Crossroads at Westfields
Building II

Chantilly, Va

STEPHEN LUMPP
Structural option

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Technical Report 2

EXECUTIVE SUMMARY

This report is a study of alternate floor systems for Crossroads at Westfields Building II. Including the existing floor system, composite metal deck with steel framing, three other systems were designed, analyzed, and compared to see whether they were viable for further investigation. The comparison consisted of many factors including architecture, effects on the lateral system, constructability, cost, fire rating, and impact on the foundation. The main architectural feature of the building is its open floor plan which is achieved by spans of over forty feet. Due to the large loads of this office building and long spans the following systems were chosen to be analyzed:

1. Composite metal Deck with steel framing (existing)
2. Two-way Flat Slab with Drop Panels
3. Hollow Core planks with steel framing
4. Two-way Post-tensioned slab

Based on the preliminary design and analysis of the 4 systems, the existing composite floor system proved to be the best design for this building, verifying the actual design. The two-way post-tensioned slab and hollow core offered the best alternatives due to the fact that they kept the bay sizes unchanged handling the large loads and long spans. The PT system achieves the least deep floor which allows for the greatest floor to ceiling heights. The Hollow Core system is very similar to the existing composite system but has the most depth of any of the floor systems. The two-way flat slab system required adding columns to split the long spans eliminating it from further consideration. Overall, the hollow core system and post-tensioned system would provide the best alternatives and other criteria such as vibration, deflection and lateral effects will be investigated in future reports.

TABLE OF CONTENTS

OVERALL INTRODUCTION

The Crossroads at Westfields are two identical office buildings mirroring each other on site. Although the project is currently on hold, these two buildings will offer over 300,000 GSF of office space to future tenants. Located in the Westfields Corporate Center in Chantilly, Virginia, the site is located at the crossing of the Stonecroft Blvd. and Lee Rd., hence the name.

Site Plan

Building II, identical to Building I, is a 5- story office building with floor plans that offer spans of over 41 feet. The large open floor plan creates long spans that require the beams to be cambered to pass deflection criteria. The structure consists of composite steel beam framing with ordinary moment connections to resist lateral loading. The roof is supported by joists and steel decking, and the future mechanical units will have composite slab pads similar to each floor.

Typical Floor Plan

EXISTING STRUCTURAL SYSTEMS

FOUNDATION SYSTEMS

The Foundation system consists of reinforced cast-in-place concrete spread footings. According to the Geotechnical report recommendations prepared by ECS, Ltd the allowable soil bearing values vary throughout the site. Foundations bearing on the natural 'weathered rock' soil classification will be designed with an allowable soil bearing of 6000 psf while foundations bearing on engineered fill will be designed for soil bearing of 3000 psf. The concrete strength shall be 3000 psi.

According to recommendations in the Geotechnical Report, the Slab on Grade will bear on the natural soil. The slab is a 4" thick cast-in-place concrete with 6x6–10/10 welded wire mesh (WWM), laid on a 6-mil fiberglass reinforced polyethylene vapor barrier and 4" of washed gravel. Interior SOG will have a compressive strength of 3000 psi, while exterior SOG will have a strength of 4500 psi.

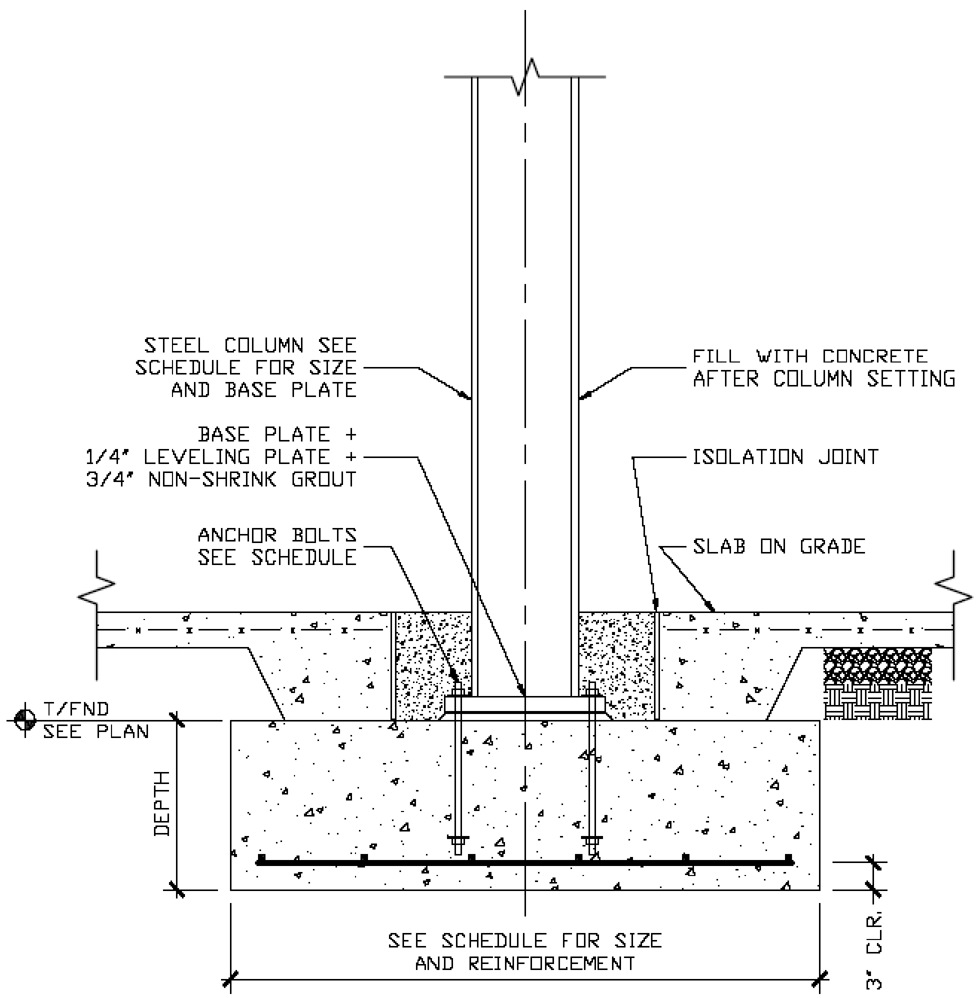


Figure 1- Typical Foundation section

FLOOR SYSTEMS

A typical floor in the Building II consists of 3” 20 gauge composite steel deck with 3-1/4” lightweight concrete slab totaling a total slab thickness of 6-1/4”. The slab shall be reinforced with 6X6-10/10 WWM and have a compressive strength of 3000 psi. The floor is supported by A992 wide flange beams with studs dimensioned at ¾” in diameter and 5 ¼” in length. The beams are spaced at 10’ o/c and span 41’-8” in a typical exterior bay and 30’-0” in a typical interior bay, as you can see in Figure 2 below. Depending on the floor, the beams will be cambered from an 1” to 1½” and will vary in size and weight. Typical interior girders are W24-62 spanning 30’-0”, while typical exterior girders vary in size and also span 30’-0”.

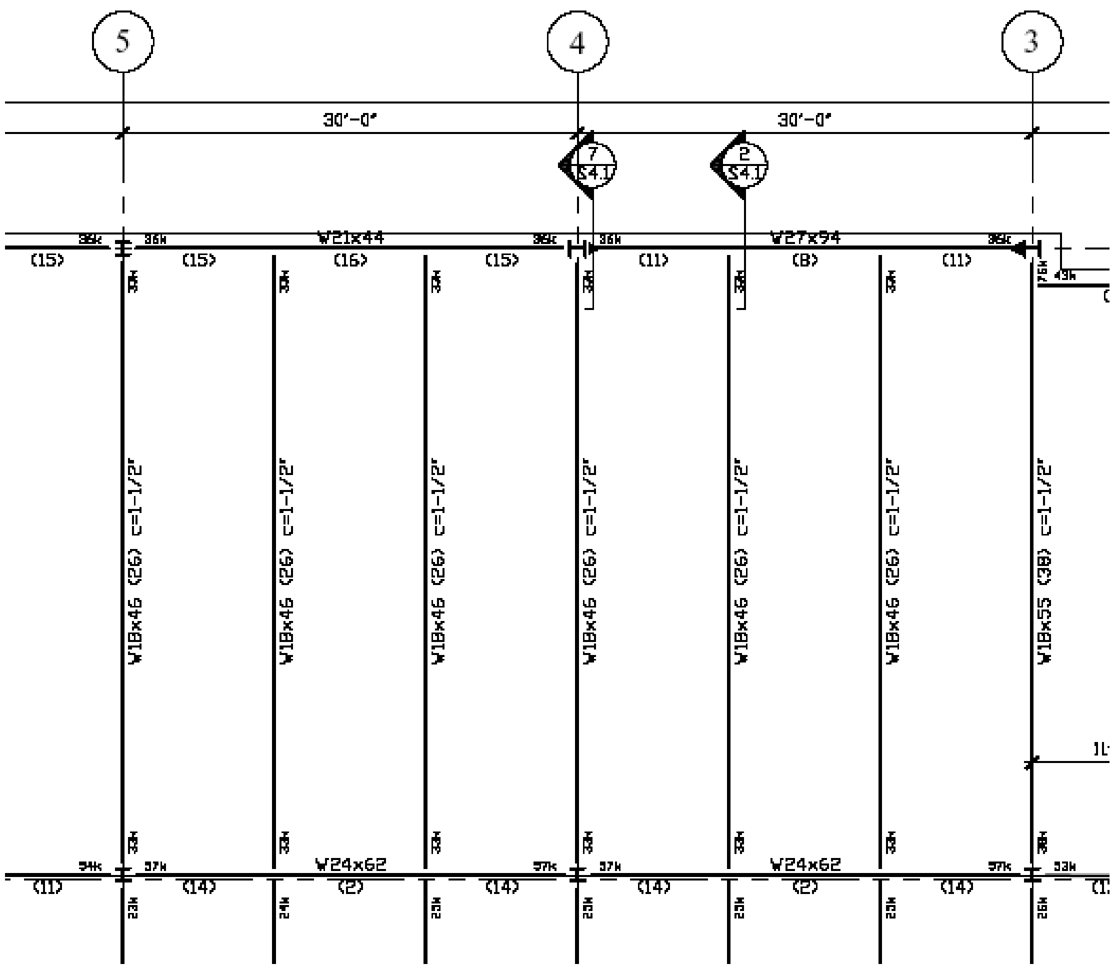


FIGURE 2 – Typical exterior floor bay

ROOF SYSTEM

As seen in Figure 3, the roof system is comprised of 1-1/2" 22 gauge Type B wide rib galvanized roof deck, on K series bar joists and steel girders. Light-gage framing makes up the 4' parapet and the screen wall encompassing the roof. Precast panels frame into each floor including the roof.

Rooftop Mechanical pads for future tenant equipment shall be constructed similar to the typical floor system consisting of 3" 20 gauge composite steel deck with 3-1/4" lightweight concrete slab totaling a total slab thickness of 6-1/4". The slab shall be reinforced with 6X6-10/10 WWM and have a compressive strength of 3000 psi.

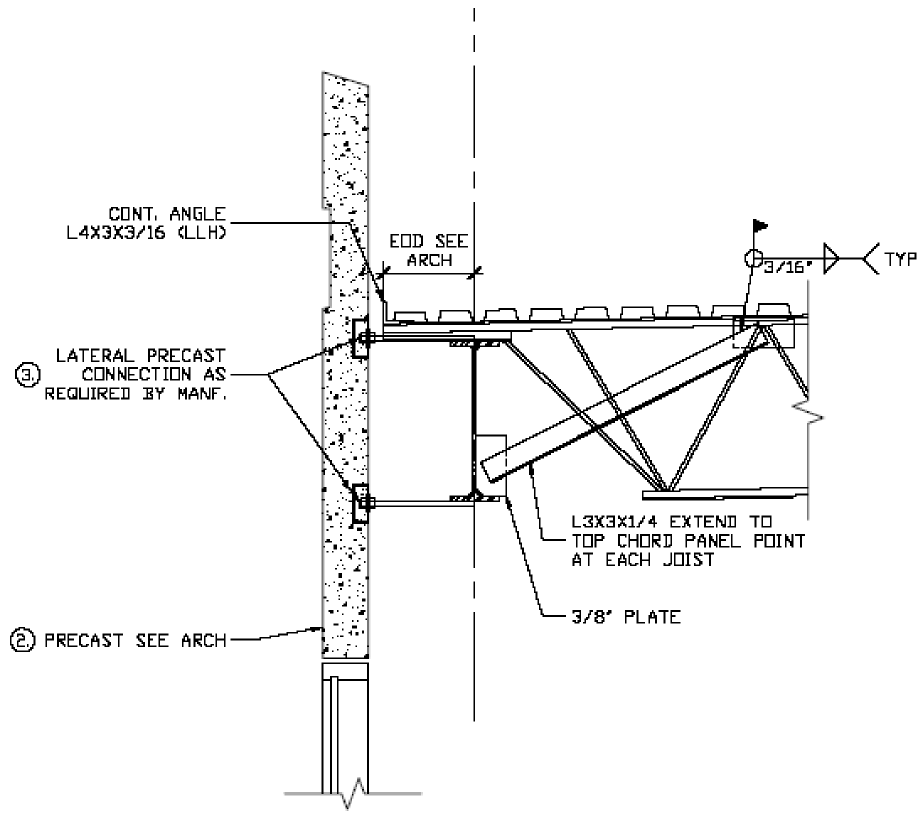


FIGURE 3 – Typical exterior roof section

LATERAL SYSTEM

The lateral resisting system for wind and seismic loads consists of a number of structural steel moment frames running in both directions. Lateral loading is transferred from precast panels (connected at each floor) to each individual floor. Once transferred into the floor system, the load is transferred into composite beams which make up the framing and then into the columns. The columns and beams are connected by a moment connection seen in Figure 4. the columns transfer the rest of the load into the foundation.

