SENIOR THESIS

TECHNICAL REPORT 2: FLOOR SYSTEM EXPLORATION

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Date Submitted: October 19th, 2011

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Executive Summary

The objective of Technical Report 2 is to explore three alternative floor systems and compare them to the existing floor system of the Judicial Center Annex (JCA) that was analyzed in Technical Report 1: Existing Conditions. This was accomplished through hand calculations, using the 33'x41' bay spanning East-West between column grids 4 and 5 and North-South between column grids D and E, as it has the largest spans and would likely control the design. The systems were compared on the basis of cost, weight, depth, as well as architectural, structural, serviceability, and construction impacts. The existing system is a post tensioned slab with wide-shallow beams running in the NS direction which extend 8" below the adjacent slab. The three alternatives considered were a Two Way Slab with Drop Panels, One Way Slab with Beams, and Composite Deck on Composite Steel Beams.

The Two Way Slab was designed to be 13" thick with drop panels extending 4" below the slab that are approximately 10' square and offset on the columns toward the thicker spans per 13.2.5 ACI 318-08. This system doesn't add any depth, but adds significant weight and is more expensive. In terms of vibration and constructability the two way slab performs comparatively, while it would be anticipated to deflect slightly more.

The One Way Slab with Beams was considered the least feasible of the alternatives. The shallow 6" slab lightened the weight of the floor, though not significantly as an infill beam had to be added to make the one way action feasible. The 24x24 beams and girders that were designed resulted in a depth that is almost 1.5 the first two systems, which would have a significant architectural effect. This, incorporated with the large cost increase and difficulty in achieving the desired slab cantilever on the East Elevation make it a poor choice.

The Composite Deck on Composite Steel Beams was the only system considered with steel framing. 2VLI18 with a 3" LW topping rests upon W16x31 beams framing into W21x68 girders. The Total system depth comes to 26", just as detrimental as the one way system with the exception that mechanical systems can be run through the beams if need be. The system only weighs 44 psf though, which could have potential foundation and lateral savings for the structural system due to a reduction in forces. This system is likely the fastest to construct as well, due to the lack of formwork and unshored assumptions made.

Building Introduction

The Judicial Center Annex (JCA) is a modern addition to the existing Montgomery County Judicial Center. Located on the corners of Maryland Avenue and East Jefferson Street in downtown Rockville, MD the JCA is set provide a bold statement through both its architecture and engineering. Construction on the addition began this past April and is projected to take two years to complete.

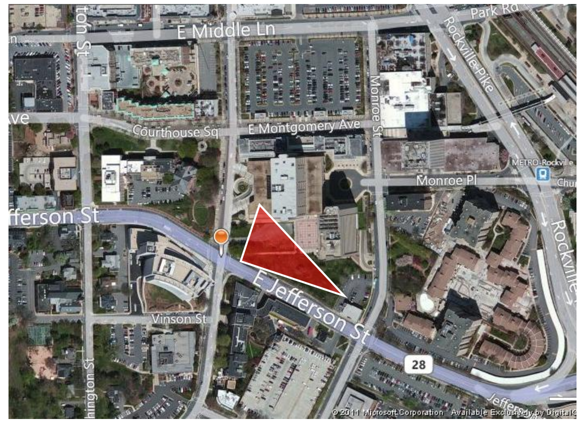


Figure 1: Site Location, Bing.com

The JCA will stand 114' tall at the crest of each of the four lanterns located on top of the building, so tall that local building codes needed waved for overall building height. Six stories rise above the ground, with garage and terrace levels located below grade, adding approximately 210,000 sq ft to the Judicial Center that will add 10 more courtrooms and administrative spaces among other spaces.

The project team, led by AECOM who provided both architectural and the majority of building engineering services, was able to achieve a unique look through both form and material. The East and West Elevations (Figure 2) are dominated by glazing, with the curtain wall that covers the East wrapping around the South corner. This curtain wall system is unique in that it uses glass stabilizing fins instead of traditional aluminum mullions, which enables an all glass look that when combined with the way the slab cantilevers out from the structure gives the illusion of the floors floating without structure. On the North the addition abuts against the original Judicial Center. The elements of the façade not covered in glass are sheathed in either a powder coated aluminum that has a reddish hue or architectural pre-cast panels that are more reminiscent of the exterior of the original building.

Figure 2: West Elevation

From the roof projects four lanterns which have a translucent linear glazing system allowing them to light up the night sky in a truly dramatic manner. The roof is also the site of two of the JCA's sustainable features that enabled it to achieve a LEED Gold Rating. The tops of each of the four lanterns are covered in photovoltaic panels, while green roofs cover much of the remaining roof.

Structural Overview

The JCA sits atop core-drilled concrete piers due to the rather poor soil conditions, all columns coming to bear atop a pier. The floor systems are post-tensioned slabs, with wide-shallow beams running one-way on the typical levels framing into cast-in-place concrete columns. The lateral system consists of five concrete shear walls, which rise continuously to the penthouse level, with some continuing to support the roof.

This building was designed as Occupancy III according to Sheet 1.S001. The reason for this is thought that the holding cells in the building subject it to the "Jail and detention facilities" clause or perhaps a courtroom has the ability for "more than 300 people to congregate." This Occupancy was assumed due to one of the previously mentioned reasons for purposes of coming up with importance factors in later calculations.

Foundations

Schnabel Engineering performed the geotechnical services on the JCA project. Reports indicated that for the purposes of shallow continuous wall footings the soil has a bearing capacity of 2000 psi, with any unsuitable conditions requiring excavation and replacement with lean concrete. Core-drilled piers ranging in diameter from 2.5' to 7' are located beneath every column and support much of the shallow wall footings. Grade beams are also used in several locations, specifically beneath the five shear walls. The usage of grade beams beneath the continuous shear walls is due to the extremely large concentration of forces that need transferred into the soil as a result of both the shear walls own weight and the lateral forces that are being transferred through them. Tying into the Grade beams would help against uplift which will be investigated further in Technical Report 3. Grade beams vary from 24" to 42" in width and 36" to 72" in depth. The slab on grade is 5" thick and reinforced with WWF.

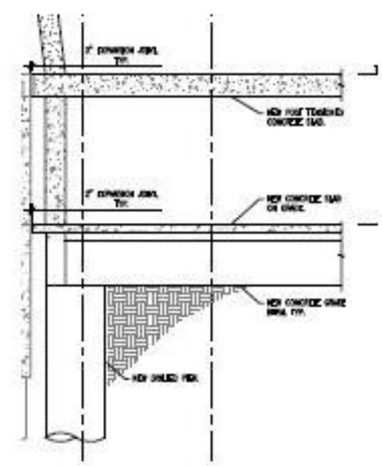


Figure 3: Column adjacent to existing Judicial Center resting on pier foundation

The garage level of the JCA is located 25' below grade. Though soil pressures on basement walls were not considered in this report they are a possible point of investigation in the future.

Framing Systems

Cast-in-place columns rise from the garage level to the roof, with the four lanterns extending the extra fourteen feet with steel framing. The column concrete has a compressive strength of 7000 psi at the base, which is reduced to 5000 psi at level 2. Typical column sizes are 24"x24"

Each lantern has a flat roof framed in structural steel with a slight slope on the edges. HSS tubes make up the columns, with the majority of the framing being small steel shapes with spans in the range of 5' and typical sizes of L3x3x1/4, HSS4x4x1/4, and C6x13. In the center of the roof are several W12x40 girders with a maximum span of 33' that are famed into by smaller wide flange shapes. These heavier shapes are intended to carry the photovoltaic panels mounted on top of the lanterns. Several HSS brace frames provide lateral stability for the lanterns. The lanterns were given an assumed weight of 40 psf to account for the steel, translucent linear glazing, and photovoltaic panels.

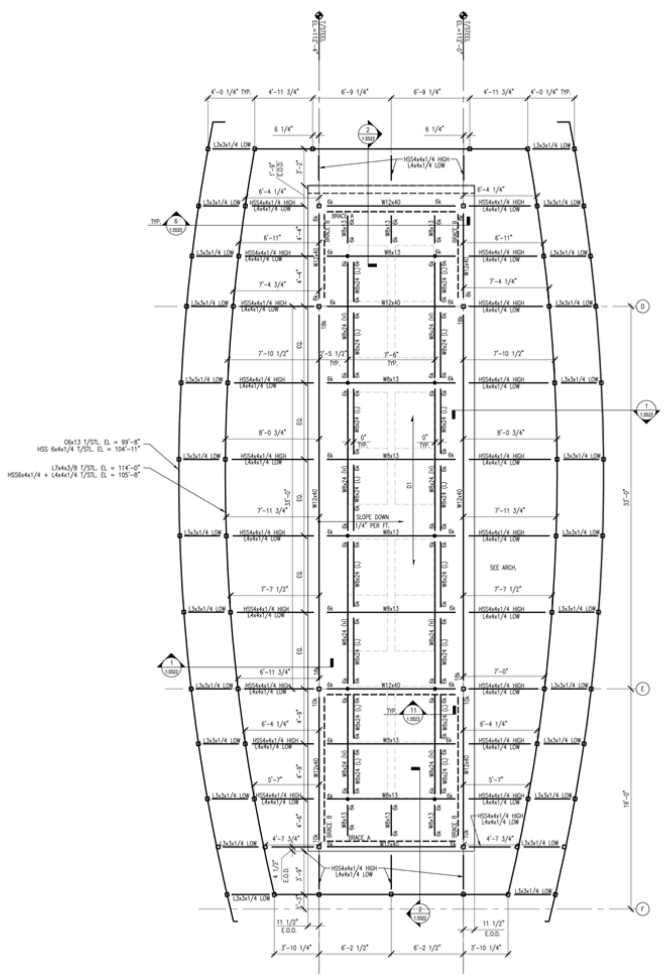


Figure 4: Lantern Framing Plan, larger plan found in Appendix A

Lateral System

The lateral system of the JCA is comprised of five cast-in-place concrete shear walls, see Figure 5. The shear walls in the NS plan direction extend to the roof, while in the EW direction they reach the penthouse level. The walls extend continuously upward and feature large openings relying on link beams to maintain the load path from the various floor heights to the foundation. The walls are all 12” thick, and assuming a rigid diaphragm (reasonable for the thick concrete slabs), the walls will take load in proportion to their stiffness. Based upon their similar thicknesses, this stiffness will then be proportional to their length, meaning that in the EW direction shear walls 4 and 5 each take half the lateral force, while in the NS direction shear wall 1 takes half the load with shear walls 2 and 3 splitting the other half between them. These assumptions will be investigated more in depth through the usage of computer software in subsequent reports. Also worth investigation is how much of the load is transferred through frame action in the concrete slab and columns, and whether overturning will be an issue for the shear walls that are tied into grade beams.

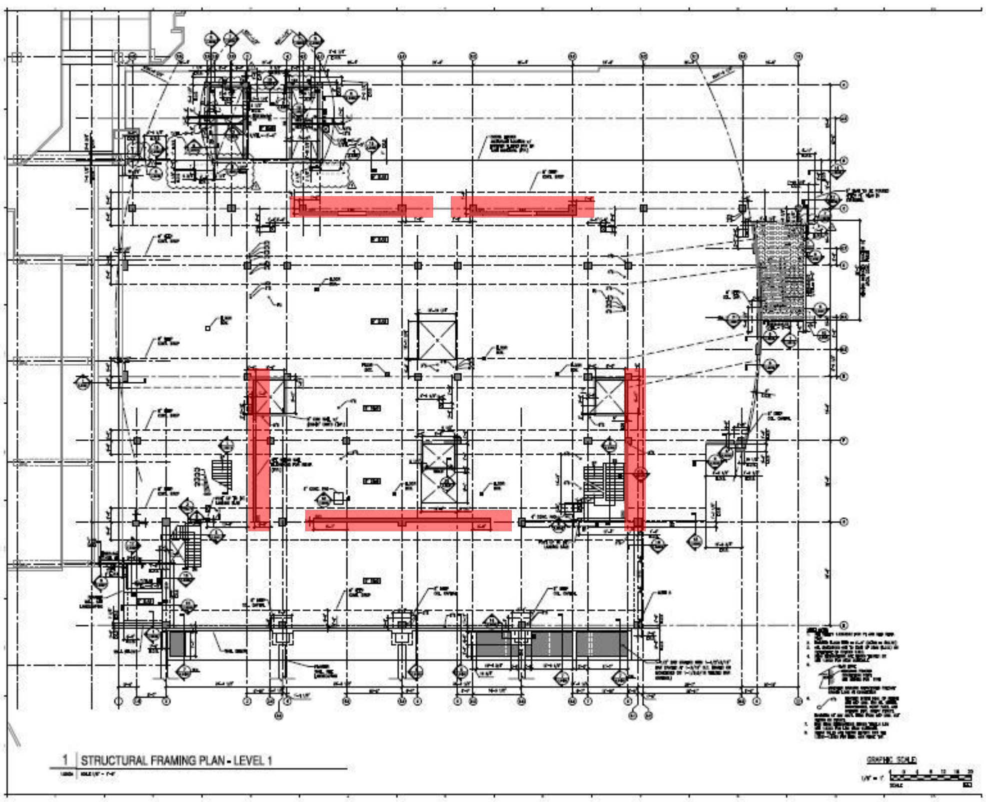


Figure 5: Shear wall layout on typical floor plan

Roof Systems

The roof varies in height in several locations with the floor slabs described earlier in *Floor Systems*. The varying heights made snow drift a concern, and the large loads associated with the penthouse floor, which is the heaviest floor on the building, add a significant contribution to both seismic base shear and overturning. The green roof and pavers on the penthouse and upper roof levels lay overtop a hot applied fluid membrane.

Design Codes

The list of Major Codes and Standards on Sheet 1.S001 is as follows:

- 2009 International Building Code
- ACI 318-08
- AISC LRFD, 13th Edition, 2005
- AWS D1.1, D1.3, D1.4, Current Edition
- ASTM, Current Edition
- Steel Deck Institute Design Manual for Composite Deck, Form Decks and Roof Decks., 2007

These are the codes being used to complete the analyses performed in this report, with heavy usage of ASCE 7-05 (Minimum Design Loads).

