel

$$A_s = \frac{M_u}{4d} = \frac{375.3}{4(23)} = 4.08 \text{ in}^2$$

Try (4) # 10 rebars $4(1.27) = 5.08 \text{ in}^2 > 4.08 \text{ in}^2$



One-Way Slab | Tech 2 | Page 10 of 11

$d = 23''$

$a = \frac{(5.08)(60)}{.85(4)(15)} = 5.98$ $\quad c = \frac{5.98}{.85} = 7.04$

$\varepsilon_s = .003\left(\frac{23 - 7.04}{7.04}\right) = .0068$

$> .005$

$\phi = .9$

$\phi M_n = .9(5.08)(60)(23 - 5.98/2)$

$= 457.5^{k \cdot ft} > 375.3^{k \cdot ft}$ ✓

Min steel area

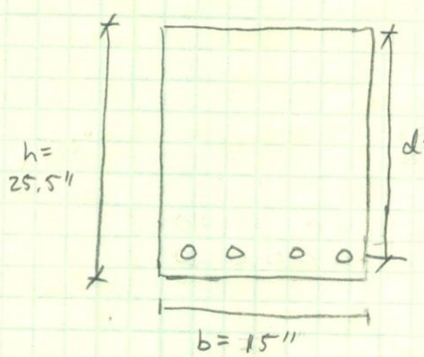
$A_{s,min} = \frac{200}{60,000}(15)(23) = 1.15 in^2 < 5.08 in^2$ ✓

max reinf

$\rho_{max} = 0.85(.85)\left(\frac{4}{60}\right)\left(\frac{.003}{.008}\right) = .0181$

$\rho = \frac{5.08}{(15)(23)} = .0147 < .0181 in^2$ ✓

$\boxed{\text{So use } 15'' \times 25.5'' \text{ concrete girder with (4) \#10 rebars}}$



Layout 2 is alot more efficient so will use layout 2 instead of layout 1 in the report.

Check Concrete (24" x 24") only pure axial.

Total Pu

$(13)(15)[103.7(1.2) + 1.2(18) + 1.6(100)](\overset{4\ floors}{4}) + 13(15)[103.7(1.2) + 1.2(90) + 1.6(150)]$

$239k + 93k$

$P_u = 332k$

One-way Slab | Tech 2 | page 11 of 11

$$\varepsilon_y = \frac{f_y}{E_s} = \frac{60}{29{,}000} = 0.00207$$

(4) #10 rebars $= (4)(1.27) = 5.08\,in^2$

$f_y = 60$ ksi

$$P_o = 0.85(4)(24 \cdot 24 - 5.08) + (5.08)(60)$$

$$P_o = 1941 + 304$$

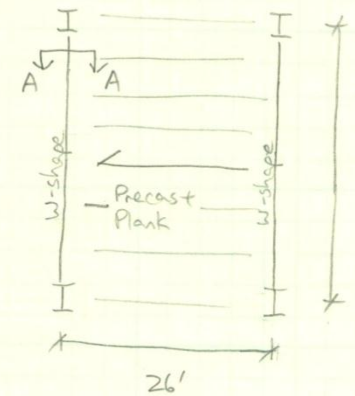
$$P_o = 2245\,k > 332\,k$$

Column 24" x 24" is more than enought for pure axial compression.

Appendix E

Precast Plank/Steel Girder Tech 2 page 1 of 3

Used on a typical Bay
Precast Plank Slab Design



Plan View

Dead Load = Self weight
Live Load = 100 psf Found on S201A
Superimposed Dead Load = 18 psf

Finishing	3 psf
Partitions	10 psf
M.E.P.	5 psf
SDL =	18 psf

Use Nitterhouse hollowcore planks.
2hr fire rating required.

