TECHNICAL REPORT 2

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Embassy Suites Hotel, Springfield Virginia

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Executive Summery

The purpose of this technical report is to delve in how alternate floor systems would function in the existing building and what impact they would have on the overall design. The current floor system, in place at the Embassy Suites Hotel project is a two way concrete flat slab with drop panels, which was found in Technical Report 1 to be exceptionally designed to carry the loads imposed on it by wind and gravity. The other potential floor systems were designed with limiting factors in mind. Additionally, a bay size of 16.5’ x 23.3’ was used on a typical guest room floor, where in some cases was rounded to accommodate for design parameters. The alternate floors systems that were taken into consideration are as follows:

- One Way Slab with Reinforced Concrete Beams
- Composite Deck System on Steel Framing
- Hollow Core Concrete Planks on Steel Framing

The existing 8 inch two way slab system was designing in accordance the ACI 318 - 11 code for concrete construction. This design takes advantage of the height restriction situation presented by zoning parameters and allows for a fairly open design. The system is supported by 14” x 30” concrete columns. The one way slab system consists of a 10 inch slab and is supported by 14” x 20” beams running both the 16.5 and 23.33 directions (4 beams total for the bay). The composite deck system was designed using The Vulcraft Deck Catalogue and the ACI Steel Manual. The deck chosen was a 2VLI20 with a 2 inch topping thickness and a total depth of 4 inches. This system was supported by (4) W10 x12 beams (2 infill beams spaced at 5.5 inches) and (2) W10 x 12 girders. The last system considered was a hollow core concrete planks on steel framing. The plank size selected was a 4 foot wide and 8 inch thick untopped hollow core plank with 58-S strands. The framing members supporting this system were found to be (4) W 12 x 58 steel beams. All systems were designed with having a minimum 2 hour fire rating in mind.

In analyzing these systems based on cost, weight, and impact to floor to floor both the pros and cons of each system were taking into consideration when comparing them to the exiting design. It was found that with all these factors in mind the existing floor system is the most efficient system that can be in place. However, the composite deck system showed promising results when compared and can be a viable option in future reports. Detailed descriptions of the systems and design calculation can be found throughout the report and in the Appendices respectively.

Introduction: Embassy Suites Hotel


Figure 1.2: **Site Map. (Photo taken from Google Earth)**

The Embassy Suites Hotels is the newest, 7 story, luxury, hotel to become part of the Miller Global, LLC family. Along with Miller Global, the owner the collaborative construction team on this venture include, Cooper Carry, architect; SK & A Structural Engineers, PLLC , structural designers; Balfour Beatty Construction, construction manager; Jordan and Skala, MEP firm; Christopher Consultants, LTD, civil engineering firm. The site is located at the junction of I-95 and Fairfax County Parkway. The location lies in the Springfield region of Fairfax County, Virginia. The site is approximately 16 miles away from the heart of downtown Washington, D.C... Patrons will also be in close proximity to both the Fort Belvoir Main Army Post and the National Geospatial-Intelligence Agency (NGA) facility. The construction delivery method was design –bid - build. Construction began in November 2011 and will be completed July 13th 2013.


Figure 2.2: **Facade. (Photo taken from Miller Global, LLC website)**

Upon its completion, this 31.5 million, 185,000 square foot, hotel will feature many amenities. These include a large open air atrium and spacious two room suites. The hotel will serve as a model for comfort and convenience. The building's architecture boasts long flowing curved lines that give it immense visual appeal and a unique flow. The hotel's ground floor will contain a 1300 square foot pool area, a fitness center along with multiple meeting areas, a bar, a lounge and over 1400 square feet of retail space.

The ground level and upper floors store front materials will be made up of manufactured masonry (adhered concrete stone veneer). It is comprised of boral cultured stone country ledge stone along with architectural adhered precast concrete panels. It also contains 1" insulated glass windows with aluminum frames and automatic entrances. The upper levels the exterior façade will feature an exterior insulation finish system (EIFS).

This report will be describing the various structural elements and systems in place at the Embassy Suite Hotel project. This report will also be delving into the design of alternative floor systems that can be viable options that would fit in this sound, cohesive building project.

Structural Systems

Existing Foundation

Prior to construction, subsurface exploration and geotechnical engineering analysis were conducted on the future Embassy Suites Hotel site and was completed in January11, 2011 by ECS Mid- Atlantic, LLC. The report indicates a number of test borings were performed on 3 separate occasions. The test borings were drilled at depths ranging from 2.5 ‘to 79’ to determine the soil composition in different areas of the site. ECS Mid- Atlantics results showed fill soil material was found in ten boring locations around the site. The fill material was composed of silty sand and clay from depths of 6.5’ to 8.5’ below the ground surface. Further down the

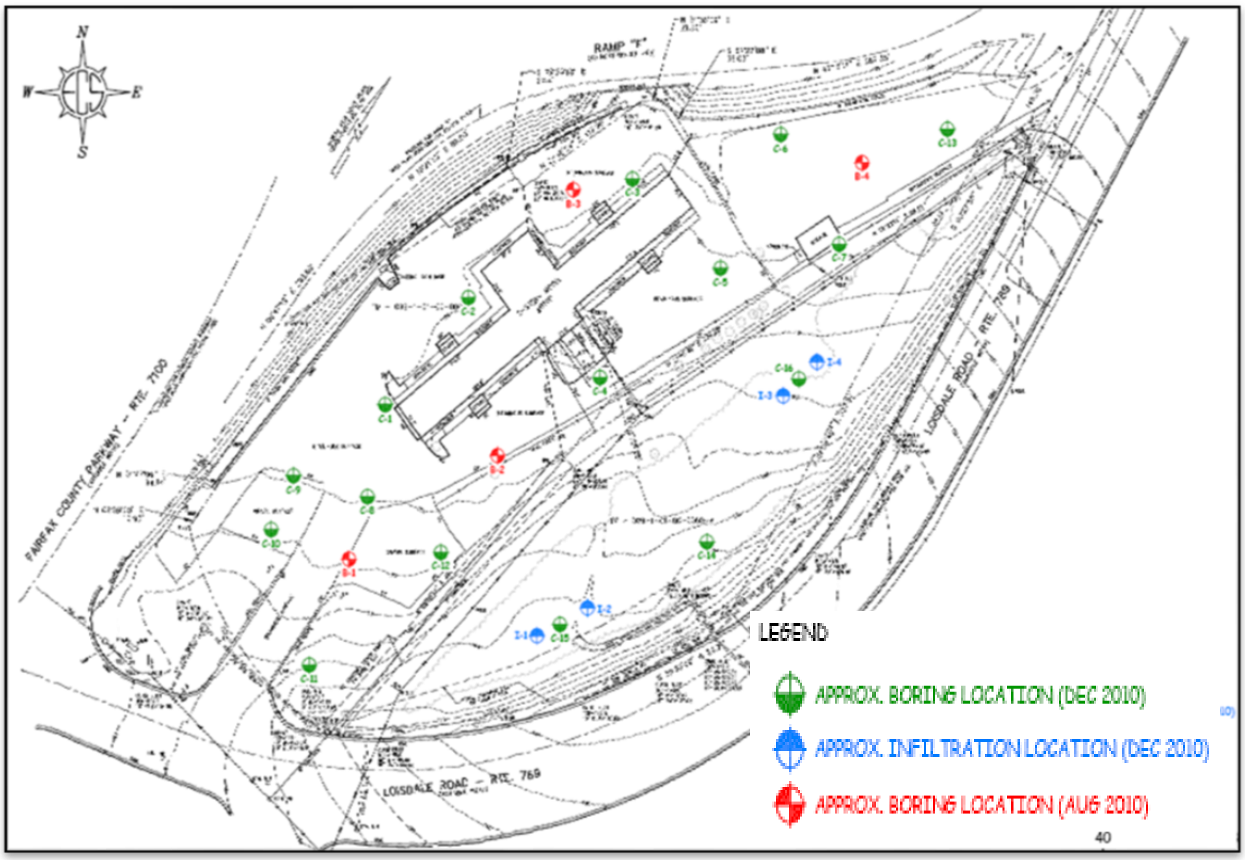


Figure 3.3: **Core Boring Locations**

borings indicated the existence of natural soils that were mainly composed of clayey sand, silt and fat clay. A weather rock material was found at 77’ to 78.6’and ground water was encounter at of 18.5’ to 65’.

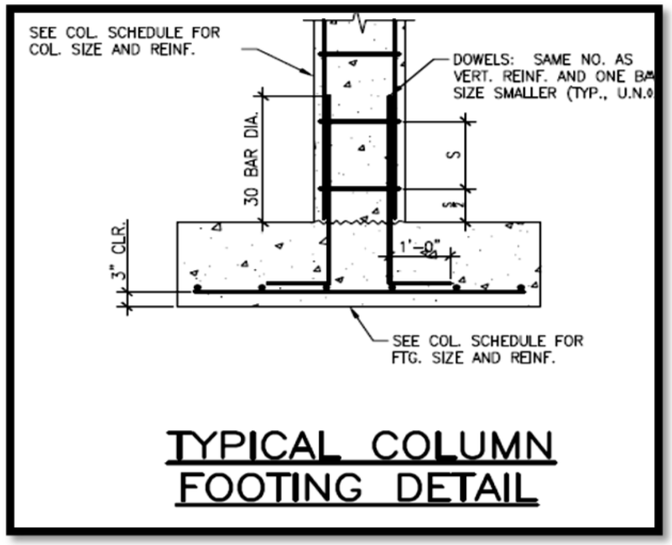
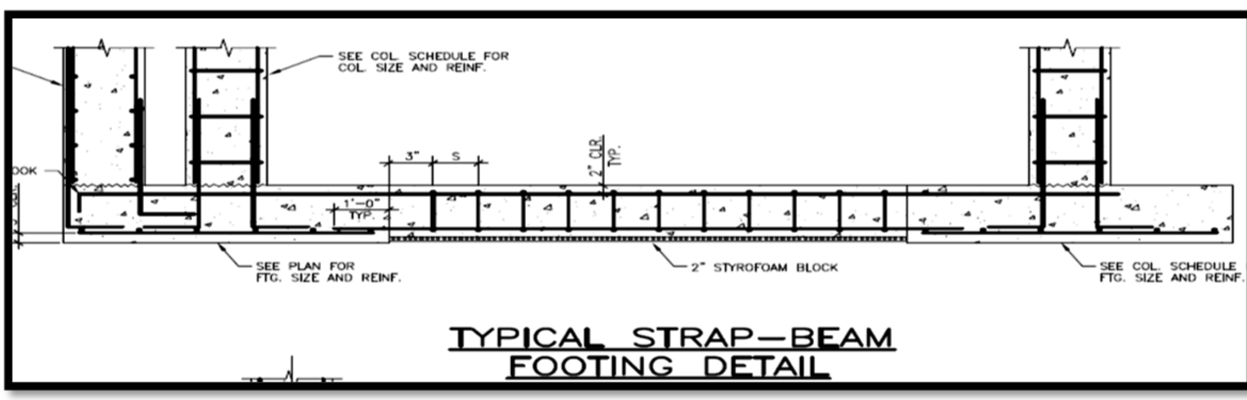


Figure 4 Footing Detail

Due to the variability in soil composition, the project team had to employ a partial mud matt system to equalize the soil capacity around the site in some areas. A mud matt system is a thin layer of lean concrete mix (in this case 2000 psi) placed over the existing soil below and allows a stable base for construction. The spread footings were designed to have an allowable bearing capacity 6000 psi. The size of footings range from 3' by 3' to 12' by 8' and extend 2' below the slab on grade. To tie the footings together, longitudinally placed strap beams ranging from 36 width x 24 depths to 42 width x 24 depth beams were used. A strap beam is a structural element used to connect to isolated footings together. These beams help distributed the building load to the footings and eventually the ground. The beams range in size and have varied vertical and horizontal reinforcing.