Materials Used

Sheet 1.S001 was used as the reference for materials used in the construction of this project and summarized in Figure 6.

Concrete		
Usage	Weight	f'c (psi)
Column (Levels 2-Rf)	Normal	5000
Column (Levels G1-1)	Normal	7000
Floor Slab	Normal	5000
Wall Footings	Normal	3000
Beams	Normal	5000
Slab on Grade	Normal	4500
Walls, Piers, & Pilasters	Normal	5000
Drilled Piers	Normal	4000
LW Concrete Fill on Deck	Light	4000
Isolation Slab @ Penthouse	Light	4000

Steel		
Type	ASTM Standard	Grade
W Shapes	A992	
Plates, Angles, Channels	A36	
High-Strength Bolts	A325 or A490	
Anchor Rods	F1554	36
Tubes	A500	B
Pipes	A53 E or S	B
Reinforcing Steel	A615	60
Reinforcing Steel, Welded	A706	60
Roof Deck	A653	A - F
Floor Deck	A653	C, D, or E
Post-Tensioned Reinforcment	A416-96	

Masonry		
Type	ASTM Standard	F'm (psi)
CMU	C90	1500
Masonry Mortar	C270	
Grout	C476	
Aggregate	C404	

Figure 6: Summary of Materials Used

Gravity Loads

This section will describe how dead, live, and snow loads were calculated and compared to loadings given on the structural drawings. Three gravity checks were performed once the loadings were determined for an interior column, the typical long span for the post tensioned slab, and a doubly reinforced beam with full hand calculations available in Appendix A.

Dead and Live Loads

The dead loads listed on 1.S001 shown in Figure 7 were used for the purposes of analyses. The non-load-bearing CMU walls were assumed to be fully grouted for the purposes of worst-case load calculations. The weight of the building was calculated neglecting voids in slabs and with an assumption of 10 psf for the steel lantern framing, which would not have much effect on the building weight were it too small an assumption. The total building weight which was used for the seismic calculations was in the order of 28000 kips.

Dead Loads		
	Design	Student
Vegetated Roof	55	55
MEP/Celing	15	15
CMU Partitions	Actual Weight	91 pcf (Fully Grouted Assumption)

Figure 7: Summary of Dead Loads

Based upon ASCE 7-05 the 100 psf typical live load was found to be correct, possibly for different reasons than the designer decided for, and the 40 psf holding cell load was neglected in favor of using the 100 psf live load in all locations except for the mechanical penthouse and the roof loading.

Live Loads		
	Design	ASCE 7-05
Typical	100	80 (Corrider Above First Floor) + 20 (Partition) = 100
Holding Cells	40	-
Mechanical Penthouse	150	150
Roof	-	20

Figure 8: Summary of Live Loads

Snow Loads

The flat roof snow load was calculated via the method outlined in Chapter 7 of ASCE 7-05. A discrepancy arose as the importance factor, I, listed on the drawings had a value of 1.0, whereas the appropriate importance factor for an Occupancy III building is 1.1. This led to flat roof snow load value of 22 psf which differs from the calculated value of 23.1 psf. Curiously the design load is higher despite the lower importance factor which may be a result of a higher design ground snow load, though this isn't available on the drawings.

Flat Roof Snow Load		
pf = .7 CeCtIpg > 20*I		
Ce	1	ASCE 7-05 Tab. 7-2
Ct	1	ASCE 7-05 Tab. 7-3
pg	25	ASCE 7-05 Fig. 7-1
I	1.1	ASCE 7-05 Tab. 7-4
pf =	0	
20*I=	500	
pf =	22	

Figure 9: Snow Load Parameters and Flat Roof Calculation

The varying roof levels led to eight different drift calculations. The calculations can be see viewing Figure 10 and 11, with an accompanying hand check for one of the drifts performed in Appendix A.

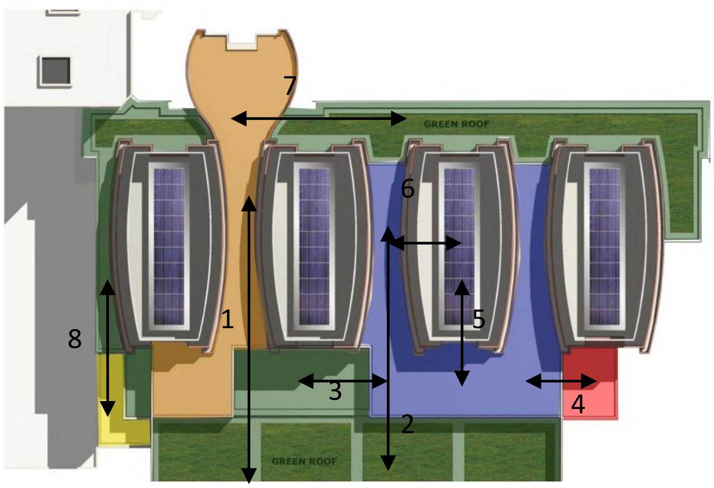


Figure 70: Rooftop Drift Diagram

Snow Drift		γ=	17.25						
	Lu	Ll	hc	hd Lee	hd Wind		hd (ft)	w (ft)	Max psf
Drift 1	130	50	16	3.79826	1.764815	3.79826	3.79826	15.19	65.52
Drift 2	93	30.33	18	3.238561	1.321269	3.238561	3.238561	12.95	55.87
Drift 3	70	50	18	2.810406	1.764815	2.810406	2.810406	11.24	48.48
Drift 4	70	20	21	2.810406	1.004234	2.810406	2.810406	11.24	48.48
Drift 5	70	20	14	2.810406	1.004234	2.810406	2.810406	11.24	48.48
Drift 6	38	12	14	2.016252	0.670866	2.016252	2.016252	8.07	34.78
Drift 7	21	147	16	1.385528	3.014862	3.014862	3.014862	12.06	52.01
Drift 8	83	24	52	3.06224	1.137649	3.06224	3.06224	12.25	52.82

Figure 81: Drift Spreadsheet

Floor Systems

The objective of this technical report is to analyze the existing floor system and compare it to three alternative floor system designs, each with different pros and cons. The systems will be compared in terms of cost (calculated using RS Means Costworks online, Appendix F), weight, depth and what impacts these and other parameters have on the architecture, structure, and construction.

The typical floor plan features relatively irregular bays. This made it difficult to choose one bay that was representative of the entire floor plan, so the bays in the plan north section of the building are focused upon as they have the largest spans and would likely control the design. The 33’x41’ bay spanning East-West between column grids 4 and 5 and North-South between column grids D and E (highlighted in Figure 12, larger plan included in Appendix A) was chosen for ease of comparison.

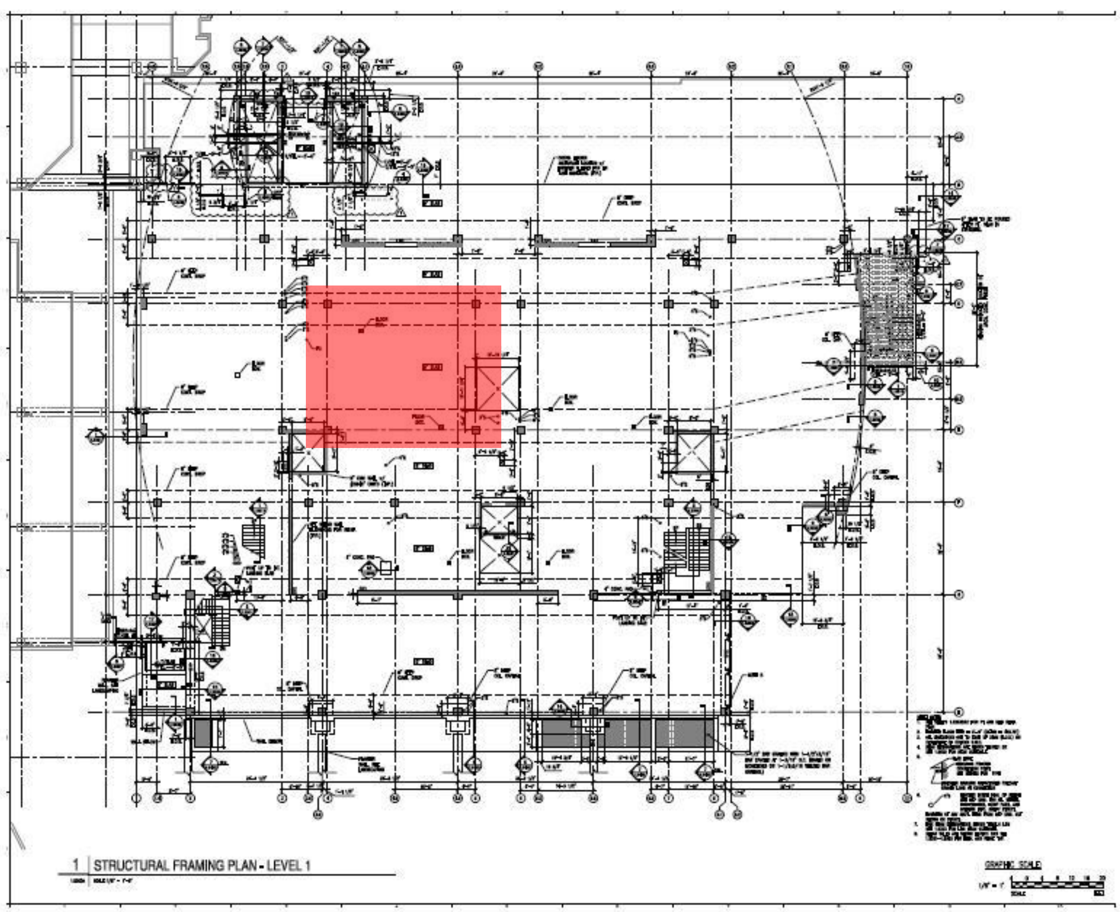


Figure 12: Typical framing plan with bay of interest highlighted

Post Tensioned Slab with Wide-Shallow Beams

The current floor system of the JCA is a post tensioned slab that ranges in depth from 8" to 9" on a typical floor.PT slabs are used to achieve greater economy over longer spans as the moment balancing allows for a shallower slab depth. The plans denote continuous drop panels which are also referred to as slab bands in the design narrative that run in the North-South direction and are approximately 8' in width with a depth of 8" beyond the adjacent slab. These are interpreted as wide-shallow beams as it is thought they may prove beneficial with regards to reducing positive moment reinforcement. According to ACI 318-08 section 13.2.5 a drop panel that is used to reduce negative moment reinforcement or a minimum slab thickness will meet two requirements: project beneath the slab at least one quarter of the adjacent slab distance and extend in each direction from the centerline of support a distance greater than one sixth the span length measured from center to center. The wide-shallow beams meet these requirements and therefore may be called continuous drop panels, though because it is assumed that they are providing aid to the positive moment they will be referred to as beams from here on out.

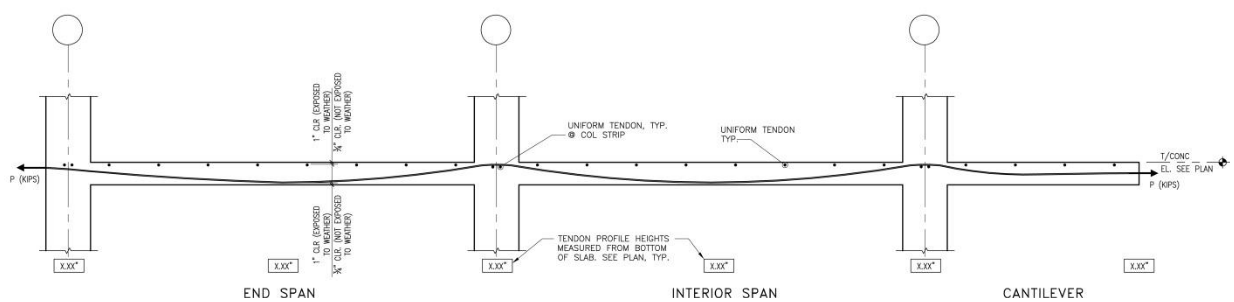


Figure 13: Section of PT slab showing tendon drape

General

The post tensioned system was found to be not only the least expensive but also the shallowest of all the systems considered costing in the order of $18.74/SF with a maximum depth of 17". Despite the high floor to floor heights, average 15'-6", the depth remains important as many of the court room spaces have ceiling heights in the order of 14' which reduces the otherwise generous ceiling cavity to that seen more typically in construction. The weight of this system is 137 psf.

Architectural

This is the base system, so its architectural impacts are none seeing how the architecture was designed with this in mind. Interesting to note is that one of the reasons the system was chosen was for its ability to cantilever out on the East Elevation with no negative impacts. It is not believed that the slab is left exposed anywhere in the building.

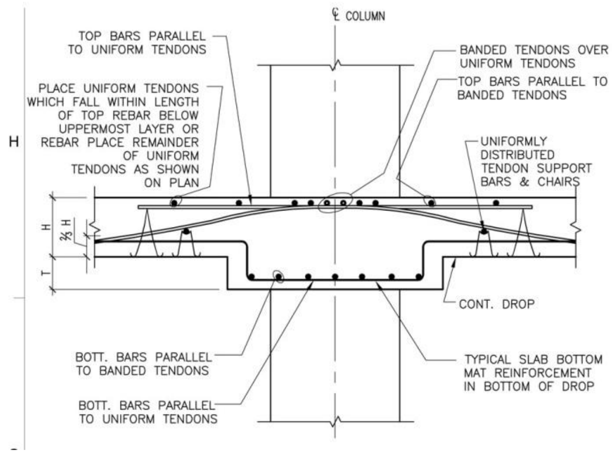


Figure 14: Section through shallow beam and column showing pre stressing tendon configuration

Structural

The existing structure with a foundation of core drilled piers and grade beams and a lateral system of cast-in-place shear walls would remain.

