Park Potomac Office Building "E"

Technical Assignment #2

Potomac, MD

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Structural Option (IP)

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Executive Summary

This report studies alternative floor systems to the post-tensioned system used in the original design of Park Potomac Office Building "E". These floor systems were chosen in an effort to maintain the column layouts used in the original design, in an effort to maintain the value of the rental spaces and promote space planning for future tenants. This resulted in the need for long spans, up to 45'. For this reason, the following alternates were proposed:

1. Steel Composite
2. Steel Noncomposite
3. Hollow-core Precast Planks on Steel Beams

The comparison was completed by analyzing the building's three most critical bays. These bays can be seen in the figure below. The criteria for comparison included cost, weight, floor depth, fireproofing, vibration, construction considerations, and potential foundation and lateral system changes.

After analysis and comparison of the alternate floor systems, it was revealed that the noncomposite system was not feasible for this project. The higher cost and larger self weight provided no advantage to the noncomposite system over the composite alternative. Additionally, the hollow-core precast planks were also found to not be beneficial for this project. The large concrete weight, high cost, and inconvenient 4' sections made this system an unattractive option. The composite system was found to be the most viable alternative. Despite a deeper floor than the existing system and requiring fireproofing, this system was light, cheap, constructible, and allowed for potential savings in foundation and superstructure designs.

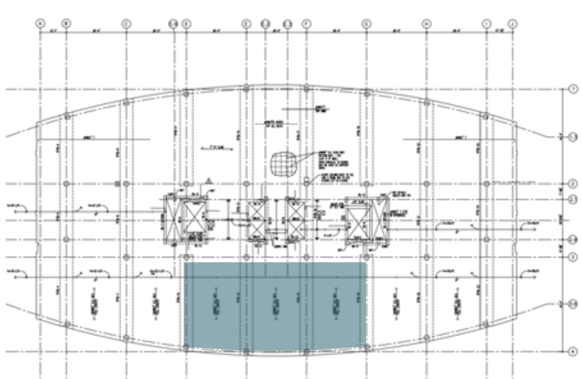

Figure 1: Portion Selected for Analysis

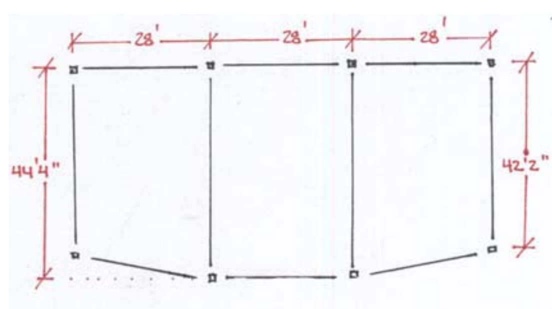

Figure 2: Bay Dimensions

Introduction

Park Potomac Office Building "E" is located prominently off I-270 at Seven Locks and Montrose Roads. It is just one of several planned office buildings that are part of an "urban village" which mixes stunning town homes, Class A office space, and a wide range of amenities including dining and shopping.

Office Building "E" is a central part of the Park Potomac Master Plan. Its central location, at the end of Cadbury Avenue, makes it a focal point for this small community (Figure 3). It also puts it right at the main courtyard that will be a retail gathering point as well.

Figure 3: View from Cadbury Ave.

Material Strength Summary

Concrete:

Footings	3000 psi
Foundation Walls	4000 psi
Columns	Varies
Slab-on-Grade	3500 psi
Reinforced Slabs & Beams	5000 psi
Parking Structure	5000 psi
P.T. Concrete	5000 psi

Structural Steel:

Wide Flanges & Tees	ASTM A992, Fy = 50 ksi
Square/Rectangular Hollow Shapes	ASTM A500, Grade B, Fy = 46 ksi

Masonry:

Compressive Strength	1500 psi

Codes & Design Standards

Original Design:

a. "The International Building Code – 2003", International Code Council

b. "Minimum Design Loads for Buildings and Other Structures" (ASCE7-02), American Society of Civil Engineers

c. "Building Code Requirements for Structural Concrete, ACE 318-02", American Concrete Institute

d. " ACI Manual of Concrete Practice- Parts 1 Through 5", American Concrete Institute

e. "Manual of Standard Practice", Concrete Reinforcing Steel Institute

f. "Post Tensioning Manual", Post Tensioning Institute

g. "Manual of Steel Construction- Allowable Stress Design", Ninth Edition, 1989, American Institute of Steel Construction (Including specifications for structural steel buildings, specifications for structural joints using ASTM A325 of A490 bolts and AISC Code of Standard Practice)

Substituted for thesis analysis:

a. "The International Building Code – 2006", International Code Council

b. "Minimum Design Loads for Buildings and Other Structures" (ASCE7-05), American Society of Civil Engineers

c. "Building Code Requirements for Structural Concrete, ACI 318-08", American Concrete Institute

Gravity Loads

Floor live loads were determined using ASCE 7-05. These loads were then compared to the design loads used in the original design. The design loads were largely the same as those from ASCE 7-05. A few of the loads used exceeded the required loadings from ASCE 7-05. These loads can be found below.

Table 1: Floor Live Loads

Area	Design Load (psf)	ASCE 7-05 Load (psf)
Assembly Areas	100	100
Corridors	100	100
Corridors Above First Floor	80	80
Lobbies	100	100
Marquees & Canopies	75	75
Mechanical Rooms	150	125
Offices	80 + 20 psf Partitions	50 + 20 psf Partitions
Parking Garages	50	40
Plaza, Top Floor Parking	Fire Truck Load or 250 psf	250
Retail- First Floor	100	100
Stairs and Exitways	100	100
Storage (Light)	125	125

The following superimposed dead loads were also considered in the design of the structure.

Table 2: Superimposed Dead Loads

Area	Design Load (psf)
Floors	5
Roof	10

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A flat roof snow load was calculated for this report as well. Beginning with a 30 psf ground snow load for Montgomery County, a flat roof snow load of 21 psf was calculated using the variables shown below from ASCE 7-05. This snow load of 21 psf was identical to the design snow load used by Cagley & Associates. Snow drift loads will occur on the roof level around the screen walls; however, this loading was not examined in this report.

Table 3: Flat Roof Snow Load			
Ground Snow Load	P_g=	30	psf
Snow Exposure Factor	C_e=	1.0	
(Terrain Category B)			
Thermal Factor	C_t=	1.0	
Importance Factor	I=	1.0	
Flat Roof Snow Load	**p_f**	**21**	**psf**

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Existing Structural System

Foundations:

Park Potomac Office Building "E" consists of a seven story office building that sits above two levels of underground parking. The parking structure levels have a footprint of over 103,000 sq. ft. This is much larger than the office structure, which has a footprint of just more than 25,000 sq. ft.