The typical slab on grade is a minimum of 5 inches in depth and sits on 4 inches of washed crushed stone. The capacity of the slab is 3500 psi for the interior portions and 5000 psi for exterior slab conditions. The slab contains 6x6 – W 2.0 x W2.0 welded wire fabric and has number 4 reinforcing steel bars spaced 12 inches on center each way.



Framing System

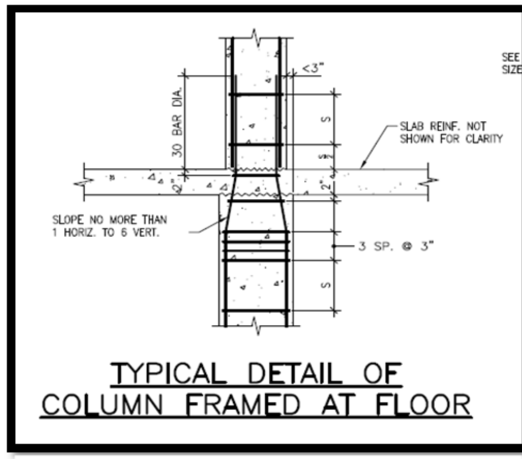


***Figure 5.4*: Strap Beam Detail**

In the image below, shown is a typical framing plan for floors of the Embassy Suites Hotel (Floors 3 to 7). A typical bay size is 23' by 18' for floors containing the guest suites. The columns chosen in for the framing plan were almost all 14 x 30 inch rectangular reinforced concrete columns. The majority of the columns have a minimum compressive strength of 6,000psi. There are no beams running in between the interior and exterior columns. The only reinforced beams found are located in stairwell openings and elevator shafts. Due to the increased load on the second floor, large concrete transfer girders had to be used to accommodate for the fitness and pool area. Level 2 also contains HSS columns along

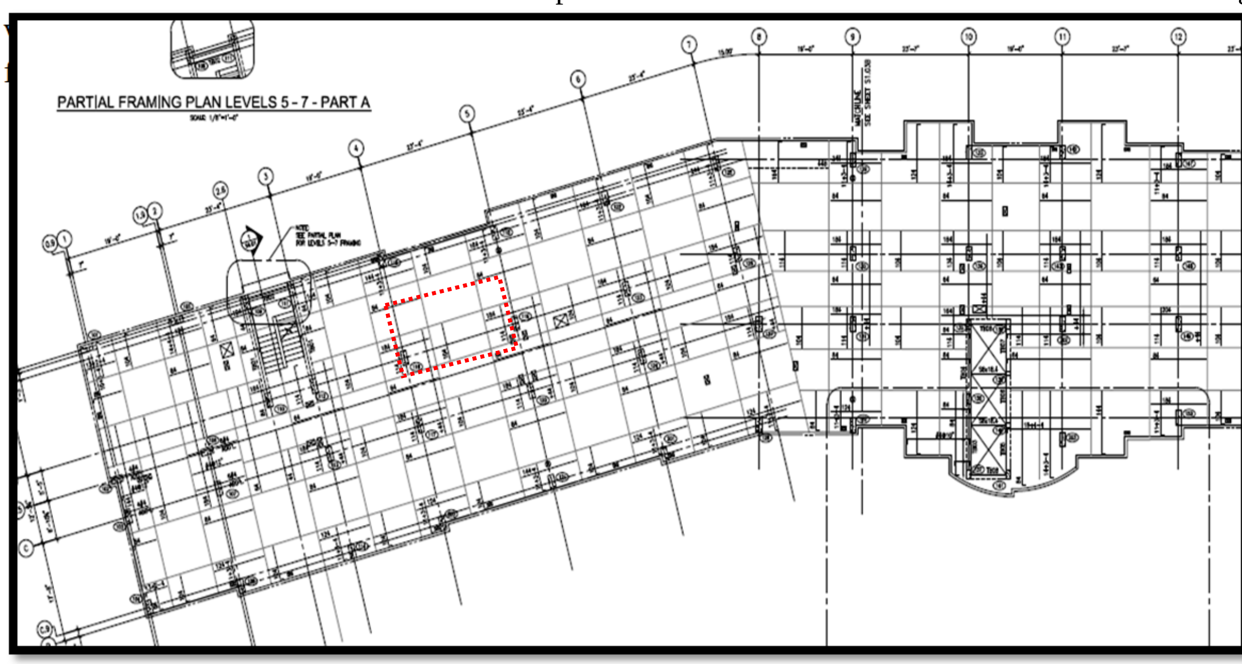
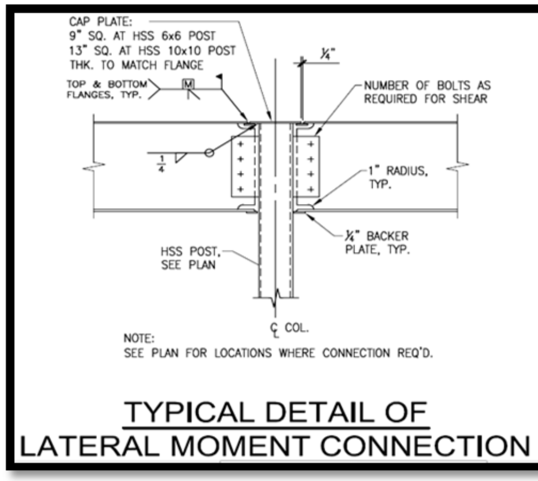
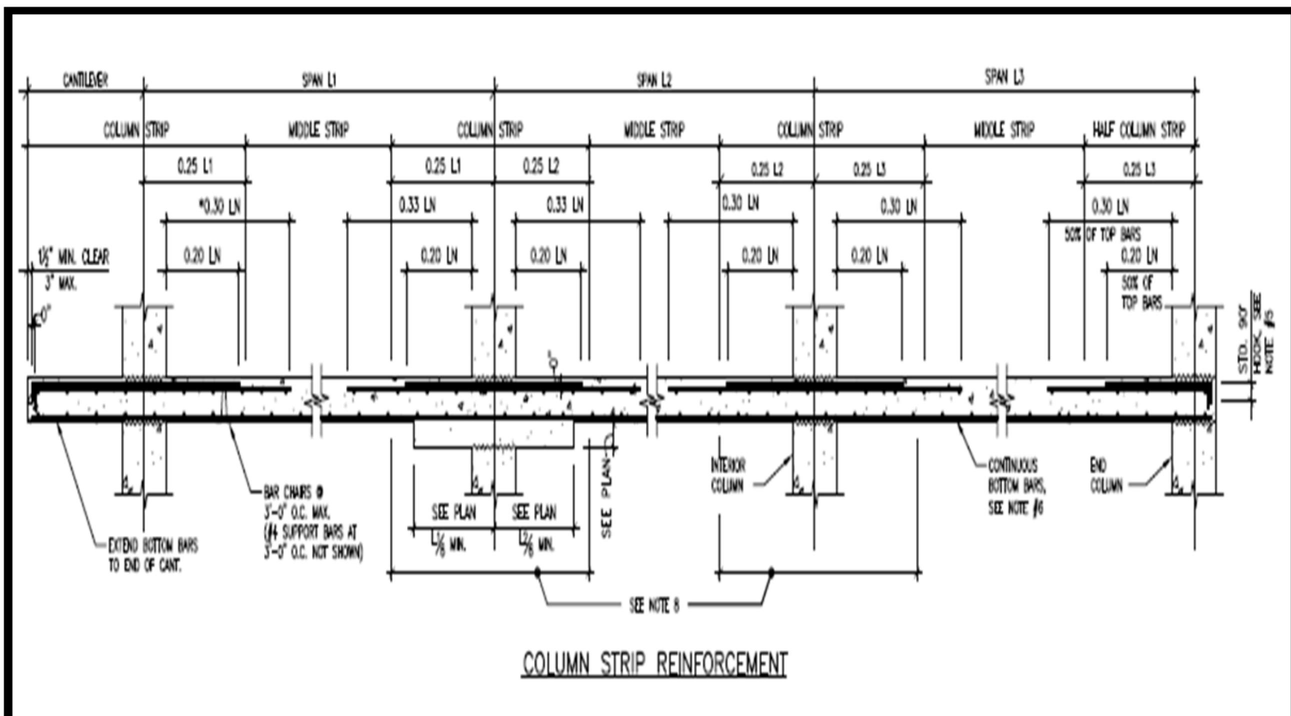


Figure 6: Typical Framing Plan Levels 3-7

Lateral System

To resist lateral forces due to wind and seismic loads the structural engineers employed reinforced concrete moment frames moment frames. A concrete moment frame load resisting system (in this case a slab and columns cast monolithically) opposes overturning moment caused by lateral loads. The concrete moment frames are the main lateral force resisting system in the building. The lower storefront levels have welded steel moment connections. The moment connections were designed to develop the full capacity of the member. The connections use high strength ¾ or 7/8 inch ASTM A325 or A490 threaded bolts. The bolts connect the ¼ x 1 inch





plates to the beams were the plates are butt and penetrate welded. Figure 9: Welded Moment Connection

Figure 7: Main Lateral Force Resisting System

Roofing System

The high level roofing system consist of 2 inch deep 20 gauge Type N cold formed metal deck. The metal deck is topped by 3.25 inch light weight concrete slab. This slab has a compressive strength of 3,500 psi. The deck holds a minimum of a 3 span condition. The lower level roof (top of retail space) is made of 1.5 inch deep 20 gauge Type B cold formed metal deck. The roof deck systems are supported by wide flange beams, concrete reinforced beams varying in size and open web steel joists. The lower level roof system is comprised of a thermoplastic membrane fully adhered with heat welded seams and vapor retarder over a metal deck. Part of the lower level roof (top of part of the second floor) contains a green roof system that includes a pre-vegetated 50 percent extensive and a 50 percent intensive system that is placed upon a protective mat.