Serviceability

Deflection calculations were typically controlled by using Tables 9.5(a) and 9.5(c) ACI 318-08, with all systems falling within the limits for deflection. However, relative deflection from system to system was hypothesized based upon the known system properties, which could have some merit for choosing a system over another. It is thought that, due to the load balancing effects of the pre stressing tendons, the post tensioned slab will have the best deflection performance.

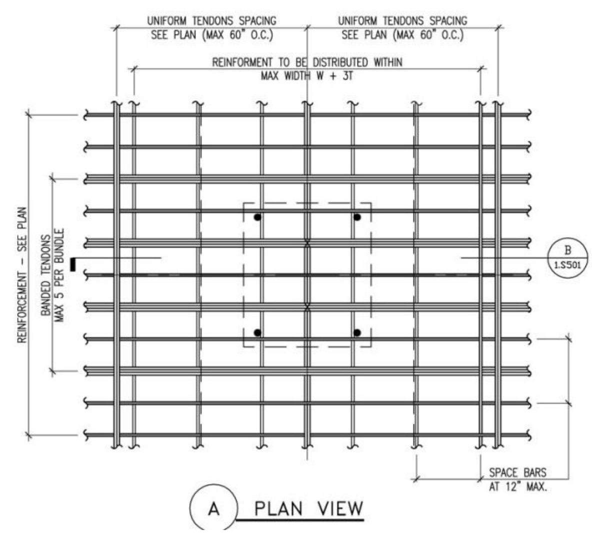


Figure 15: Plan view showing banded and uniform tendons through a column

Similarly, vibrational analyses were not performed for this report, but common knowledge of how these systems perform relative to each other was applied. The post tensioned slab has a relatively thick slab comparatively, so this additional mass plus the inherent damping properties of concrete means that it will perform well with respect to vibration control.

Construction

Constructability was similarly given a relative rating based upon the difficulty anticipated with each system. The post tensioned slab was given a rating of medium, as the proper installation of pre stressing tendons and then the jacking process require additional equipment, expertise, and precision than a typical mildly reinforced concrete slab.

Summary

The existing system is a cost effective, shallow system that has few negatives.

Two Way Slab with Drop Panels

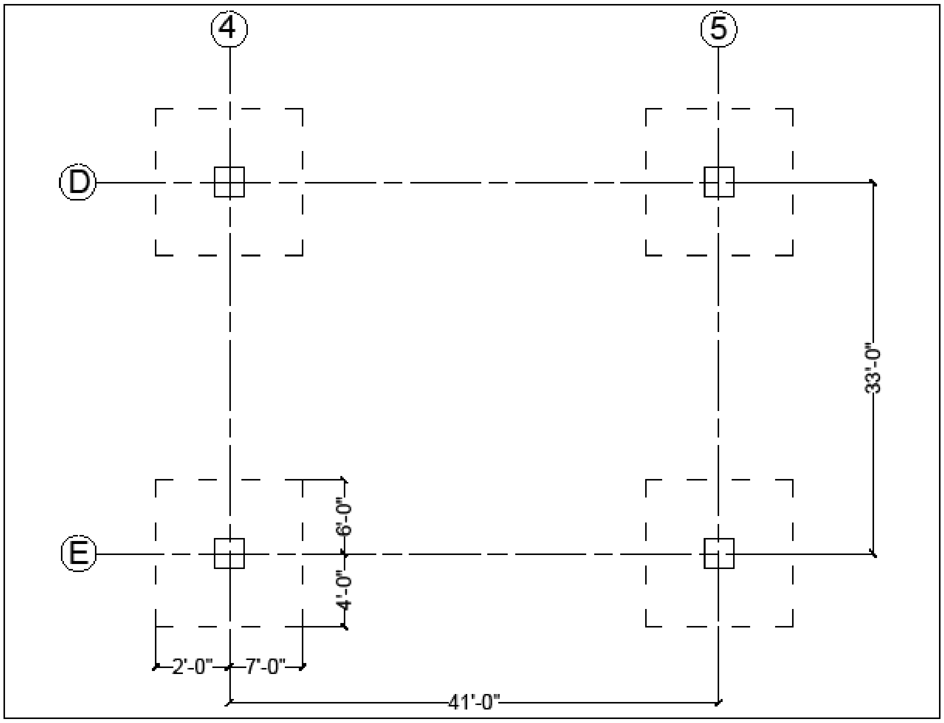


Figure 16: Plan view of two way slab with drop panels

Next a two way flat slab with drop panels was designed. A comparison between system performances without pre stressed reinforcement seemed a logical step, particularly to evaluate if the slab depth, one of the main advantages of post tensioned systems, will have a large difference. The thickness of the slab was designed as 13", drop panels were designed to extend 4" below the adjacent slab (t/4) and extend l/6 in the direction of the spans (dimensions shown in Figure can be mirrored).

General

The total depth of the flat slab with drop panel system was 17", which matches the post tensioned slab. However, the slab thickness was 13", a result of Table 9.5(c) ACI 318-08 to control deflections. It's possible that a more competitive slab thickness could have been achieved, especially with the drop panels aid in the negative moment region, though this was not explored. This overall greater thickness led this system to being the heaviest at 163 psf. The system cost a competitive $19.21/SF, though with no clear advantages over a post tensioned system to this point it would be difficult to justify the cost increase.

Architectural

This two way system would have the potential to see some architectural impact. The floor thickness is barely affected, so the height of the building would likely not be changed. The cantilevered slab however would likely prove an issue, as the slab would tend to act in a one-way manner. According to Table 9.5(a) ACI 318-08, l/10 is necessary for a cantilever which would require a thickening of the slab in this region.

Structural

This is by far the heaviest system, which means it would have one of the larger impacts structurally. The seismic force would increase due to the additional weight which may require adjustments to the lateral system. Additionally the foundations would have to be looked at for adjustment for both the additional dead load and the increased lateral load.

Serviceability

This system will likely perform as well if not better with regards to vibration control than the post tensioned system due to its additional mass. With regards to deflection it was hypothesized that relative to the other systems it would be ranked 3rd, the two way deflection over these long spans with the heavy self-weight are considered in this, as is the lack of beams to stiffen the slab.

Construction

This would be perhaps the easiest system to construct. No additional fireproofing would be required and no post tensioning expertise combined with little formwork would make this a very easy system to put up. The lack of post tensioning may also make it quicker to build and shorten the schedule slightly.

Summary

While the two way slab with drop panels does not perform better than the post tensioned system in any category except perhaps vibration control and construction schedule, the extent of these out performances are not easily quantifiable and would not justify the increase in cost, weight, and the architectural effects with the cantilever slab, though the system is in theory viable, especially if more in depth analysis on deflections were performed to thin the slab thickness.

One Way Slab with Beams

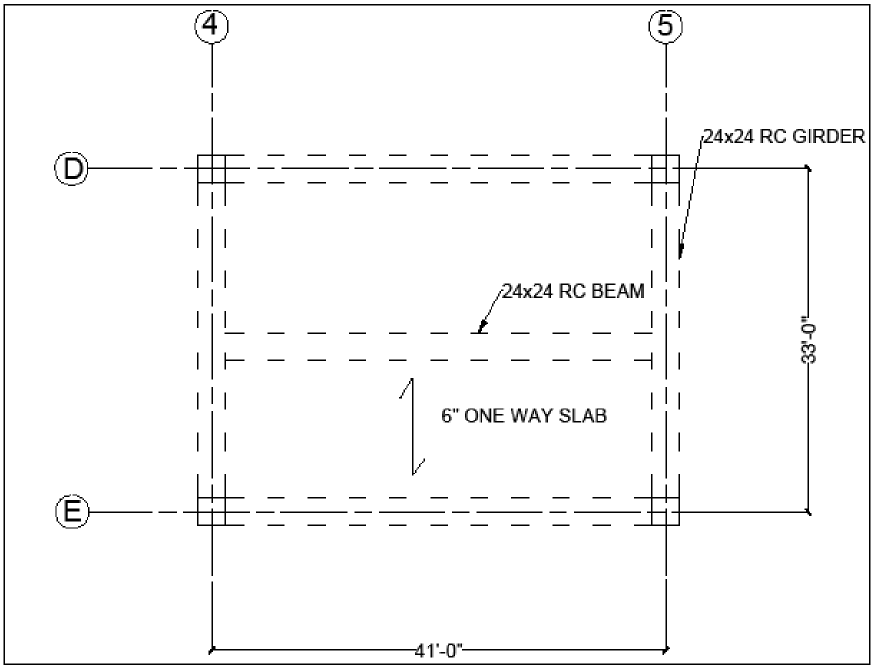


Figure 17: Plan view of one way slab with beams

A one way slab system with beams was considered a second viable replacement. While the cost would increase the system had potential savings with weight, which could reduce foundation and lateral system sizes. It was determined that this bay size was too large to practically consider a one way slab unless an infill beam was added. The beam was designed in the long direction, so that the girders would have the shorter span and so that the one-way slab would also be spanning in a shorter direction and be able to be kept to a minimal thickness. A width of 24" was assumed for both beams and girders considering formwork as the columns are typically 24x24.

General

This system had a slab depth of only 6", achieved by checking deflection, but when factoring in the infill beam that had to be added to make the one way span possible the system weight was approximately 125 psf, not a drastic weight reduction. Additionally the system cost an expensive $22.30/SF and had a total depth of 24" due to the beams and girders. This means that there is the potential of adding 8" per floor should the plenum space be required.

Architectural

The one way slab would have one of the largest architectural impacts. Not only would the height of the building have the potential to be increased by some 8" per floor, but the cantilever slab would likely not be possible unless the span was reduced. The code requirement to control deflection would lead to a 17" slab that might still have trouble dealing with the negative moment at the face of the cantilever.

Structural

This system compares similarly to the original system in terms of weight, though as it is lighter there may be some potential to reduce certain aspects of the foundation and/or lateral system, though unlikely.

Serviceability

This system has comparable mass to the first two systems, most of which is in the beams however so while it is felt the system would still do well for controlling vibrations it wasn't quite as good comparably as the first two due to its thin slabs.

The large infill beam that breaks up the span into two smaller more manageable distances is believed to create a system that will have the second best relative deflection.

Construction

As the system is still concrete, fire proofing does not need added. The construction process would be slowed down by the amounts of formwork, and while an effort was made to keep these consistent, this would also impact the ease of construction which was deemed 'medium'.

Summary

The one way slab saves on weight, but adds too much depth and cost to be viable. Cantilevering the slab edge 14' would likely prove too much of a challenge for this system.

Composite Deck on Composite Steel Beams

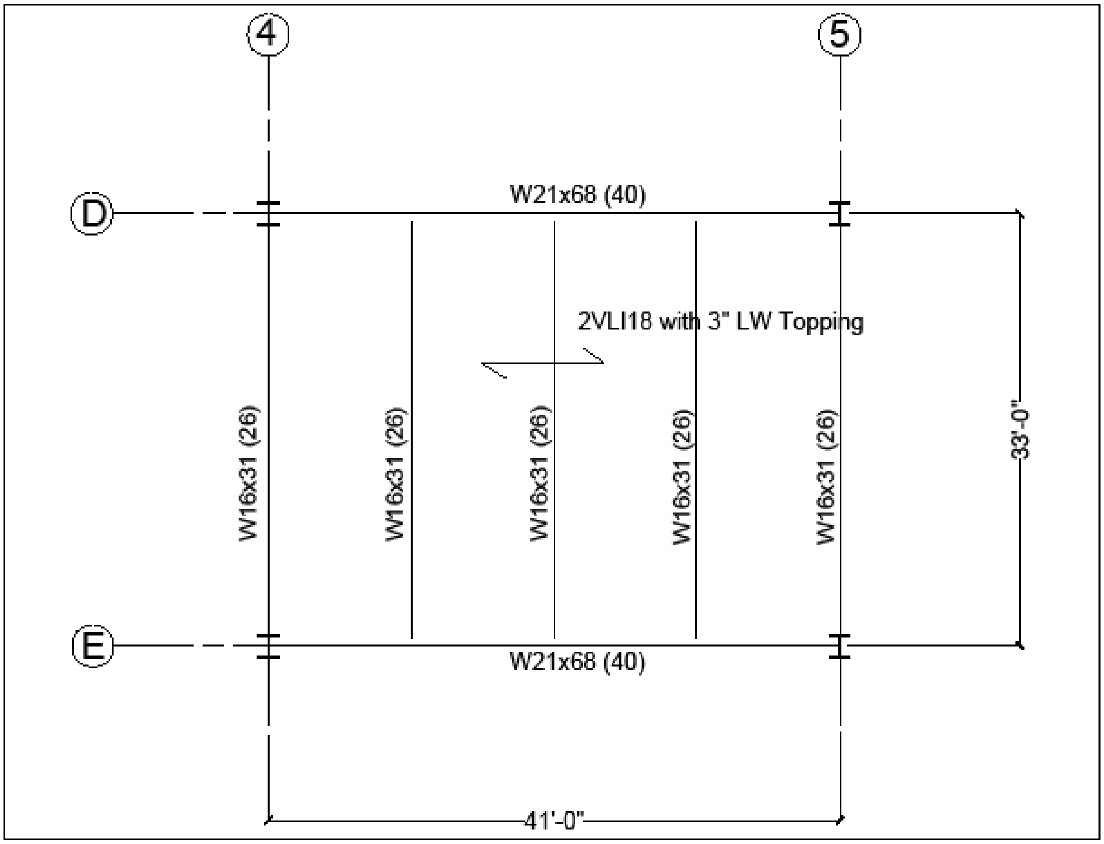


Figure 18: Plan view of composite slab on composite steel beams

The final system analyzed was a composite deck on composite steel beam and girders. As Rockville is in an area typically dominated by concrete construction this was looked at with interest from both the cost per square foot and the overall depth of the floor system versus how much lighter this would make the system. Composite beams were chosen in an effort to get smaller sizes and depths for the long spans as the slab would work integrally with the beams.

General

The deck selected was 2 VLI18 with a 3" light weight topping. This was chosen as the deck and slab work compositely, which is hoped would keep the slab as thin as possible. The total cost was $23.34/SF which is the most of any system. The total system depth is also the worst of all four systems, at 26", which could add `10" onto each floor potentially. The advantage of this system was found in its weight, only 44 psf.