This relationship has a large impact on the design of the foundation as well. The net allowable bearing pressures for the site are 4000 psi for undisturbed soil and 3,000 psi for foundations place on compacted structural fill. Over 150 spread footings are used throughout the project (Figure 4). All footings are 3000 psi concrete, and foundation walls are 4000 psi concrete. Spread footings, mostly ranging from 10' x 10' to 12' x 12', are used beneath the two levels of parking with no office building above. The majority of these footings are between 28" and 34" deep.

Larger mat footings are used in the center of the project, taking load from the two parking levels and also from the office building above. These larger foundations are up to 52' x 64' in size and can be up to 62" deep.

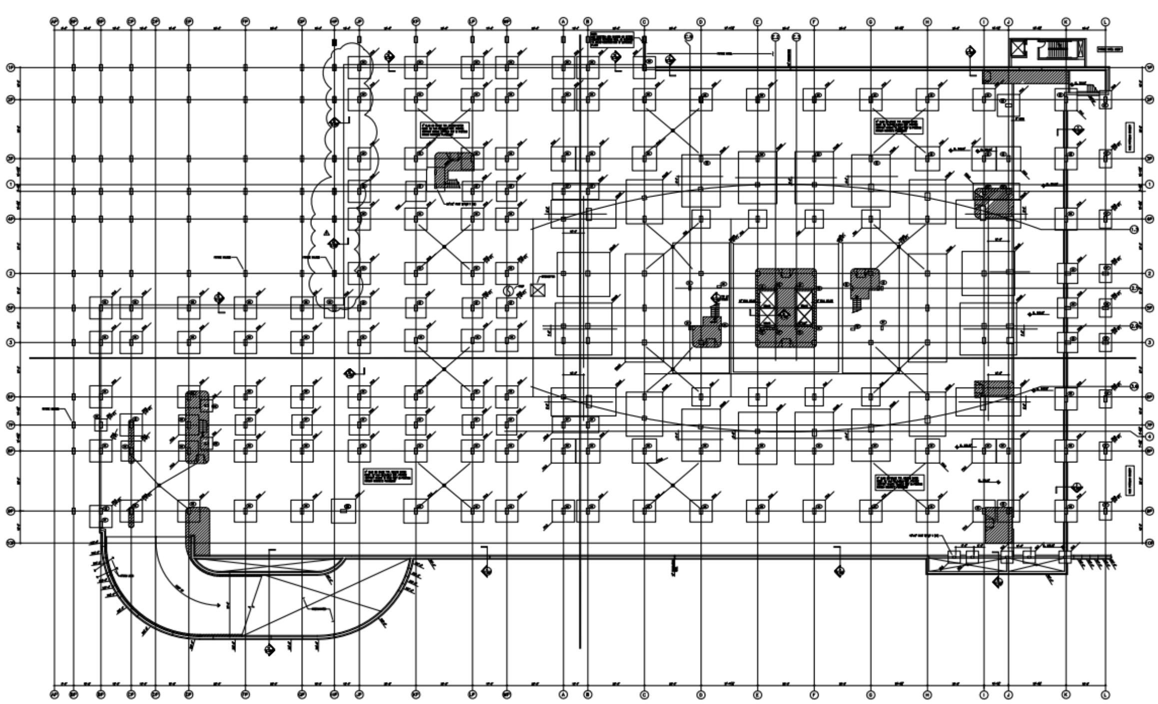

Figure 4: Foundation Plan

Floor System:

The slab on grade at the P2 Parking Level is a 5" thick, 3500 psi concrete slab. It is reinforced with 6x6 – W2.0 x W2.0 welded wire fabric. All other slabs contain 5000psi concrete. Two-way flat slabs are used at the P1 Parking level and the Plaza/First Floor Level as well. The slab is 8" thick at the P1 Level and 12" thick at the Plaza/First Floor Level. These slabs are reinforced as needed to resist negative moment at the columns and positive moments at midspan. Post-tensioning is not used on the parking levels. Tying a post-tensioned slab into foundation walls or other fixed structure does not allow the post-tensioned slab to shrink when stressed. This would result in cracking of the slab if post-tensioning was used below grade. Using this method for the parking garage would also lead to difficulty in stressing the tendons as well. The designers of Office Building "E" use mild reinforcing below grade, and post-tensioning for the slabs above grade.

Above the Plaza Level, Office Building "E" has seven levels of office floors. These floors are 7" thick post-tensioned slabs. The post-tensioning cables induce forces in the slab ranging from 12.5 k/ft up to 35 k/ft. The post-tensioning system uses banded tendons in the 20" beams in the E-W direction, and a one way slab with uniform tendon layout in the N-S direction. This design allows for ease of construction when laying out the tendons. The post-tensioned slab also allows for cantilevers that exist at the North and South ends of the structure. The load from a 12' cantilever on each end is taken by the uniformly spaced tendons that run through the slab.

Post-tensioning is key to achieving several main goals on this project. The first main goal is that it allows for large spans in the floor layout. The design of this project requires that columns be placed around the exterior walls of the building and the interior core as well. This requires the beams and slab to span long distances over the floor. Post-tensioning achieves these span requirements while maintaining a slab thickness of just 7 inches. Deflection over these spans is controlled effectively, while cracking is reduced as well.

Several steel shapes are utilized on the second floor slab to frame out the canopies above the East and West building entrances. This framing consists of TS5x2 shapes that are welded to ¾" plates and hung from the bottom of the slab by L4x4 angles. Steel shapes (W8x10) are also utilized as elevator rail supports throughout all floors.