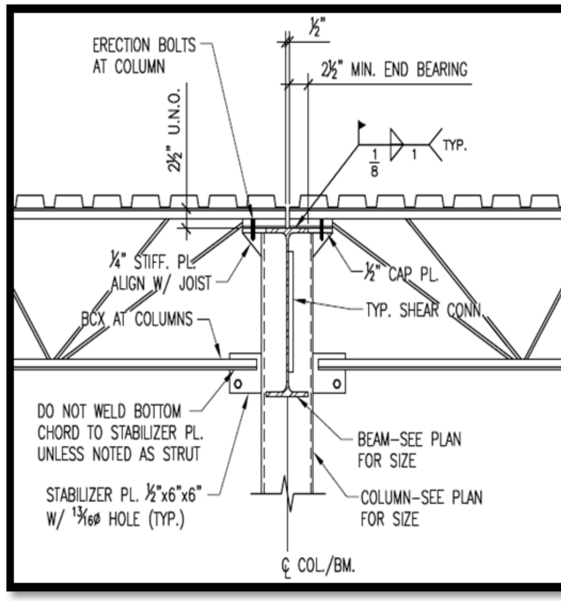


Figure 8: Lower Roof System Connection

Codes and Requirements

- 2009 Virginia Construction Code (IBC 2009) with the Virginia Statewide Building Code
- 2009 Virginia Mechanical Code (IMC 2009)
- 2008 International Electric Code
- 2009 International Pluming Code (IPC 2009)
- 2009 Virginia Fire Prevention Code (IFC 2009) with the Statewide Fire Prevention Code
- American Society of Civil Engineers (ASCE 7- 05)
- Publication #4 "Standard Recommended Practice for Concrete Formwork" (ACI 347)
- American Concrete Institute Specifications for Reinforced Cast-In-Place Concrete (ACI 318-08)
- American Concrete Institute Specifications for Structural Concrete (ACI 301)
- American Institute of Steel Construction (AISC 325 -11)
- American Iron and Steel Institute Specification for the Design of Light Gage Cold Formed Structural Steel Members (A.I.S.I)
- Steel Deck Institute Design Specifications (S.D.I)

Codes Used in Analysis

ASCE 7-05, Minimum Design Loads for Buildings

ACI 318-08, Building Code Requirements for Structural Concrete

International Building Code (IBC), 2009

Materials

Concrete		
Element	Weight	Strength (psi)
Footings	Normal	4000
Grade Beams	Normal	4000
Retaining Wall	Normal	4000
Retaining Wall Footing	Normal	4000
Interior Slab-On-Grade	Normal	3500
Exterior Slab-On-Grade	Normal	5000
Formed Slabs	Normal	5000
Formed Beams	Normal	5000
Columns	Normal	6000
Foundation Walls	Normal	4000
CMU Grout	Normal	2500
All Other	Normal	3000

Table 1: **Concrete Material Summery**

Steel		
Element	Standard	Grade
Reinforced Bars	ASTM 615	60
Welded Wire Reinforcement	ASTM 185	N/A
Pre-stressed Steel Wire	ASTM 416	N/A
Wide Flange Shapes (Beams, Girders, Columns etc.)	ASTM A992	50
Stiffener Plates	ASTM A572	50
Hollow Structural Sections	ASTM 500	B
Steel Pipe	ASTM A53	B
Angles, Channels, S-Shapes etc.	ASTM A36	36
Nuts, Bolts	ASTM A325, A490	N/A
Misc. Steel	ASTM A36	36

Table 2: Steel Material Summery

Gravity Loads

Dead and Live Loads

In this section, gravity loads (dead, live, and applicable) are presented. These loads are compared to actual building load calculations used in Embassy Suites Hotel. Assumptions for superimposed dead load are offered in Tables 3 and 4.

Live Load

Live Load		
Element	Design Live Load (psf)	Thesis Load (psf)
Guestroom Floors	40	40
Mechanical Rooms	150	150
Partitions	15	15
Elevator Machine Room	125	125
Stairs and Exit Ways	125	125
Slab on Grade	125	125
Balconies	125	125

Table 3: Live Load Values

Dead Load

Dead Load		
Element	Design Dead Load (psf)	Thesis Load (psf)
MEP	-	10
8" Reinforced Concrete Slab	100	100
Deck and Slab (Vulcraft)	-	39
Precast Hollow Core Panels	-	56

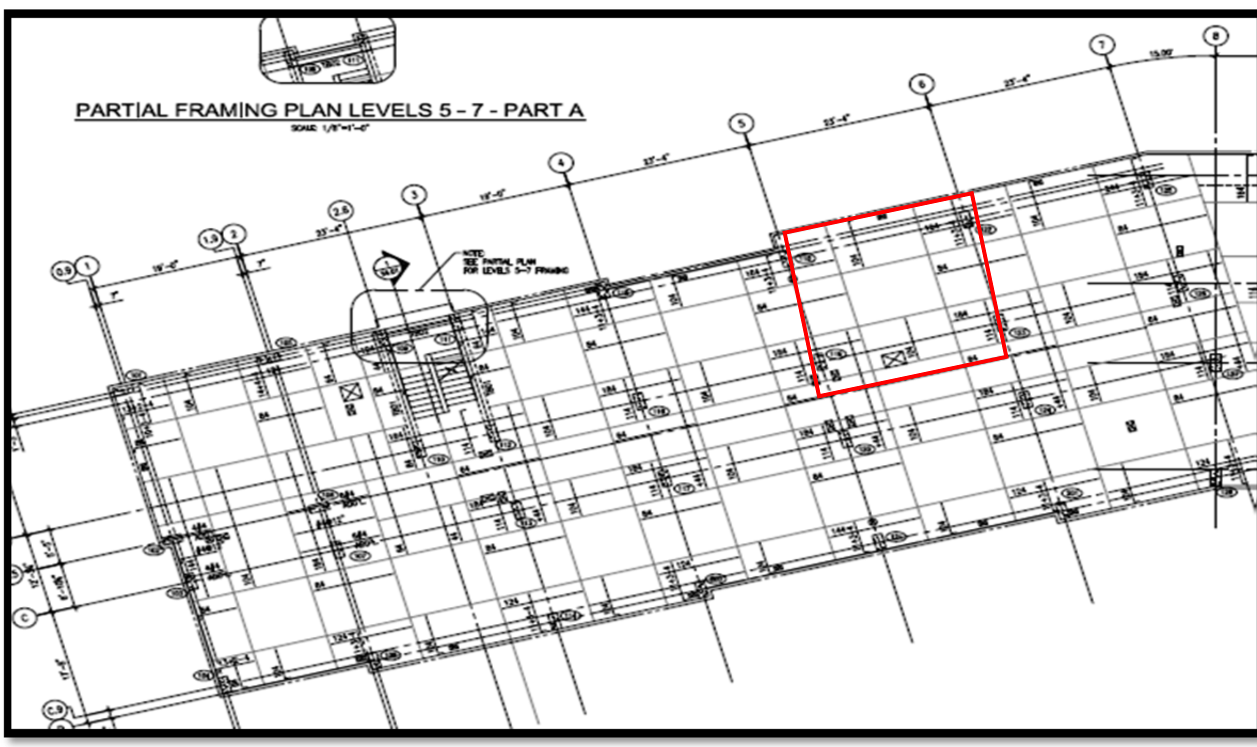
Table 4: Dead Load Values

Flooring Systems

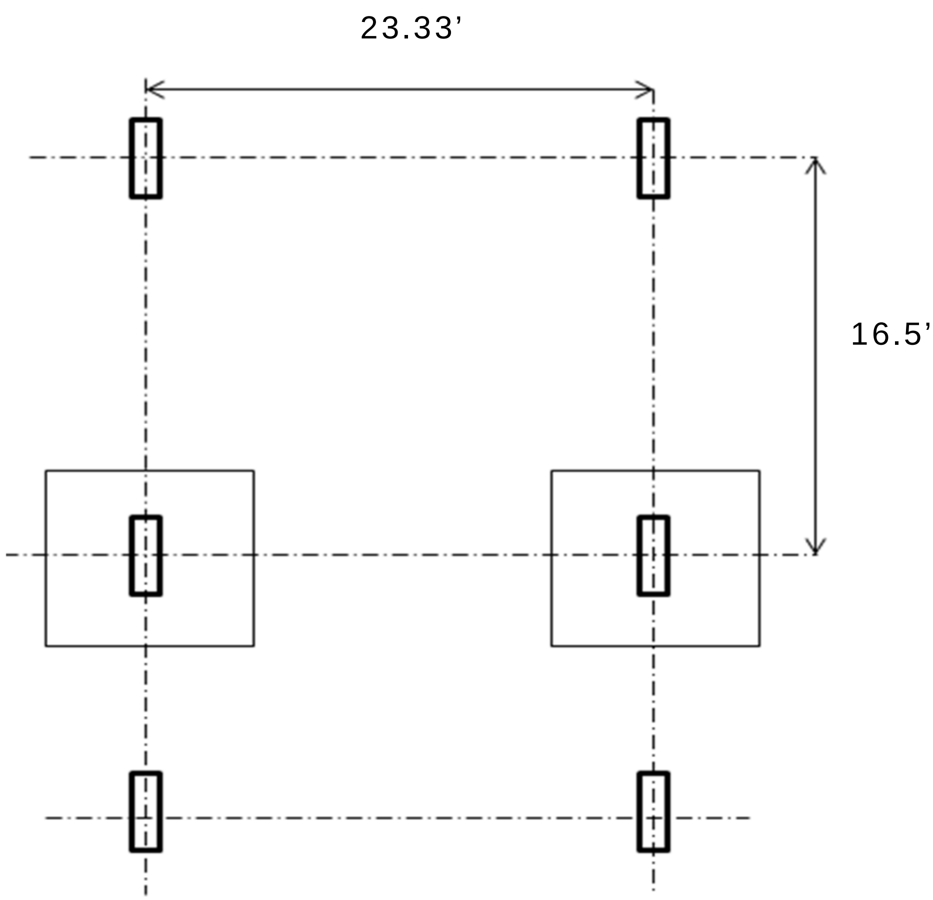
The purpose of this technical report is to design possible alternative floor systems that could have been viable solutions in the initial design of the Embassy Suites Hotel. These alternative systems are to be compared to the existing floor stems to gain a better understanding of how the actual floor systems was chosen based on factors like fire protection, weight, cost, and depth, examining the benefits and drawbacks of each system. The alternative systems comparisons will also look at how the structure, design, and construction of building components can be affected if these other systems were in use. The systems to be compared are as follows:

- Two Way Flat Slab with Drop Panels
- One Way Slab with Reinforced Concrete Beams
- Composite Deck with Steel Framing
- Hollow Core Panels with Steel Framing

On a typical floor, (floors 3 to 7) a bay size of 23' – 4" by 16' – 5" was chosen for the alternate floor designs due to the fact it has the largest span sizes and would potentially control the design. In some cases the bays size was rounded for ease of calculation.



Existing Floor System: Two Way Flat Slab with Drop Panels



The existing flour system in place at Embassy Suites Hotel is a two way flat slab construction with drop panels along column line B. The slab thickness is 8 inch and the compressive strength of the normal weight concrete is 5000 psi. The slab reinforcing includes number 4 reinforcing bars spaced at 10 inches on center, either way and run the full length from column to column. The floor system also uses a thickened slab or drop panel system around the columns to protect against punching shear and increased resistance to moment. Punching shear is a failure mechanism were the slab separates from the column due to concentrated shear force. Drop panels are 3.5 inches thick (total slab thickness around column on typical floor is 11.5 inches) and extend 5 feet from either side of the columns.