Architectural

The added depth would have the potential to increase the height of the building significantly, as was the case with the one-way slab, though the steel beams can have mechanical systems run through them which is not the case with the one way slab. Additionally the cantilever section would not be possible without adding steel beams for support that would require expensive moment connections.

Structural

This system would have the largest structural impact. The overall weight of the floor system decreased 70%, and since this is where most of the mass in the building is found the building would be considerably lightened. This would have the possible impact of lightening the foundation and/or lateral system which would see reduced seismic force. Additionally, as steel is being used there would be the possibility to explore steel braced frames and/or moment frames instead of the existing cast-in-place shear walls.

Serviceability

Steel is known to have vibration problems, so this system was hypothesized to be the worst when it comes to controlling vibrations.

This system was also thought to be the worst at deflection, as the composite deck system is almost at its load capacity for the span of 12'-0", and the inertia of the steel beams and girders is relatively small compared to the concrete beams and girders. Additionally the steel beams are assumed as pinned connections, which imply greater rotation at the joint and greater deflection at midspan than a fixed case, as is assumed with the concrete beam system.

Construction

This system would be easy to construct, with possible time saved on the schedule due to the ease of erection of steel and the lack of formwork required with the metal deck. The metal deck was also designed unshored, which would allow for greater speed and economy. The steel beams and girders would need fireproofing to be brought up to the necessary fire rating.

Summary

The reason this structure is viable is because of the drastic reduction in weight. The potential cost savings in foundation and the lateral system would potentially out-weigh the vibration, depth, and cost issues; though this is seen as unlikely.

System Summary

Consideration		System			
		Post-Tensioned	Two-Way Flat Slab with Drop Panels	One-Way Slab with Beams	Composite Steel
General	Weight(psf)	137	163	125	44
	Cost($/SF)	18.74	19.21	22.3	23.34
	Floor Depth (in) (Slab) Total	(9) 17	(13) 17	(6) 24	(5) 26
Architectural	Fire Rating	2 hr	2 hr	2 hr	2 hr
	Other	N/A	Cantilever section would likely require a thickened slab due to one-way action	Could add 8" of height per floor, could not cantilever off the edge without significant adjustment	Could add 9" of height per floor, cantilever would require moment connections
Structural	Foundation	Core-drilled piers and grade beams	Require heavy foundations, larger core drilled piers and grade beams	Potential for a slight reduction in foundation sizes	Likely to reduce the foundation size to some degree
	Lateral System	Cast-in-Place Shear Walls	Shear Walls	Shear Walls	Exploration of steel braced/moment frames
Servicability	Deflection (rated on an anticipated relative scale)	1	3	2	4
	Vibration Control	Very Good	Very Good	Good	Poor
Construction	Additional Fire Protection	None	None	None	Spray on for beams/deck
	Schedule	N/A	May reduce schedule	Potentially increase schedule	May reduce schedule
	Constructability	Medium	Easy	Medium	Easy
Viability		N/A	Yes	No	Yes

Figure 19: System comparison

AECOM Floor Exploration

In the design narrative there was a portion of the structural systems report dedicated to the discussion of how a post tensioned slab was decided upon and what other systems were considered. AECOM looked at a composite steel and a skip-joist system in addition to the post tensioned slab, with the three main criterion of cost, vibration, and ease of future modification. The skip joist system was outdated, so it came down to the composite steel and post tensioned systems. The driving factor in the design became the addition of the large cantilever which would require moment connections and a thicker slab depth which would affect the floor to floor height at this portion of the building, while the post tensioned slab deals favorably with the cantilevers.

Conclusion

In conclusion, Technical Report 2 analyzed the existing floor system and compared it to three alternative floor systems, each selected under the pretense that their merits would make them a viable option conceptually that could be further evaluated through more in depth analyses. The systems were compared on several factors, the most important of which being cost, system depth, and weight of the system.

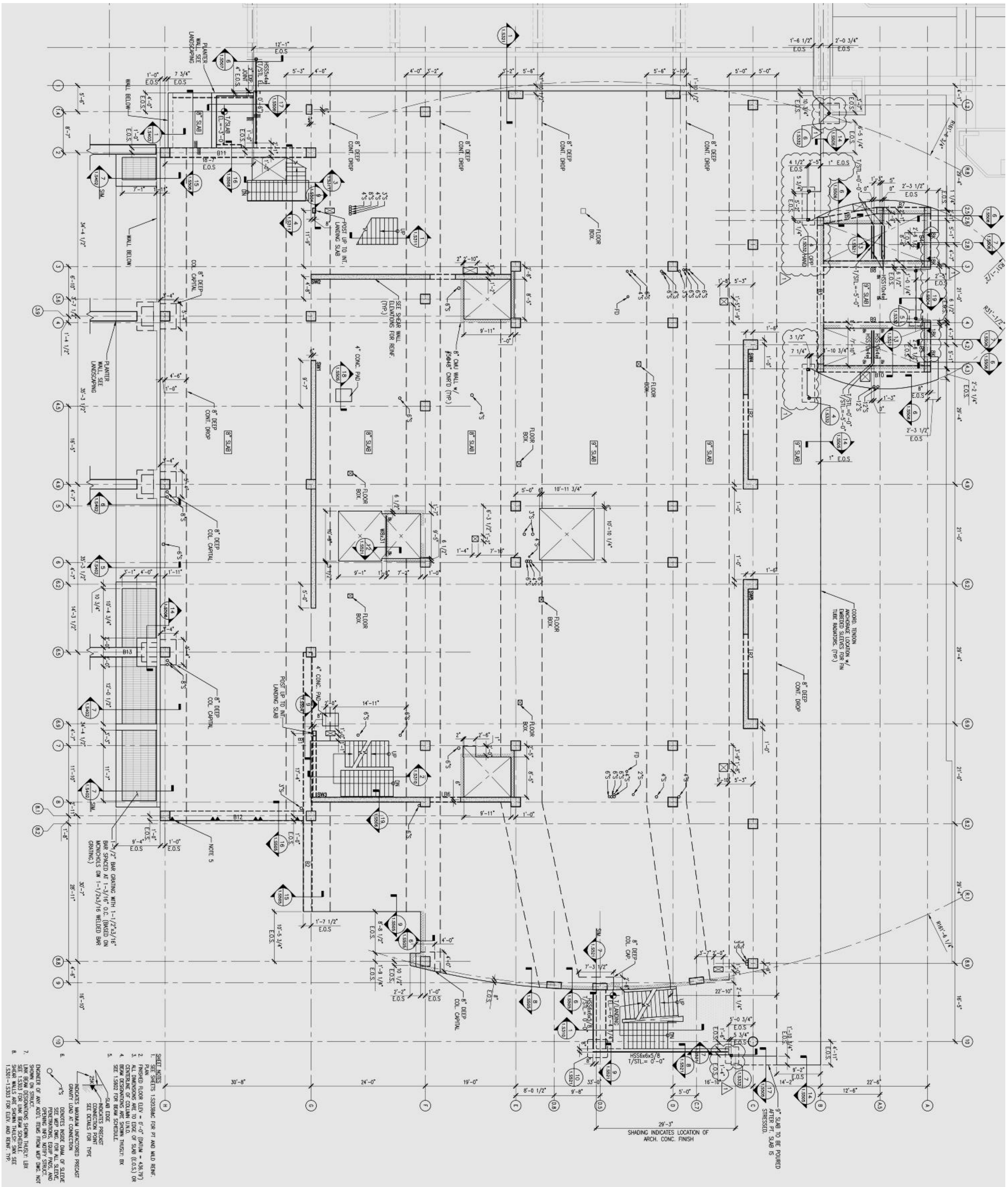
The existing post tensioned slab with wide-shallow beams was found to be the most favorable system as it kept the lowest depth at 17" and was the least expensive. The nature of the system made the 14' cantilever easily achievable without an increase in slab depth. It was hypothesized that it would be effective for both minimal deflection and a large amount of vibration control. The system was one of the heaviest of the four, weighing 137 psf, which is the only conceived negative.

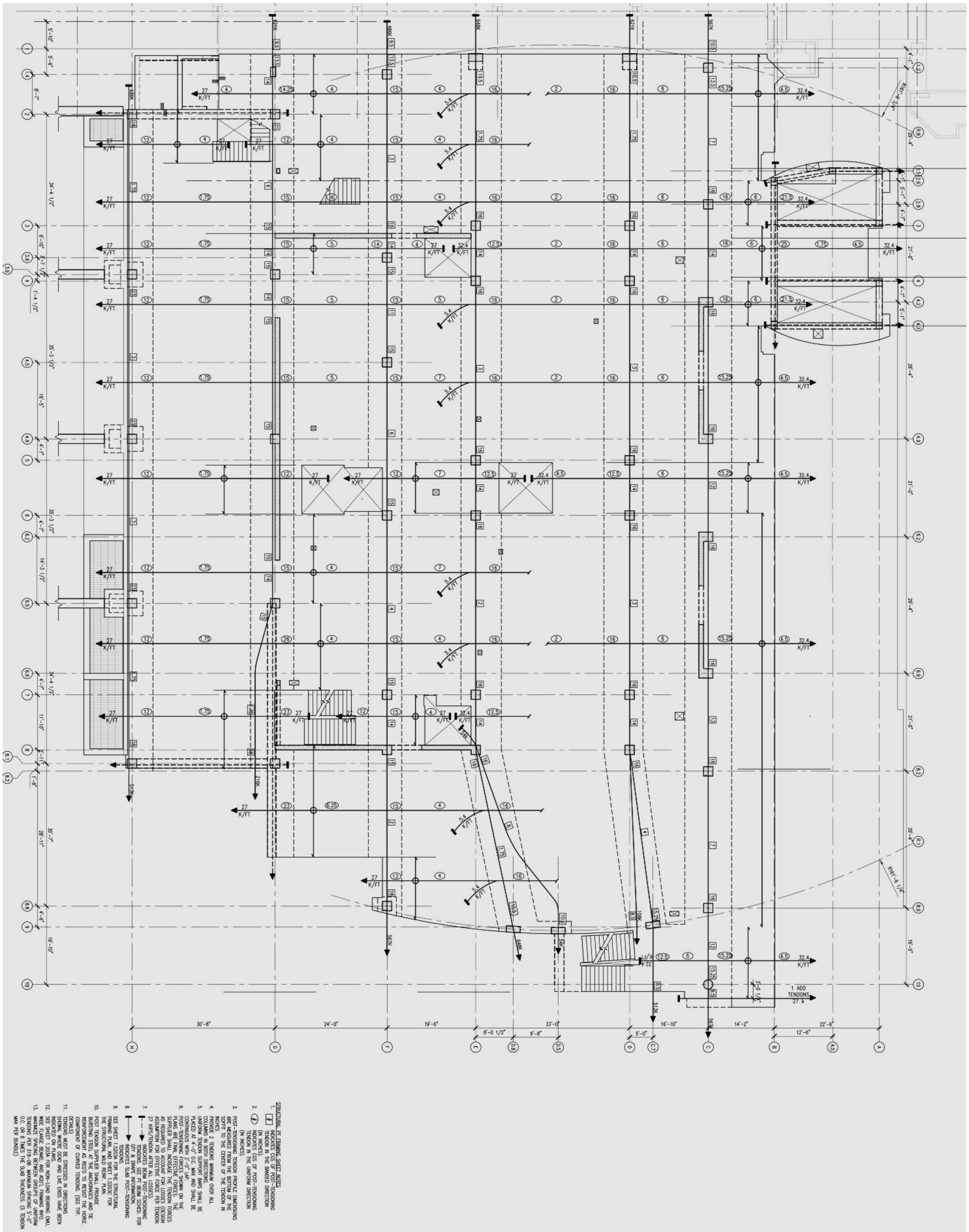
The two way slab had a remarkable total system depth of only 17", however the slab is approximately 50% thicker leading to a much heavier system that would have negative effects on the lateral and foundation systems. The system otherwise compared favorably and would be viable with an increase in vibration control.