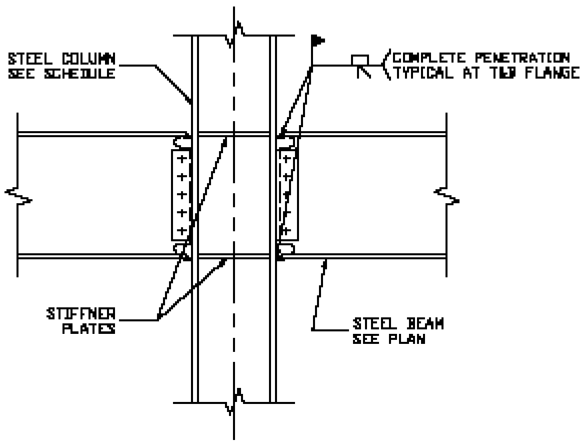


FIGURE 4 – Typical Beam to Column Moment connection

Figure 5 clearly shows the four moment frames positioned in each direction, North-South and East-West, supporting the building laterally. In both directions the moment frames are positioned symmetrically about the center axis. The North-South lateral system is 2 sets of parallel moment frames anchoring each end bay. The East-West lateral system is a set of 2 moment frames on each exterior side of the building. The beam sizes vary.

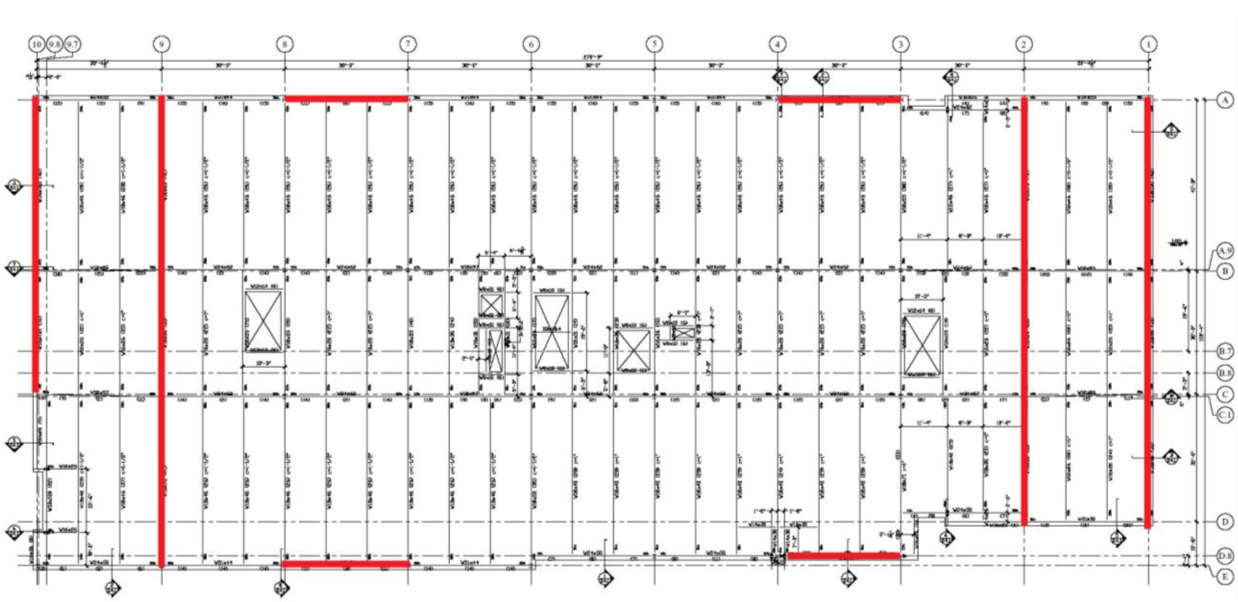


FIGURE 5 – Typical Floor plan with moment frames

COLUMN SYSTEM

Having a very uniform design layout the column system consists of typical exterior bays of 30'-0" x 41'-8" and interior bays of 30'-0" x 30'-0". All of the columns consist of either a gravity resisting member or a combined lateral and gravity resisting member. Each columns is spliced at 4 feet past the third floor, regardless of its resisting system. All columns vary in size depending on location and load resistance capabilities.

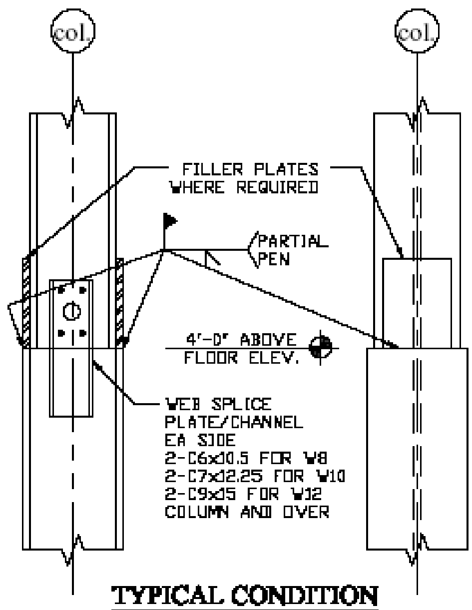


FIGURE 6 – Typical splice connection

APPLICABLE CODE

Design Codes used for Original Design:

- International Building Code, 2003 Edition
- Viginina Uniform State Building Code, 2003
- American Society of Civil Engineers (ASCE)
  - ASCE 7 – 02, Minimum Design Loads for Buildings and Other Structures
- American Institute of Steel Construction (AISC)
  - Steel Construction Manual, Ninth Edition (LRFD)
- American Concrete Institute (ACI)
  - Building Code Commentary 318-02

Code Substitutions/ Additional References used for Thesis Design:

- International Building Code, 2006 Edition
- American Society of Civil Engineers (ASCE)
  - ASCE 7 – 05, Minimum Design Loads for Buildings and Other Structures
- American Institute of Steel Construction (AISC)
  - Steel Construction Manual, Thirteenth Edition (LRFD)
- American Concrete Institute (ACI)
  - Building Code Commentary 318-08

MATERIALS AND PROPERTIES

Steel:

Wide flange shapes	50 ksi (A992)
Square or Rectangular Tubes	46 ksi (A500 Grade B)
Round Pipes	42 ksi (A500 Grade B)
Miscellaneous Steel	36 ksi (A36)
Bolts	36/45 ksi (A325N/A490N)
Steel Studs	60 ksi (A108)
Weld Strength	70 ksi (E70XX)

Concrete:

Foundations, Int. Wall & Int. SOG	f'c = 3000 psi
Ext. SOG and Pads	f'c = 4000 psi
Deck supported slabs (lightweight)	f'c = 3000 psi

Reinforcement:

Stirrups and Ties	40 ksi (A615)
All other	60 ksi (A615)
Welded Wire Fabric:	(A185)

Cold-Formed Steel Framing:

20 Gage	33 ksi (A653)
18 Gage	33 ksi (A653)
16 Gage	50 ksi (A653)

Note: Material strengths are based on American Society for Testing and Materials (ASTM) Standard ratings.

ALTERNATIVE FLOOR SYSTEMS

Composite Metal Deck (Existing System)

The composite metal deck system is viable floor system for the Crossroads at Westfields considering it is the existing floor system of the building. One of the main architectural features of the building is the 41'-8" spans that are in the typical exterior bays, allowing for maximum office space, as seen in figure 8. The composite system is a very effective system for this because of its ability to span long lengths and resist heavy loads, while meeting deflection criteria. The fire code for Building II requires a 1-hour rating for floor systems structural members. The 6 ¼" slab satisfies the 1-hour rating and the steel framing members require fireproofing to meet the criteria. Although larger wide flanges are needed to meet the deflection criteria, the overall weight of the floor system is approximately 66 PSF which is relatively light compared to the other floor systems proposed.

The Construction process of the composite system is very efficient and is one of the main reasons for this is the existing system for Building II. The erection of the steel members is much quicker than having a concrete structure where formwork and shoring is required. The slab can be poured at a much faster rate because the slab does not require many breaks. The cost of the floor system is $27.85 per SF according to RS Means and is very similar in price range to the other alternate floor systems. The one negative to this system is the depth of the floor system which is over 30" deep (24" steel sections and 6 ¼" slab) reducing floor to ceiling heights.

Advantages

- Long spans and capable of resisting large loads
- Meets architectural and structural criteria
- Relatively light weight system allowing for smaller foundations
- Efficient construction process
- Relatively cost effective

Disadvantages

- Larger steel members reducing the floor to ceiling height

The number of advantages clearly outweighs the disadvantages showing why this system is not only viable but was chosen as the existing floor system for the Crossroads at Westfields Building II.

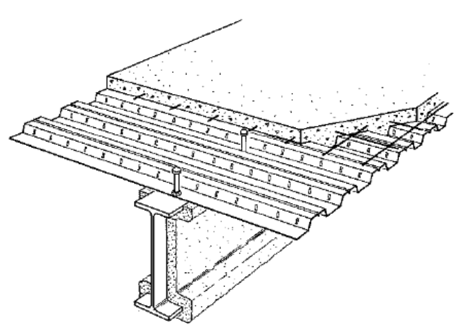


FIGURE 7 – COMPOSITE FLOOR SECTION

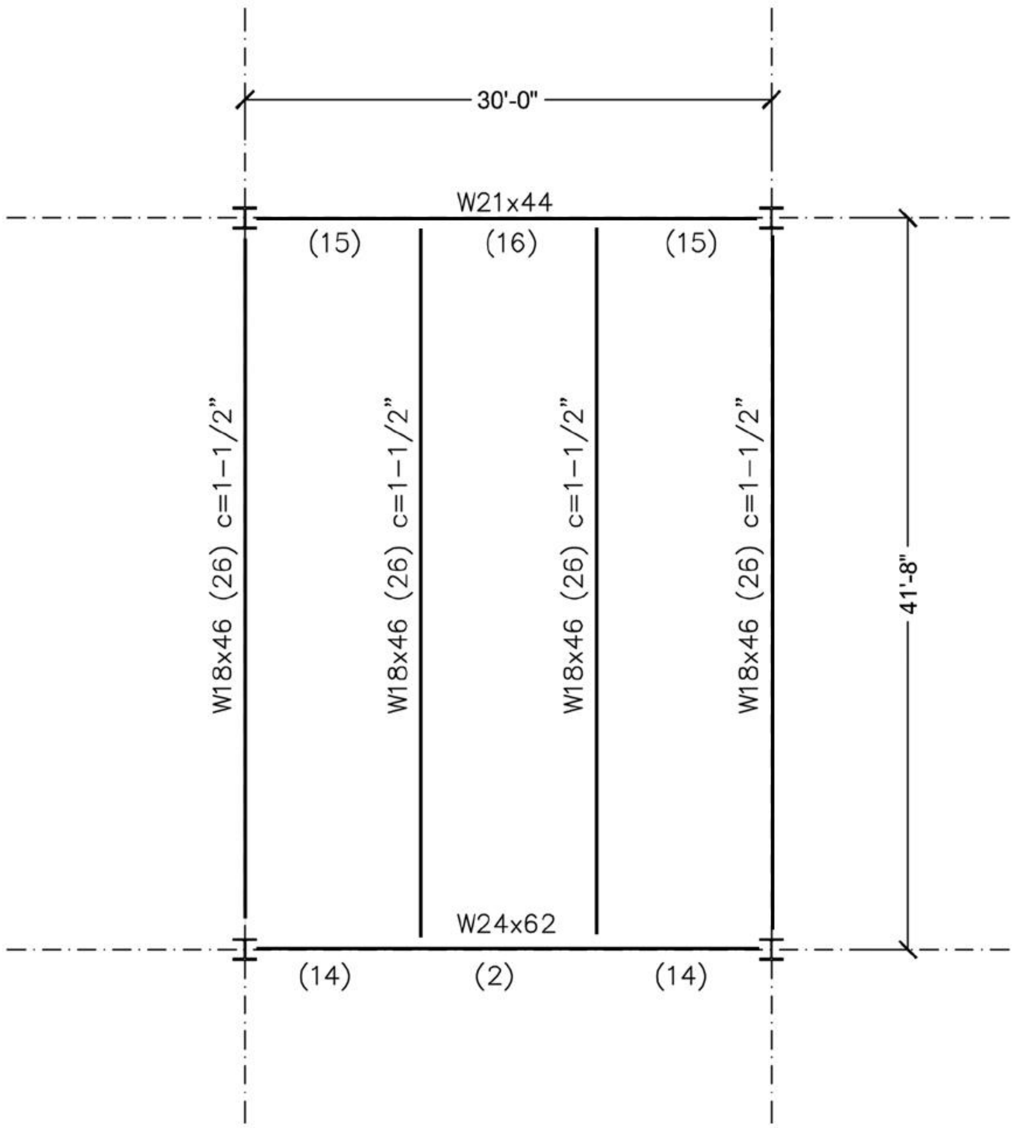


FIGURE 8 – TYPICAL COMPOSITE LAYOUT

2-way Flat Slab with Drop Panels

The initial goal as stated in the executive summary was to maintain the original column grid which is the main architectural feature of the floor plan allowing for an open office floor. To keep the original span of 41'-8" in the exterior bays the slab thickness would have needed to be a minimum of 16" thick which would not have been very economical. Therefore, additional columns were added in the middle of the long spans cutting the span length to 20'-10" and creating two 30'x21' bays in lieu of one 42'x30' bay. Unfortunately, this takes away from the "open" floor plan but is more economical resulting in an 11" thick slab instead of a 16" thick slab. The columns chosen were 24" circular with capital and drop panel. The drop panel is used to reduce the slab thickness and remove punching shear. The reason for the circular columns in lieu of rectangular is strictly for architectural aesthetic and would be analyzed for further feasibility if this system was considered a viable solution.

This system requires a totally different lateral system than the existing moment frame. Shear walls would most likely be used on the exterior faces of the building and in the main core around the elevator and stair shafts. Although the slab thickness is only 11" and the drop panels add an additional 4" the floor depth will increase with the addition of other building systems such as mechanical ducts. The weight of the floor system is approximately 137 PSF which is somewhat heavy and coupled with the added shear walls and columns the foundation would need to be redesigned. Although the construction time for this system is especially long due to shoring and formwork, the cost of the system is relatively cheap according to RS Means totaling only $21.05.

Advantages

- Cost is relatively cheap
- Fireproofing easily meets criteria
- Floor depth is only 15+ inches allowing for greater ceiling heights

Disadvantages

- Architectural floor plan is altered resulting in less "open space"
- Weight of floor system is high
- Construction time is very long due to formwork and shoring

Overall, I would not consider this system viable as an alternate solution mostly because it requires a change to the architectural floor plan. Getting rid of the long spans defeats the purpose to have an "open" floor plan for office use.