Advantages

Having a two way flat slab system with drop panels in place is a cost effective method that allows for shallow floor depths and low floor height if building height in a zoning area is an issue. Concrete is usually more cost effective than steel construction and it is almost always readily available. Additionally, concrete has natural fire resistive qualities and this system in particular allows for the achievement a two hour fire rating without any additional fire resistant material being added.

Disadvantages

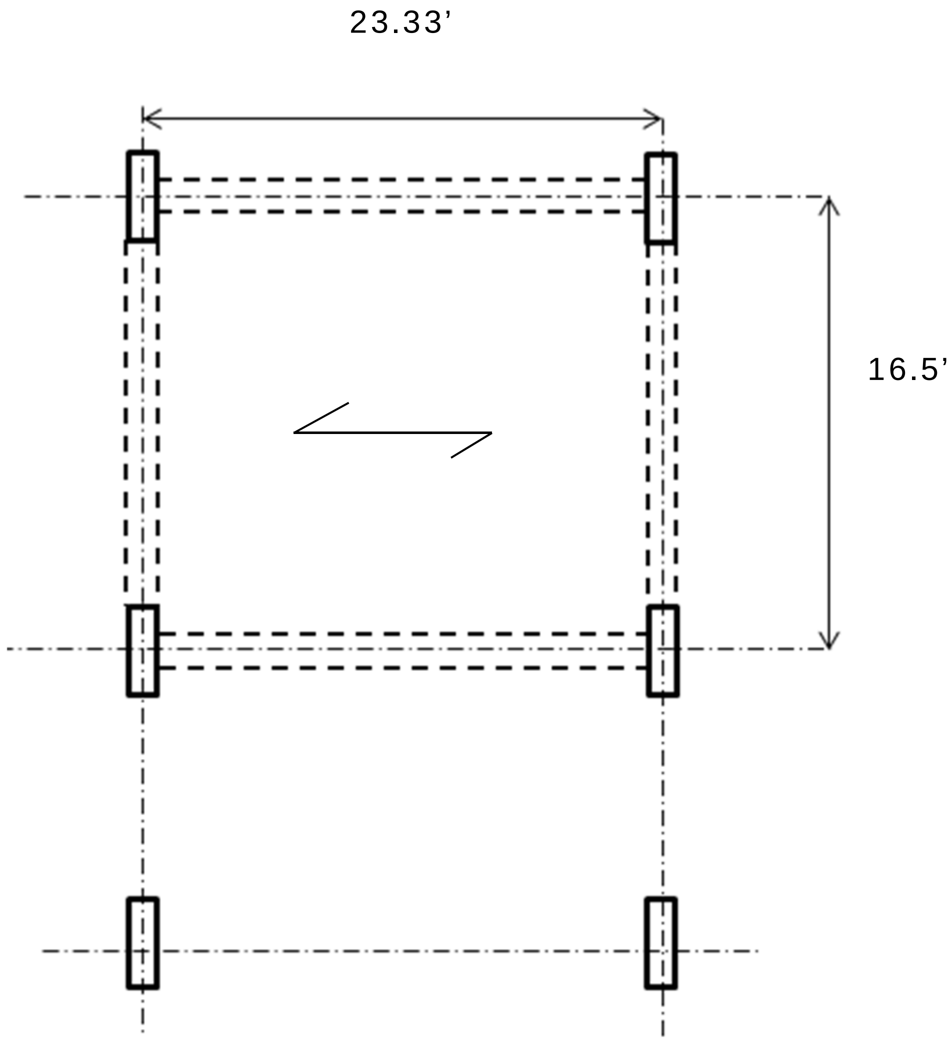
One disadvantage of using a two way slab system like this is the cost of large amounts formwork that need to be placed for the concrete to set up or its 28 day cycle. In general, concrete construction usually takes more time to complete due to the fact the concrete has to reach a portion of its full capacity for any load to placed on it. Also there are some reinforcing issues that have to be addressed with the required correct placement of the bars to develop the full moment capacity of the slab.

Results

The two ways slab was found to have and overall depth of 11.5 inches and have a total weight of 188 psf. This system by far has the shallowest at 11.5 inches. This is an important factor to consider due to zoning restrictions in the Fairfax county region of Virginia which limits the overall building height above grade. The slab itself is sufficient for a minimum fire rating of 2 hours.

To calculate the estimated cost per square foot RS Means 2011 Square Foot Cost manual was used. Values for costs were chosen based on a similar bay size square footage, total load and total depths. With these factors chosen a $17.35 per square foot cost was determined. This value was not adjusted for location or for the bay only having 2 drop panels in it. RS Means assumes that every column will have a thickened slab. If the actual bay size were used and the other limiting factors addressed the cost could potentially be reduced making this the most cost effective system.

Alternative System 1: One Way Slab with Beams



The first alternative floor system that was explored was a one way slab system with reinforced concrete beams. The existing bay size for a typical floor was not altered in this calculation. The slab was chosen to span the 23'-4" (across N-S Direction) length with the beams running in the 16'- 5" direction. The existing 14" x 30' reinforced concrete columns were not altered in the consideration for the alternated design. The slab thickness for a one way slab is usually governed by deflections and is some cases fire rating. In this case, it was established that a slab thickness 10" was required and the beam dimensions were found to be 14" x 20" spanning the

23’ – 4” direction. The compressive strength of the concrete was assumed to be 5000 psi in calculations.

Advantages

There are several benefits to a one way slab system in a building structure. Concrete is typically able to be accessed without any problems and can be formed and erected on site with relatively low levels of difficulty. The system also allows for the option of having a decreased slab depth while still having at least a fire rating of 2 or more hours.

Disadvantages

One way slab may have to be more heavily reinforced due to their nature being less stiff because the system is only one directional. This may cause cracks to develop more rapidly. If height is an issue another disadvantage would be increases total system depth. Lastly, again cast in place concrete can be extensive due to the amount of formwork required for the concrete to set.

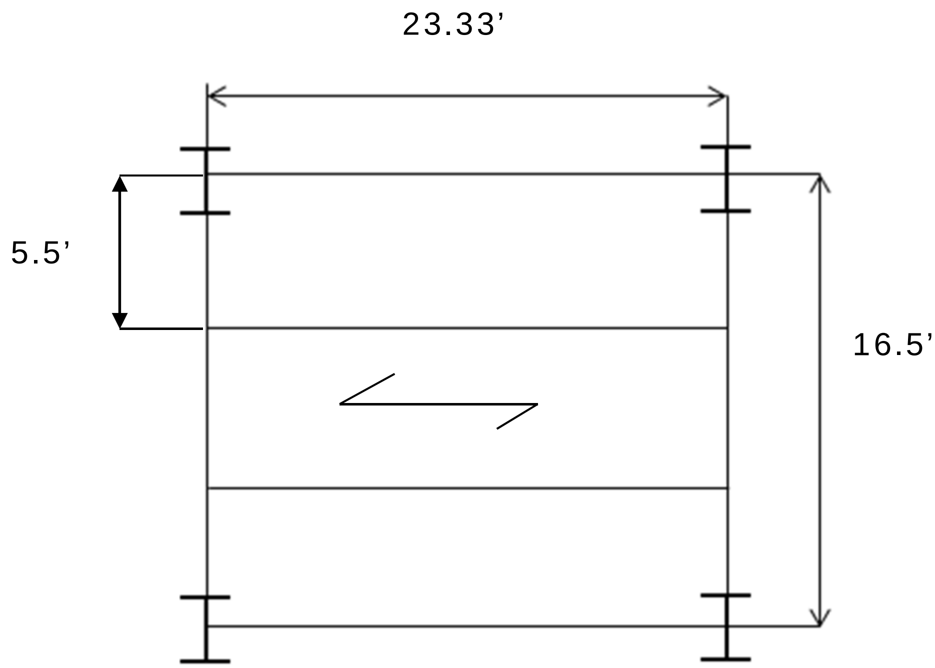
Results

Throughout the examinations of the alternate floor systems the one way slab had a total system depth of 30 inches making it the deepest floor system. Along being the deepest the one way slab it also was the heaviest being 1125 psf. This is almost 5 times the existing weight per square foot. Before the start of construction the site was found to have differential soil capacities forcing the design professions employ mud matt systems to equalize the capacity around site. With the increased weight of this alternate system, the existing foundation would in turn have to be redesigned to compensate for the extra weight potentially having to employ an entirely new system. Along with the increased, weight the system will greatly reduce the floor to ceiling height. With their being height restrictions, the building would not be able to accommodate a compensation for the increased depth their not being able to increase the floor to floor height.

The total cost of the system was determined to be $22.10 per square foot, only being slightly less than the most expensive system, yet still being almost $5 dollars more per square foot compared to the two way slab system.

With all factors considered, the increased weight, cost impact on structure and architecture this would not be viable alternative option to be in place and will not be considered in future reports.

Alternate System 2: Composite Floor System on Metal Deck



A composite floor system with metal deck was chosen for the second alternate design. Composite floors systems work well to reduce floor depth without losing any capacity due to the concrete and steel working integrally as one system. The infill beams are spaced at 5.5 inches and the deck spans the 23.3 foot direction. The size for the beams and girders were determined to be a W10 x 12. Other sizes were considered but ultimately with depth being a factor in the design a W10 x 12 was selected. Additionally, this type of construction was chosen to get a cost comparison between a dominantly steel system with the existing system being all concrete.

Advantages

Overall, the main advantages of having this type of system in place are reduced slab heights that can potentially allow for the ease of placement of mechanical and electrical systems under the floor and the reduction in the self weight of the structural components themselves.

Disadvantages

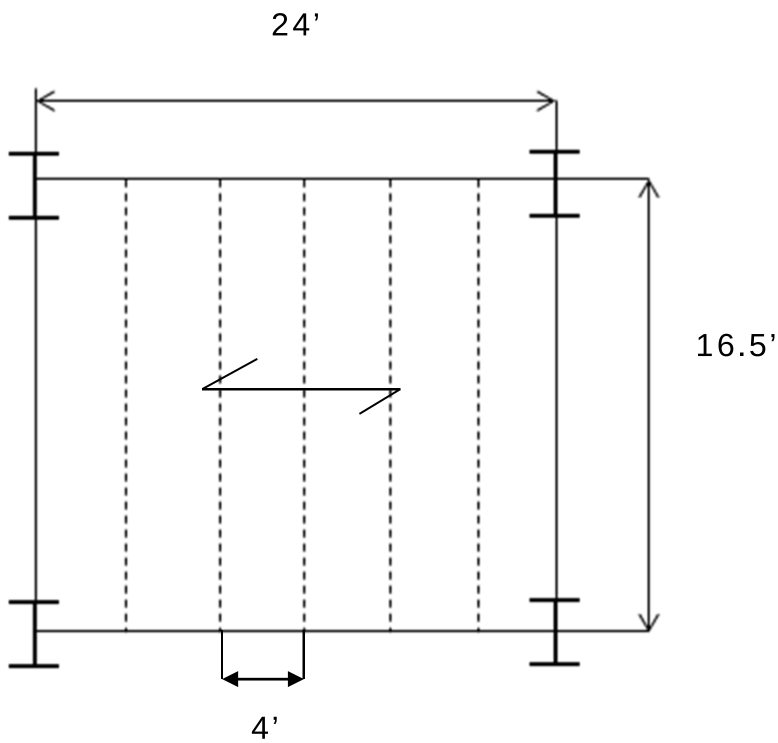
With a dominantly steel floor system cost is always a factor. For one, the addition of fire proofing to achieve a 2 hour fire rated system is mandatory since steel itself has no natural fire resistive properties itself. Also, due to the increased depth of the beams the total depth of the ceiling would have to be increased causing a problem if there are height restrictions and could have architectural impacts.