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Gravity System:

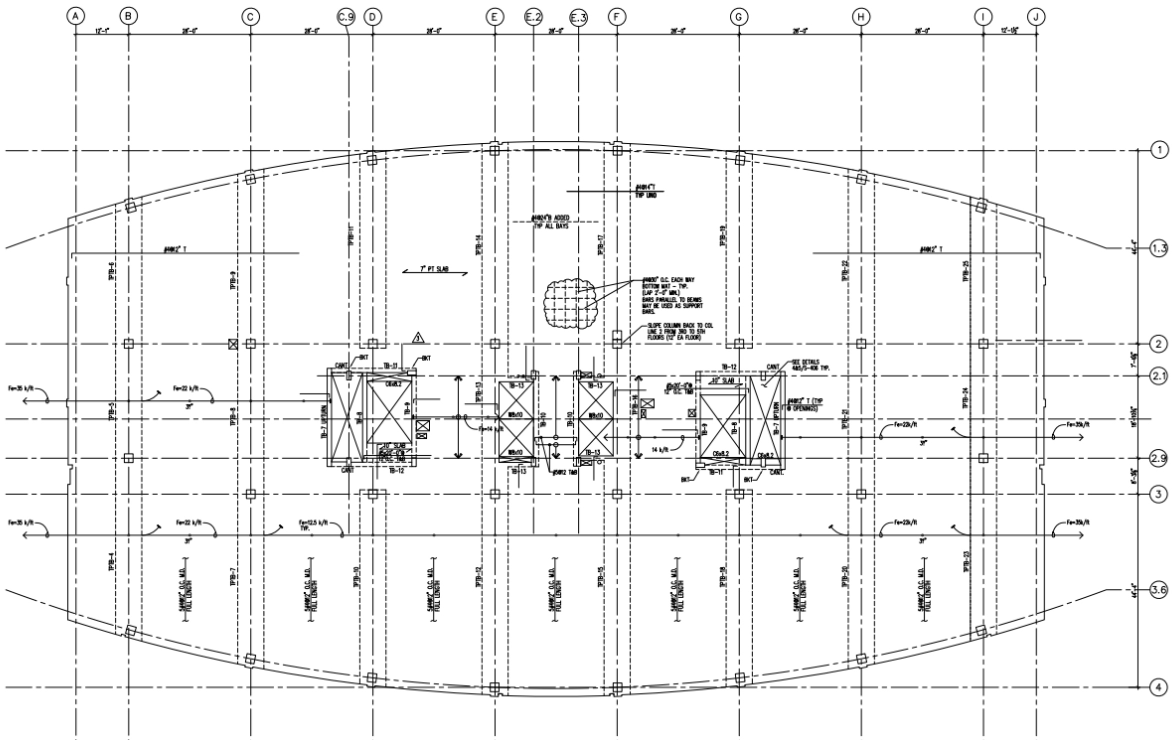

Figure 5: Typical Framing Plan

The majority of the columns in the two levels of parking are 18" x 36" columns reinforced with 10 #9 bars. These columns are typically spaced between 15' and 30' apart. Columns supporting only the two parking levels consist of 4000 psi concrete, while 6000 psi concrete is utilized where load from the office building portion above is carried. Columns in the parking levels utilize drop panels to spread the load and resist punching shear.

In the office portion of the project, a relatively repetitive column layout is achieved. Excluding the central building core, 32 columns are used to transfer the load down through all seven levels. Long span post-tensioned beams are used to transfer load from the floor to the columns. At typically 20" x 72" in size, these shallow, wide beams span in the E-W direction and continue the entire building width. In order to minimize the amount of columns in the tenant spaces and promote flexible space planning, large spans up to nearly 45' exist on each floor.

Columns on the office levels are 24" x 24" at every level and the concrete strength is varied throughout the levels to support an increased load as required. The plaza level through the fourth floor use 5000 psi concrete, while 4000 psi concrete is used above the fourth floor.

Lateral System:

Park Potomac Office Building "E" uses concrete moment frames to resist lateral forces. In the E-W direction, the wide post-tensioned beams on each floor create a series of parallel frames that run up through all seven floors. These frames resist any lateral forces on the building in the parallel direction.

Similarly, forces in the N-S direction are resisted essentially by concrete moment frames as well. The concrete columns and the 7" slab, which is post-tensioned in the N-S direction, combine to create a frame that resists later forces in this direction as well.

In both directions, the lateral forces are taken by the slab or beams and is transferred to the columns and down through the building.

Roof System:

The main roof system consists of a 7" to 8" structural slab. This slab varies in order to create the required roof slopes throughout. The roof contains a Penthouse/Mechanical space, as well as an elevator machine room. The penthouse roof is an 8" two way flat plate system, while the elevator machine room utilizes a 12" thick slab.

TS8x8 posts and TS 6x6 supports are used to frame a 16' tall screen-wall on the roof level to isolate the mechanical spaces from view.

Alternate Floor Systems

Steel Composite Floor System

While maintaining the identical column layout for the bays analyzed, RAM Structural System was used to design the composite floor system shown below. Larger girders were used to span 42' and 45' from column to column, while beams spanned from girder to girder spaced at 5' on center. For this analysis, a 5" total slab thickness with 2" metal deck was assumed. It was also assumed that 3.5" long, ¾" diameter shear studs were used. This slab thickness was larger than that required by the Vulcraft Catalog, however, it was assumed that the final design the floor system would exceed the thickness recommended in the catalog due to the inclusion of lateral forces in the actual slab. Further calculations will occur to determine specific slab information after floor system comparisons are completed. These assumptions resulted in a relatively clean design, with W12x14 members having 1" of camber acting as the typical interior beam. Deflections were all acceptable and can be found in Appendix A. Larger members (W24x55 and W30x90) transfer the load over the most critical spans to the columns.