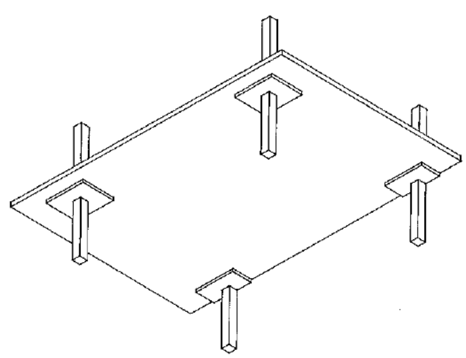


FIGURE 9 – VIEW OF FLAT SLAB WITH DROP PANELS

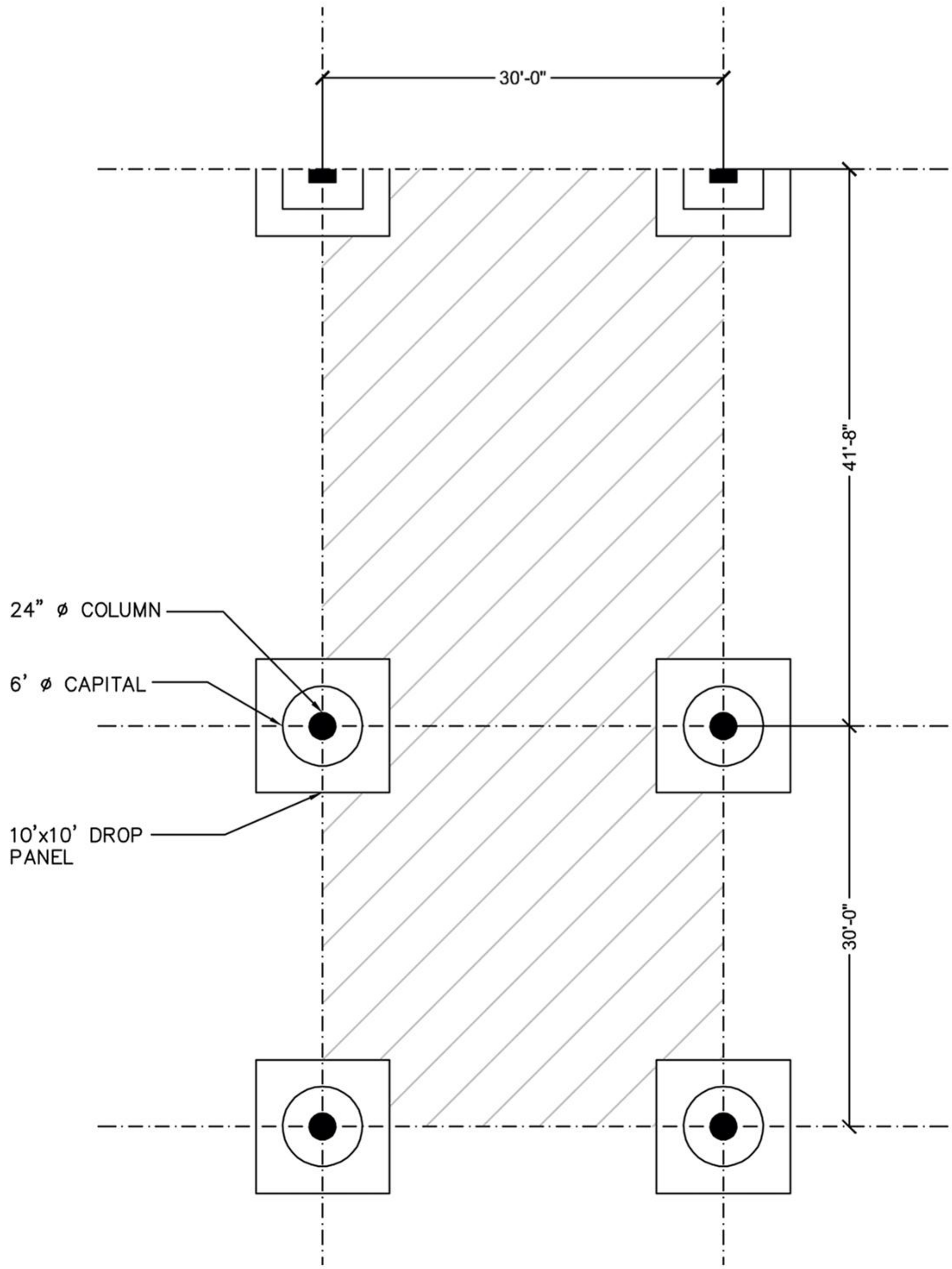


FIGURE 10 – TYPICAL FLAT SLAB LAYOUT

Hollow Core Plank

This system meets the goal to keep the architectural floor plan the unchanged. The column grid was altered and the steel framing plan was slightly altered by the subtraction of one beam running in the long direction of the typical bay. The reason why a beam was able to be removed was because the hollow core plank is able to span further distances than the composite steel deck. One negative is that beam spanning that long direction is 30" deep alone, not to mention the additional 6" for the plank itself. That results in a 36" deep floor system minimizing floor to ceiling heights but also meeting the deflection criteria for the system. The weight of the building results in 59 PSF which is relatively light in weight and will not effect the existing foundation.

This system, similar to the existing system, easily meets the 1-hour fire rating for the slab and requires fireproofing for the steel members. The constructability of the system is very efficient and fast, including the erection of the steel and installation of the precast planks. One negative is that the lead time for this system is slower because of the ordering and shipping of the system. The Cost of the planks is $10.59 while the cost of the steel framing is approximately $17 totaling $27.59 which is very comparable to the existing system.

Advantages

- Architectural plan remained unchanged
- Weight of the building is lighter
- Construction time is very fast and efficient

Disadvantages

- Floor depth is 36" minimizing ceiling to floor heights
- Lead time is long due to ordering and shipping

After analyzing this floor system, many similarities were noticed to the existing floor plan with the exception that the lead time is much longer. The other disadvantage is the floor depth is greater than that of the existing system. Overall, the similarities to the existing make this a possibility as an alternative for the Crossroads and Westfields Building II.

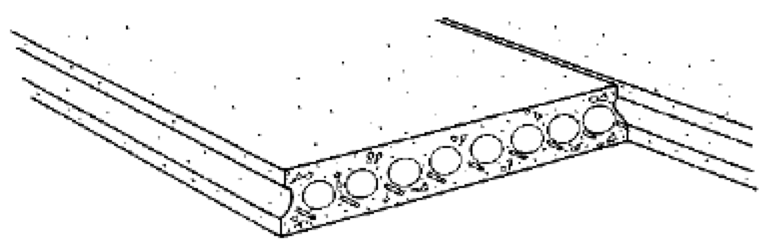


FIGURE 11 – HOLLOW CORE SLAB SECTION

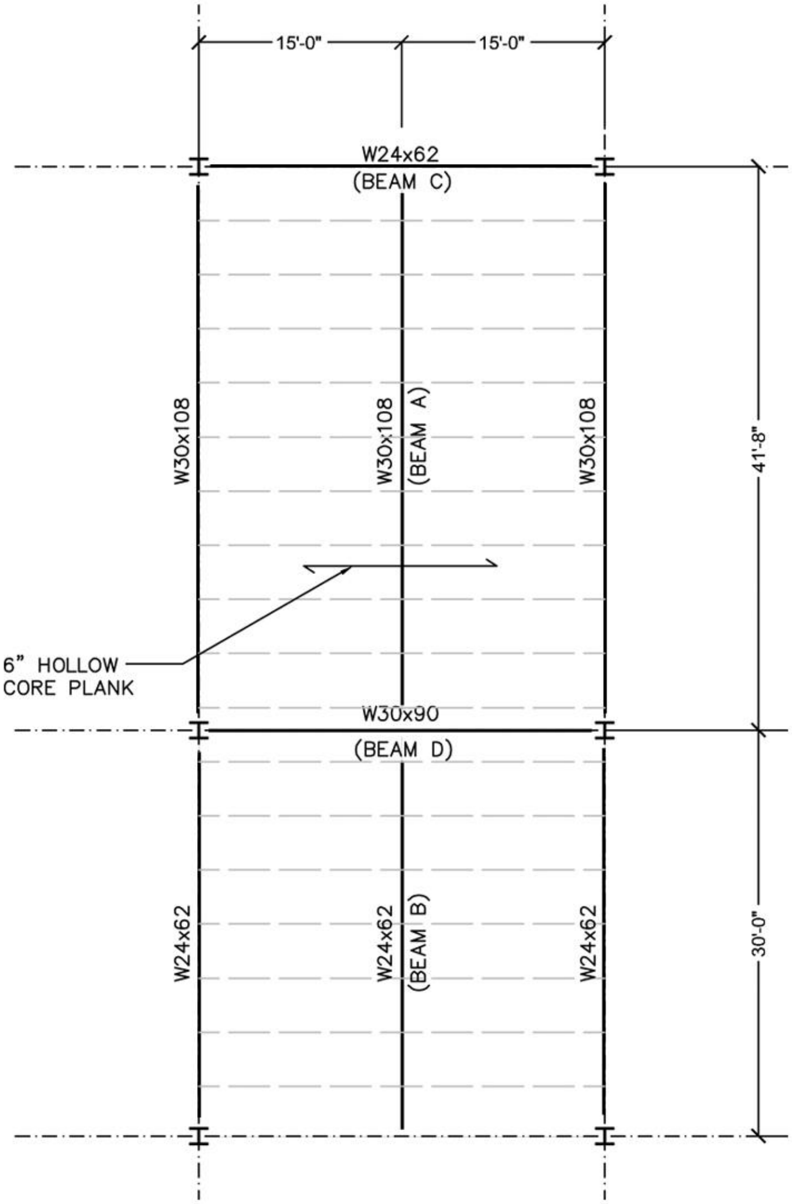


FIGURE 12 – TYPICAL HOLLOW CORE LAYOUT

2-way Post-Tensioned Slab

One of the many advantages of a post-tensioned slab is its ability to achieve long spans economically, and that was the goal for this floor system: minimize the thickness of the slab and keep the long spans. The minimum slab thickness is 12" but unfortunately, due to punching shear a 1" deep, 43" x 43" drop panel was required. Since the main architectural feature of the building is to keep "open" floor plans, this system is probably worth looking into further. The 12" slab easily meets the 1-hour fire rating and the weight of the floor system is 150 PSF which is relatively heavy compared to the composite system with the same number of columns. A new lateral system would have to be designed which may also add weight to the structure.

Post-Tensioned slab are good in deflection and vibration control as well as crack control. The cost is similar to the other systems totaling $32 per SF with additional costs possible. These additional costs come with construction process. The laying of the tendons and placing of formwork require addition time. Due to the high jacking forces during installation specialized supervision and safety precautions are highly recommended.

Advantages

- Reduced structural depth and longer spans
- Can carry much higher loading
- Great in deflection, vibration and crack control

Disadvantages

- A little expensive due to many safety precautions during installation
- Construction takes a longer for several reasons

Overall, this system is viable solution for an alternate floor system of the Crossroads at Westfileds Building II because it can achieve long spans while maintaining a relatively thin floor depth.