Results

In contrast to the concrete one way and two way slab floors, a composite system was examined. The deck and slab thickness was calculated to be 4.5 inches, a little more than half the depth of the existing system. The overall depth was determined to be 13.87 inches, only slightly larger than the two way slab. This will not have a significant effect on the ceiling and overall floor to floor height. Of all the alternative floor systems, the composite system was shown to have to be lightest, having a weight of 114 psf. This will in turn not affect the existing foundation system in terms of being able to be carried by the soil capacity on the sight.

Along with being the lightest system, the composite floor is also the most cost effective. The cost per square foot is $16.80, which is .55 cent less that the existing system. In calculating the price in RS Means, again assumptions had to be made because of the irregularity of the bay size. Additionally, to achieve a minimum fire rating of 2 hours the extra cost of spray fireproofing was incorporated in the price analysis. While examining the components of this floor design with its low cost, light weight, minimal affect on building height is a promising option and will be looked at further for a design option in upcoming assignments.

Alternative System 3: Hollow Core Precast Planks on Metal Deck



The final alterative floor system considered was a hollow core concrete plank on steel framing. The system consists of 4 foot wide planks sitting atop W 12x 58 beams and girders. The planks themselves have a dead load of 56 psf and contain 5 strands with half in diameters. No topping was selected for this system. For this floor system the bay size had to be increased for 23.33 feet to 24 feet to allow for ease of construction and even spacing. The planks span the 24 foot direction and have a capacity of 160 psf.

Advantages

The main advantage to using precast hollow core planks is construction time. They can easily be erected quickly and require no time to cure on site. The hollow for planks have the capacity to span longer distances and usually have a shallow depth it a good option if height restrictions are an issue. Also, the light weight of the plank and with the addition of small amounts of additional fireproofing can potentially reduce cost.

Disadvantages

Even with its quick erection time, a drawback of a hollow core system is it has to use heavy equipment to hoist it into place that can take a lot of project location. The hollow core planks themselves cannot simply support the floor alone and have to employ a framing system wither steel or masonry walls. With this the framing can result in added floor thickness and could be a problem if floor to floor height is an issue. Hollow core floors come in standard lengths and do not work well if there are irregular bay sizes or bays that are not rectangular shaped.

Results

To support the loads present a 4 foot, 8 inch 4HC8 with 58-S stand was selected. The plank was chosen using the Precast Concrete Association's (PCA) design aids and was not required to have any topping thickness. The floor depth was determined to be 20. 2 inches, being the second deepest system. Due to deflection purposes the framing size had to be increased significantly resulting in a heavier and larger section when compared to the other steel framed system.

Using RS Means, the cost per square floor was found to be $22.27, being the most expensive system and much more costly when compare to the existing one way slab system. Again with the Embassy Suites Hotel having zoning restrictions, limiting the height above grade the increased floor system depth would cause construction difficulties. This coupled with increased cost would not make this a feasible option for a design in future reports.

System Comparison Summary

Comparison Criteria	Two Way Slab with Drop Panels	One Way Slab with Reinforced Concrete Beams	Composite Deck System on Steel Framing	Hollow Core Concrete Planks on Steel Framing
Slab Self Weight (psf)	100	125	39	56
Slab Depth (inches)	8	10	4	8
Overall Weight (psf)	187.5	1125	48.7	67.9
Overall Depth (inches)	11.5	30	13.87	20.2
Fire Rating	2 Hr.	2 Hr	2 Hr.	2 hr
Fire Protection	Sufficient Alone	Sufficient Alone	Spray	Spray
Modification of Building Design	Existing	Increase Floor to Floor Ht.	Increase Floor to Floor Ht.	Increase Floor to Floor Ht.
Constructability	N /A	Moderate	Easy	Easy
Cost Per Square Foot	$17.35	$22.10	$16.80	$22.27
Feasibility	Yes	No	Yes	No

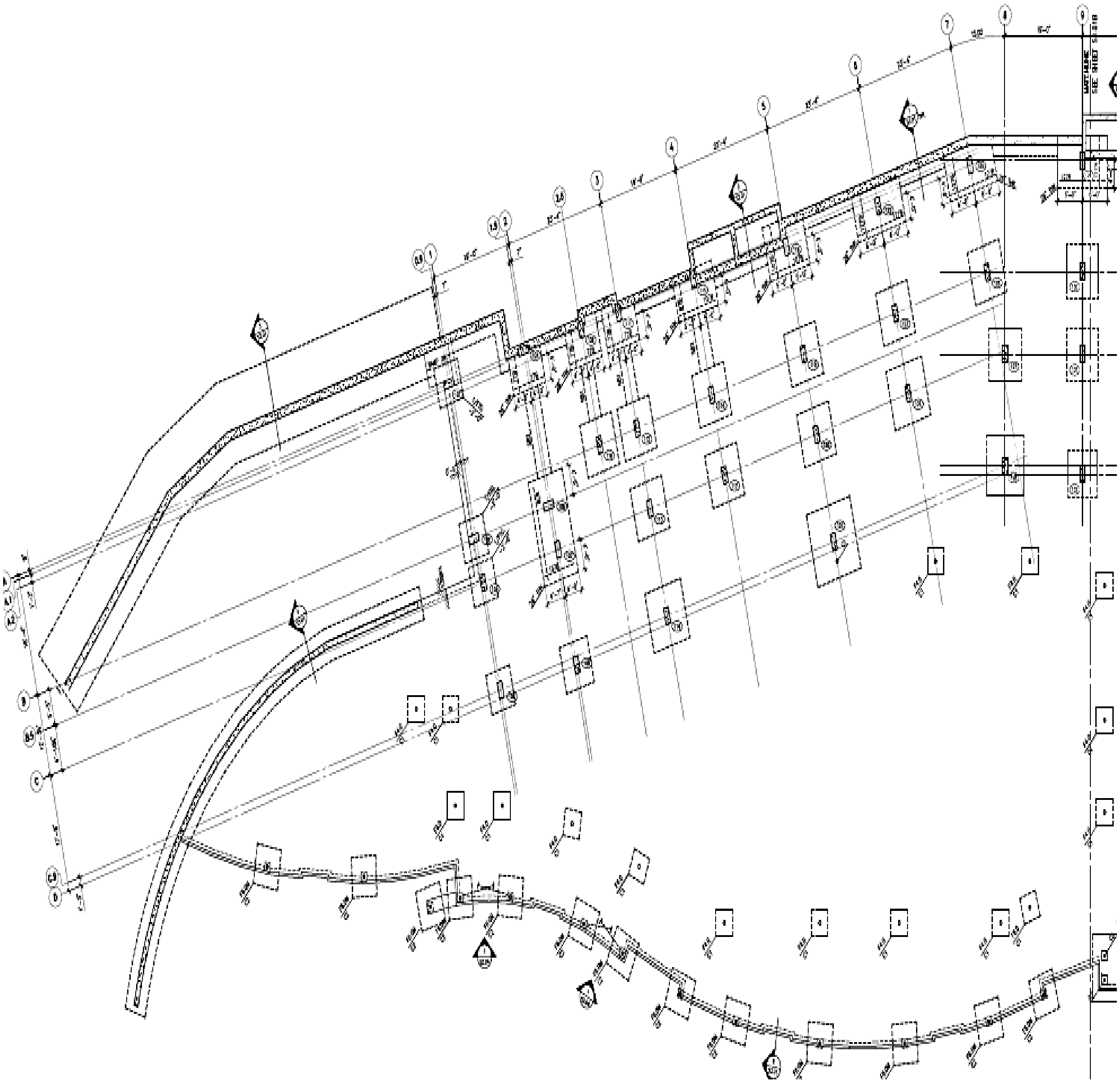
Conclusion

Overall, developing alternative design options for the flooring systems in the Embassy Suites Hotel, one can gain a better understanding of how structural design options are determined using limiting factors that present potential problems. The alternative flooring systems were designed with trying to achieve the shallowest depth due to the fact that there are zoning restrictions for the building in its existing location. The existing system in place at the Embassy Suites Hotel project is a two way flat slab system. The alternatives consists of a one way slab system with reinforced concrete beams, a composite deck system with steel framing and a hollow core plank system on steel framing. In comparing the alternate floors, factors such as impact on floor to floor height, system weight, floor depth, and cost were considered.

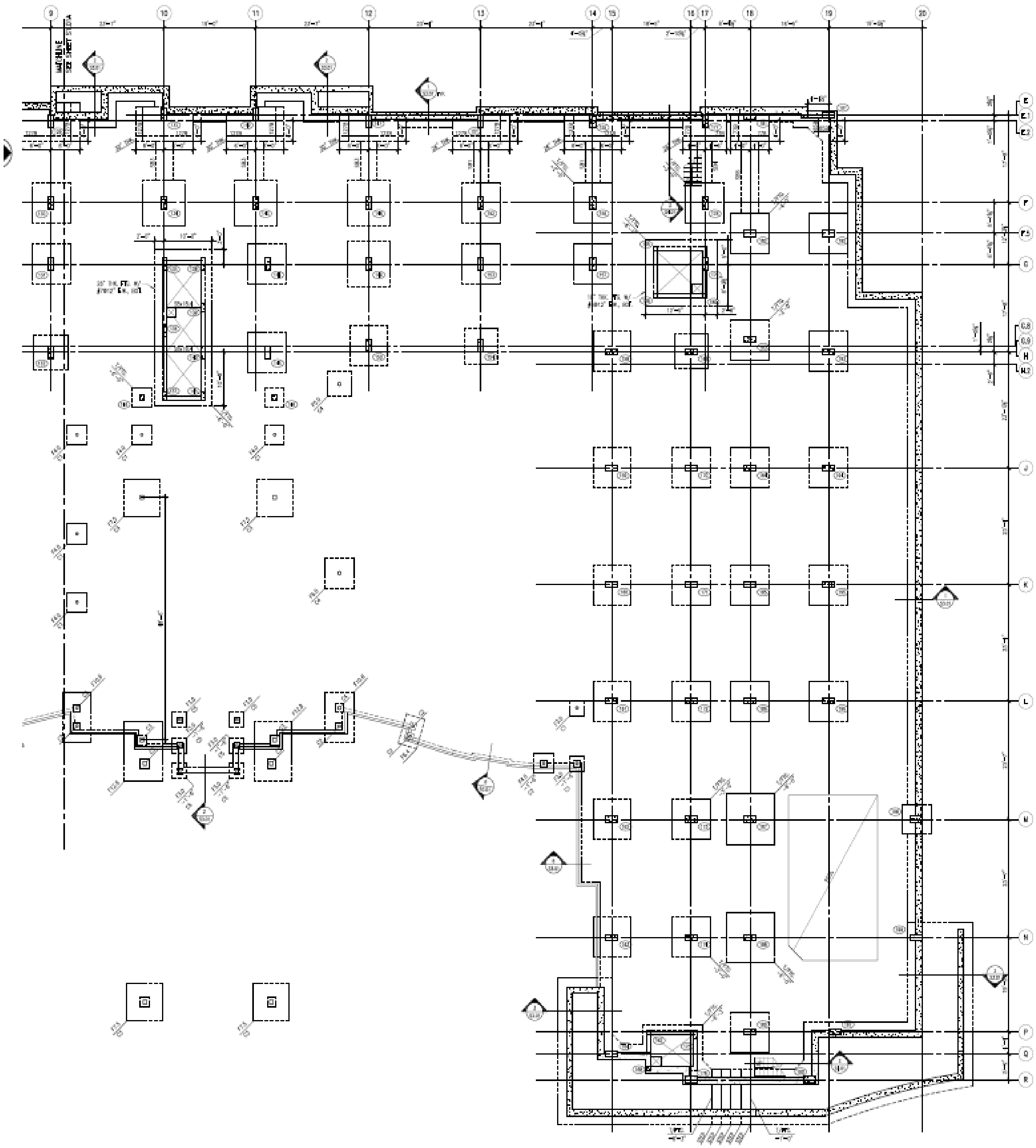
In analyzing the existing system and comparing it to the alternate designs it was found that due to height limitations and relative low cost the existing floor system is the best solution. The one way slab system was by far the most out of reach for an alternative system to be implemented With its high cost, large depth and considerable weight this system cannot be feasible to due limitations present when compared to the existing system. The precast hollow core planks offers value when it is compared to overall construction type, having the planks premade off site and easily installed. This system turned out to contain the largest cost and the second largest depth having almost a foot of difference when compared to the existing system. If that 1 foot was compounded on every floor the building would not meet the zoning codes requirements for height.