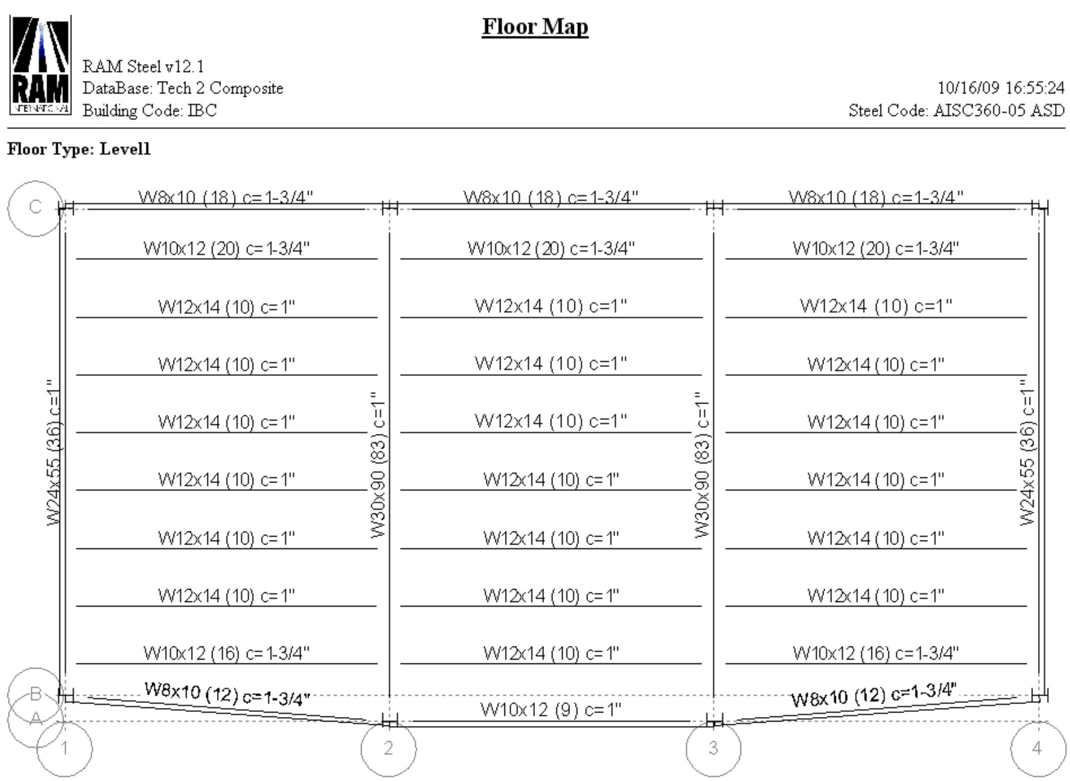

Figure 6: Composite Design

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Steel Noncomposite Floor System

RAM Structural System was again used to design a floor system in order to compare with the other proposed alternatives. All applicable design assumptions remained the same from the previous analysis. This will allow for appropriate comparisons later in this report. It is clear from the design below that not taking advantage of the composite action in the slab has lead to much larger members. The typical internal beam size increased from a W12x14 to a W16x26. This is a significant variation. It is also clear that there is no camber in the design. This is due to the acceptable deflections that occur in the given members. The specific deflection values can be seen in Appendix B; however, these values do not exceed the required limit of L/360. Perhaps these members could be downsized further. These issues, along with a comparison with the composite floor design will be analyzed later in the report. The costs associated with shear stud installation will be compared with differing beam sizes in a later section.

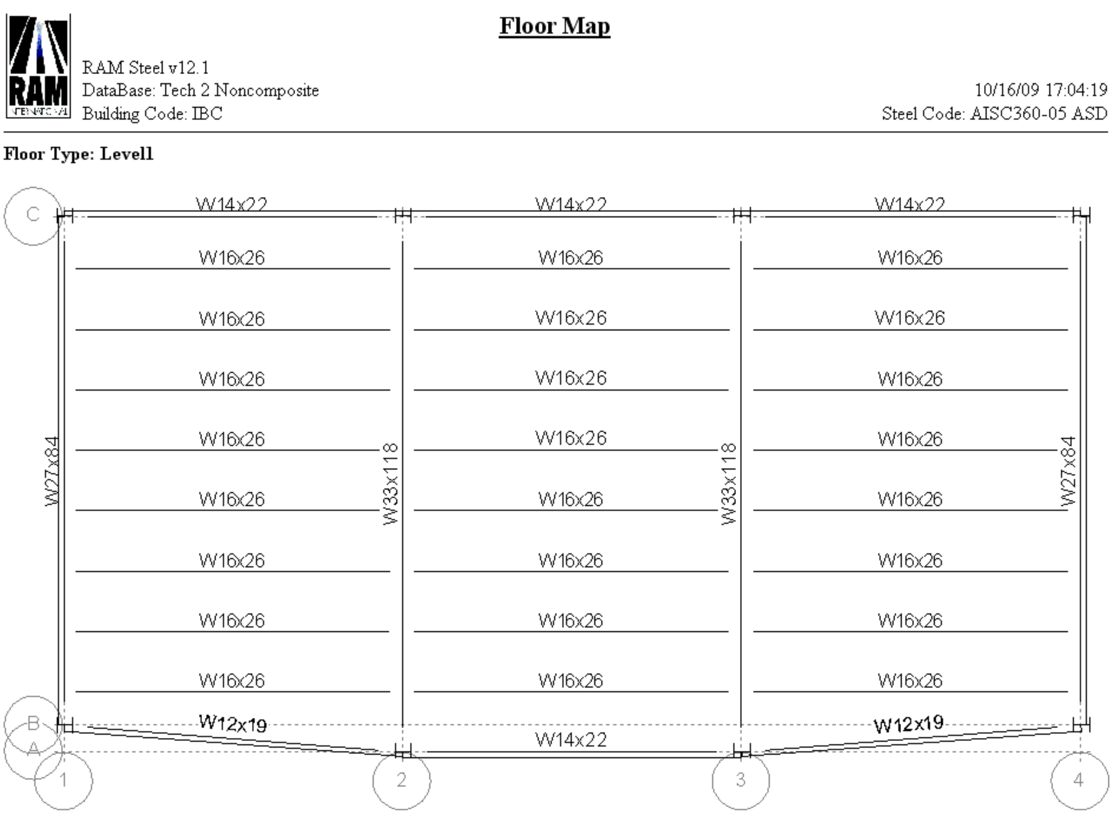

Figure 7: Noncomposite Design

Hollow-core Precast Planks

One concrete system that could be a potential option for this building while maintaining the current column layout is the use of hollow-core concrete planks. The design of these planks and the reduction of self weight through the hollow-cores could make this system an option, unlike other standard concrete systems, which may not be practical for the span distances required to maintain a desirable column layout for this space.

Using the PCI Design Handbook, an 8" thick hollow-core precast plank with a 2" topping slab was selected. This selection was made using a conservative 120 psf required service load determined from the live load, superimposed floor dead load, and an additional 15 psf due to the 2" topping slab. These loads resulted in the selection of a 4' x 8" normal weight concrete (5000 psi) plank with a strand designation of 58-S and a capacity of 126 psf for a 28' span. This designation references the number of strands (5), strand diameter in 1/16ths (8), and straightness of the strands (S). Because these slabs are only available in 4' sections, constructability issues may arise due to curved slab edges and bay sizing. If this floor system was used, these issues would need to be considered during the design of the project.

The system layout is shown below, including steel girder sizes, as well as plank layout. Planks that will need to be cut are shown in red.