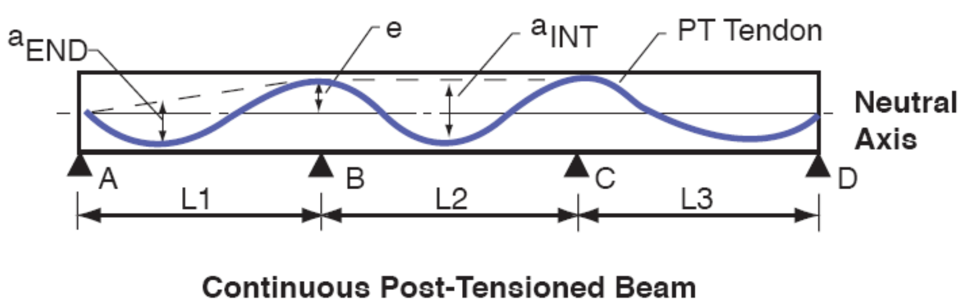


FIGURE 13 – PT TENDON LAYOUT

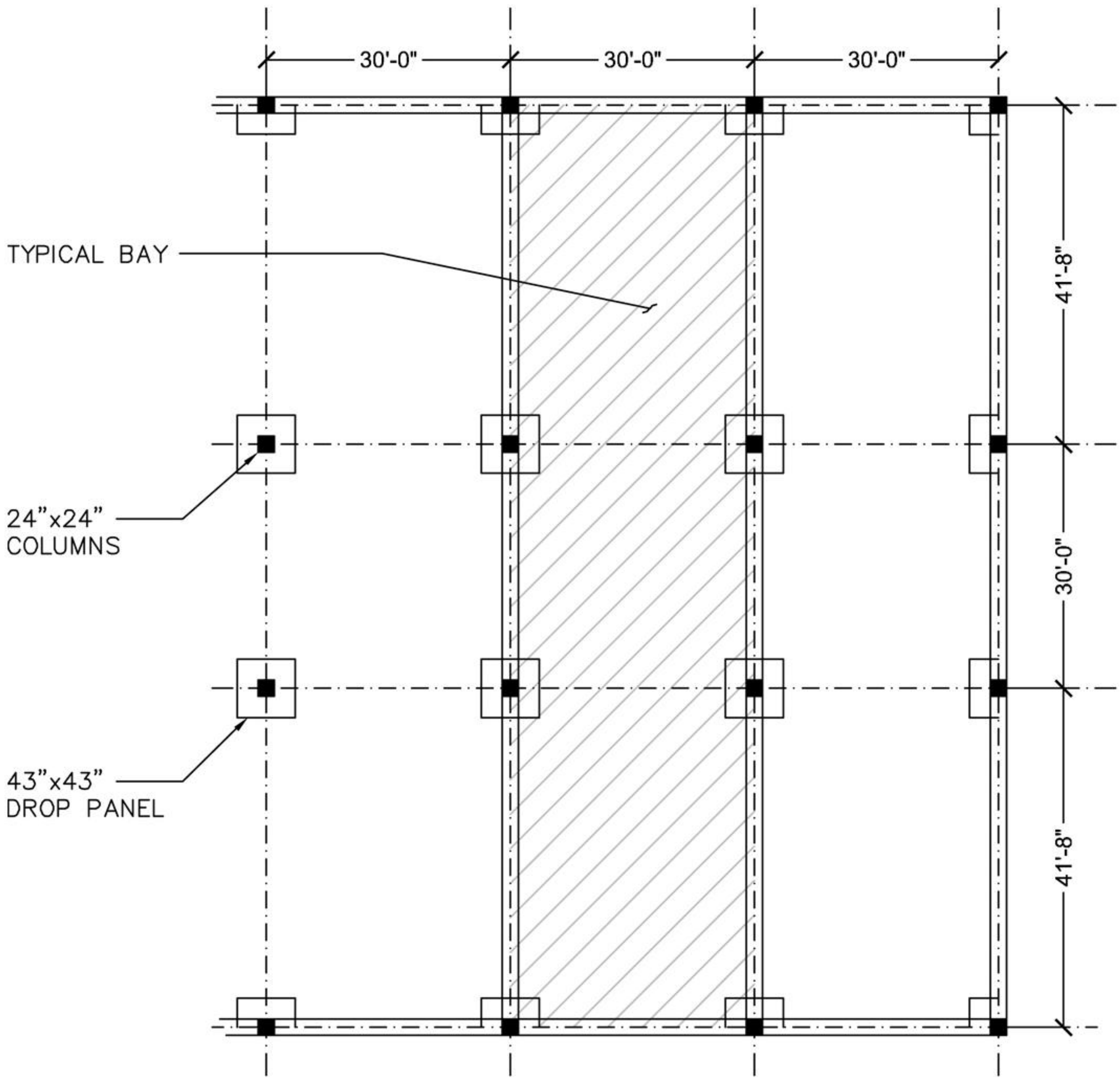


FIGURE 14 – TYPICAL POST-TENSION SLAB LAYOUT

Table 1				
Floor Systems - Comparisons				
Item	**Composite Slab (Existing)**	**2-Way Flat Slab w/ Drop Panels**	**Hollow Core Plank w/ Steel Framing**	**Post-Tensioned Slab**
Architectural Requirements (Bay Dimensions unchanged)	Yes	No	Yes	Yes
Lateral System	No changes	Shear Walls - Both Directions	No changes	Shear Walls - Both Directions
Fire Ratings	Slab - 1-hour Rating Framing - fireproofing	Slab - 1-hour Rating	Slab - 1-hour Rating Framing - fireproofing	Slab - 1-hour Rating
Slab Depth (in.)	6.25"	11" (+ 4" Drop Panels)	6"	12" (+ 1" Drop Panels)
Depth of floor sytem (in)	**30"** (6.25" slab + 24" steel members)	**15"+** (11" slab + 4" drop panels + possible ductwork)	**36"** (6" slab + 30" steel members)	**13"+** (12" slab + 1" drop panel + possible ductwork)
Weight (PSF)	66 PSF	137 PSF	59 PSF	150 PSF
Foundation Impact	None	Re-design necessary	Very Little	Re-design necessary
Construction - Process	Efficient	Inefficient (more time and labor)	Efficient, but requires longer lead time	Inefficient (more time, labor and additional supervision)
Material Cost (SF)	21	11.1	21 (8.5+12.5)	19.4
Installation Cost (SF)	6.85	9.95	6.59 (2.09 +4.5)	11.4 (+1.2 for equip.)
Overall Cost (Per SF)	27.85	21.05	27.59	32
Deflection	Meets Criteria	Further investigation	Meets Criteria	Further investigation
Vibration	Further investigation	Further investigation	Further investigation	Further investigation
Viable System for Future consideration	Yes	No	Yes	Yes

COMPARISONS AND CONCLUSION

The goal of this report was investigate viable alternatives for the floor system of the Crossroads at Westfields Building II. Including the existing floor system, a composite design, three schematic designs of additional systems were conducted to test the feasibility of each. Each system was compared to the others through a variety of criteria which can be found in Table 1 located on page 20 in the report. After weighing all of the comparisons it was concluded that the two-way post-tension slab and hollow core floor systems were the best alternatives to the existing system, although the existing composite slab proved to be the best choice for the design. The two-way flat slab will no longer be considered in future reports because it required the addition of extra columns splitting the exterior bays in half.

The three viable choices can all span long lengths and resist heavier loads. The hollow core and the existing composite floor systems are very similar when compared, both are very easy to construct, both require additional fire proofing of their steel members, both are relatively the same cost per square foot, and both have little impact on the foundation and lateral system in place now. The one negative of the two systems is that they both require very deep floors overall, reducing the floor to ceiling height. The PT system on the other hand, maximizes the floor to ceiling height having the least deep floor system. It requires no additional fire proofing and is probably meets vibration criteria easily because its only concrete (further investigation will be conducted on vibration). Some flaws to the PT system are cost and constructability. It costs more than the other two systems per square foot and takes much longer to construct.

After this preliminary design it is concluded that three systems, composite, post-tension, and hollow core will be further investigated. Other criteria will be considered such as deflection, vibration and the effects the lateral system will have on the building and foundation.

APPENDIX

Appendix A – *Composite Metal Deck on steel framing*

Gravity Beam Design

RAM Steel v12.1
DataBase: D07024 Westfields II(new) — 10/21/08 23:18:13
Building Code: IBC — Steel Code: AISC LRFD

Floor Type: TYP — **Beam Number = 53**

SPAN INFORMATION (ft): I-End (90.00,114.16) J-End (120.00,114.16)

Beam Size (Optimum) = W21X44 — Fy = 50.0 ksi
Total Beam Length (ft) = 30.00

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Concrete thickness (in)	3.25	3.25
Unit weight concrete (pcf)	115.00	115.00
f'c (ksi)	3.00	3.00
Decking Orientation	parallel	parallel
Decking type	USD 3" Lok-Floor	USD 3" Lok-Floor

beff (in) =	57.00	Y bar(in) =	18.18
Mnf (kip-ft) =	740.74	Mn (kip-ft) =	715.65
C (kips) =	389.02	PNA (in) =	20.30
Ieff (in4) =	2237.31	Itr (in4) =	2379.47
Stud length (in) =	5.00	Stud diam (in) =	0.75

Stud Capacity (kips) Qn = 17.7
\# of studs: Full = 78 Partial = 44 Actual = 44
Number of Stud Rows = 1 Percent of Full Composite Action = 56.92

POINT LOADS (kips):

Dist	DL	CDL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	CLL
10.000	15.62	13.55	20.66	20.0	0.00	0.00	0.0	0.00	Snow	4.13
20.000	15.62	13.55	20.66	20.0	0.00	0.00	0.0	0.00	Snow	4.13

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	CLL
1	0.000	0.071	0.061	0.100	20.0%	Red	0.020
	30.000	0.071	0.061	0.100			0.020
2	0.000	0.044	0.044	0.000	---	NonR	0.000
	30.000	0.044	0.044	0.000			0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 49.18 kips 0.90Vn = 195.62 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.2DL+1.6LL	246.5	15.0	10.0	1.00	0.90	275.21
	Init DL	1.4DL	206.3	15.0	---	---		
	Max +	1.2DL+1.6LL	481.8	15.0	---	---	0.85	608.30
Controlling		1.2DL+1.6LL	246.5	15.0	10.0	1.00	0.90	275.21

REACTIONS (kips):

	Left	Right
Initial reaction	19.56	19.56
DL reaction	17.35	17.35
Max +LL reaction	17.73	17.73
Max +total reaction (factored)	49.18	49.18

Gravity Beam Design

RAM Steel v12.1
DataBase: D07024 Westfields II(new)
Building Code: IBC

Page 2/2
10/21/08 23:18:13
Steel Code: AISC LRFD

DEFLECTIONS:

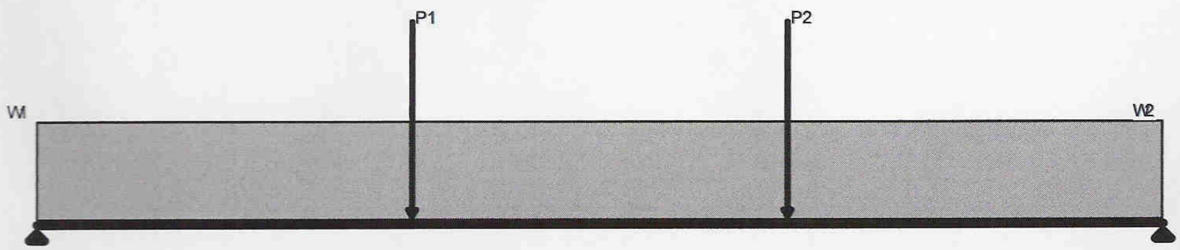
Initial load (in)	at	15.00 ft	=	-0.996	L/D	=	361
Live load (in)	at	15.00 ft	=	-0.444	L/D	=	810
Post Comp load (in)	at	15.00 ft	=	-0.500	L/D	=	720
Net Total load (in)	at	15.00 ft	=	-1.496	L/D	=	241

Load Diagram

RAM Steel v12.1
DataBase: D07024 Westfields II(new)
Building Code: IBC

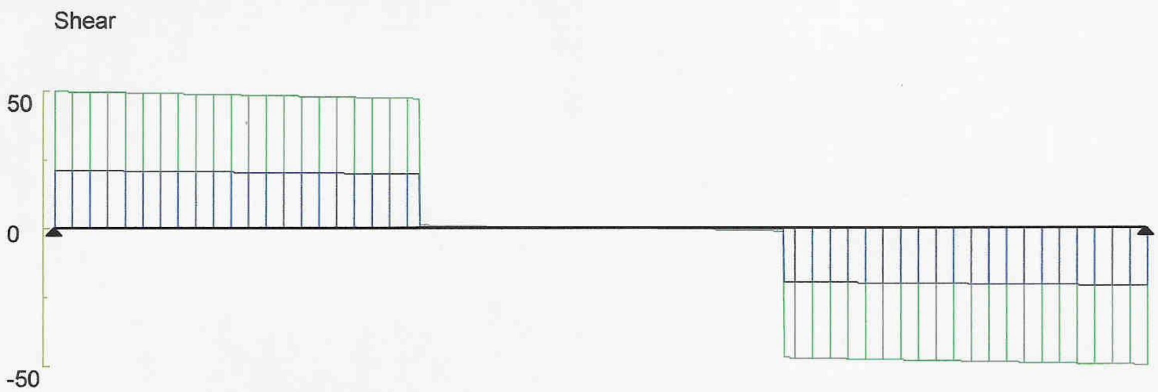
10/21/08 23:18:13

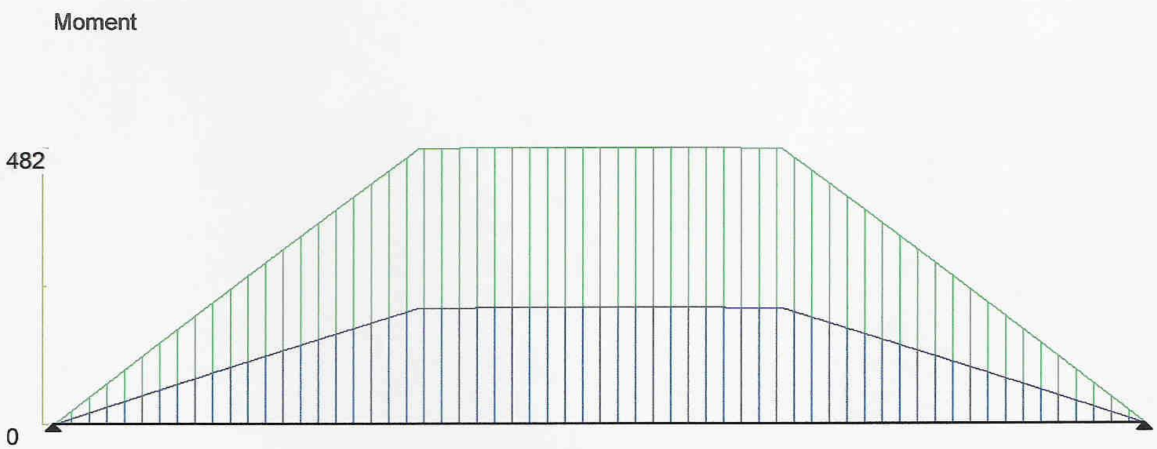
Floor Type: TYP **Beam Number = 53**
Span information (ft): I-End (90.00,114.16) J-End (120.00,114.16)

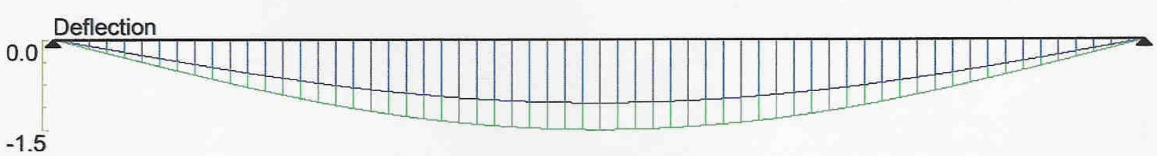


Load	Dist	DL	LL+	LL-	Max Tot
	ft	kips	kips	kips	kips
P1	10.000	15.618	16.529	0.000	32.147
P2	20.000	15.618	16.529	0.000	32.147
	ft	k/ft	k/ft	k/ft	k/ft
W1	0.000	0.115	0.080	0.000	0.195
W2	30.000	0.115	0.080	0.000	0.195

Ram Steel v11.0 Shear, Moment, and Deflection Diagrams
DataBase: D07024 Westfields II(new) 10/21/08 23:18:13
Building Code: IBC
Floor Type: TYP Beam Number = 53
Span information (ft): I-End (90.00,114.16) J-End (120.00,114.16)







Max DL Shear = 20.82 kips
Max Shear = 49.18 kips

Max Pos Moment = 481.84 kip-ft at 15.000 ft

Gravity Beam Design

RAM Steel v12.1
DataBase: D07024 Westfields II(new) — 10/21/08 23:18:13
Building Code: IBC — Steel Code: AISC LRFD