The most promising and potentially feasible alternative system was the composite floor system with steel framing. This system with its low cost, low floor to floor height only being a little more than 2 and a half inches larger than the existing system and the lightest weight can a great alternative design to consider. This system would allow for mechanical and electrical equipment to be easily placed underneath the floor without any substantial effect to the ceiling height or over-all floor to floor height. The main potential drawback to this system was its lack of natural fireproofing qualities but this can easily be rendered by the used of spray fire proofing partial. Further examination of the impact on the foundation and conversion of the concrete columns to steel have to be taken in consideration in the future. With all the factors present the composite deck with steel framing system is viable option and logical choice for the Embassy Suites Hotel project.

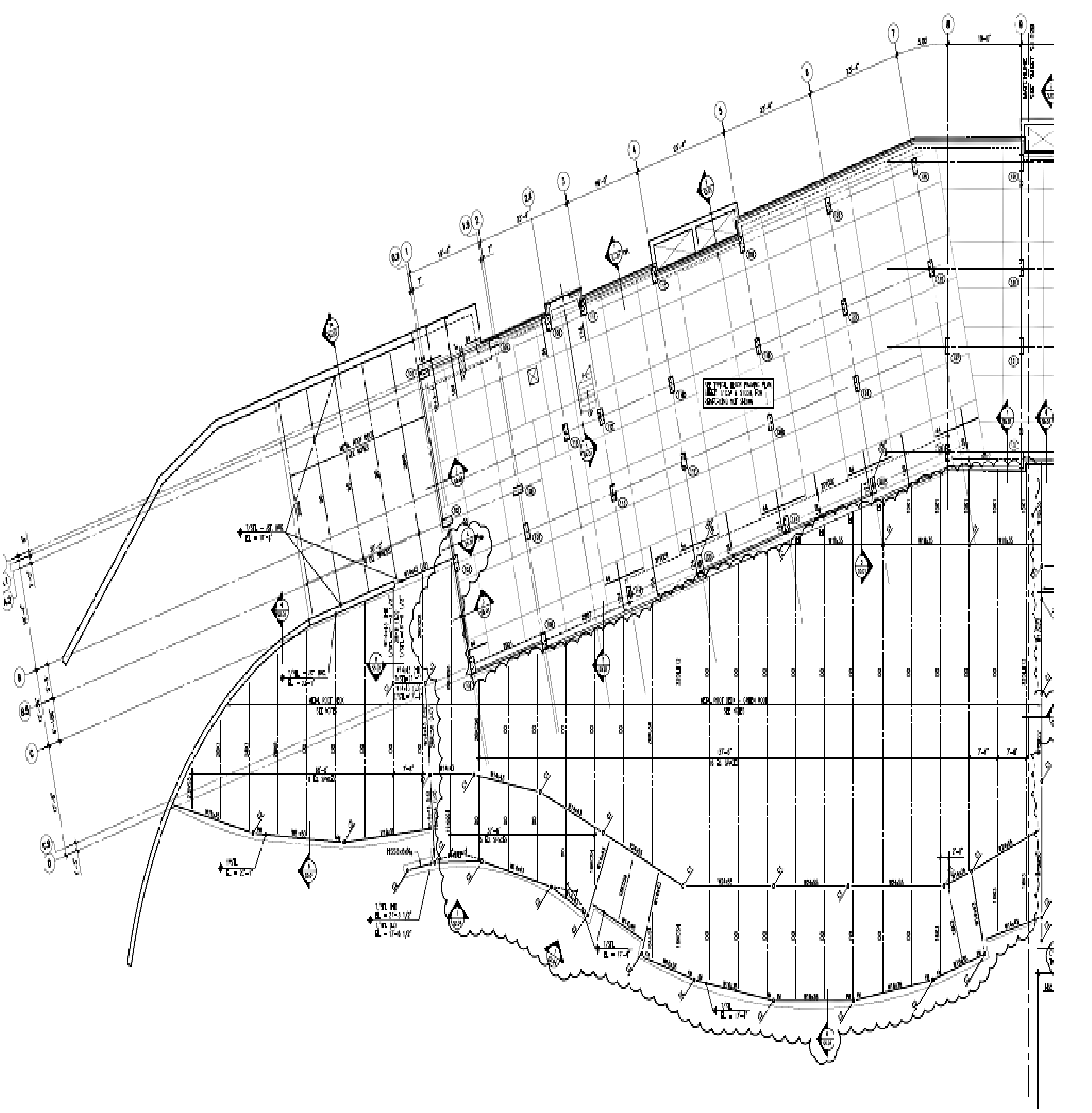
Appendix A: Plans



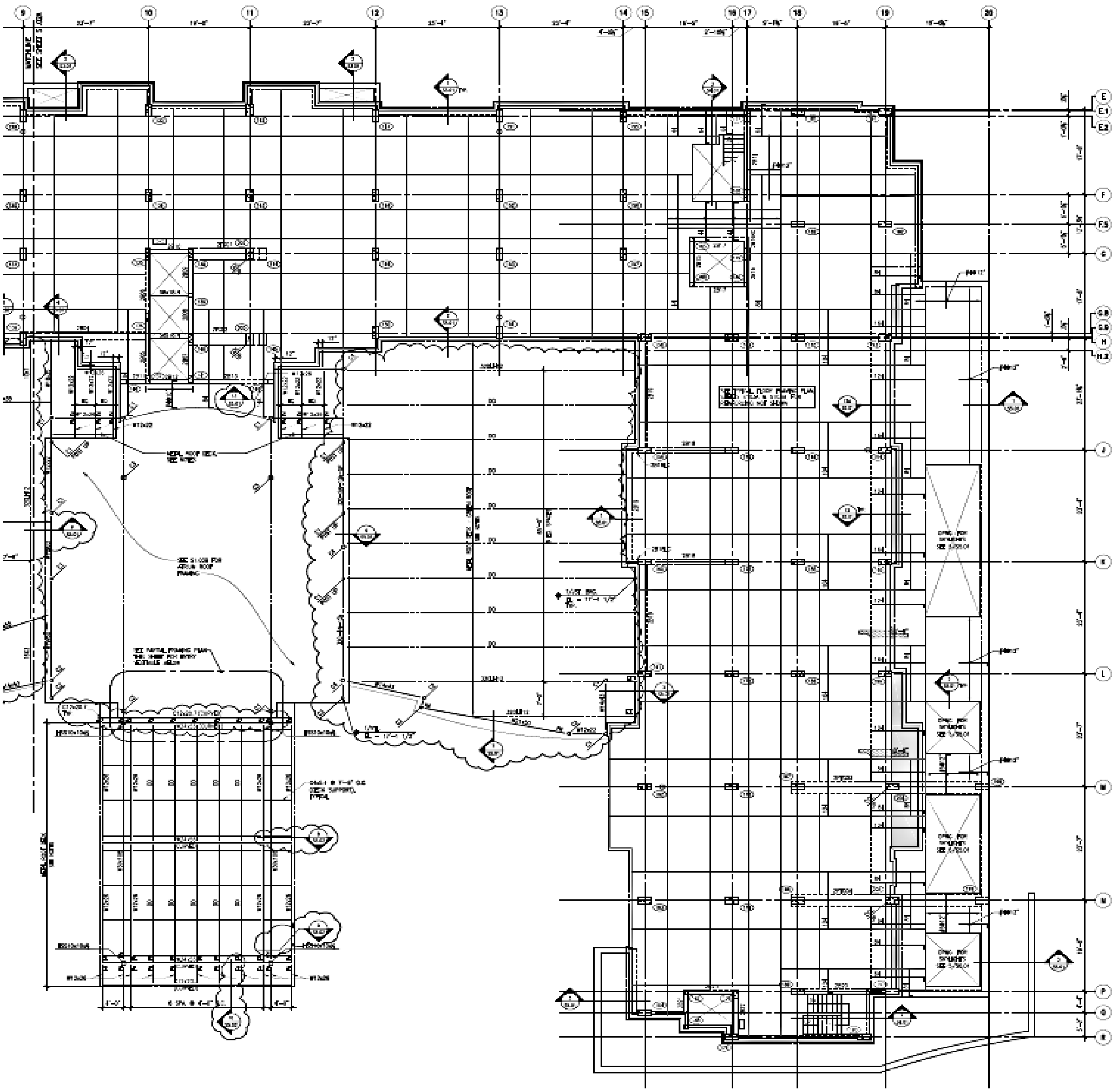
Foundation Plan A



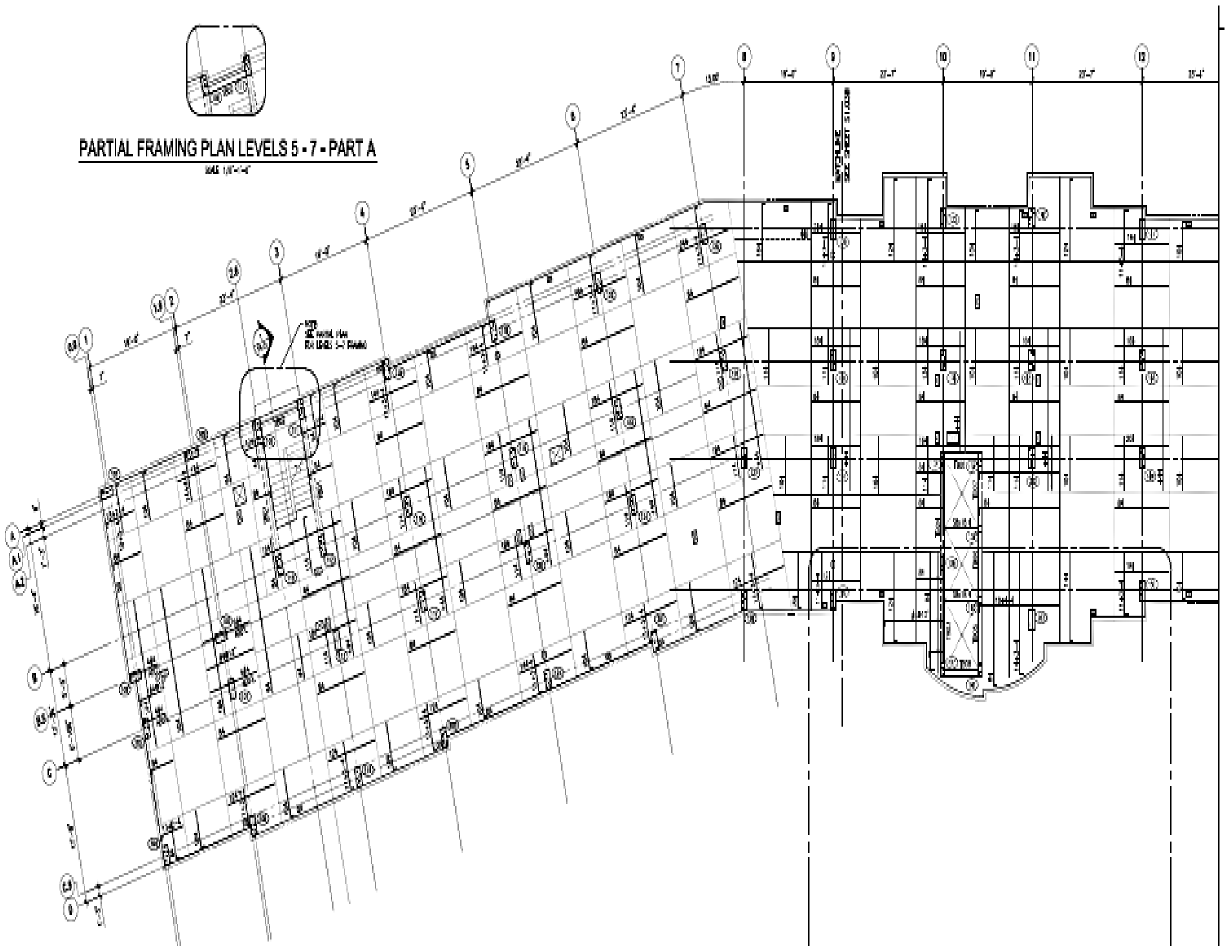
Foundation Plan B



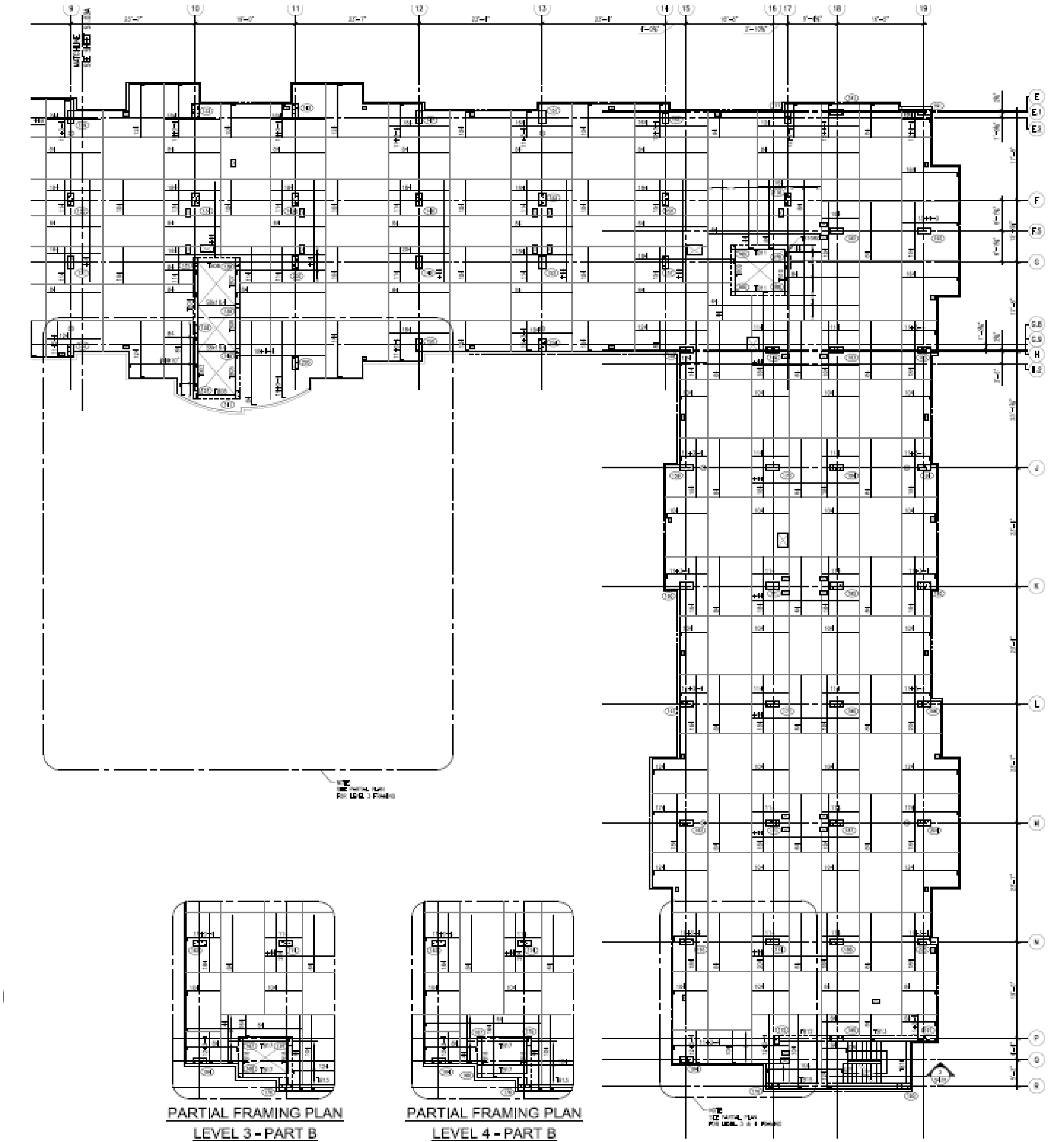
Floor Plan 2 Part A



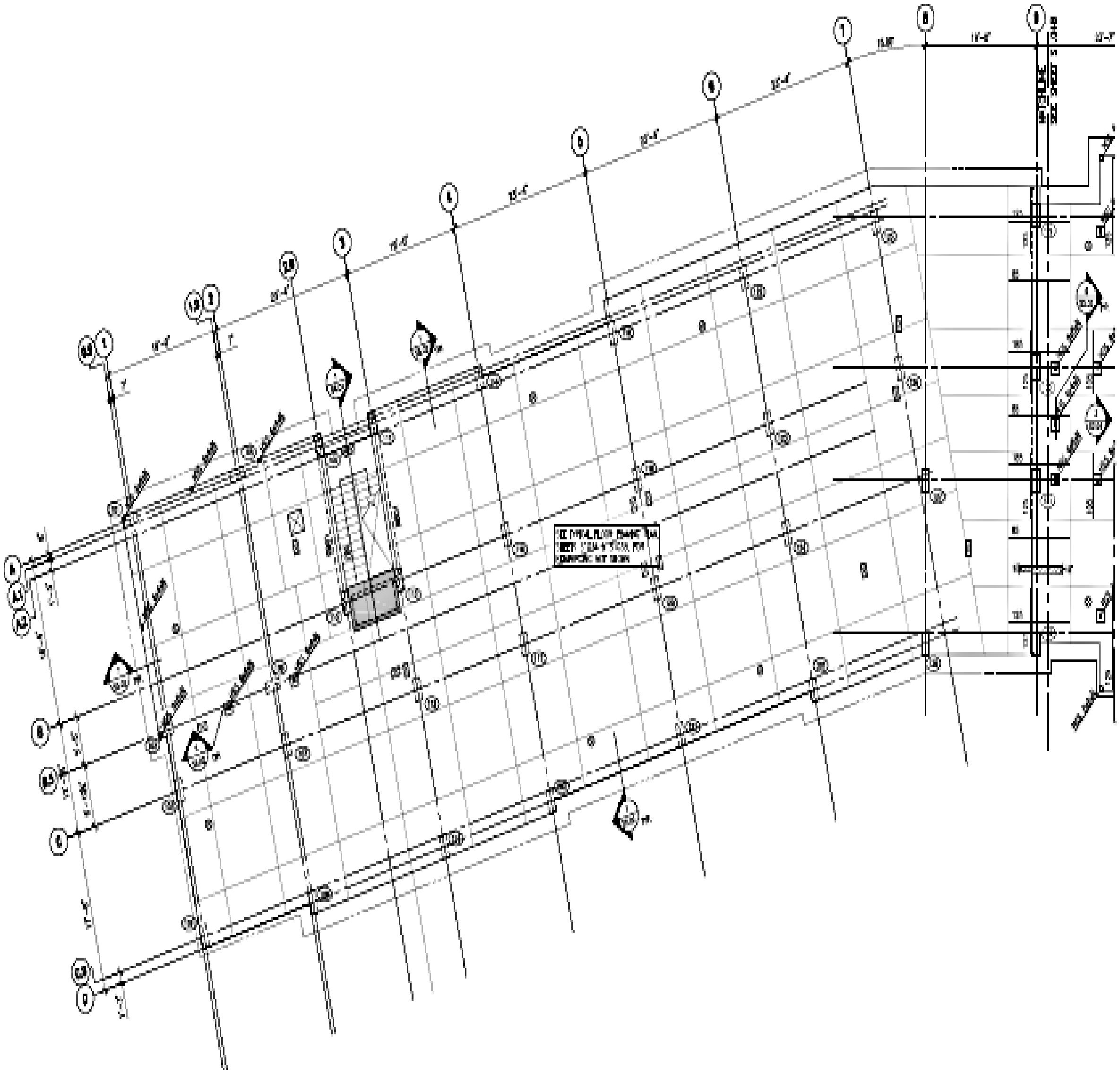
Floors Plan 2 Part B



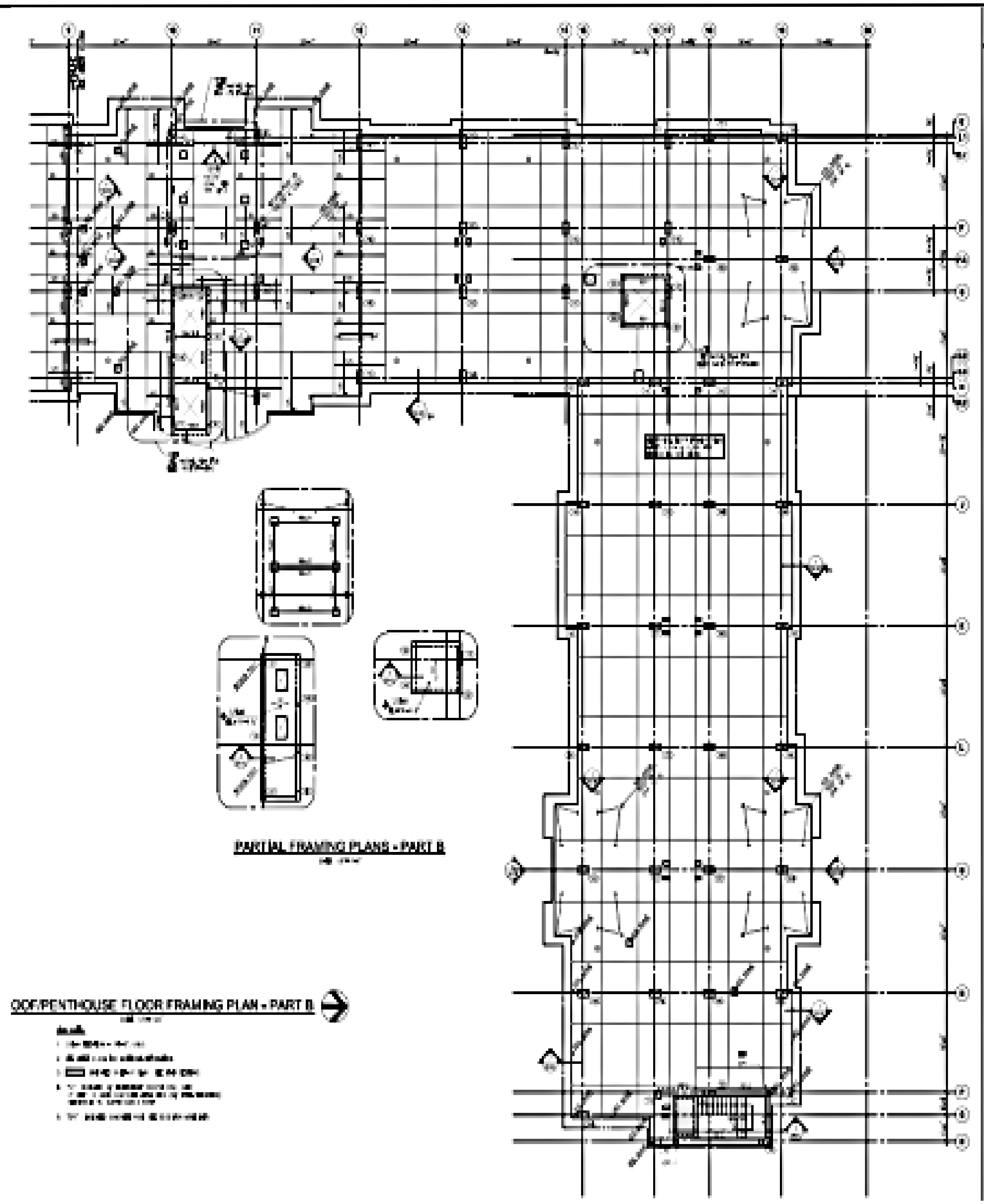
Floors Plan 3 to 7 Part A



Floor Plan 3-7 Part B



Main Roof Level Part A

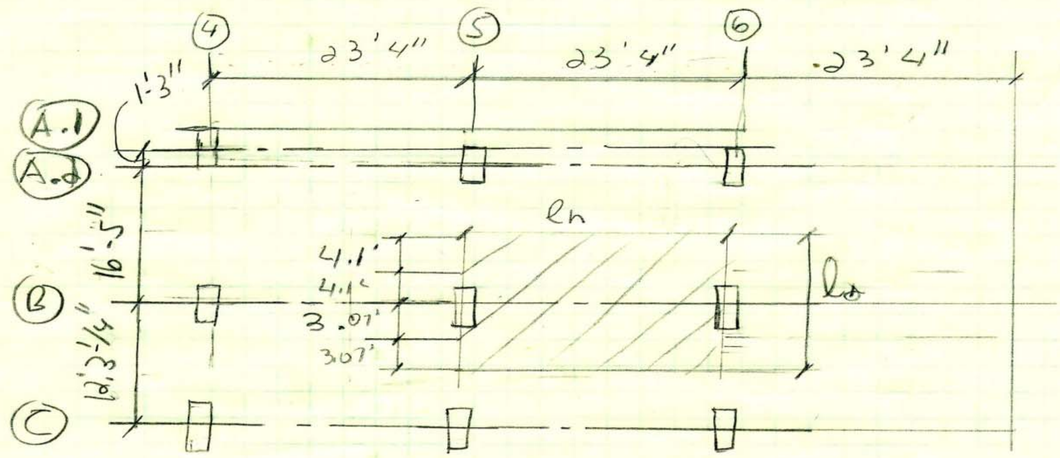


Main Roof Level Part B

Appendix B: Two Way Slab with Drop Panels