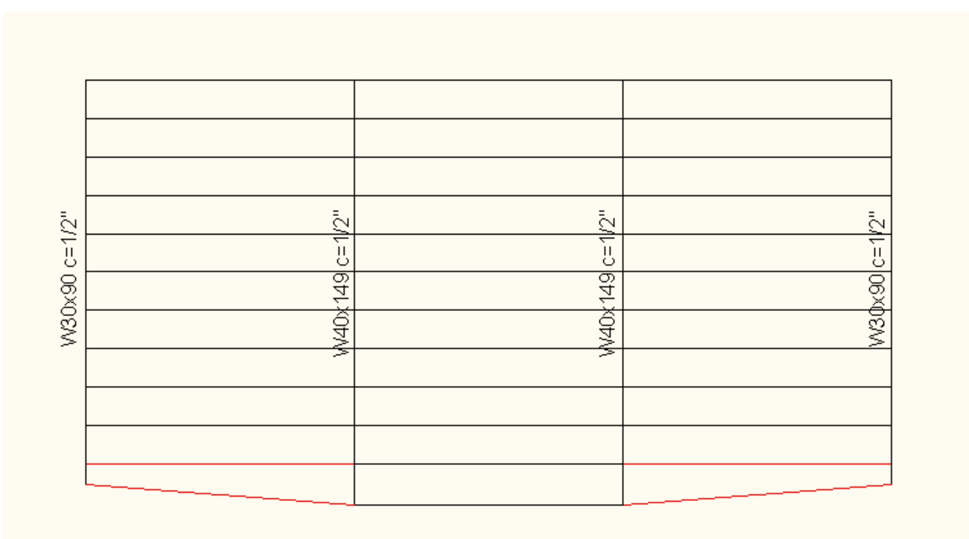

Figure 8: Hollow-core Design

Floor System Comparison

There are obviously a variety of criteria that are used to measure the benefits of a particular floor system over another. In the planning phases of a project, all options must be considered in order to provide the system that is not only most economical, but most beneficial and practical for that project in particular. In this analysis, nine categories were selected to measure the feasibility of a system. The systems analyzed were examined and compared based on cost, weight, floor depth, fireproofing, vibration, construction considerations, and potential foundation and lateral system changes.

Cost

On every project, cost is considered an important, if not the most important factor. Every party involved in a project, including the designer, contractor and owner, want to deliver the best project for the lowest cost. That being said, analyzing the cost of a floor system alone cannot be the determining factor in the decision. The choice of a floor system creates a ripple effect throughout the project, affecting all other trades as well as the rest of the structural system. All the facts must be considered when making a decision. R.S. Means 2009 data was used to determine the costs shown in the table below. Location was not taken into account here and numbers are for comparison only.

Floor Cost (per SF)			
System	**Material Cost**	**Labor Cost**	**Total Cost**
Post-Tensioned Concrete	-	-	15.70
Steel Composite	18.00	5.95	23.95
Steel Noncomposite	24.50	8.45	32.95
Precast Hollow-Core Planks	31.58	9.92	41.50

The data obtained from the cost analysis is somewhat surprising. The post-tensioned cost seems somewhat low, while the others seem fairly high. Steel composite is fairly attractive at this low price, also taking into account the potential savings throughout the rest of the project due to its lower self weight. The hollow-core planks were also more costly than anticipated, and will be even more expensive due to the fact that planks need to be cut to fit. This is a large negative for this system. It is also noteworthy that the noncomposite system is more expensive than the composite, even when looking at labor alone. Due to the large spans needed, noncomposite will not likely be feasible.

Weight

The weight of a particular floor system can have significant repercussions on the overall structural system. A heavier floor system creates a need for larger columns to support this system. This, in turn, creates an even heavier building that needs to be carried by the foundations. Upsizing of the lateral system may be required as well if seismic design is determined as the controlling lateral force.

The use of normal weight concrete was utilized in the design of the composite and noncomposite systems. This was due to the use of normal weight concrete in the initial design. The use of lightweight concrete may be considered in the future if these designs are deemed feasible for this project.

Floor system weights for each system were calculated by hand calculations seen in the appendix. For the steel decking, a 2" 20 gage deck was used, resulting in a slab weight (including deck and concrete) of 51 psf. Shear studs were also added in for the composite system and were assumed to be 10 lb per shear stud for a total of 607 shear studs (6070 lb). A floor weight of 81 psf was used for the hollow-core planks.

Floor Weight				
System	**Framing Weight (lbs)**	**Floor Weight (lbs)**	**Total Weight (lbs)**	**Total Weight (psf)**
Post-Tensioned Concrete	21787	322000	343787	93.4
Steel Composite	23149	193750	216899	58.9
Steel Noncomposite	38550	187680	226230	61.5
Precast Hollow-Core Planks	20806	298080	318886	86.7

The existing post-tensioned system was the heaviest. The precast hollow-core planks were slightly lighter, weighing 86.7 psf. The steel composite and noncomposite systems provided substantial improvements in reducing the overall building weight. These systems were both over 30 psf lighter than the original system. These systems also have the potential to become even lighter with the potential use of lightweight concrete as well. This large reduction in building weight could have a significant impact on the foundation design in this case, potentially resulting in overall savings.

Floor Depth

Floor depth is an important overall consideration in the selection of a practical floor system. The floor depths for the existing system and the three alternates can be seen in the table below. The depths were compared at the middle of each bay, as well as at the girders that support the slab and occur every 28' in the N-S direction.

Floor Depth			
System	**Slab Depth (in)**	**Max Depth at Midspan (in)**	**Max Depth at Girder (in)**
Post-Tensioned Concrete	7"	7"	20"
Steel Composite	5"	17"	35"
Steel Noncomposite	5"	21"	38"
Precast Hollow-Core Planks	8"	8"	48"

At the middle of the bay, it is clear that both of the concrete systems have very shallow depths. The post-tensioned system remains the thinnest, closely followed by the 8" concrete planks. The steel systems are deeper here, due to the steel wide flanges that support the slab.

At the girder, all of the proposed systems are significantly deeper than the original. This comparison is where the post-tensioned design really shows its benefits. Post-tensioning allows for large spans (almost 45' here) while maintaining a very shallow profile. The 45' spans are the building's most critical, which greatly increases the depth at this girder. Because the girders only occur every 28' and only run in one direction, it is possible that the mechanical equipment could be designed to minimize the interruption and maintain a reasonable ceiling height for the alternate systems.

Using the alternate systems would result in reducing the floor to ceiling heights or increasing the overall height of the building. Reducing the floor to ceiling heights would result in less desirable rental spaces, which would result in lower prices for the space. Increasing the building height is also a feasible option. No code limitations would prevent this increase. Negative impacts of this increase would include increasing the lateral loads on the building, increasing the overall volume for heating/cooling, increased costs for more building materials (envelope, etc.) and other possible architectural considerations.