Floor Type: TYP **Beam Number = 52**

SPAN INFORMATION (ft): I-End (90.00,72.83) J-End (120.00,72.83)

Beam Size (Optimum) = W24X62 Fy = 50.0 ksi
Total Beam Length (ft) = 30.00

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Concrete thickness (in)	3.25	3.25
Unit weight concrete (pcf)	115.00	115.00
f'c (ksi)	3.00	3.00
Decking Orientation	parallel	parallel
Decking type	USD 3" Lok-Floor	USD 3" Lok-Floor

beff (in)	=	90.00	Y bar(in)	=	20.95
Mnf (kip-ft)	=	1184.54	Mn (kip-ft)	=	940.08
C (kips)	=	247.56	PNA (in)	=	17.61
Ieff (in4)	=	3134.17	Itr (in4)	=	4299.78
Stud length (in)	=	5.00	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 17.7

\# of studs per stud segment: Full = 43,1,42
Partial = 14,2,14
Actual = 14,2,14

Number of Stud Rows = 1 Percent of Full Composite Action = 33.19

POINT LOADS (kips):

Dist	DL	CDL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	CLL
10.000	15.62	13.55	20.66	20.0	0.00	0.00	0.0	0.00	Snow	4.13
10.000	11.35	9.81	15.42	20.0	0.00	0.00	0.0	0.00	Snow	3.08
20.000	15.62	13.55	20.66	20.0	0.00	0.00	0.0	0.00	Snow	4.13
20.000	11.05	9.56	14.91	20.0	0.00	0.00	0.0	0.00	Snow	2.98

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	CLL
1	0.000	0.062	0.062	0.000	---	NonR	0.000
	30.000	0.062	0.062	0.000			0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 79.32 kips 0.90Vn = 275.16 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.2DL+1.6LL	402.0	13.0	10.0	1.00	0.90	463.87
	Init DL	1.4DL	335.1	13.6	---	---		
	Max +	1.2DL+1.6LL	789.5	10.5	---	---	0.85	799.07
Controlling		1.2DL+1.6LL	789.5	10.5	---	---	0.85	799.07

REACTIONS (kips):

	Left	Right
Initial reaction	31.39	31.27
DL reaction	27.80	27.69

Gravity Beam Design

RAM Steel v12.1
DataBase: D07024 Westfields II(new)
Building Code: IBC

Page 2/2
10/21/08 23:18:13
Steel Code: AISC LRFD

	Left	Right
Max +LL reaction	28.73	28.59
Max +total reaction (factored)	79.32	78.98

DEFLECTIONS:

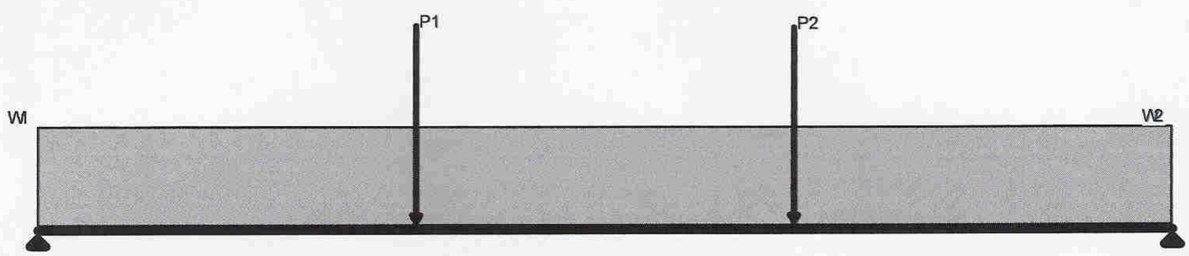
Initial load (in)	at	15.00 ft	=	-0.881	L/D	=	409
Live load (in)	at	15.00 ft	=	-0.522	L/D	=	689
Post Comp load (in)	at	15.00 ft	=	-0.587	L/D	=	613
Net Total load (in)	at	15.00 ft	=	-1.468	L/D	=	245

Load Diagram

RAM Steel v12.1
DataBase: D07024 Westfields II(new)
Building Code: IBC

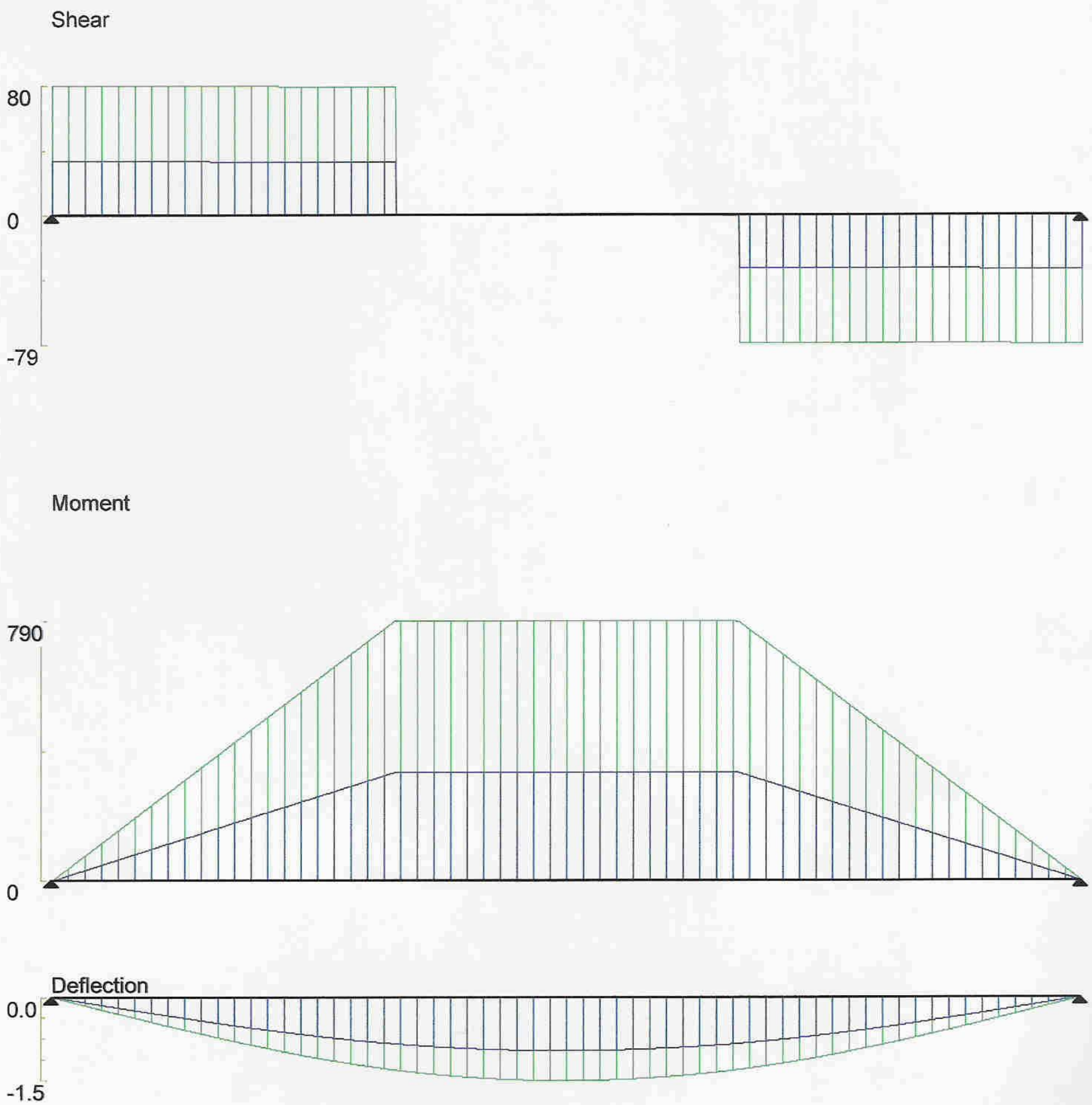
10/21/08 23:18:13

Floor Type: TYP **Beam Number = 52**
Span information (ft): I-End (90.00,72.83) J-End (120.00,72.83)



Load	Dist	DL	LL+	LL-	Max Tot
	ft	kips	kips	kips	kips
P1	10.000	26.967	28.862	0.000	55.829
P2	20.000	26.665	28.457	0.000	55.122
	ft	k/ft	k/ft	k/ft	k/ft
W1	0.000	0.062	0.000	0.000	0.062
W2	30.000	0.062	0.000	0.000	0.062

Ram Steel v11.0 Shear, Moment, and Deflection Diagrams
DataBase: D07024 Westfields II(new) 10/21/08 23:18:13
Building Code: IBC
Floor Type: TYP Beam Number = 52
Span information (ft): I-End (90.00,72.83) J-End (120.00,72.83)



Max DL Shear = 33.35 kips
Max Shear = 79.32 kips

Max Pos Moment = 789.47 kip-ft at 10.469 ft

Gravity Beam Design

RAM Steel v12.1
DataBase: D07024 Westfields II(new) 10/21/08 23:18:13
Building Code: IBC Steel Code: AISC LRFD

Floor Type: TYP **Beam Number = 56**

SPAN INFORMATION (ft): I-End (100.00,72.83) J-End (100.00,114.16)

Maximum Depth Limitation specified = 19.50 in
Beam Size (Optimum) = W18X46 Fy = 50.0 ksi
Total Beam Length (ft) = 41.32

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Concrete thickness (in)	3.25	3.25
Unit weight concrete (pcf)	115.00	115.00
f'c (ksi)	3.00	3.00
Decking Orientation	perpendicular	perpendicular
Decking type	USD 3" Lok-Floor	USD 3" Lok-Floor

beff (in)	=	120.00	Y bar(in)	=	18.48
Mnf (kip-ft)	=	798.58	Mn (kip-ft)	=	558.77
C (kips)	=	172.41	PNA (in)	=	13.84
Ieff (in4)	=	1605.05	Itr (in4)	=	2479.06
Stud length (in)	=	5.00	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 13.3
\# of studs: Max = 82 Partial = 26 Actual = 26
Number of Stud Rows = 1 Percent of Full Composite Action = 25.54

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	CLL
1	0.000	0.710	0.610	1.000	20.0%	Red	0.200
	41.322	0.710	0.610	1.000			0.200
2	0.000	0.046	0.046	0.000	---	NonR	0.000
	41.322	0.046	0.046	0.000			0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 45.19 kips 0.90Vn = 175.93 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu	@	Lb	Cb	Phi	Phi*Mn
			kip-ft	ft	ft			kip-ft
Center	PreCmp+	1.2DL+1.6LL	236.3	20.7	0.0	1.00	0.90	340.12
	Init DL	1.4DL	196.0	20.7	---	---		
	Max +	1.2DL+1.6LL	466.8	20.7	---	---	0.85	474.96
Controlling		1.2DL+1.6LL	466.8	20.7	---	---	0.85	474.96

REACTIONS (kips):

	Left	Right
Initial reaction	17.68	17.68
DL reaction	15.62	15.62
Max +LL reaction	16.53	16.53
Max +total reaction (factored)	45.19	45.19

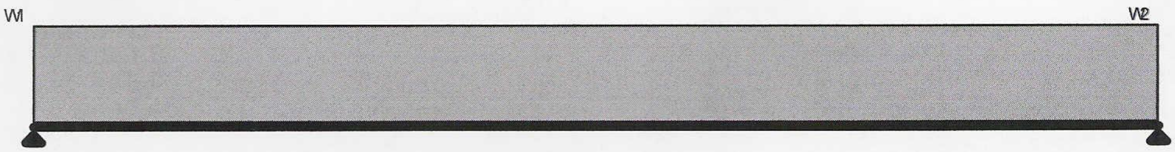
DEFLECTIONS: (Camber = 1-1/2)

Initial load (in)	at	20.66 ft	=	-2.084	L/D	=	238
Live load (in)	at	20.66 ft	=	-1.128	L/D	=	440
Post Comp load (in)	at	20.66 ft	=	-1.269	L/D	=	391
Net Total load (in)	at	20.66 ft	=	-1.853	L/D	=	268

Load Diagram

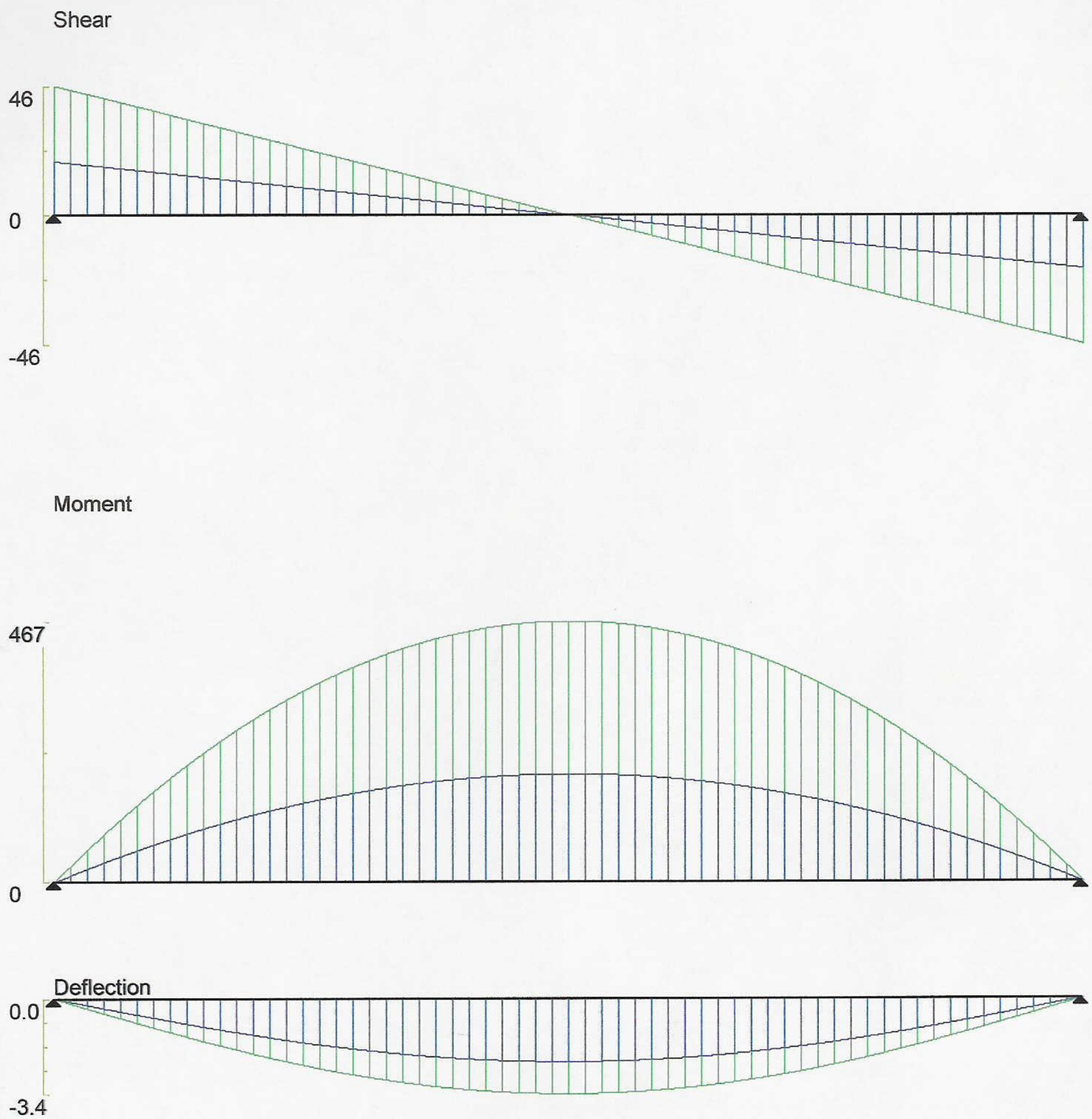
RAM Steel v12.1
DataBase: D07024 Westfields II(new) 10/21/08 23:18:13
Building Code: IBC