D1 | Slab check | TECH 1 pg 1/5

2 Way Slab Spot Check



8" - Concrete Slab

#4 reinforcing spaced 10" O.C each way

$f'_c = 5000psi$, $f_y = 60,000psi$

$l_n = 23'.33$, $l_2 = 14.34'$

- Dead Load
  - concrete = 150pcf (8/12) = 100psf
  - MEP/ = 10psf
- Live Loads
  - Guestroom floor → 40psf
  - partitions → 15psf

DL | Slab Check | TECH 1 pg 4/5

$q_u = 1.2(110) + 1.6(55) = 220\ psf$

$M_o = \frac{q_u l_2 l_n^2}{8} = \frac{220(14.34)(23.33)^2}{8} = 214.6\ ^{k\text{-}ft}$

- Positive & Negative Moments (interior span)

$M^- = .65 M_u = .65(214.6^{\text{k-ft}}) = 139.5^{\text{k-ft}}$

$M^+ = .35 M_u = .35(214.6^{\text{k-ft}}) = 74.4^{\text{k-ft}}$

- Column Strip Moments

$l_1/l_2 = \frac{23.33}{14.34} = 1.63$

$M^-_{col} = .75(M^-) = .75(139.5^{\text{k-ft}})$
$= 104.6^{\text{k-ft}}$

$M^+_{col} = .60(M^+) = .6(74.4^{\text{k-ft}}) = 44.9^{\text{k-ft}}$

- Middle Strip Moments

$M^-_{mid} = .25(M^-) = (.25)(139.5) = 42.25^{\text{k-ft}}$

$M^+_{mid} = .4(M^+) = (.4)(74.4) = 36.4^{\text{k-ft}}$

- for column strip

$b = 86.04 \quad h = 8'' ,\ d = 8'' - .5'' - .25'' = 7.25$

$(18)\ \#4\ @\ 10''\ O.C.\ (bottom)$

$A_s = (18)(.2) = 3.6$

$a = \frac{A_s F_y}{.85 f'_c b} \quad \frac{(3.6)(60)}{.85(5)(86.04)} = .591$

$c = a/\beta_1 = \frac{.591}{.8} = .739$

$\varepsilon_y = \frac{\varepsilon_c}{c}(d-c) ,\quad \varepsilon_y = \frac{.003}{.739}(7.25 - .739)$

$\varepsilon_y > .005$ use $\phi = .9$

DL | Slab check | TECH 1 pg 3/5

$\phi M_N = .9(3.6)(60)(7.25 - \frac{.738}{2}) = 111.5^{k\text{-}ft}$