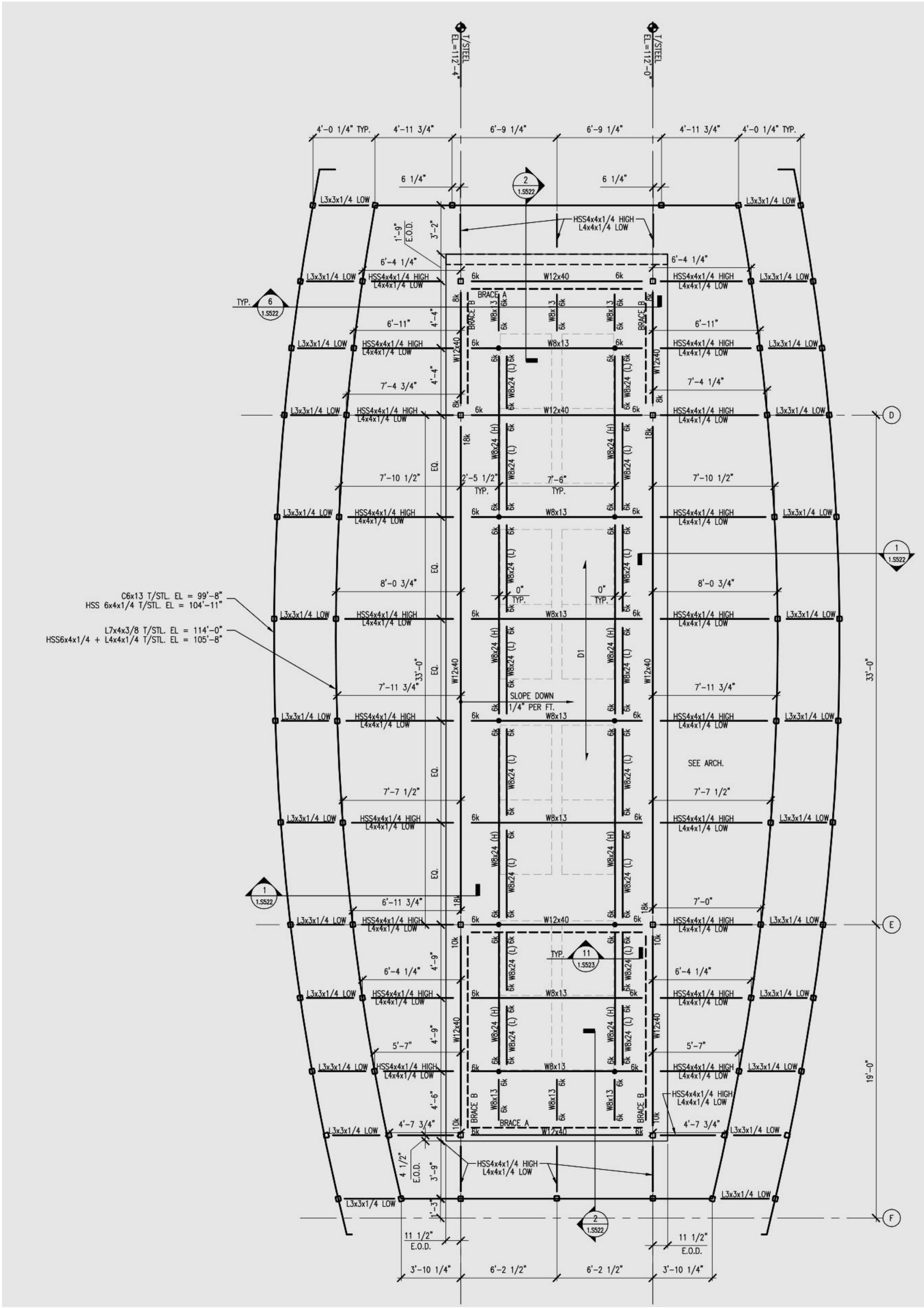
The one way slab system was the only system that was deemed unfeasible due to the much larger system depth, 24", which does not justify its slight decrease in weight and cost.

The composite steel system was considered viable as it had a drastic reduction in weight, only 44 psf, which could give cost savings in the foundation and lateral systems to offset the cost of the floor assembly. The system would be quick to construct, but would likely have difficulty with the slab cantilever sections that make it a poor choice.

Appendix A: Typical Plans





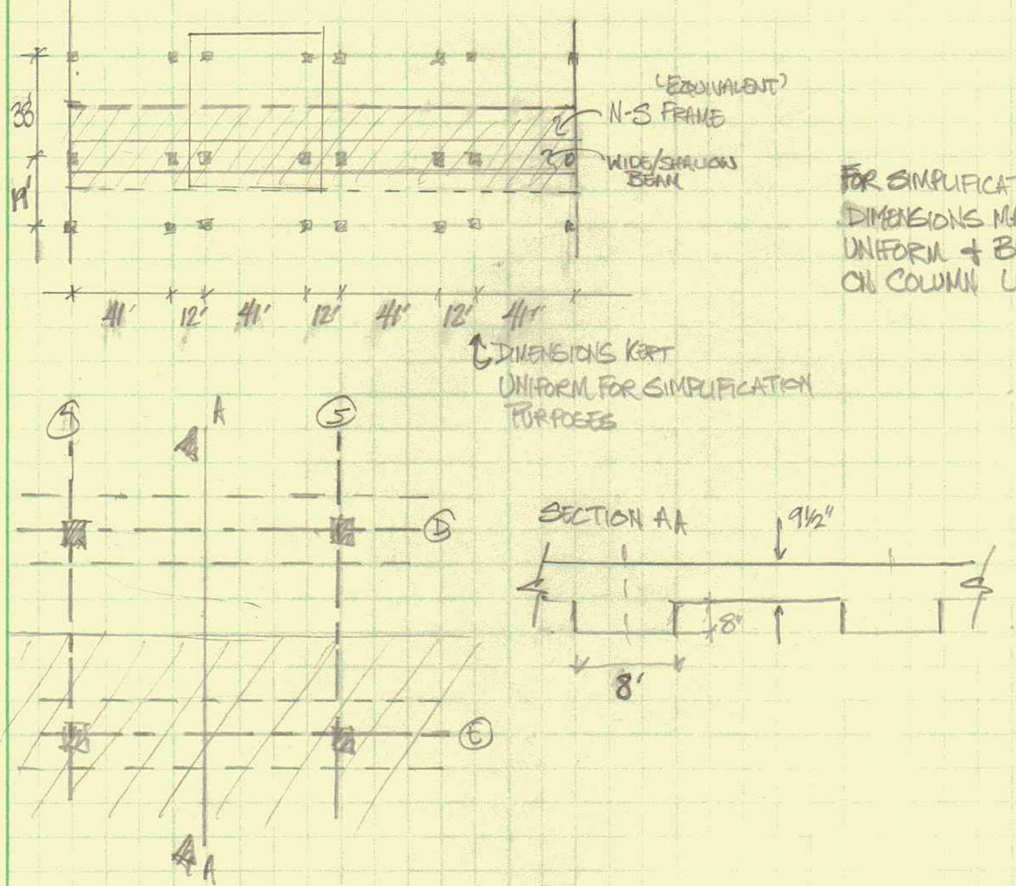


Appendix B: Post Tensioned Slab

JAKE WIEST | GRAVITY CHECK

IDEALIZED SLAB PLAN (LEVELS TERRACE–5)

COLUMNS – 24"x24"
FLR TO FLR – 16'



FOR SIMPLIFICATION DIMENSIONS MADE UNIFORM + BMS CTR'D ON COLUMN LINE

SLAB THICKNESS

$h = l_n/45 = \frac{39 \times 12}{45} = 10.4 \rightarrow 10\frac{1}{2}"$ DESIGNER USED 9"

IN SOME INSTANCES SPAN TO DEPTH FOR PT $\rightarrow l_n/50 = 9\frac{1}{2}"$, CLOSER TO 9"

USE 9"

$W_D = \frac{9}{12} \times 150 = 112.5$ psf

W_D OF BM' $= \frac{8}{12} \times 150 = 100$ psf (OVER 8') $\rightarrow \frac{100 \times 8}{27} = 30$ psf

$W_L = 100$ psf

$\frac{LL}{DL} \frac{100}{142.5} = .7 \leq \frac{3}{4}$

FULL LIVE LOAD (NO PATTERNS)

APPROX FLEXURAL STIFFNESS OF COLUMNS

$K_c = \frac{4E_c I_c}{L-2t}$ $E_c = E_s = 1.0$ $I_c = \frac{24(24)^3}{12} = 27{,}648 \text{ in}^4$

$L = 16 \times 12$

$= \frac{4(27648)}{16\times12 - 2(9)} = 636\, E_c$ in-lb

TORSIONAL STIFFNESS OF WIDE/SHALLOW BM + SLAB

IN THIS CALC WIDE SHALLOW BM NEGL.

$C = (1 - .63\ 9/24)(9^3(24)/3) = 4454$

$K_t = \frac{9E_c\, 4454}{(19\times12)[1 - 24/(19\times12)]^3} + \frac{9E_c\, 4454}{(33\times12)[1 - 24/(33\times12)]^3} = 368\, E_c$ in-lb

SLAB STIFFNESS

$K_s = \frac{4E_{cs} I_s}{l_n - c_1/2}$ $I_s = I_{BM} + I_s - I_{s\text{ WHERE BM WAS}}$

$= \frac{(8\times12)(17^3)}{12} + \frac{(26\times12)(9^3)}{12} - \frac{(8\times12)(9^3)}{12}$

$= 52426 \text{ in}^4$

$K_{s,\,12'\text{SPAN}} = \frac{4E_c(52426)}{(12\times12) - 24/2} = 1588\, E_c$

$K_{s,\,41'} = \frac{4E_c(52426)}{(41\times12) - 24/2} = 437\, E_c$

DISTRIBUTION FACTORS

(4) → ← (5)

$D_F = \frac{K_s}{\Sigma K}$ $\Sigma K = K_{ec} + K_{sl} + K_{sr}$ $K_{ec} = \frac{1}{\Sigma K_c} + \frac{1}{K_t} = \frac{1}{2(636)} + \frac{1}{368} = 285\, E_c$

$$D_F = \frac{437}{437+285+1589} = .189$$

$\leftarrow 4 \quad 5 \rightarrow$

$$D_F = \frac{1589}{437+285+1589} = .688$$

SECTION PROPERTIES (ACCOUNTING FOR WIDE/SHALLOW BM)

$A = bh = 12 \times 18(9) + 8 \times 12(17) = 3576 \text{ in}^2$

$S = bh^2/6 = \frac{(12 \times 18)(9)^2}{6} + \frac{(8 \times 12)(17)^2}{6} = 7540 \text{ in}^3$

DESIGN PARAMETERS

AT TIME OF JACKING
$f'_{ci} = 3000$ psi

COMP: $= .6 f'_c = 1800$ psi
TENS $= 3\sqrt{f'_c} = 164$ psi

AT SERVICE
$f'_c = 5000$ psi

COMP: $.45 f'_c = 2250$ psi
TENS: $6\sqrt{f'_c} = 424$ psi

AVG. PRECOMPRESSION LIMITS

$P/A = 125$ psi MIN
$= 300$ psi MAX

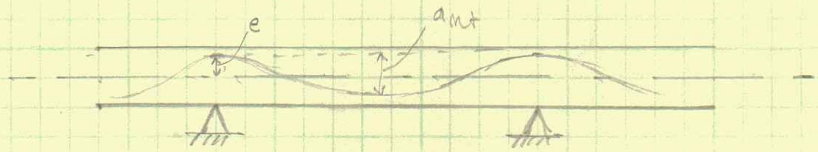
COVER REQ'D

RESTR: 3/4" BOTTOM
UNRESTR: 1.5" BOTTOM
3/4" TOP

TARGET LOAD BALANCE

$3/4\, w_D = 142.5(.75) = 107$ psf

TENDON PROFILE



LOCATION FROM BOTTOM
INT. SUPPORT : 16"
INT. SPAN : 1"

$a_{int} = 16 - 1 = 15"$

$a_{end} = (16 + 8.5)/2 - 1" = 11.25$

No e @ end

ASSUMED ENDSPAN

FORCE REQ'D TO BALANCE 75% OF DL.

$W_b = 107 \times 26' = 2.8$ klf

FORCE NEEDED IN TENDONS

$P = w_b L^2 / 8 a_{end} = 2.8(41)^2 / 8(\frac{11.25}{12}) = 628$ k

ASSUMING
$d = 1/2"$, 7 WIRE-STRAND
$A = .153 in^2$
$f_{pu} = 270$ ksi
$f_{py} = 243$ ksi
$f_{pe} = 159$ ksi

$.153 \times 159 = 24.3$ k/tendon

$628/24.3 = 25.8$ TENDONS → 26 TENDONS

$P_{ACTUAL} = 631.8$ k

$W_B = \frac{631.8}{628}(2.8) = 2.82$ k/ft

$\frac{P_{ACTUAL}}{A} = \frac{631.8 \times 1000}{3576} = 177$ psi > 125 psi, < 300 psi ∴ ok

INTERIOR

$P = 2.8(41^2)/(8 \times 16/12) = 442$ k < 682 k ✓

$W_b = 682(8)(16/12)/41^2 = 3.82$ klf

$W_D = 3.7$ klf < 3.82 NO GOOD

BALANCE 70% = $.7(142.5)(26) = 2.6$ klf

$P = 2.6(41^2)/8(11.25/12) = 583$ k

USE 24 TENDONS

$W_b = 583(8)(16/12)/41^2 = 3.52$ ✓

INTERIOR = $P = 2.6(41^2)/8 \times 16/12 = 409 < 583$

$\frac{583 \times 1000}{3576} = 163$

$FEM = w l^2/12$

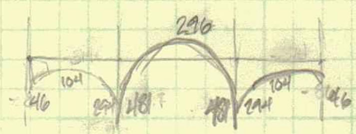
DL MOMENT

$\frac{3.7 \times 41^2}{12} = 518 \text{ k-ft}$

$\frac{3.7 \times 12^2}{12} = 44.4 \text{ k-ft}$

		4		5		
	.688	.688	.189	.189	.688	.688
FEM	-44.4	+44.4	-518	+518	-44.4	+44.4
DISTR.	+80.5 →	15.25			-5.25 ←	-80.5
		-443		443		
	152.5 ←	+305	+88.7	-83.7	-805 →	-157.5
	-104.9 →	-53	41.9	+41.9	53 ←	104.9
		+293.8		-293.8		
	101 ←	202.1	56	+56	+202.1 →	101
	+70 →	+35	28	-28	35 ←	-70
		-251		+251		
	86 ←	172	47	-47	-172 →	-86
	-43 →	-28	-24	24	22 ←	+43
	-45.9	+294	-481	+481	-294	+45.7

$M^+ = \frac{3.7(41^2)}{8} - 480.7 = 296 \text{ k-ft}$



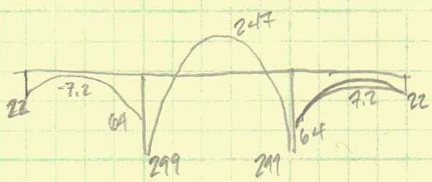
LL MOMENT

$\frac{2.6 \times 41^2}{12} = 364 \text{ k-ft}$

$\frac{2.6 \times 12^2}{12} = 31.2 \text{ k-ft}$

	4		5		
.688	.688	.189	.189	.688	.688
-31.2	+31.2	-364	+364	-31.2	+31.2
21.5 →	10.7			-10.7 ←	-21.5
	+322.1		+322.1		
+111 ←	+222	+60.9	-60.9	222 →	+111
-76 →	-38	30.5	-30.5	+38 ←	+76
	276		-276		
-95 ←	-190	-52	52	190 →	95
48 →	28	26	-26	-24 ←	-48
-22	64	-299	299	-64	22