Fireproofing

One major advantage to the existing post-tensioned concrete system is the fact that concrete itself is a fireproof material. This means that no further fireproofing was needed to achieve the two hour fire rating required by the IBC. This is an advantage with regards to cost as well as constructability. The three alternate systems proposed would all require some degree of additional fireproofing material. Due to the fact that the layouts are the same, the steel composite and noncomposite systems would require the identical amount of fireproofing material. The hollow-core plank system would require significantly less fireproofing, as only the girders would need fireproofed. This additional fireproofing required to some degree by all of the alternate systems would likely be a spray-on fireproofing used on all the steel members. Another option would be to use a fire rated drop ceiling. This is probably a less desirable option due to the fact that fire dampers would be required at any openings, which would increase the MEP costs.

Vibration

Floor vibration is an important consideration when considering which type of system to select. A variety of factors can create vibrations, including building occupants walking or the operation of mechanical equipment. Long spans with extremely light floor systems can create vibration issues for building occupants. It becomes a larger issue if a building contains sensitive laboratory or medical equipment. Although no specific calculations were performed for this report, it is estimated that vibration will not be a factor for the four systems in question. The steel composite and noncomposite systems have roughly the same mass and stiffness. It is likely that vibration will not be an issue for these systems; however, this may become a consideration if lightweight concrete is used. This may have to be considered further if that is the case. The hollow-core plank and post-tensioned systems will be even less susceptible to vibration issues due to their larger weights.

Construction Considerations

Post-tensioning creates several issues that can arise during construction. The tendon layout before casting the slab is essential to success. Incorrect placement of the tendons can induce localized forces in the slab that can lift the slab or can blow out portions of the slab if the drape becomes slightly curved in the wrong direction during placement.

Post-tensioning also creates shrinkage over time which can slightly impact the slab edge locations of the slab. This must be taken into account for the fitting of the building envelope.

It is not likely that any significant construction issues would arise with regards to the alternate systems being proposed. The steel composite and noncomposite systems are both common construction systems, which makes construction fairly routine. For this project, ample site area provided space for material storage and the cranes required for construction. Casting the slab is also made easier by these alternate systems, as metal decking is used rather than having to form all the slab pours. One issue that does have to be considered is the installation of shear studs in the composite system, which can prove somewhat labor intensive. Lead times would have to be considered for the steel members needed, but should remain reasonable as long as common shapes are selected.

The hollow-core precast planks would also not present any detrimental construction issues for this project. Abundant storage space would accommodate the storage and placement of the planks. A potentially serious issue for this system would be the fact that the 4' wide planks would need to be cut at the slab edges. Lead time would also need to be considered for these items; however, it is estimated that the lead times would be reasonable for these materials as well.

Foundation Changes

As seen in Figure 2, many foundations are used in the project, as the parking levels encompass a very large square footage. The existing system uses smaller spread footings to take most of the load from the parking levels alone. The footings directly under the building are much larger mat foundations and are required to take much larger loads from the building above. The proposed alternate floor systems would all result in significantly lower overall building loads, and in turn, would allow for downsizing of the large mat foundations. This could result in significant cost benefits for the redesigned project.

Lateral System Changes

The current post-tensioned system uses concrete moment frames to resist lateral forces in each direction. A series of frames are formed in the short direction by beams and columns, while the post-tensioned slab and columns form a frame that resists lateral forces in the building's long direction.

The use of this type of lateral system is not practical for the alternative systems proposed in this report. A more likely lateral force resisting system might be the use of braced frames or shear walls in the buildings core. This will need to be researched and designed further in later reports; however, it is assumed that a redesign of the lateral system will need to take place with all of the proposed alternative systems.

Conclusion

Overall Comparison				
	Post-Tensioned Concrete	**Steel Composite**	**Steel Noncomposite**	**Precast Hollow-core Planks**
Cost ($ per SF)	15.70	23.95	32.95	41.50
Weight (psf)	93.4	58.9	62	87
Floor Depth (max at girder)	20"	35"	38"	48"
Fireproofing	Not Required	Needed	Needed	Needed
Vibration	Not Critical	Not Critical	Not Critical	Not Critical
Construction Considerations	Moderate	Low	Low	Moderate
Foundation System Changes	-	Yes	Yes	Probable
Lateral System Changes	-	Yes	Yes	Yes

Based on the above analysis, it is clear that the steel noncomposite system is not a feasible option for this project. With a higher cost and weight than the composite system, it is unclear why a noncomposite system would be used here. This system can be eliminated as an option.

The precast hollow-core planks do not appear to be a better alternate than the existing system either. The relatively heavy weight of the planks would require large steel members to support them. This greatly increases the floor depth from the original design. Additionally, this system has the largest of the floor system costs. It is unlikely that this cost would be offset by savings elsewhere either. Moreover, the fact that the self weight is not greatly reduced would likely not allow for savings in other areas of the project. All of this, in addition to the fact that many of these planks would need to be cut, does not provide much hope for this alternative.

The steel composite system seems to be the best alternative for this project. It is clear from looking at the above chart that this system seems to be the most beneficial in almost every category. Despite an increase in floor depth from the existing system, and the need for fireproofing, it appears that the composite system could potentially be a better alternative. This is supported by a relatively common construction method. Additionally, the low floor system cost and low self weight support the potential for large project savings through a redesign of the superstructure and of the large mat foundations supporting the building. All of these considerations will be investigated further in future reports.