Try 8" x 4'-0" Hollow Core Plank
- 2hr fire resistance rating with 2" topping

Self weight noted in catalog (attached in report)

$W_{sw} = 61.25\ psf + \underbrace{150pcf\left(2/12\right)}_{25psf\ for\ 2''\ NWC\ top} = 86.25$

Service load

$w = 86.25 + 18 + 100 = 204.25\ psf$

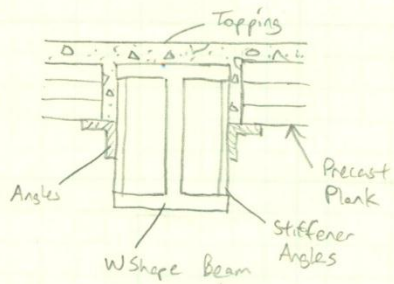
Superimposed load

$w = 18 + 100 = 118\ psf$

Check Mu   Planks are 4' wide

$M_u = \frac{(204.25)(4)(26)^2}{8} = 69.04\ k\cdot ft$

$= 828.5\ k\cdot in$

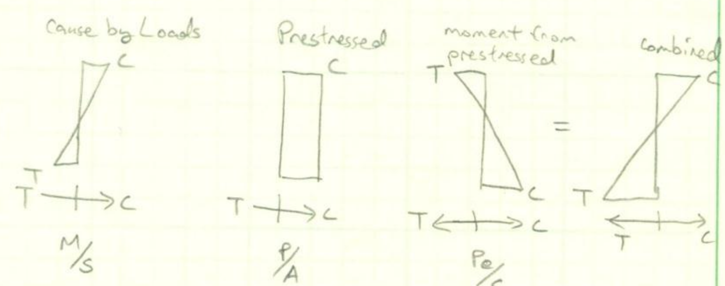


Section A-A

Check for Stresses

$P_{eff} = .6(270ksi)(0.153in^2) = 24.8\ k/strand$

$0.153 = A_{wire},\ f_{pu} = 270ksi$



Precast $S_{bcp} = 616\ in^3$

Topping $S_{tcp} = 1076\ in^3$

$A_c = 301\ in^2$

$f'c = 6000\ psi$

$Y_{bcp} = 5.09\ in$

$e = 5.09 - 1.75 = 3.34$

$I_c = 3134\ in^4$

$f_T = \frac{-828.5}{1076} - \frac{6(24.8)}{301} + \frac{6(24.8)(3.34)}{1076} = 802\ psi\ comp$ ✓

$< .45f'c = 2700\ psi$

$f_b = \frac{828.5}{616} - \frac{6(24.8)}{301} - \frac{6(24.8)(3.34)}{616} = 44\ psi\ ten$

$< 10\sqrt{f'c} = 775\ psi$ ✓

Precast Plank/Steel Girder | Tech 2 | page 2 of 3

Check for deflections

$$\Delta_{LL} = \frac{5(1.6)(0.1\,ksf)(4ft)(26ft)^4(1728)}{384(29{,}000\,ksi)(3134\,in^4)} = .073\,in < \Delta_{LLmax} = 0.7\,in \checkmark$$

Since its post-tensioned, it won't even deflect this much.

$$\Delta_{TL} = \frac{5[1.2(0.018+0.08625)+1.6(0.1)](4)(26)^4(1728)}{384(29000)(3134)} = 0.129\,in < \Delta_{TLmax} = 1.05\,in \checkmark$$

Factored Mu

$$M_u = \frac{[1.2(.018+.08625)+1.6(.1)]\,4(26)^2}{8} = 96.4\,k\cdot ft = 1157\,k\cdot in$$

Choosing a strand pattern of 6 – ½" ϕ,

M = 130.6 k-ft @ 60% jacking force

M = 130.6 k·ft > 96.4 k·ft ✓ this is good.