$M^+ = \frac{2.6(41^2)}{8} - 299 = 247$



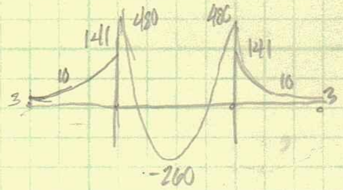
BALANCING MOMENT

$W_b = -3.52\ klf$

$\frac{Wl^2}{12} = \frac{-3.52(41^2)}{12} = -493$

$\frac{-3.52(12^2)}{12} = -42$

		4		5		
.688	.688	.189		.189	.688	.688
-42	+42	-493		+493	-42	+42
~~29~~ →	15				-15	← -29
		-436		436		
150 ←	300	82		-82	-300	→ -150
-103 →	-52	-41		41	52	← +103
		289		-289		
-100 ←	-199	-55		55	199	→ 100
69 →	35	28		-28	-35	← -69
3	141	-480		480	-141	3



$M = \frac{3.52(41)^2}{8} - 480 = 260\ ft\,k$

STAGE 1 (DL + Pi) ONLY CONSIDERING 41' SPAN

MIDSPAN

$f_{TOP} = (-296 + 260)\frac{12 \times 1000}{7540} - 163 = -220$ COMP < 1800 ok

$f_{BOT} = (296 - 260)\frac{12 \times 1000}{7540} - 163 = -105$

SUPPORT

$f_{top} = (481 - 480)\frac{12 \times 1000}{7540} - 163 = -161$ COMP < 1800 ok

$f_{bottom} = (480 - 481)\frac{12 \times 1000}{7540} - 163 = -164$

STAGE 2 (DL+LL+Pi)

MIDSPAN

$f_T = (-296 - 247 + 260)\frac{12 \times 1000}{7540} - 163 = -613 < 2250\ psi$ ok

$f_B = (296 + 247 - 260)\frac{12 \times 1000}{7540} - 163 = 287 < 424$

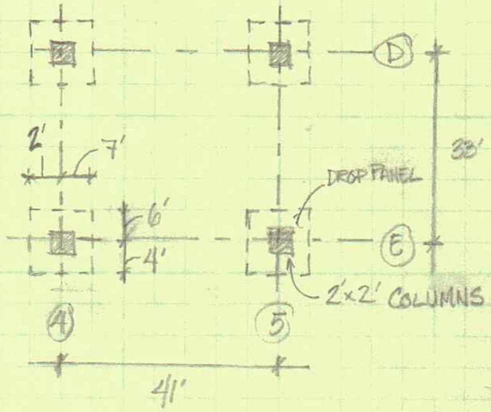
SUPPORT

$f_T = (481 + 299 - 480)\frac{12 \times 1000}{7540} - 163 = 314 < 424\ psi$

$f_B = (-481 - 299 + 480)\frac{12 \times 1000}{7540} - 163 = -640 < 2250\ psi$ ok

Appendix C: Two Way Slab with Drop Panels

JAKE WIEST | TWO-WAY FLAT SLAB W/ DROP PANELS



DROP PANELS – 1/6 SPAN LENGTH, 1/4 SLAB DEPTH

TABLE 9.5(c)

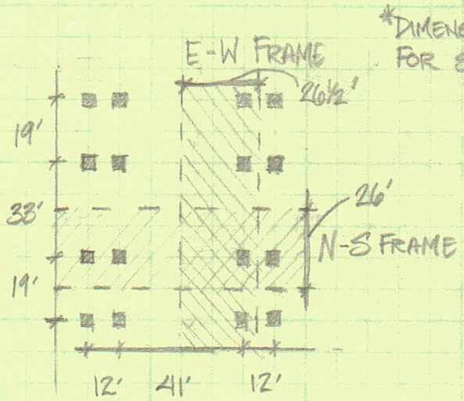
$l_n/36 = 39\times12/36 = 13''$

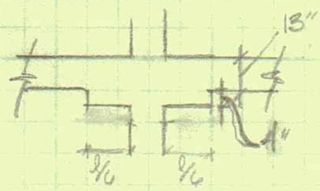
MAKE $t = 13''$

*DIMENSIONS MADE UNIFORM FOR SIMPLICITY

FLR to FLR = 16'

$f'_c = 5000$ psi





DROP PANEL TOTAL DEPTH = 17"

$W_D = 13/12 \times 150 = 162.5\text{ psf} + 15 = 177.5\text{ psf}$

MEP

$W_L = 100$ psf

$\frac{LL}{DL} = \frac{100}{177.5} = .56 > 3/4 \therefore$ No PATTERNS

N-S FRAME

FLEXURAL STIFFNESS OF COLUMNS

$K_c = \frac{4E_cI_c}{L-2t}$ $E_c = E_s = 1.0$ $I_c = \frac{(24)(24)^3}{12} = 27,648\text{ in}^4$

$\frac{4(27648)}{16\times12-2(13)} = 667$ in-lb

TORSIONAL STIFFNESS OF SLAB

$C = (1 - .63\,^{13}/_{24})(13^3(24)/3) = 11578$

$K_t = 2 \times \dfrac{9E_c(11578)}{(26\times12)\left[1-\frac{24}{(26\times12)}\right]^3} = 849 \text{ in-lb}$

$\dfrac{1}{K_{ec}} = \dfrac{1}{\Sigma K_c} + \dfrac{1}{K_t} = \dfrac{1}{2\times667} + \dfrac{1}{849} \rightsquigarrow K_{ec} = 519 \text{ in-lb}$

SLAB STIFFNESS

$K_s = \dfrac{4E_{cs}I_s}{l_n - c_1/2}$ $\qquad I_s = \dfrac{(26\times12)(13^3)}{12} = 57{,}122 \text{ in}^4$

$K_{s_{12'\text{SPAN}}} = \dfrac{4E_c(57{,}122)}{(12\times12) - 24/2} = 1731$ $\qquad K_{s_{41'}} = \dfrac{4E_c(57{,}122)}{(41\times12) - 24/2} = 476$

E-W FRAME

$K_c = 667$

$K_t = 2\times\dfrac{9E_c(11578)}{(26.5\times12)\left[1-\frac{24}{26.5\times12}\right]^3} = 829$ $\qquad I_s = \dfrac{(26.5\times12)(13^3)}{12} = 58221 \text{ in}^4$

$K_{ec} = \dfrac{1}{1/829 + 1/2\times667} = 511 \text{ in-lb}$

$K_{s_{19'\text{SPAN}}} = \dfrac{4E_c(58221)}{(19\times12) - 24/2} = 1078$ $\qquad K_{s_{33'\text{SPAN}}} = \dfrac{4E_c(58221)}{(33\times12) - 24/2} = 606$

DISTRIBUTION FACTORS

N-S

4 → ← 5

$D_F = \frac{K}{\Sigma K} = \frac{476}{476+1781+519} = .175$

← 4 5 →

$D_F = \frac{1781}{476+1781+519} = .635$

E-W

D → ← E

$D_F = \frac{606}{1078+606+511} = .276$

← D E →

$D_F = \frac{1078}{1078+606+511} = .491$

$w_u = 373$ psf ← FT. WIDTH STRIP METHOD

N-S

$M_u = \frac{w_u l_n^2}{12} = \frac{373(41)^2}{12} \times \frac{12}{1000} = 627 \frac{k\text{-in}}{ft}$

$M_u = \frac{w_u l_n^2}{12} = \frac{373(12^2)}{12} \times \frac{12}{1000} = 53.7\ k\text{-in}/ft$

E-W

$M_u = \frac{373(33)^2}{12} \times 12/1000 = 406\ k\text{-in}/ft$

$M_u = \frac{373(19)^2}{12} \times 12/1000 = 135\ k\text{-in}/ft$

	4		5		
.635	.635	.175	.175	.635	.635
-53.7	+53.7	-627	+627	-53.7	+53.7
+34.1	+17.0			-17.0	-34.1
	-539		+539		
+171	+342	+94.3	-94.3	-342	-171
-109	54.3	-47.2	+47.2	+54.3	+109
	+335		-335		
-107	-213	-58.6	+58.6	+213	+107
+67.9	+34.0	+29.3	-29.3	-34.0	-67.9
	-208		+208		
+66	+132	+36.4	-36.4	-132	-66
-41.9	-21.0	-18.2	+18.2	+21.0	+41.9
	+129		-129		
+41.0	-81.9	-22.6	+22.6	+81.9	+41.0
+26.0	+13	+11.3	-11.3	-13	-26.0
	-80.2		+80.2		
+25.5	+50.9	+14.0	-14.0	-50.9	-25.5
-16.2	-8.1	-7.0	+7.0	+8.1	+16.2
	+49.8		-49.8		
-15.8	-31.6	-8.7	+8.7	+31.6	+15.8
+10.0	+5	+4.4	-4.4	-5	-10.0
+15.9	+237.7	-600	+600	-237.7	-15.9

SHORT SPAN 12'		LONG SPAN 41'	
-460	-240	600	+340 k

← POSITIVE MOMENT $\frac{wl^2}{8}$ FEM

ASSUME THAT NEG MOMENTS ON INTERIOR FACE OF SHORT SPAN WOULD BE APPROX EQUAL

DID MOMENT DISTRIBUTION OVER 3-SPANS TO SIMPLIFY IT

	D	D	E	E	
.491	.491	.261	.261	.491	.491
-135	+135	-406	+406	-135	+135
+66.3	+33.1			-33.1	-66.3
	-238		+238		
+58.5	+117	+62.1	-62.1	-117	-58.5
-28.7	+14.4	-31.1	+31.1	+14.4	+28.7
	+184		-134		
-32.9	-65.8	-35.0	+35.0	+65.8	+32.9
+16.5	+8.3	+17.5	-17.5	-8.3	-16.5
	-75.0		+75.0		
+18.4	+36.8	+19.6	-19.6	-36.8	-18.4
-9.0	+4.3	-9.8	+9.8	+4.5	+9.0
	+42.1		-42.1		
-10.4	-20.7	-11.0	+11.0	+20.7	+10.4
+5.1	+2.6	+5.5	-5.5	-2.6	-5.1
	-23.6		+23.6		
+5.8	+11.6	+6.2	-6.2	-11.6	+5.8
-2.8	-1.4	-3.1	+3.1	+1.4	+2.8
	+13.3		-13.3		
-3.3	-6.5	-3.5	+3.5	+6.5	+3.3
+1.6	+.8	1.8	-1.8	-.8	-1.6
-49.9	+232	-387	+387	-232	+49.9

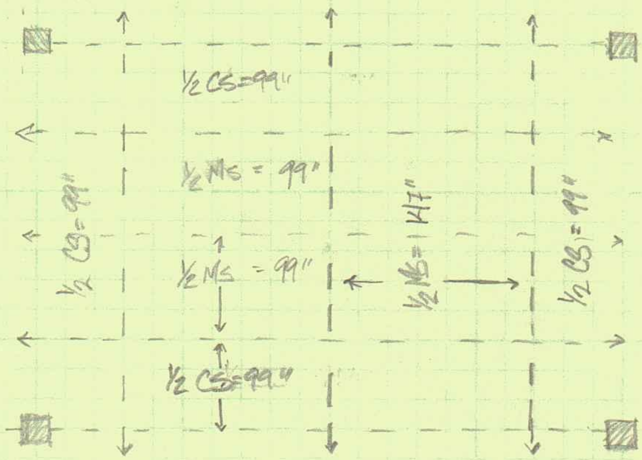
19'		33'	
-32	-232	-387	+222

NO BEAMS, SO 13.6.4.1 : NEG: COLUMN STRIP GETS 75%
MIDDLE STRIP GETS 25%

POS.
CS → 60%
MS → 40%

ONLY DESIGN REINFORCEMENT FOR 33'x41' BAY

$CS = \ell_2/2 = 33 \times 12/2 = 198''$



	NS COL. FACE	NS MIDSPAN	EW COL. FACE	EW MIDSPAN		NS	NS	EW	EW
MS	-150	+136	-97	+89	k-in/ft → k-ft TOTAL IN STRIP :	-103	94	-99	91
CS	-450	+204	-290	+133		-309	140	-199	136

USE #6 BARS : Diam=.75" $a = .44 in^2$

+ NS $d = 13 - 3/4 - .75/2 = 11.875$ EW = 11.125"

– COLUMN STRIP

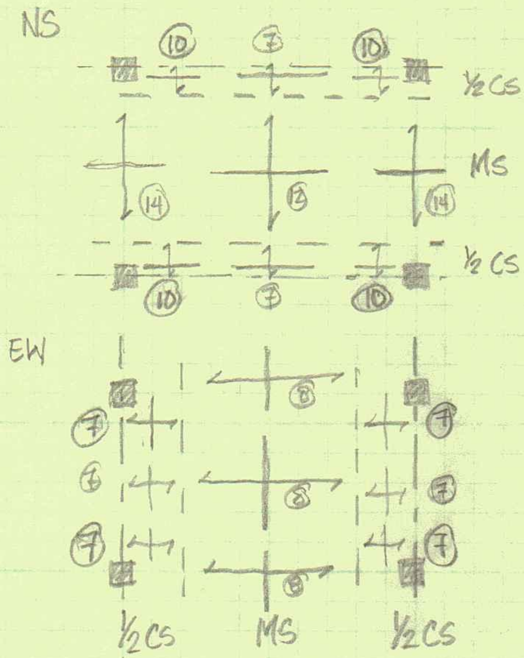
NS $d = 11.875 + 3 = 15.875$ EW = 15.125"

NS

	½ Col Strip M⁻	½ Col Strip M⁺	½ Middle Strip M⁻	½ Middle Strip M⁺
M_u	-309	+140	-150	+136
Width (b)	99"	99"	99"	99"
Effect Depth (d)	15.875	11.875	11.875	11.875
$M_u \times 12/b$	-37.5	+17	-18.2	+16.5
$M_n = M_u/\phi$	-343	+156	-167	+151
$R = M_n/bd^2$	1165	134	144	128
ρ: Table A.5a	.0028	.0023	.0025	.0022
$A_s = \rho bd$	4.4 in²	2.7 in²	2.94 in²	2.59 in²
$A_{s,min} = .0018bt$	3.02	← 2.31 →		
N	10	7	7	6
N_{min} : b/2t	3	← 4 →		3