Appendix A: Composite

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Beam Deflection Summary

RAM Steel v12.1
DataBase: Tech 2 Composite 10/16/09 16:55:24
Building Code: IBC Steel Code: AISC360-05 ASD

STEEL BEAM DEFLECTION SUMMARY:

Floor Type: Level1

Composite / Unshored

Bm #	Beam Size	Initial in	PostLive in	PostTotal in	NetTotal in	Camber in
7	W24X55	1.556	1.108	1.163	1.719	1
10	W8X10	2.195	0.901	0.946	1.391	1-3/4
129	W10X12	2.201	0.894	0.939	1.390	1-3/4
130	W12X14	1.503	0.802	0.842	1.344	1
131	W12X14	1.503	0.802	0.842	1.344	1
132	W12X14	1.503	0.802	0.842	1.344	1
133	W12X14	1.503	0.802	0.842	1.344	1
134	W12X14	1.503	0.802	0.842	1.344	1
135	W12X14	1.503	0.802	0.842	1.344	1
136	W10X12	2.299	0.801	0.841	1.390	1-3/4
1	W8X10	2.344	0.756	0.793	1.388	1-3/4
6	W30X90	1.356	0.967	1.015	1.371	1
9	W10X12	1.516	0.787	0.826	1.343	1
121	W12X14	1.503	0.802	0.842	1.344	1
122	W12X14	1.503	0.802	0.842	1.344	1
123	W12X14	1.503	0.802	0.842	1.344	1
124	W12X14	1.503	0.802	0.842	1.344	1
125	W12X14	1.503	0.802	0.842	1.344	1
126	W12X14	1.503	0.802	0.842	1.344	1
127	W12X14	1.503	0.802	0.842	1.344	1
128	W10X12	2.299	0.801	0.841	1.390	1-3/4
2	W8X10	2.344	0.756	0.793	1.388	1-3/4
5	W30X90	1.356	0.967	1.015	1.371	1
8	W8X10	2.195	0.901	0.946	1.391	1-3/4
113	W10X12	2.201	0.894	0.939	1.390	1-3/4
114	W12X14	1.503	0.802	0.842	1.344	1
115	W12X14	1.503	0.802	0.842	1.344	1
116	W12X14	1.503	0.802	0.842	1.344	1
117	W12X14	1.503	0.802	0.842	1.344	1
118	W12X14	1.503	0.802	0.842	1.344	1
119	W12X14	1.503	0.802	0.842	1.344	1
120	W10X12	2.299	0.801	0.841	1.390	1-3/4
3	W8X10	2.344	0.756	0.793	1.388	1-3/4
4	W24X55	1.556	1.108	1.163	1.719	1

Technical Assignment #2

Gravity Beam Design Takeoff

RAM Steel v12.1
DataBase: Tech 2 Composite 10/16/09 16:55:24
Building Code: IBC Steel Code: AISC360-05 ASD

STEEL BEAM DESIGN TAKEOFF:

Floor Type: Level1
Story Level 1
Steel Grade: 50

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
W8X10	5	140.17	1412
W10X12	6	168.00	2024
W12X14	19	532.00	7531
W24X55	2	84.33	4649
W30X90	2	88.67	7965
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	34		**23580**

Total Number of Studs = **607**

Appendix B: Noncomposite

Technical Assignment #2

Beam Deflection Summary

RAM Steel v12.1
DataBase: Tech 2 Noncomposite 10/16/09 17:04:19
Building Code: IBC Steel Code: AISC360-05 ASD

STEEL BEAM DEFLECTION SUMMARY:

Floor Type: Level1

Noncomposite

Bm #	Beam Size	Dead in	Live in	NetTotal in	Camber in
7	W27X84	0.198	1.238	1.435	
10	W12X19	0.116	0.911	1.026	
129	W16X26	0.077	0.706	0.783	
130	W16X26	0.081	0.792	0.873	
131	W16X26	0.081	0.792	0.873	
132	W16X26	0.081	0.792	0.873	
133	W16X26	0.081	0.792	0.873	
134	W16X26	0.081	0.792	0.873	
135	W16X26	0.081	0.792	0.873	
136	W16X26	0.078	0.739	0.818	
1	W14X22	0.085	0.639	0.724	
6	W33X118	0.204	1.405	1.609	
9	W14X22	0.089	0.719	0.808	
121	W16X26	0.081	0.792	0.873	
122	W16X26	0.081	0.792	0.873	
123	W16X26	0.081	0.792	0.873	
124	W16X26	0.081	0.792	0.873	
125	W16X26	0.081	0.792	0.873	
126	W16X26	0.081	0.792	0.873	
127	W16X26	0.081	0.792	0.873	
128	W16X26	0.078	0.739	0.818	
2	W14X22	0.085	0.639	0.724	
5	W33X118	0.204	1.405	1.609	
8	W12X19	0.116	0.911	1.026	
113	W16X26	0.077	0.706	0.783	
114	W16X26	0.081	0.792	0.873	
115	W16X26	0.081	0.792	0.873	
116	W16X26	0.081	0.792	0.873	
117	W16X26	0.081	0.792	0.873	
118	W16X26	0.081	0.792	0.873	
119	W16X26	0.081	0.792	0.873	
120	W16X26	0.078	0.739	0.818	
3	W14X22	0.085	0.639	0.724	
4	W27X84	0.198	1.238	1.435	

Gravity Beam Design Takeoff

RAM Steel v12.1
DataBase: Tech 2 Noncomposite 10/16/09 17:04:19
Building Code: IBC Steel Code: AISC360-05 ASD

STEEL BEAM DESIGN TAKEOFF:

Floor Type: Level1
Story Level 1
Steel Grade: 50

SIZE	#	LENGTH (ft)	WEIGHT (lbs)
W12X19	2	56.17	1065
W14X22	4	112.00	2473
W16X26	24	672.00	17562
W27X84	2	84.33	7117
W33X118	2	88.67	10469
	-----		-------------
	34		**38686**

Total Number of Studs = **0**

Appendix C: Hollow-Core Planks

Technical Assignment #2

Strand Pattern Designation
76-S

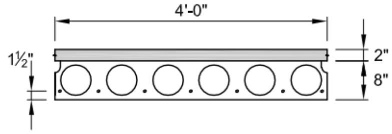
S = straight
Diameter of strand in 16ths
No. of Strand (7)

Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Capacity of sections of other configurations are similar. For precise values, see local hollow-core manufacturer.

Key
458 – Safe superimposed service load, psf
0.1 – Estimated camber at erection, in.
0.2 – Estimated long-time camber, in.

HOLLOW-CORE
4'-0" x 8"
Normal Weight Concrete



f'_c = 5,000 psi
f_{pu} = 270,000 psi

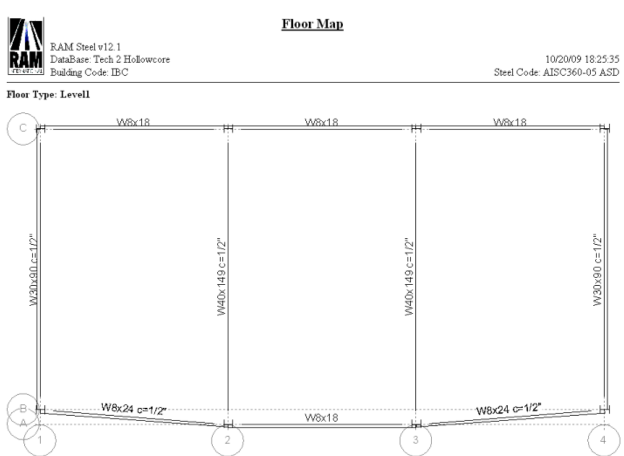
Section Properties

		Untopped		Topped	
A	=	215	in.^{2}	311	in.^{2}
I	=	1,666	in.^{4}	3,071	in.^{4}
y_b	=	4.00	in.	5.29	in.
y_t	=	4.00	in.	4.71	in.
S_b	=	417	in.^{3}	581	in.^{3}
S_t	=	417	in.^{3}	652	in.^{3}
wt	=	224	plf	324	plf
DL	=	56	psf	81	psf
V/S	=	1.92	in.		