| So use 8" x 4'-0" 2 hr rating, 2" topping, 6-½" ϕ strands is good |

Choosing the Girder required * assume 10 psf for framing

$$w_D = (18 + 86.25 + 10)(26) = 2970.5\,plf$$

$$LL_r = 0.25 + \frac{15}{\sqrt{2(26)(30)}} = .63$$

$26 \cdot 30 = 780 > 400 \checkmark$

$$w_L = 0.630(100)(26) = 1638\,plf$$

$$w_u = 1.2(2.97) + 1.6(1.64) = 6.19\,klf$$

$$I_{req,LL} = \frac{5(1.6)(1.64)(30)^4(1728)}{384(29000\,ksi)(30\times12/360)} = 1650\,in^4$$

$$I_{req,TL} = \frac{5(6.19)(30)^4(1728)}{384(29{,}000)(30\times12/240)} = 2593\,in^4$$

Check for Moment and Shear required

$$M_u = \frac{6.19(30)^2}{8} = 697\ k\cdot ft$$

$$V_u = \frac{6.19(30)}{2} = 92.9k$$

Going to table 3-2 in steel manual to look up some beams.

	(k·ft)	(k)	(in⁴)	
W 24x68	$\phi M_n = 664$	$\phi V_u = 295$	I = 1830 x	
W 24x84	$\phi M_n = 840$	$\phi V_u = 340$	I = 2370 x	
W 27x84	$\phi M_n = 915$	$\phi V_u = 368$	I = 2850 ✓	most economical that meets all criteria
	915 > 697 ✓	368 > 92.9 ✓	2850 > 2593 ✓	

* If ceiling height is more important, ⇒ use W 18 to match existing W18s.

W 18x130	$\phi M_n = 1090$	$\phi V_n = 388$	I = 2460 x
W 18x143	$\phi M_n = 1210$	$\phi V_n = 427$	I = 2750 ✓
	1210 > 610 ✓	427 > 81.3 ✓	2750 > 2271 ✓

$\boxed{\text{Using W27x84 is the most economical.}}$

Check for deflection:

$w_{LL} = 100(26) = 2.6\ klf$

$w_{TL} = 2600 + (18 + 86.25)(26) + 84 = 5.4\ klf$

$$\Delta_{TL} = \frac{5(5.4)(30)^4(12)^3}{384(29{,}000)(2850)} = 1.19'' < 1.5'' \checkmark$$

$$\Delta_{LL} = \frac{5(2.6)(30)^4(12)^3}{384(29000)(2850)} = 0.57'' < 1.0'' \checkmark$$

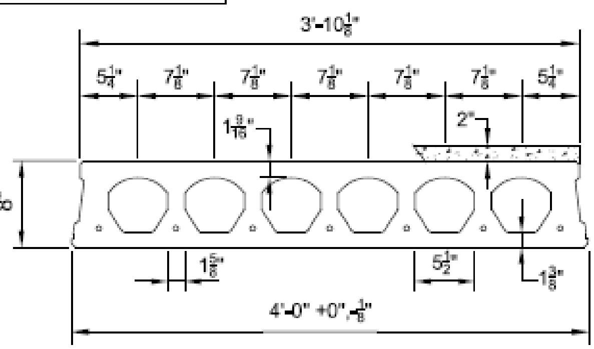
Prestressed Concrete
8"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 301 in.^{2}	Precast b_w = 13.13 in.
I_c = 3134 in.^{4}	Precast S_{bcp} = 616 in.^{3}
Y_{bcp} = 5.09 in.	Topping S_{tct} = 902 in.^{3}
Y_{tcp} = 2.91 in.	Precast S_{tcp} = 1076 in.^{3}
Y_{tct} = 4.91 in.	Precast Wt. = 245 PLF
	Precast Wt. = 61.25 PSF

DESIGN DATA



1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   4-1/2"Ø, 270K = 92.3 k-ft at 60% jacking force
   6-1/2"Ø, 270K = 130.6 k-ft at 60% jacking force
   7-1/2"Ø, 270K = 147.8 k-ft at 60% jacking force
7. Maximum bottom tensile stress is $10\sqrt{f'c}$ = 775 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS — IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)