EW

	½ CS M⁻	½ CS M⁺	½ MS M⁻	½ MS M⁺
M_u	-199	136	-99	91
b	99"	99"	147"	147"
d	15.125"	11.125"	11.125"	11.125
M_n	221	151	110	101
R=	117	148	73	67
ρ=	.0020	.0025	.0012	.0011
A_s	3.00	2.75	1.96	1.80
$A_{s,min}$	3.02	2.32	3.44	3.44
N	7	7	8	8
N_{min}	3	4	6	6


NS
10
7
10
½ CS
MS
14
13
14
½ CS
10
7
10
EW
8
7
7
6
8
7
7
8
7
½ CS
MS
½ CS

PUNCHING SHEAR TWO-WAY SLAB, SO SHOULD CONTROL OVER WIDE BM

$d = \frac{14.875 + 14.125}{2} = 14.5 \longrightarrow d/2 = 7.25$

$V_u = .373\,[26.5 \times 26] = 257\,k$

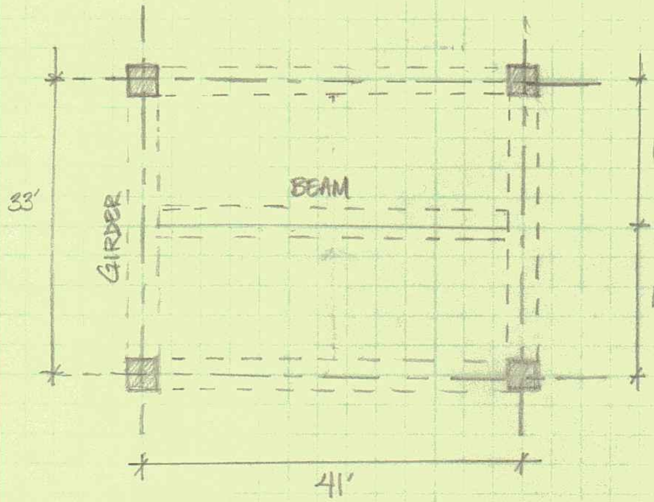
$b_o = 2\,[10 \times 12 + 9 \times 12 + 2 \times 14.5] = 514''$ $\alpha_s = 40,\ \beta_c = 1,\ \frac{b_o}{d} = 35 = 10.6$

$\phi V_c = \left(\frac{\alpha_s}{b_o/d} + 2\right)\sqrt{f'_c}\, b_o d = \left(40/35 + 2\right)\sqrt{5000}\,(514)(14.5)\,1/1000\,(.75)$

$= 1242\,k > 257\,k$

Appendix D: One Way Slab with Mildly Reinforced Beams

JAKE WIEST ONE-WAY SLAB, NON PRESTRESSED BMS



ONE-WAY SLAB SPANNING IN THE SHORT DIRECTION

$f'_c = 5000$ psi

TABLE 9.5(a) → $\ell/28$: $\frac{16.5 \times 12}{28} = 7.07"$ → USE 6" AND CHECK DEFLECTION $d \approx 5"$

$W_L = 100$ psf

$W_D = 15$ psf + $\frac{6}{12} \times 150 = 90$ psf

$W_u = 268$ psf

UNIT STRIP

ASSUMING BMS 24" WIDE

$M_u = \frac{w_u \ell_n^2}{11}$: ACI COEFFICIENT : $\frac{.268\,(14.5)^2}{11} = 5.12$ k-ft

$A_s = M_u/4d = \frac{5.12}{4 \times 5} = .256\ \text{in}^2/12"$ #4 = .2 in² @ 8" = .300 in²/12"

$d = 6 - 3/4 - .5/2 = 5"$

ASSUME $\varepsilon_s > \varepsilon_y$

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{60(.3)}{.85(5)(12)} = .353$ $c = a/\beta_1 = .353/.8 = .441"$

$\varepsilon_s = \frac{.003}{.441}(5 - .441) = .025 > .005 \rightarrow \phi = .9$

$\phi M_n = .9(.300)(60)(5 - .353/2) = 6.5$ k-ft > 5.12 ✓

CRACK CONTROL: $8 \le 15(40/60) - 2.5(.75) = 8.125 > 8$ ✓

TEMP. & SHRINKAGE: $A_T = .0018(12)(6) = .1296\ \text{in}^2$ USE #4 @ 18"

DEFLECTION:

$n = \frac{E_s}{E_c} = \frac{29{,}000{,}000}{57{,}000\sqrt{5000}} = 7.2$ $B = \frac{12}{7.2(.3)} = 5.56$ $I_g = \frac{bh^3}{12} = \frac{12(6^3)}{12} = 216$

$Kd = \left(\sqrt{2(5)(5.56)+1} - 1\right)/5.56 = 1.17$

$I_{CR} = 12\left(\frac{1.17^3}{3}\right) + 7.2(.3)(5-1.17)^2 = 38.1$

DEFLECTION OCCURS @ MIDSPAN

$M = \frac{w l_n^2}{16}$: ACI MOMENT COEFFICIENT

$M_{CR} = \frac{f_r I_g}{y_{bot}} = \frac{7.5\sqrt{5000}(216)}{2.95} = 3.24$ k-ft

$\bar{y} = \frac{12(6)(3) + (7.2-1)(.3)(5)}{12(6) + (7.2-1)(.3)} = 3.05''$

	MIDSPAN	CONT. END
	$M_D = 1.18$ k-ft	$M_D = 1.72$ k-ft
	$M_{D+L} = 2.50$ k-ft	$M_{D+L} = 3.63$ k-ft

$M_{CR} > I_D \;\&\; I_{D+L} \longrightarrow I_e = I_g$ @ POS

$M^{NEG}_{D+L} > M_{CR}$

$I_{e_{D+L}} = \left(\frac{3.24}{3.63}\right)^3 216 + \left[1 - \left(\frac{3.24}{3.63}\right)^3\right] 38.1$

$= 164.6$

$I_{e_{d}} = .7(216) + .15(164.6 \times 2)$

$= 200 \text{ in}^4$

$K = 1.2 - .2\left(\frac{16}{11}\right) = .91$ $\Delta i_{D+L} = \frac{.91(5/48)(2.5)(14.5^2)(12^3)}{57{,}000\sqrt{5000}\,(200)} = .107''$

$\Delta i_d = \frac{1.18}{2.5}(.107) = .051 = \Delta_o$ $\rho' = \frac{A_s}{b \cdot d} = 0$

$\Delta i_L = .107 - .051 = .056''$ $\Delta_{\infty D} = \frac{2}{1+50\rho'}\Delta i_D = .102''$

$\Delta_\infty = \Delta i_D + \Delta i_L + \Delta_{\infty D} + \Delta_{\infty LS}$ $\Delta_{\infty LS} = 2(.056)(.5) = .056''$

$\Delta_\infty = .265''$ $\Delta_a = .214''$

$\frac{16.5' \times 12''}{480} = .4125'' > .214''$ ✓

BEAM DESIGN

SPAN = 41' TABLE 9.5(a) = $\ell/21 \rightarrow \frac{41 \times 12}{21} = 23.4''$ USE $24'' = h$

ASSUME $b = 24''$, FORMWORK FOR COLUMNS

$W_u = 1.2[90 \times 16.5 + 4 \times 150] + 1.6[100 \times 16.5] = 5.14$ klf

EST. SW. WT.

$M_u = \frac{w_u \ell_n^2}{11} = \frac{5.14(39^2)}{11} = 711$ k-ft — ASSUMING 24" GIRDER

NEG MOMENT

$d = 24 - 1.5 - .5 - 1.128/2 = 21.5$

#4 STIRRUP

$M_u^+ = 489$ k-ft

$A_{s\,\text{ENDSPAN}}: \frac{M_u}{4d} = \frac{711}{4(21.5)} = 8.26\,\text{in}^2$ (3)#8 + (6)#9 → 8.37 in²

$A_{s\,\text{MIDSPAN}}: \frac{M_u}{4d} \rightarrow 5.69\,\text{in}^2$ (6)#9 → 6 in²

END

$a = \frac{60(8.37)}{.85(5)(24)} = 4.92$ $c = 6.2$

$\varepsilon_s = \frac{.003}{6.2}(21.5 - 6.2) = .0077 > .005$
$\phi = .9$

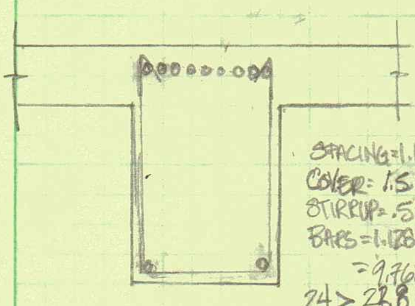
$\phi M_n = .9(8.37)(60)(21.5 - 4.92/2)$
$= 717$ k-ft ✓

MIDSPAN

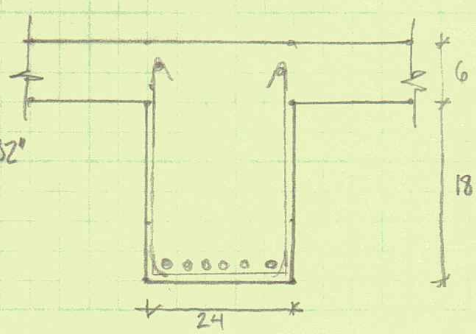
$a = \frac{60(6)}{.85(5)(24)} = 3.53$ $c = 4.41$

$\varepsilon_s = \frac{.003}{4.41}(22.2 - 4.41) = .012 > .005$
$\phi = .9$

$\phi M_n = .9(6)(60)(22.2 - 3.53/2)$
$= 582$ k-ft ✓



SPACING = 1.128"(8) = 9.02"
COVER: 1.5 × 2
STIRRUP = .5 × 2
BARS = 1.128(6) + 3
= 9.768 in
24 > 22.8 ✓



$V_u = \frac{5.12}{2}(39) = 100\,k \longrightarrow 90.5\,k\ @\ d$

$V_s = V_u/\phi - V_c = 90.5/.75 - 2\sqrt{5000}(24)(21.5)/1000 = 48\ k$

$48 = A_v f_{yt}\, d/s \qquad s = \frac{(.2\times2)(60)(21.5)}{48} = 10.75\,in \longrightarrow 10''$

$\phi V_n = 94\ k > 90.5\,k \longrightarrow (2)\#4$ LEGS @ $10''$

GIRDER DESIGN

ASSUME: 24x24, FORMWORK

$P_U = 136k$

SLFWT = .6 klf → .72 klf (factored)

POSITIVE

$M_U = \frac{.72(33)^2}{16} + \frac{136(33)}{8} = 610\ k\text{-}ft$

$A_{S\,MID} = \frac{610}{4(22.2)} = 6.87\ in^2$

(9) #8 = 7.11 in²

NEG.

$M_U = \frac{.72(33)^2}{11} + \frac{136(33)}{8} = 632\ k\text{-}ft$

$A_{S\,END} = \frac{632}{(4)(22.2)} = 7.12\ in^2$

(6) #10 = 7.62 in²

END

$a = \frac{60(7.62)}{.85(5)(24)} = 4.48 \quad c = 5.6$

NOT 22.2 B/ONLY #8's

$\varepsilon_s = \frac{.003}{5.60}(21.4 - 5.60) = .008 > .005$

$\phi = .9$

$\phi M_n = .9(7.62)(60)(21.4 - 4.48/2)$

$= 656\ ft\text{-}k$ ✓

MIDSPAN

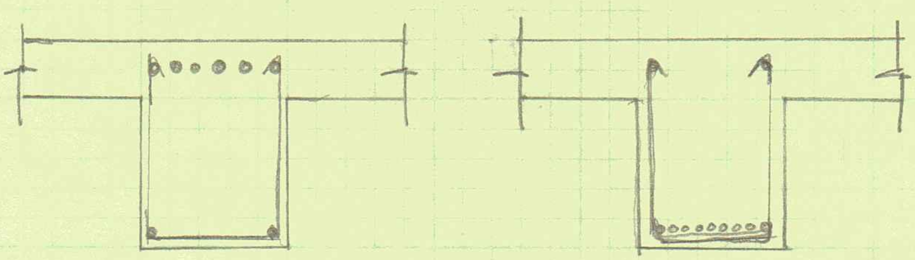
$a = \frac{60(7.11)}{.85(5)(24)} = 4.18 \quad c = 5.23$

$\varepsilon_s = \frac{.003}{5.25}(21.5 - 5.28) = .009 > .005$

$\phi = .9$

$\phi M_n = .9(7.11)(60)(21.5 - 4.18/2)$

$= 621\ k\text{-}ft$



$V_u = \frac{.72(31)}{2} = 11.2k + \frac{136}{2} = 79.2k \rightarrow 77.9k$ @d

$V_s = V_u/\phi + V_c = 77.9/.75 - 2\sqrt{5000}(24)(21.5)/1000 = 81k$

$s = \frac{(.2 \times 2)(60)(21.5)}{31} = 16.6in \rightarrow 16''$

$\phi V_n = 78.9k > 77.9$ ✓

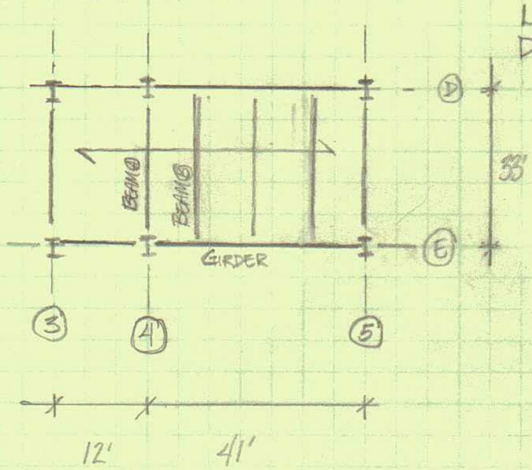
Appendix E: Composite Deck on Composite Steel Beams

JAKE WIEST

COMPOSITE SLAB + MTL DECK ON COMPOSITE BMS + GIRDERS

USING BAY FROM PT DESIGN

LIVE LOAD = 100 psf
DEAD LOAD = 15 psf MEP → SUPERIMPOSED LOAD = 115 psf



AS UNSHORED CONSTR. IS MORE ECONOMICAL, WOULD LIKE TO PICK A DECK THAT CAN BE PLACED 12' UNSHORED SO NO INFILL BEAMS NEED ADDED IN THE SMALLER BAYS.