4HC8 + 2

Table of safe superimposed service load (psf) and cambers (in.) **2 in. Normal Weight Topping**

Strand Designation Code	Span, ft																											
	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
66-S	489	445	394	340	294	256	224	197	173	153	135	119	105	93	82	68	56	45	36	26								
	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.2	0.1	0.0	–0.0	–0.1	–0.2	–0.3								
	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.4	–0.6	–0.7	–0.9	–1.2	–1.4								
76-S	498	457	420	387	347	304	267	235	208	184	164	146	130	116	103	88	74	62	51	41	31							
	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.3	0.3	0.3	0.3	0.2	0.2	0.1	–0.0	–0.1	–0.2							
	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.5	–0.7	–0.9	–1.2	–1.4							
58-S	492	451	414	384	357	333	310	293	274	245	219	196	177	159	143	126	110	95	82	70	59	49	40	32				
	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.5	0.1	0.3	0.2	0.1	0.0	–0.1				
	0.3	0.3	0.4	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.3	0.2	0.1	–0.1	–0.2	–0.4	–0.6	–0.9	–1.2	–1.5	–1.8				
68-S		463	426	393	366	342	319	299	282	267	251	239	216	195	177	158	140	124	110	97	84	73	62	53	44	36	28	
		0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.2	0.1	–0.1	
		0.4	0.5	0.5	0.6	0.6	0.6	0.6	0.7	0.7	0.7	0.6	0.6	0.6	0.5	0.4	0.3	0.2	0.0	–0.2	–0.4	–0.6	–0.9	–1.2	–1.6	–2.0	–2.4	
78-S		472	435	402	375	348	325	305	288	273	257	245	232	220	207	186	167	149	133	119	106	94	83	73	64	55	46	38
		0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.0	0.9	0.9	0.7	0.6	0.5	0.3
		0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.8	0.9	0.9	0.9	0.9	0.8	0.8	0.7	0.7	0.6	0.4	0.3	0.1	–0.1	–0.3	–0.6	–0.9	–1.3	–1.7	–2.2



Appendix D: Comparison Calculations

Technical Assignment #2

KYLE WAGNER | TECH 2

FLOOR WEIGHTS

AREA = 3680 SF

- POST TENSIONED:

FRAMING WT: $45' + 45' + \frac{2(42')}{2} = 132'$

$132' \times \left(\frac{13''}{12}\right) \times \left(\frac{27''}{12}\right) = 145.25 \text{ ft}^3$

$145.25\ (150 \text{ pcf}) = 21787 \text{ lb}$

FLOOR WT: $3680 \text{ SF} \left(\frac{7''}{12}\right)(150 \text{ pcf}) = 322000 \text{ lb}$

TOTAL WT: 343787 lb → $\boxed{93.4 \text{ psf}}$

- STEEL COMPOSITE

FRAMING WT:
(19) W12x14 → (19)(28')(14 lb/ft) = 7448 lb
(5) W10x12 → (5)(28')(12 lb/ft) = 1680 lb
(5) W8x10 → (5)(28')(10 lb/ft) = 1400 lb
(2) W24x55 → (2)(42.2)(55 lb/ft) = 4642 lb
(2) W30x90 → (2)(44.33)(90 lb/ft) = 7979 lb

23149 lb

FLOOR WT: 3680 SF (51 psf) = 187680 lb

SHEAR STUDS: 607 STUDS (10 lb) = 6070 lb

TOTAL WT: 216899 → $\boxed{58.9 \text{ psf}}$

Technical Assignment #2

KYLE WAGNER TECH 2

FLOOR WEIGHTS

• STEEL NONCOMPOSITE

FRAMING WT: (24) W16x26 → (24)(28')(26 lb/ft) = 17472
(4) W14x22 → (4)(28')(22) = 2464
(2) W12x19 → (2)(28)(19) = 1064
(2) W27x84 → (2)(42.2)(84) = 7090
(2) W33x118 → (2)(44.33)(118) = 10462

38550 lb

FLOOR WT: 3680 (51 psf) = 187680 lb

TOTAL WT: 226230 lb → $\boxed{61.5 \text{ psf}}$

• PRECAST HOLLOW-CORE PLANKS

FRAMING WT: (2) W30x90 → (2)(42.2)(90) = 7596
(2) W40x149 → (2)(44.33)(149) = 13210

20806 lb

FLOOR WT: 3680 (81 psf) = 298080 lb

TOTAL WT: 318886 lb → $\boxed{86.7 \text{ psf}}$

Technical Assignment #2

KYLE WAGNER TECH 2

COST ESTIMATES

- POST TENSIONING

CONCRETE $575/CY

$$\frac{3680 \cdot 7/12 \cdot 1/27\,(575)}{3680} = \$12.42\ /SF$$

PT TENDONS $4.60/LB

35 k/ft REQUIRED OVER 45' → 1575 k

26.5 k/TENDON (RICHARD APPLE PRESENTATON)

$$1575k \cdot \frac{1\ TENDON}{26.5k} = \rightarrow 60\ TENDONS$$

$60 \cdot (3 \cdot 28') = 5040$ ft

FOR 1/2" ϕ TENDON → 0.52 lb/ft

↳ 2620.8 lb

$$\frac{2620.8 \cdot 4.60}{3680} = \$3.28/SF$$

$$12.42 + 3.28 = \boxed{\$15.70}$$

		MAT	INST	TOTAL
• COMPOSITE		18.00	5.95	$\boxed{\$23.95}$
• NONCOMPOSITE		24.50	8.45	$\boxed{\$32.95}$
• HOLLOWCORE				
	CONC PLANK	11.08	4.42	15.50
	STEEL	20.50	5.50	26
		31.58	9.92	$\boxed{\$41.50}$