Floor Type: TYP **Beam Number = 56**
Span information (ft): I-End (100.00,72.83) J-End (100.00,114.16)



Load	Dist	DL	LL+	LL-	Max Tot
	ft	k/ft	k/ft	k/ft	k/ft
W1	0.000	0.756	0.800	0.000	1.556
W2	41.322	0.756	0.800	0.000	1.556

Ram Steel v11.0 Shear, Moment, and Deflection Diagrams
DataBase: D07024 Westfields II(new) 10/21/08 23:18:13
Building Code: IBC
Floor Type: TYP Beam Number = 56
Span information (ft): I-End (100.00,72.83) J-End (100.00,114.16)



Max DL Shear = 18.74 kips
Max Shear = 45.19 kips

Max Pos Moment = 466.83 kip-ft at 20.662 ft

Appendix B – *Two-way flat slab with drop panels*

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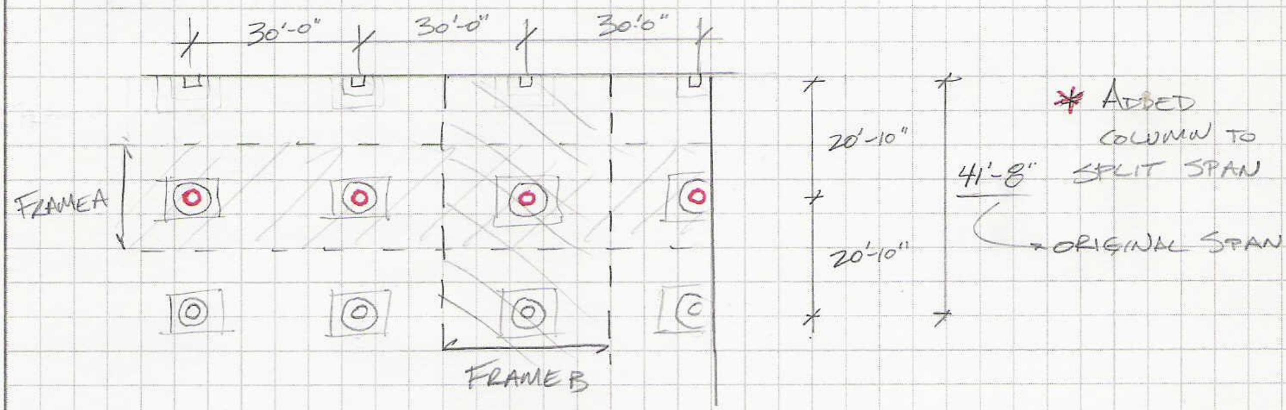
TWO-WAY SLAB ANALYSIS



· DIRECT DESIGN METHOD CHECK:

1. 3 CONTINUOUS SPANS IN EACH DIRECTION ✓

2. $\frac{l_2}{l_1} = \frac{30}{20.83} = 1.44 < 2$ ✓

3. $l_2 - l_1 < \frac{1}{3} l_2 \Rightarrow 30 - 20.83 < \frac{1}{3}(30)$
   $9.17 < 10$ ✓

→ REFERENCE: ACI 318-08

· MINIMUM THICKNESS OF SLAB (TABLE 9.5c)

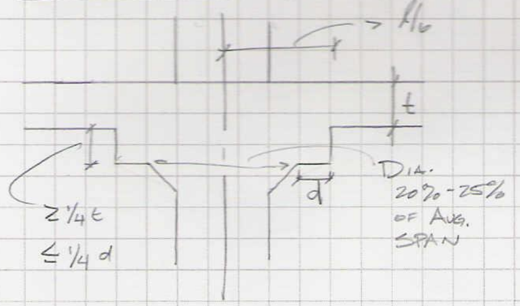
$f'_c = 3000$ psi
$f_y = 60000$ psi

· EXT. PANEL W/o EDGE BEAM $l_n/33 = \frac{30(12)}{33} = 10.91$ ←

· INT. PANEL $l_n/36 = \frac{30(12)}{36} = 10$

SLAB THICKNESS = 11" (MINIMUM)

· DROP PANEL (13.25)



* ASSUME 24" ∅ COLUMNS

· $\frac{30(12)}{6} = 60" \rightarrow 10'$ DROP PANEL

· AVG SPAN = 25.4 → 20% = 5 ; → 25% = 6.3

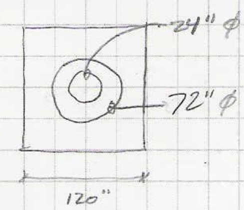
COLUMN CAPITAL = 6' DIAMETER

· $d = 60" - 36" = 24"$

· DROP PANEL DEPTH

$\frac{1}{4}(11") \leq D \leq \frac{1}{4}(24")$

$D = 4"$



· LOADS :

· LIVE LOADS = 100 PSF

· REDUCED LIVE LOAD = $100\left(.25 + \frac{15}{\sqrt{30(20.83)(1)}}\right)$ — 2 WAY SLAB

$= 85$ PSF

· DEAD LOAD = 25 PSF (SUPER IMPOSED)

$= (11/12)(150 \text{ PCF}) = 137.5$ PSF (SELFWEIGHT)

$w_u = 1.2(137.5 + 25) + 1.6(85) = 331 \text{ PSF} = \underline{.331 \text{ KSF}}$

MOMENTS : $\frac{1}{8} w \ell_2 \ell_1^2 \left(1 - \frac{2c}{3\ell_1}\right)^2 = M_0$

→ COL. CAP ∅

FRAME A : $\frac{1}{8}(.331)(20.83)(30-2)^2\left(1 - \frac{2(6')}{3(30-2)}\right)^2 = \underline{496}\ ^{'k}$

FRAME B : $\frac{1}{8}(.331)(30)(18.83)^2\left(1 - \frac{2(6')}{3(18.83)}\right)^2 = \underline{273}\ ^{'k}$

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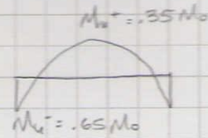
Distribution of Moment – Frame A

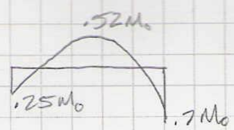
Interior Span



$M^- = 496(.65) = -322\ ^{\prime K}$

$M^+ = 496(.35) = 174\ ^{\prime K}$

End Span w/o Edge Beam



$M_{ext}^- = 496(.25) = -124\ ^{\prime K}$

$M^+ = 496(.52) = 258\ ^{\prime K}$

$M_{INT}^- = 496(.70) = -347\ ^{\prime K}$

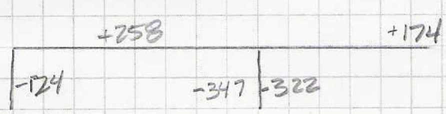
Frame B

INT
$M^- = 273(.65) = -178\ ^{\prime K}$
$M^+ = 273(.35) = 96\ ^{\prime K}$

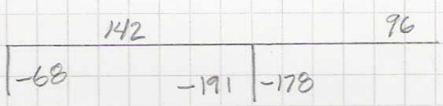
END
$M_{ext}^- = 273(.25) = -68\ ^{\prime K}$
$M^+ = 273(.52) = 142\ ^{\prime K}$
$M_{INT}^- = 273(.70) = -191\ ^{\prime K}$

* No beams, therefore no torsional constant or stiffness ratios.

Frame Summary



Frame A



Frame B

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MOMENT DISTRIBUTION TO COLUMN/MIDDLE STRIP

FRAME A

COLUMN STRIP = 10.42' MIDDLE STRIP = 2 @ 5.21'

TOTAL MOMENT	−124	+258	−347	−322	+174
% TO COL STRIP	100%	60%	75%	75%	60%
MOMENT IN C.S	−124	+155	−260	−242	+104
MOMENT IN M.S	0	+103	−87	−80	+70

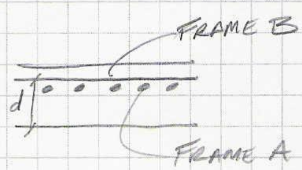
FRAME B

(10'-5")
COLUMN STRIP = 10.42' MIDDLE STRIP = 2 @ 9.79'

TOTAL MOMENT	−68	+142	−191	−178	+96
% TO COL. STRIP	100%	60%	75%	75%	60%
MOMENT IN C.S.	−68	+85	−143	−134	+58
MOMENT IN M.S.	0	+57	−48	−44	+38

REINFORCEMENT: (ASSUME #6 BARS, $d_B > d_A$

DEPTHS, d



B
$$d = 15 - .75 - \frac{.75\text{ in}}{2} = 13.88''$$
$$d = 11 - .75 - \frac{.75}{2} = 9.88''$$

A
$$13.88'' - .75'' = 13.13''$$
$$9.88'' - .75'' = 9.13''$$

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FRAME A – DESIGN REINF FOR COLUMN STRIP & MIDDLE STRIP

		COLUMN STRIP					MIDDLE STRIP				
ITEM	DESCRIPTION	M^-_{EXT}	M^+	M^-_{INT}	M^-	M^+	M_{EXT}	M^+	M^-_{INT}	M^-	M^+
1	M_u	-124	+155	-260	-242	+104	0	+103	-87	-80	+70
2	WIDTH, b DROP	120"	125"	120"	120"	125"	125"	125"	125"	125"	125"
3	EFF. DEP, d	13.13"	9.88"	13.13"	13.13"	9.88"	9.88"	9.13"	9.88"	9.88"	9.13"
4	$M_n = M_u/.9$	-138	172	-289	-269	116	0	114	-97	-89	+78
5	$R = M_n/bd^2$	80	169	168	156	114	0	131	95	88	+90
6	ρ (TABLE A5.A)	.00136	.0029	.0029	.00275	.00195	—	0.0225	.0016	.0015	.0015
7	$A_s = \rho bd$	2.14	3.55	4.57	4.33	2.41	0	2.78	1.98	1.85	1.71
8	$A_{s\,min} = .002bt$	3.71	2.75	3.71	3.71	2.75	2.75	2.75	2.75	2.75	2.75
9 LARGER GOVERNS	LARGER 7 OR 8/.44 #6	8.43 = 9	8.07 = 9	10.39 = 11	9.84 = 10	6.25 = 7	6.25 = 7	6.32 = 7	6.25 = 7	6.25 = 7	6.25 = 7
10.	$N_{min} = \frac{WIDTH}{2t}$	4.00 = 4	5.68 = 6	4.0 = 4	4.0 = 4	5.68 = 6	5.68 = 6	5.68 = 6	5.68 = 6	5.68 = 6	5.68 = 6

FRAME B – DESIGN REINF FOR COLUMN STRIP & MIDDLE STRIP

		COLUMN STRIP					MIDDLE STRIP				
ITEM	DESCRIPTION	M^-_{EXT}	M^+	M^-_{INT}	M^-	M^+	M^-_{EXT}	M^+	M^-_{INT}	M^-	M^+
1	M_u	-68	+85	-143	-134	+58	0	+57	-48	-44	+38
2	WIDTH, b DROP	120"	125"	120"	120"	125"	235"	235"	235"	235"	235"
3	EFF. DEP, d	13.88"	9.88"	13.88"	13.88"	9.88"	9.88	9.88	9.88	9.88	9.88
4	$M_n = M_u/.9$	-76	+94	-159	-149	+64	0	63	-53	-49	+42
5	$R = M_n/bd^2$	39	+92	83	77	+63	0	33	28	26	22
6	ρ (TABLE A5.A)	.00065	.00155	.00145	.0013	.0011	—	.00055	.0005	.00045	.0004
7	$A_s = \rho bd$	1.08	1.91	2.42	2.16	1.36	0	1.28	1.16	1.04	.93
8	$A_{s\,min} = .002bt$	3.71	2.75	3.71	3.71	2.75	5.17	5.17	5.17	5.17	5.17
9. LARGER GOVERNS	LARGER 7 OR 8/.44 #6	8.43 = 9	6.25 = 7	8.43 = 9	8.43 = 9	6.25 = 7	11.75 = 12	11.75 = 12	11.75 = 12	11.75 = 12	11.75 = 12
10.	$N_{min} = \frac{WIDTH}{2t}$	4.00 = 4	5.68 = 6	4.0 = 4	4.0 = 4	5.68 = 6	10.68 = 11	10.68 = 11	10.68 = 11	10.68 = 11	10.68 = 11