2VLI 18 w/3" LW TOPPING CHOSEN
MAX UNSHORED: 12'9" > 12' ✓ (3 SPAN COND)
CARRIES 122 psf @ 12'-0" SPAN > 115 ✓

TOTAL SLAB DEPTH = 5"
TOTAL SLAB WT. = 39 psf

GIVEN SLAB CONSTRAINTS MEAN 3 INFILL BEAMS IN LARGE BAY @ 10.25' SPACING

BEAM DESIGN

LL = 100 psf

$LL_{RED} = .25 + \frac{15}{\sqrt{33 \times 20.5}} = .83 = 83$ psf (>400 ✓)

DL = 39 + 15 + 10 = 64 psf (BM WT)

BEAM A

$W_U = [1.2(64) + 1.6(83)]\frac{10.25+12}{2}$

$= 2.33$ klf

BEAM B

$W_U = [1.2(64) + 1.6(83)]\,10.25$

$= 2.15$ klf

$M_U = \frac{2.33(33)^2}{8} = 317$ k-ft

$M_U = 293$ k-ft

$b_{eff} = \frac{SPAN}{8} = \frac{33 \times 12}{8} \times 2 = 99" \leq 2 \times \frac{1}{2} SPACING = \left(\frac{10.25+12}{2}\right) \times 12 = 133.5"$

USE 99" FOR BOTH

AS DEFLECTION WILL LIKELY CONTROL, DESIGN 1 BM FOR LARGER TRIB.

ASSUME $a \approx 1'' \therefore Y2 = 5 - .5 = 4.5''$

W14×38 : $\phi M_n = 425$ $\Sigma Q_n = 386$ $386/17.2 = 22.4 \rightarrow 46$ STUDS/BM

W16×36 : $\phi M_n = 389$ $\Sigma Q_n = 229$ $229/17.2 = 13.3 \rightarrow 28$ STUDS/BM

W18×35 : $\phi M_n = 397$ $\Sigma Q_n = 194$ $194/17.2 = 11.3 \rightarrow 24$ STUDS/BM

W16×31 : $\phi M_n = 340$ $\Sigma Q_n = 213$ $213/17.2 = 12.4 \rightarrow 26$ STUDS/BM TRY

CHECK a : $a = \frac{213}{.85(4)(4)(.85)} = .74'' \checkmark$

f'c, λ FOR LW

UNSHORED

$w_u = [1.2(39) + 1.6(20)]\left(\frac{10.25+12}{2}\right) + 1.2(31) = .914$ klf

NO MEP OR BM, CONSTR. LL, BM

$M_u = \frac{.914(33)^2}{8} = 124 < 203$ k-ft $\checkmark$

WET CONC. DEFL.

$w = 39\left(\frac{10.25+12}{2}\right) + 31 = .465$ klf

$\Delta = \frac{5(.465)(33)^4(1728)}{384(29000)(375)} = 1.14''$

$\Delta_{max} = \frac{33(12)}{240} = 1.65 \therefore$ ck

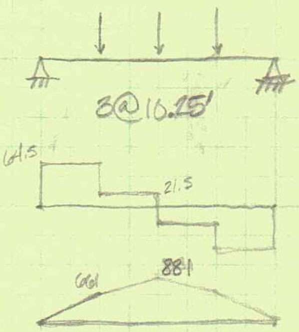
LL DEFL.

$w = 83\left(\frac{10.25+12}{2}\right) = .923$

$\Delta = \frac{5(.923)(33)^4(1728)}{384(29000)(825)} = 1.02$

$\Delta_{max} = \frac{33(12)}{360} = 1.1 \therefore$ ck

GIRDER DESIGN



19'-0" SPAN ON OTHER SIDE OF GIRDER

$P_D = 15.2k \leftarrow [64 \times 10.25) + 31]26'$

$P_L = 15.3k \leftarrow LL_{RED} = .25 + \frac{15}{\sqrt{4(1\times(38+19))}} = .5748$

57.5psf × 10.25' × 26' = 15.3k

$P_u = 43k$

$M_u = 881$ k-ft

$b_{eff} = \frac{41 \times 12}{8} \times 2 = 123'' \leq \left(\frac{19+83}{2}\right)12 = 812''$ ∴ 123" CONTROLS

$Y2 = 4.5''$ $(a \approx 1'')$

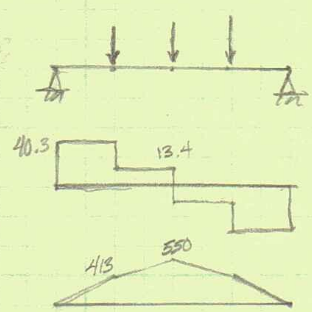
TRY W21×68:

$\phi M_n = 881$ k-ft $\Sigma Q_n = 342$ 342/17.2 = 19.9 → 40 STUDS/BM

$a = \frac{342}{.85(123)(4)(.85)} = .96$ ∴ ok

UNSHORED

$P_u = 1.2(15.3) + 1.6\left(\frac{20 \times 10.25 \times 26'}{1000}\right) = 26.9$



+

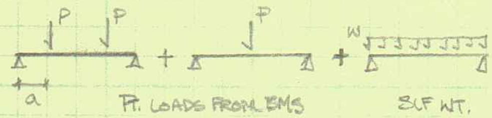
MOMENT FROM SLF WT.

W21×68: $\phi M_n = 600$ k-ft > 564.3 ∴ ok

$\frac{.068(41)^2}{8} = 14.3 + 550 = 564.3$ ft-k

WET CONC. DEFL.

ASSUMPTION: SUPER POSITION OF DEFL IS EQUAL TO REAL



$P = [(39 \times 10.25) + 35]26' = 11.3\,k$

$$\frac{Pa}{24EI}(3l^2 - 4a^2) + \frac{Pl^3}{48EI} + \frac{5wl^4}{384EI}$$

$$1728 \times \left[\frac{11.3(10.25)}{24(29000)(1480)}(3(41)^2 - 4(10.25)^2) + \frac{11.3(41)^3}{48(29000)(1480)} + \frac{5(.068)(41)^4}{384(29000)(1480)}\right]$$

$\Delta = 1.65''$

$\Delta_{Max} = \frac{12 \times 41}{240} = 2.05''$ ✓

LL DEFL.

SAME ASSUMPTION:

$P = [(57.5) \times 10.25]26' = 15.3\,k$

$$\frac{1728}{29000(2680)}\left[\frac{15.3(10.25)}{24}(3(41)^2 - 4(10.25)^2) + \frac{15.3(41)^3}{48}\right]$$

$\Delta = 1.18''$

$\Delta_{max} = \frac{12 \times 41}{360} = 1.37''$ ✓

Appendix F: Cost Analysis

Assembly B10102237600						**Based on National Average Costs**
Flat plate, concrete, 10" slab, 24" column, 25'x25' bay, 125 PSF superimposed load, 250 PSF total load						
Description	**Quantity**	**Unit**	**Material**	**Installation**	**Total**	
C.I.P. concrete forms, elevated slab, flat plate, plywood, to 15' high, 4 use, includes shoring, erecting, bracing, stripping and cleaning	0.977	S.F.	1.1	5.37	6.48	
C.I.P. concrete forms, elevated slab, edge forms, alternate pricing, to 6" high, 1 use, includes shoring, erecting, bracing, stripping and cleaning	0.032	SFCA	0.02	0.2	0.22	
Reinforcing Steel, in place, elevated slabs, #4 to #7, A615, grade 60, incl labor for accessories, excl material for accessories	3.468	Lb.	1.77	1.49	3.26	
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, sand, Portland cement and water, delivered, excludes all additives and treatments	0.834	C.F.	3.36	0	3.36	
Structural concrete, placing, elevated slab, pumped, 6" to 10" thick, includes strike off & consolidation, excludes material	0.834	C.F.	0	1.06	1.06	
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 and 4, to achieve a Composite Overall Floor Flatness & Levelness value up to F35/F25, bull float, machine float & steel trowel (walk-behind), excludes placing, striking	1	S.F.	0	0.82	0.82	
Concrete surface treatment, curing, sprayed membrane compound	0.01	C.S.F.	0.06	0.09	0.15	
Pre-Stressing Tendons	1.00	Lb.	2.38	1.01	3.39	
Total			**$6.30**	**$9.03**	**$18.74**	

Assembly B10102229600						Based on National Average Costs
Flat slab, concrete, with drop panels, 12" slab/11" panel, 24" column, 35'x35' bay, 125 PSF superimposed load, 290 PSF total load						
Description	**Quantity**	**Unit**	**Material**	**Installation**	**Total**	
C.I.P. concrete forms, beams and girders, exterior spandrel, plywood, 12" wide, 4 use, includes shoring, erecting, bracing, stripping and cleaning	0.036	SFCA	0.03	0.36	0.39	
C.I.P. concrete forms, elevated slab, flat slab with drop panels, to 15' high, 4 use, includes shoring, erecting, bracing, stripping and cleaning	0.993	S.F.	1.27	5.66	6.93	
Reinforcing Steel, in place, elevated slabs, #4 to #7, A615, grade 60, incl labor for accessories, excl material for accessories	5.432	Lb.	2.77	2.34	5.11	
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, sand, Portland cement and water, delivered, excludes all additives and treatments	1.091	C.F.	4.4	0	4.4	
Structural concrete, placing, elevated slab, pumped, 6" to 10" thick, includes strike off & consolidation, excludes material	1.091	C.F.	0	1.39	1.39	
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 and 4, to achieve a Composite Overall Floor Flatness & Levelness value up to F35/F25, bull float, machine float & steel trowel (walk-behind), excludes placing, striking	1	S.F.	0	0.82	0.82	
Concrete surface treatment, curing, sprayed membrane compound	0.01	C.S.F.	0.06	0.09	0.15	
Total			**$8.55**	**$10.66**	**$19.21**	

Assembly B10102199400						Based on National Average Costs
Cast-in-place concrete beam and slab, 9" slab, one way, 26" column, 35'x40' bay, 125 PSF superimposed load, 273 PSF total load						
Description	**Quantity**	**Unit**	**Material**	**Installation**	**Total**	
C.I.P. concrete forms, beams and girders, exterior spandrel, plywood, 12" wide, 4 use, includes shoring, erecting, bracing, stripping and cleaning	0.164	SFCA	0.15	1.63	1.78	
C.I.P. concrete forms, beams and girders, interior, plywood, 12" wide, 4 use, includes shoring, erecting, bracing, stripping and cleaning	0.34	SFCA	0.37	2.79	3.16	
C.I.P. concrete forms, elevated slab, flat plate, plywood, to 15' high, 4 use, includes shoring, erecting, bracing, stripping and cleaning	0.856	S.F.	0.97	4.71	5.68	
Reinforcing Steel, in place, elevated slabs, #4 to #7, A615, grade 60, incl labor for accessories, excl material for accessories	5.834	Lb.	2.98	2.51	5.48	
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, sand, Portland cement and water, delivered, excludes all additives and treatments	0.983	C.F.	3.96	0	3.96	
Structural concrete, placing, elevated slab, pumped, 6" to 10" thick, includes strike off & consolidation, excludes material	0.983	C.F.	0	1.25	1.25	
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 and 4, to achieve a Composite Overall Floor Flatness & Levelness value up to F35/F25, bull float, machine float & steel trowel (walk-behind), excludes placing, striking	1	S.F.	0	0.82	0.82	
Concrete surface treatment, curing, sprayed membrane compound	0.01	C.S.F.	0.06	0.09	0.15	
Total			**$8.50**	**$13.80**	**$22.30**	

Assembly B10102568000						Based on National Average Costs
Floor, composite metal deck, shear connectors, 5.5" slab, 35'x40' bay, 29.5" total depth, 125 PSF superimposed load, 171 PSF total load						
Description	**Quantity**	**Unit**	**Material**	**Installation**	**Total**	
Welded wire fabric, sheets, 6 x 6 - W1.4 x W1.4 (10 x 10) 121 lb. per C.S.F., A185	0.011	C.S.F.	0.15	0.39	0.54	
Structural concrete, placing, elevated slab, pumped, less than 6" thick, includes strike off & consolidation, excludes material	0.333	C.F.	0	0.5	0.5	
Structural concrete, ready mix, lightweight, 110 #/C.F., 3000 psi, includes local aggregate, sand, portland cement and water, excludes all additives and treatments	0.333	C.F.	2.41	0	2.41	
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 and 4, to achieve a Composite Overall Floor Flatness & Levelness value up to F35/F25, bull float, machine float & steel trowel (walk-behind), excludes placing, striking	1	S.F.	0	0.82	0.82	
Concrete surface treatment, curing, sprayed membrane compound	0.01	C.S.F.	0.06	0.09	0.15	
Weld shear connector, 3/4" dia x 4-7/8" L	0.153	Ea.	0.11	0.29	0.4	
Structural steel project, apartment, nursing home, etc, 100-ton project, 3 to 6 stories, A992 steel, shop fabricated, incl shop primer, bolted connections	8.34	Lb.	10.51	3.5	14.01	
Metal floor decking, steel, non-cellular, composite, galvanized, 3" D, 18 gauge	1.05	S.F.	2.31	1.06	3.37	
Metal decking, steel edge closure form, galvanized, with 2 bends, 12" wide, 18 gauge	0.027	L.F.	0.09	0.06	0.15	
Sprayed cementitious fireproofing, sprayed mineral fiber or cementitious for fireproofing, beams, 1 hour rated, 1-3/8" thick, excl. tamping or canvas protection	0.654	S.F.	0.38	0.63	1.01	
Total			**$16.00**	**$7.34**	**$23.34**	