Strand Pattern		SPAN (FEET)																		
		17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
4 - 1/2"ø	LOAD (PSF)	280	248	214	185	159	138	118	102	87	74	62	52	42						
6 - 1/2"ø	LOAD (PSF)	366	341	318	299	271	239	211	187	165	146	129	114	101	88	77	67	58	50	42
7 - 1/2"ø	LOAD (PSF)	367	342	320	300	282	265	243	221	202	181	161	144	128	114	101	90	79	70	61

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

NITTERHOUSE
CONCRETE PRODUCTS

2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17202-9203
717-267-4505 Fax 717-267-4518

11/03/08

8SF2.0T

Appendix F

Weight of Systems

* Note, for one-way slab, there is calculations for both layout, 1 and 2. Only layout 2 will be used to compared with the other 3 floor system because its more efficient.

Composite System:

slab/deck = 75 psf
Beam = 55 plf
Girder = 94 plf

$W_{sw} = 75\text{psf} + 55/8.66 + 94/35$
$= \boxed{84\text{ psf}}$

Non composite on Joists and Joist Girders:

slab/deck = 31 psf
Joist = 12.2 plf
Girder Joist = 82 plf

$W_{sw} = 31\text{psf} + 12.2\text{plf}/8.66' + 82\text{plf}/35'$

$= \boxed{34.8\text{psf}}$

One-way Slab on Concrete Beams:

Layout 1

slab/deck = 143.8 psf for 11.5" NWC Slab
Beam = $150(33.5/12)(21/12) = 733$ plf
$\text{Rebar}_{slab} = 1.502\text{ lb/ft}\,(12/7) = 2.58$ psf
$\text{Rebar}_{beam} = 8(4.303) = 34.42$ plf

$W_{sw} = 143.8\text{psf} + 733/26' + 2.58\text{psf} + 34.42/30'$
$= \boxed{175.7\text{ psf}}$

Layout 2

75 psf for 6" NWC
$150(13.5)(22.5)/144 = 316$ plf
$150(15)(25.5)/144 = 399$ plf
Rebars $- (4)(3.4)/(30) + (4)(4.3)/26 = 1.12$ psf
$\text{Rebars}_{slab} = 1.502(12/11) = 1.64$ psf

$W_{sw} = 75 + 316/30 + 399/26 + 1.64 + 1.12$
$\boxed{= 103.7\text{psf}}$

Precast Plank on wide Flange Girders:

slab = 86.25 psf
Girder = 84 plf

$W_{sw} = 86.25\text{psf} + 84/26' =$
$= \boxed{89.5\text{ psf}}$

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System Depths

Composite System:

slab = 7.5"
beam = 18.1"
girder = 26.9" ⇒ controls

$d = 7.5'' + 26.9'' = \boxed{34.4''}$

Noncomposite on Joists and Joist Girders:

slab = 3.0"
joist = 26"
Girder joist = 28" ⇒ controls.

$d = 3.0 + 28'' = \boxed{31''}$

One-way slab on Concrete beams:

Layout 1
Slab = 11.5"
beam = 33.5"

$d = \boxed{33.5''}$

Layout 2
Slab = 6"
beam = 22.5"
girder = 25.5"

$\boxed{d = 25.5''}$

Precast Plank on wide Flange Girders:

slab = 10"
girder = 24.7"

$d = 10'' + 24.7'' = \boxed{34.7''}$

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Deflections

Composite System:

Beam: 0.59 in $= \Delta_{TL}$, 0.29 in $= \Delta_{LL}$

Girder: 0.53 in $= \Delta_{TL}$, 0.22 in $= \Delta_{LL}$

.59 + .53 = 1.12 in

$\boxed{1.12 \text{ in}}$

Noncomposite on joists and joist girders:

Joists: 0.843 in $= \Delta_{TL}$, 0.566 in $= \Delta_{LL}$

Joist Girders: 0.601 in $= \Delta_{TL}$, 0.403 in $= \Delta_{LL}$

$.843 + .601 = \boxed{1.45 \text{ in}}$

One-Way Slab on Concrete Beams:

Layout 1

Beam: $\Delta_{TL} = \frac{5 w_{TL} l^4}{384 EI} = \frac{5(7.54)(28)^4(1728)}{384(3605)(65,800)}$

$I = \frac{1}{12}bh^3 = \frac{1}{12}(21)(33.5)^3 = 65,800 \text{ in}^4$

$E = 57000\sqrt{4000} = 3605 \text{ ksi}$

$\Delta_{TL} = 0.440''$

$\Delta_{LL} = 0.152''$

beam: $W_{TL} = [(18+143.75)+100]26' + 733 = 7.54 \text{ klf}$

Slab: $\Delta_{TL} = \frac{5(.244)(26)^4(1728)}{384(3605)(1521)}$

beam: $W_{LL} = 100(26) = 2.6 \text{ klf}$

$\Delta_{TL} = 0.458''$

slab: $W_{TL} = (143.75+100)(1') = 243.75 \text{ plf}$

$\Delta_{LL} = 0.188''$

slab: $W_{LL} = 100(1') = 100 \text{ plf}$

$I = \frac{1}{12}bh^3 = \frac{1}{12}(12)(11.5)^3 = 1521 \text{ in}^4$

$\Delta_T = .44 + .458 = \boxed{.898 \text{ in}}$

Layout 2

Beam: $\Delta_{TL} = \frac{5(6.1)(30)^4(1728)}{384(3605)(12,814.5)} = .14'' < 1.5''$

beam: $W_{TL} = [(18+75)+100]30' + 317 = 6.1 \text{ klf}$

$\Delta_{LL} = \frac{5(3)(30)^4(1728)}{384(3605)(12,814.5)} = .106 < 1''$

beam: $W_{LL} = 100(30) = 3.0 \text{ klf}$

$I = \frac{1}{12}(13.5)(22.5)^3 = 12,814.5$

Girder: $\Delta_{TL} = \frac{Px^2(3l-4x)}{48EI} = \frac{115.5(12)^2[3(26)-4(12)](1728)}{48(3605)(20,726)}$

$I = \frac{1}{12}(15)(25.5)^3 = 20,726$

$= .24 \text{ in}$ ✓

$\Delta_{LL} = .06 \text{ in}$ ✓

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– Deflections (continue)

Slab: $\Delta_{TL} = \frac{5(.175)(13)^4(1728)}{384(3605)(216)} = .144'' < 1.5''$ ✓

slab: $w_{TL} = (75+100)(1') = 175\,plf$

$w_{LL} = 100(1') = 100\,plf$

$I = \frac{1}{12}(12)(6)^3 = 216$

$\Delta_{LL} = \frac{5(.1)(13)^4(1728)}{384(3605)(216)} = .0826'' < 1''$ ✓

$\Delta_T = .14 + .144 + .24 = \boxed{.524\text{ in}}$

Precast Plank on wide Flange Girders:

Slab: $0.129'' = \Delta_{TL}$, $0.073'' = \Delta_{LL}$

Joist Girder: $1.19'' = \Delta_{TL}$, $0.53'' = \Delta_{LL}$

$0.129 + 1.19 = \boxed{1.32\text{ in}}$

Fire Ratings 2^+hr fire rating for all systems required

Composite System:

Using Unprotected Deck, 4½" NWC Topping, 3VLI = 2hr fire rating

Beams and girders are 2 hr spray fire proofing

Noncomposite on Joists and Joist Girders:

2 hr spray fire proofing for deck, joists and joist girders.

One-way Slab on Concrete Beams:

Layout 1 Using an 11.5" concrete slab is thick enought to reach a fire rating of 2hr. No additional protection is required.

Layout 2 6.0" concrete slab > 5" for 2hr fire rating. No additional protection is required.

Precast Plank on wide Flange Girders:

Hollow Core Planks are 2 hr fire ratings with 2" Concrete Topping.

Girders are 2 hr spray fire proofing.