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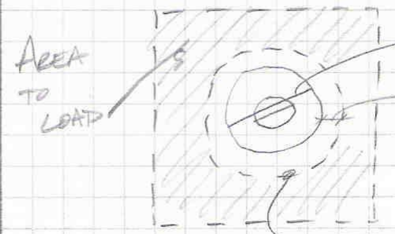
CHECK SHEAR (2-WAY OR PUNCHING ACTION)



DIAMETER = 6'

$d_{AVG} = \frac{13.88 + 13.13}{2} = 13.5''$

$d/2 = \frac{13.5}{2} = 6.75''$

DIAMETER OF CRITICAL SECTION : $6.75(2) + 6'$

$\phi = 7.125'$

AREA TO LOAD : $V_u = \omega_u \cdot AREA$

$= .331\left(30' \times 20.83' - \frac{\pi(7.125)^2}{4}\right)$

$V_u = 193.65^K$

PERIMETER ; $b_o = \pi(7.125 \times 12) = 286.6''$

$\frac{b_o}{d} = \frac{286.6}{13.5} = 21.23$; $\beta_c = 1$ (CIRCULAR) ; $\alpha_s = 40$ (INT. COLUMN)

LESSER

Eq 1 : $V_c = 4\sqrt{f'_c}\, b_o d = 4\sqrt{3000}\,(286.6)(13.5)\left(\frac{1}{1000}\right) = 847.7^K$

Eq 2 : $V_c = \left(2 + \frac{4}{\beta_c}\right)\sqrt{f'_c}\, b_o \cdot d = \left(2 + \frac{4}{1}\right)\sqrt{3000}\,(286.6)(13.5)\left(\frac{1}{1000}\right) = 1271.5^K$

Eq 3 : $V_c = \left(\frac{\alpha_s}{b_o/d} + 2\right)\sqrt{f'_c}\, b_o \cdot d = \left(\frac{40}{21.23} + 2\right)\sqrt{3000}\,(286.6)(13.5)\left(\frac{1}{1000}\right) = 823.1^K$

GOVERNS

$\phi V_c = .75(823.1^K) = 617.3^K > 193.65^K$ $\therefore$ OK

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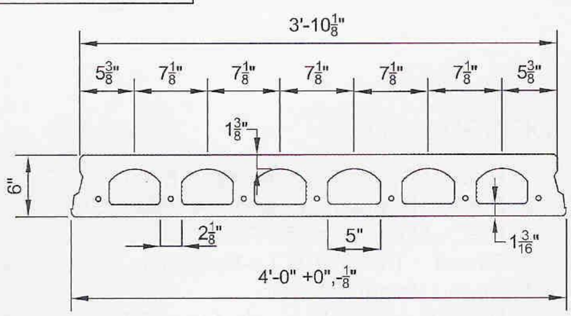
Appendix C – *Hollow Core Plank with Steel Framing*

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

1 Hour Fire Resistance Rating (Untopped)

PHYSICAL PROPERTIES Precast	
$A = 187$ in.^{2}	$S_b = 245$ in.^{3}
$I = 757$ in.^{4}	$S_t = 260$ in.^{3}
$Y_b = 3.09$ in.	Wt.= 195 PLF
$Y_t = 2.91$ in.	Wt.= 48.75 PSF
$e = 1.34$ in.	



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 = 47.7 k-ft
   7-1/2"Ø, 270K = 76.0 k-ft
7. Maximum bottom tensile stress is $7.5\sqrt{f'c} = 580$ 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. Load values to the left of the solid line are controlled by ultimate shear strength.
12. Load values to the right are controlled by ultimate flexural strength or allowable service stresses.
13. Load values will be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
14. 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 2003 & ACI 318-02 (1.2 D + 1.6 L)

Strand Pattern		SPAN (FEET)																		
		14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32
4 - 1/2"ø	LOAD (PSF)	219	191	165	150	131	124	110	97	86	76	66	58	51	45					
7 - 1/2"ø	LOAD (PSF)	345	318	285	262	231	214	197	175	156	138	123	110	98	88	78	70	62	55	49

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 1 Hour & 0 Minute fire resistance rating.

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CONCRETE PRODUCTS
2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17201-0813
717-267-4505 Fax 717-267-4518

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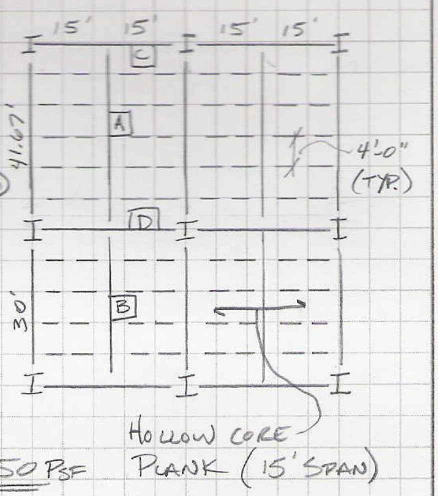
PRESTRESSED CONCRETE – HOLLOW CORE PLANK

- 6" × 4'-0" HOLLOW CORE PLANK
- 1 HOUR RATING (UNTOPPED)



LOADS: · LIVE LOAD: 100 PSF
· DEAD LOAD: 25 PSF (SUPERIMPOSED)
· PLANK: 48.75 PSF (SELFWEIGHT)

SERVICE LOAD:

$1.2(25) + 1.6(100) = 190$ PSF

$318 > 190 \therefore$ OK (FROM TABLE)

$w_u = 1.2(25 + 48.75) + 1.6(100) = 248.5 \approx \underline{\underline{250}}$ PSF

DEFLECTION LIMITS:

$\Delta_{LL} = L/480$, $\Delta_{TL} = L/240$

BEAM "A" DESIGN

TRIB WIDTH = 15', SPANNING 41.67'

$\Delta_{LL}: \dfrac{5(.100(15))(41.67)^4 1728}{384(29000) I_{REQ}} = \dfrac{(41.67)(12)}{480} \Rightarrow I_{REQ} = 3374 \text{ in}^4$

$\Delta_{TL} = \dfrac{5(.250(15))(41.67)^4 1728}{384(29000) I_{REQ}} = \dfrac{41.67(12)}{240} \Rightarrow I_{REQ} = 4217 \text{ in}^4$ ✓ CONTROLS

$M_u = \dfrac{(.250(15)(41.67)^2}{8} = 814'^K$

CHOOSE $\boxed{W30 \times 108}$ $I = 4470 \text{ in}^4 > 4217 \text{ in}^4 \therefore$ OK

$\phi M_n = 1300'^K > 814 \text{ in}^4 \therefore$ OK

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BEAM "B" DESIGN

TRIB WIDTH = 15', SPANNING 30'

$$\Delta_{LL}: \frac{5(.100(15))(30)^4 1728}{384(29000) I_{req}} = \frac{30(12)}{480} \Rightarrow 1257 \text{ in}^4 = I_{req}$$

$$\Delta_{TL}: \frac{5(.250(15))(30)^4 1728}{384(29000) I_{req}} = \frac{30(12)}{240} \Rightarrow 1571 \text{ in}^4 = I_{req} \checkmark \text{ CONTROLS}$$

$$M_u = \frac{.250(15)(30')^2}{8} = 422'^K$$

CHOOSE $\boxed{W24 \times 68}$ $I = 1830 > 1571 \therefore ok$
$\phi M_n = 664'^K > 422'^K \therefore ok$

GIRDER "C" DESIGN

TOTAL LOAD $= \frac{250 \text{ PSF} (15 \cdot 41.67)}{2} = 78.1^K$ @ MIDSPAN

LIVE LOAD $= \frac{100 \text{ PSF} (15 \cdot 41.67)}{2} = 31.3^K$ @ MIDSPAN

$$\Delta_{LL}: \frac{31.3^K (30)^3 1728}{48(29000) I_{REQ}} = \frac{30(12)}{480} \Rightarrow I_{REQ} = 1399 \text{ in}^4$$

$$\Delta_{TL}: \frac{78.1^K (30)^3 1728}{48(29000) I_{REQ}} = \frac{30(12)}{240} \Rightarrow I_{REQ} = 1745 \text{ in}^4 \checkmark \text{ CONTROLS}$$

$$M_u = \frac{78.1^K(30')}{4} = 586'^K$$

CHOOSE $\boxed{W24 \times 68}$ $I = 1830 > 1745 \therefore ok$
$\phi M_n = 664'^K > 586'^K \therefore ok$

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GIRDER "D" DESIGN

TOTAL LOAD $= \dfrac{250\,(15\,(41.67))}{2} + \dfrac{250\,(15\cdot 30)}{2} = 134.5^{K}$

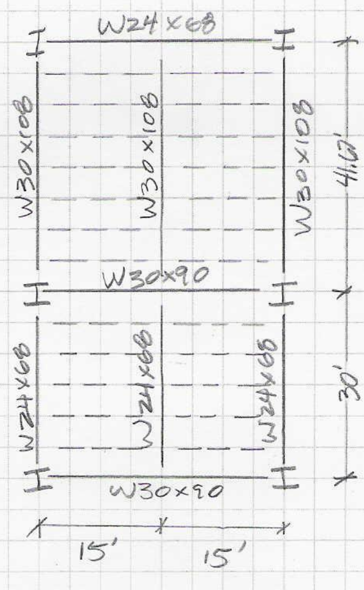
LIVE LOAD $= \dfrac{100\,(15\cdot 41.67)}{2} + \dfrac{100\,(15\cdot 30)}{2} = 53.8^{K}$

$\Delta_{LL}: \dfrac{53.8\,(30)^3\,1728}{48\,(29000)\,I_{REQ}} = \dfrac{30\,(12)}{480} \Rightarrow I_{req} = 2404\ in^4$

$\Delta_{TL} = \dfrac{134.5\,(30)^3\,1728}{48\,(29000)\,I_{req}} = \dfrac{30\,(12)}{240} = 3006\ in^4$ ✓ CONTROLS

$M_u = \dfrac{134.5\,(30')}{4} = 1009'^{K}$

CHOOSE $\boxed{W30 \times 90}$ $I = 3610 > 3006\ in^4$
$\phi M_n = 1060'^{K} > 1009'^{K}$ $\therefore$ OK



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Appendix D – *Two-way post-tension slab*

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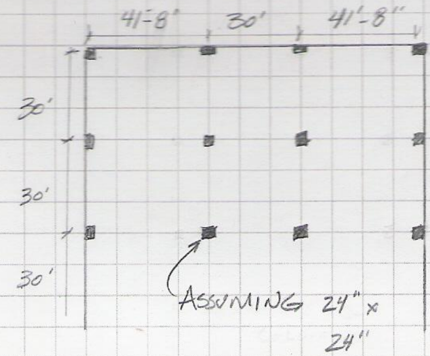
TWO-WAY POST TENSIONED

LOADS: · DEAD LOAD : 25 PSF (SUPERIMPOSED)
· LIVE LOAD: 100 PSF (UNREDUCED)



MATERIALS:

CONCRETE : NORMAL WEIGHT 150 PCF
$f'_{ci} = 3000$ psi
$f'_c : 5000$ psi

REBAR : $f_y = 60{,}000$ psi

UNBOUND TENDONS : 1/2" ϕ, 7-WIRE STANDS, $A = .153 \text{ in}^2$
$f_{pu} = 270$ ksi

ESTIMATED PRESTRESS LOSSES: 15 KSI (ACI 18.6)
$f_{se} = .7(220) - 15 = 174$ ksi (ACI 18.5.1)

$P_{eff} = A \cdot f_{se} = (.153)(174) = 26.6$ KIPS/TENDON

SLAB THICKNESS: · $L/H = 45$; LONGEST SPAN $= 41.67'$

$h = (41.67' \cdot 12)/45 = 11.11$ (TRY 12" SLAB)

LOADING: SUPERIMPOSED = 25 PSF
SELFWEIGHT : $\dfrac{(12 \text{ in})(150 \text{ PCF})}{12} = 150$ PSF

LIVE LOAD = 100 ; $K_{LL} = 1.0$ (2-WAY SLAB)

EXTERIOR SPAN: ↳ REDUCED $= 100\left(.25 + \dfrac{15}{\sqrt{1 \cdot (41.67 \cdot 30)}}\right) = 67.42$ PSF

INTERIOR SPAN: $100\left(.25 + \dfrac{15}{\sqrt{1 \cdot (30 \times 30)}}\right) = 75$ PSF

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CALCULATE SECTION PROPERTIES:

· 2-WAY DESIGNED AS CLASS U

$A = bh = (30' \cdot 12)(12) = 4320 \text{ in}^2$

$S = \frac{bh^2}{6} = (30 \cdot 12)\, 12^2/6 = 8640 \text{ in}^3$

SET DESIGN PARAMETERS:

ALLOWABLE STRESSES

· AT TIME OF JACKING:

· $f'_{ci} = 3000$ psi
· COMPRESSION: $.6 f'_{ci} = .6(3000) = 1{,}800$ PSI
· TENSION: $3\sqrt{f'_{ci}} = 3\sqrt{3000} = 164$ psi

· AT SERVICE LOADS

· $f'_c = 5000$ psi
· COMPRESSION $= .45 f'_c = .45(5000) = 2{,}250$ PSI
· TENSION $= 6\sqrt{f'_c} = 6\sqrt{5000} = 424$ psi

· AVERAGE COMPRESSIBLE LIMITS:

$P/A = 125$ psi (min) ; 300 psi (max)

· TARGET LOAD BALANCES:

→ 75% OF SELF WEIGHT = .75(150 psf) = 112.5 psf

· COVER REQUIREMENTS - IBC 2003 (2-HOUR FIRE RATING)

→ RESTRAINED SLABS = 3/4" (BOTTOM)
→ UNRESTRAINED SLABS = 1 1/2" & 3/4" (BOTTOM & TOP RESPECTIVELY)

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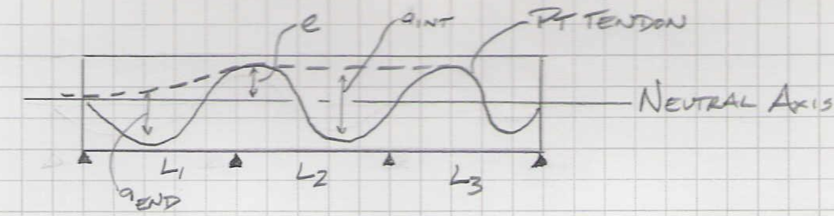
TENDON PROFILE:



CONTINUOUS PT BEAM

TENDON ORDINATE	TENDON LOCATION	
· EXT. SUPPORT - ANCHOR	6"	
· INT. SUPPORT - TOP	11"	* MEASURED FROM BOTTOM OF SLAB
· INT SPAN - BOTTOM	1"	
· END SPAN - BOTTOM	1.75"	

$a_{INT} = 11" - 1" = 10.0"$
$a_{END} = (6" + 11")/2 - 1.75" = 6.75"$
e = VARIES

PRESTRESS FORCE REQUIRED TO BALANCE TARGET LOAD

$\omega_b = .75 w_{DL}$

$w_b = .75(150)(30') = 3375$ PLF $= 3.375$ KLF

FORCE IN TENDONS TO COUNTERACT THE LOAD IN END BAY

$P = w_b L^2 / 8 a_{end}$

$= (3.375)(41.67)^2 / (8(\frac{6.75}{12})) = 1302.3^K$

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CHECK PRECOMPRESSION ALLOWANCE

• DETERMINE # OF TENDONS TO ACHIEVE 1302^K

$$\# \text{ OF TENDONS} = 1302^K / 26.6\ ^K/_{TEND.}$$
$$= 48.9$$

↳ (USE 48 TENDONS)

• ACTUAL FORCE FOR BANDED TENDONS

$$P_{ACTUAL} = 48 \text{ TENDONS } (26.6\ K) = 1277^K$$

• BALANCED LOAD FOR END SPAN IS SLIGHTLY ADJUSTED

$$w_b = \left(\frac{1277}{1302}\right)(3.375) = 3.31\ K/FT$$

• DETERMINE ACTUAL PRECOMPRESSION STRESS

$$P_{ACTUAL}/A = \frac{1277\ (1000)}{4320\ in^2} = 295.6\ psi$$

$$125 (min) < 295.6 \leq 300\ (max) \quad \therefore OK$$

EFFECTIVE PRESTRESS FORCE ; $\boxed{P_{eff} = 1277^K}$

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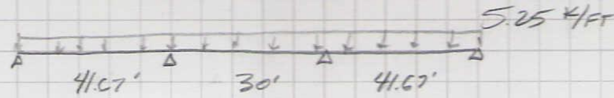
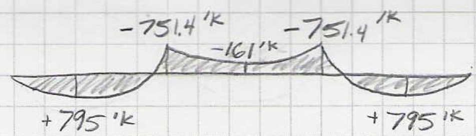
DEAD LOAD MOMENTS:

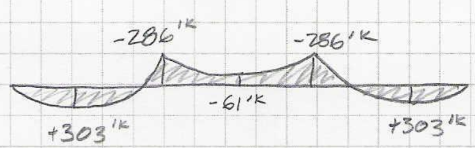
$$W_{DL} = \frac{175\text{ PSF }(30')}{1000} = 5.25\text{ KLF}$$





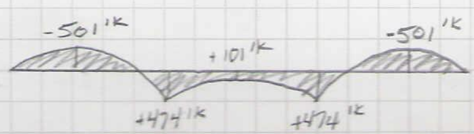
* STAAD ANALYSIS TO DETERMINE MOMENTS

$$W_{LL} = \frac{67.4\text{ PSF }(30')}{1000} = 2.0\text{ KLF}$$



TOTAL BALANCING MOMENTS, M_{bal}

$W_b = -3.31$ K/FT



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STAGE 1: STRESSES IMMEDIATELY AFTER JACKING (DL+PT)

· MIDSPAN STRESSES

· $f_{TOP} = (-M_{DL} + M_{BAL})/S - P/A$

· $f_{BOT} = (+M_{DL} - M_{BA})/S - P/A$

INTERIOR SPAN

$f_{TOP} = \dfrac{(-161 + 101)\,12\,(1000)}{8640\ in^3} - 295.6 = -379$ psi (COMP.) < 1800 psi $(.6f'_{ci})$ ∴ OK

$f_{BOT} = \dfrac{(161 - 101)\,12\,(1000)}{8640\ in^3} - 295.6 = -212$ psi (COMP.) < 1800 psi ∴ OK

END SPAN

$f_{TOP} = \dfrac{(-795 + 501)\,12(1000)}{8640\ in^3} - 295.6 = -704$ psi (COMP.) < 1800 psi ∴ OK

$f_{BOT}\ \dfrac{(795 - 501)\,12\,(1000)}{8640\ in^3} - 295.6 = 113.3$ psi (TENS) < 164 psi $(3\sqrt{f'_{ci}})$ ∴ OK

SUPPORT STRESSES

· $f_{TOP} = (+M_{DL} - M_{BAL})/S - P/A$

· $f_{BOT} = (-M_{DL} + M_{BAL})/S - P/A$

$f_{TOP} = \dfrac{(751 - 474)\,12\,(1000)}{8640\ in^3} - 295.6 = 89$ psi (TENS.) < 164 ∴ OK

$f_{BOT} = \dfrac{(-751 + 474)\,12\,(1000)}{8640\ in^3} - 295.6 = -680$ (COMP.) < 1800 psi ∴ OK

STAGE 2: STRESSES @ SERVICE LOAD (DL + LL + PT)

MIDSPAN STRESSES

$$f_{TOP} = (-M_{DL} - M_{LL} + M_{BAL})/S - P/A$$

$$f_{BOT} = (+M_{DL} + M_{LL} - M_{BAL})/S - P/A$$

INTERIOR SPAN

$$f_{TOP} = \frac{(-161 - 61 + 101)\ 12\ (1000)}{8640\ in^3} - 295.6 = -464_{psi} < 2250\ psi\ (.45f'_c)$$

$\therefore$ OK

$$f_{BOT} = \frac{(161 + 61 - 101)(12)(1000)}{8640\ in^3} - 295.6 = -127.5 < 2250\ psi \quad \therefore OK$$

END SPAN

$$f_{TOP} = \frac{(-795 - 303 + 501)\ 12\ (1000)}{8640\ in^3} - 295.6 = -1125\ psi < 2250\ psi \quad \therefore OK$$

$$f_{BOT} = \frac{(795 + 303 - 501)\ 12\ (1000)}{8640\ in^3} - 295.6 = 533.6 > 424\ psi\ (6\sqrt{f'_c})$$

$\therefore$ .6fy REINF NEEDED

SUPPORT STRESSES

$$f_{TOP} = (+M_{DL} + M_{LL} - M_{BAL})/S - P/A$$

$$f_{BOT} = (-M_{DL} - M_{LL} + M_{BAL})/S - P/A$$

$$f_{TOP} = \frac{(751.4 + 286 - 501)\ 12\ (1000)}{8640\ in^3} - 295.6 = \underset{psi}{\overset{(TENS)}{449.4}} > 424\ psi\ (6\sqrt{f'_c})$$

$\therefore$ .6fy REINF NEEDED

$$f_{BOT} = \frac{(-751.4 - 286 + 501)\ 12\ (1000)}{8640} - 295.6 = \underset{(COMP)}{-1040.6}\ psi < 2250$$

$\therefore$ OK

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ULTIMATE STRENGTH

· DETERMINE FACTORED MOMENTS

→ PRIMARY PT MOMENTS, M_1

$M_1 = P \cdot e$

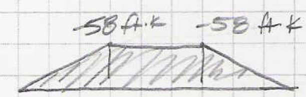
$e = 0$ in @ the EXTERIOR SUPPORT
$e = 5$ in @ THE INTERIOR SUPPORT

$$M_1 = \frac{1277^{k}(5'')}{12} = 532 \text{ ft}\cdot\text{k}$$

→ SECONDARY PT MOMENT, M_{sec}

$M_{sec} = M_{bal} - M_1$

$= 474 - 532 = -58$ ft·k
@ INT SUPPORTS



LOAD COMBINATION ($M_u = 1.2 M_{DL} + 1.6 M_{LL} + 1.0 (M_{sec})$)

@ MID SPAN: $M_u = 1.2(795) + 1.6(303) + -29(1.0) = 1410$ ft·k

@ SUPPORT: $M_u = 1.2(-751) + 1.6(-286) + 1.0(474) = -885$ ft·k

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DETERMINE MINIMUM BONDED REINFORCEMENT

- POSITIVE MOMENT REGION
  - INTERIOR SPAN, f IS COMPRESSIVE
    ∴ NO POSITIVE REINFORCEMENT NEEDED
  - EXTERIOR SPAN

$$f_t = 533.6 \text{ psi} > 2\sqrt{5000} = 141 \text{ psi}$$

- MIN. POSITIVE REINF. REQUIRED (ACI 18.9.3.2)

$$A_{s\,min} = \frac{N_c}{.5 f_y}$$

$$y = \frac{f_t}{(f_t+f_c)h} = \frac{533.6}{(533.6+1125)(12'')} = 3.86''$$

$$N_c = \frac{M_{DL+LL}}{S} \cdot (.05)(y)\,l_2 = \frac{795+303\,(12)}{8640} \cdot (.5)(3.86) \cdot (30 \cdot 12)$$

$$N_c = 1060 \text{ K}$$

$$A_{s\,min} = \frac{1060}{.5(60\text{ksi})}$$

$$\boxed{A_{s\,min} = 35.3 \text{ in}^2}$$

$$A_{s\,min} = \frac{35.3 \text{ in}^2}{30 \text{ ft}} = 1.18 \text{ in}^2/\text{ft} \Rightarrow \boxed{\text{USE \#10 @ 12 in o/c (BOTTOM)}}$$

* MIN LENGTH SHALL BE 1/3 CLEAR SPAN & CENTERED IN POSITIVE MOMENT REGION

- NEGATIVE MOMENT REGION

$$A_{s\,min} = .00075\,A_{cf} \quad (\text{ACI } 18.9.3.3)$$

- INTERIOR SUPPORTS

$$A_{cf} = (12'')\left(\frac{41.67+30}{2}\right)12 = 5160(.00075) = 3.87 \text{ in}^2 \Rightarrow \boxed{\text{USE (9) \#6 TOP } (3.96 \text{ in}^2)}$$

- EXTERIOR SUPPORTS

$$A_{cf} = 12\,(30 \text{ ft})\,12 = 4320(.00075) = 3.24 \text{ in}^2 \Rightarrow \boxed{\text{USE (8) \#6 TOP } (3.52 \text{ in}^2)}$$

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- MUST SPAN MIN. OF 1/6 CLEAR SPAN, EACH SIDE OF SUPPORT (ACI 18.9.4.2)
- AT LEAST 4 BARS REQUIRED IN EACH DIRECTION (ACI 18.9.3.3)
- PLACE TOP BARS WITH 1.5h AWAY FROM THE FACE OF THE SUPPORT SIDE (ACI 18.9.3.3)

$= 1.5(12) = 18''$

- MAXIMUM BAR SPACING IS 12" (ACI 18.9.3.3)

CHECK MINIMUM REINF SUFFICIENT FOR ULTIMATE STRENGTH

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

- $A_{ps} = .153 in^2$ (48 TENDONS
  $A_{ps} = 7.34 in^2$
- $f_{ps} = f_{se} + 10{,}000 + \frac{(f'_c bd)}{A_{ps}(300)}$

d: effective length
$a = (A_s f_y + A_{ps} f_{ps}) / .85 f'_c b$

$= 174{,}000 + 10{,}000 + \frac{5000(30 \cdot 12)d}{7.34(300)}$

$f_{ps} = 184{,}000 + 817.4d$

- AT SUPPORTS

$d = 12'' - 3/4'' - 1/4'' = 11''$
$f_{ps} = 184{,}000 + 817.4(11) = 192{,}991$ psi
$a = \frac{3.96 in^2 (60 ksi) + 7.34 in^2 (193 ksi)}{.85(5 ksi)(30 \cdot 12)} = 1.08$

$\phi M_n = .9[3.96 in^2 (60 ksi) + 7.34 in^2 (193 ksi)](11 - \frac{1.08}{2}) = 15573/12 = 1298'^k$

$1298 > 885$ ∴ MINIMUM REINF OK

AT MIDSPAN (END SPAN)

$d = 12'' - 1\frac{1}{2}'' - 1/4'' = 10.25''$
$f_{ps} = 184{,}000 + 817.4(10.25) = 192{,}378$ psi
$a = \frac{35.3 in^2 (60 ksi) + 3.06(192.3 ksi)}{.85(5 ksi)(30 \cdot 12)} = 1.77$

$\phi M_n = .9[35.3 in^2 (60 ksi) + 3.06(192.3 ksi)](10.25 - \frac{1.77}{2}) = \frac{22811}{12} = 1901'^k > 1410'^k$

∴ MINIMUM REINF OK

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CHECK PUNCHING SHEAR

$d = 12 - 1 = 11''$

$b_o = \frac{11}{2} = 5.5(2) + 24'' = 35(4) = 140''$

$\alpha_s = 40$ INT. COL , $\beta_c = 1$

$$V_c = \min \begin{cases} 4\sqrt{f'_c}\, b_o d = 4\sqrt{5000}\cdot(140)(11) = 436^K \checkmark \\ \left(2 + \frac{4}{\beta_c}\right)\sqrt{f'_c}\, b_o \cdot d = (2+4/1)\sqrt{5000}\,(140)(11) = 653^K \\ \left(\frac{\alpha_s}{b_o/d} + 2\right)\sqrt{f'_c}\, b_o d = \left(\frac{40}{140/11} + 2\right)\sqrt{5000}\,(140)(11) = 560^K \end{cases}$$

$\phi V_c = .75(436) = 327^K$

$\omega_u = (1.2)(25 + (150(\frac{12}{12}))) + 1.6(100) = 370 = .370$

$V_u = \omega_u \cdot AREA_{TRIB}$

$V_u = .370\left(30 \times \left(\frac{41.67+30}{2}\right) - \left(\frac{35}{12}\right)^2\right) = 394.6^K > 327 \therefore$ NEED DROP PANELS

$(394.6^K / .75) = 4\sqrt{5000}\cdot b_o(11) \Longrightarrow b_o = 169''$

$169''/4 = 24 \cdot d \Longrightarrow d = 18.25 = 19'' / SLAB = 12''$

NEED 13'' DROP PANEL

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