$\phi M_N = 115^{k\text{-}ft} > M^-_{col} = 104.6^{k\text{-}ft}$ ok ✓

- using same reinforcement (positive reinforcement)
  (18) #4 (bottom & top)
  $\phi M_N = 111.5^{k\text{-}ft} > M^+_{col} = 44.9^{k\text{-}ft}$ ok ✓

- for middle strip
  * parameters same as above
  $A_S = (9)(.2) = 1.8$
  $AS = \frac{(1.8)(60)}{(.85)(5)(86.04)} = .295$, $c = \frac{.295}{.8} = .369$

  $\varepsilon_Y = \frac{.003}{.369}(7.25 - .369)$, $\varepsilon_Y > .005$, so $\phi = .9$

  $\phi M_N = .9(1.8)(60)(7.25 - \frac{.295}{2}) = 57.5^{k\text{-}ft}$

  $\phi M_N = 57.5^{k\text{-}ft} > M^-_{mid} = 42.25^{k\text{-}ft}$ ok ✓

- using same reinforcement (positive reinforcement)
  $\phi M_N = 57.5^{k\text{-}ft} > M^+_{mid} = 36.4^{k\text{-}ft}$ ok ✓

- * slab is ok for flexure

DL | Slab check | TECH 1 pg 4/5

- per ACI 318-08 Tbl. 9.5c

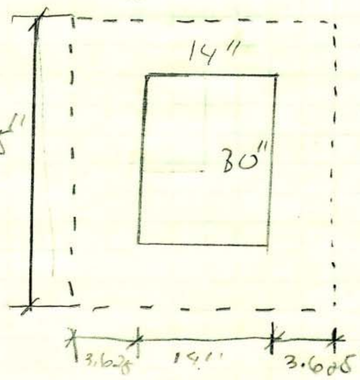
thickness $> \frac{l_n}{36}$, $t > \frac{279.96}{36} = 7.78''$

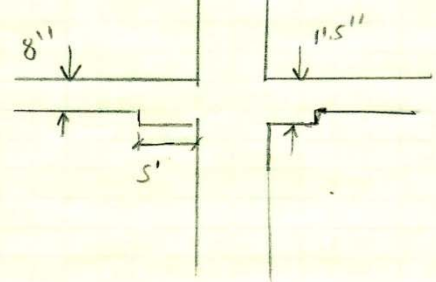
- actual slab thickness 8" ok ✓
- no need to check deflection

- Punching Shear

$h = 11.5''$
$d = 7.25''$

14X30
COL B19





self wt $- 1.2\left[\left(\frac{11.5}{12}\right)(150) + 10_{psf}\right] + 1.6(55\,psf) = .261\,ksf$

$V_u = .261\,ksf\left[(23.33')(14.34') - \left(\frac{21.25}{12}\right)\left(\frac{37.25}{12}\right)\right] = 85.9^k$

$b_o = 2(21.25 + 37.25) = 117''$

$V_c = 4\sqrt{f'_c}\, b_o d = 4\sqrt{5000}\,(117)(7.25) = 239.9^k$

DL | Slab check | TECH 1 pg 5/5

$$V_c = \left(2 + \frac{4}{(30/14)}\right)\sqrt{5000}\,(117)(7.25) = 291.4^{k}$$

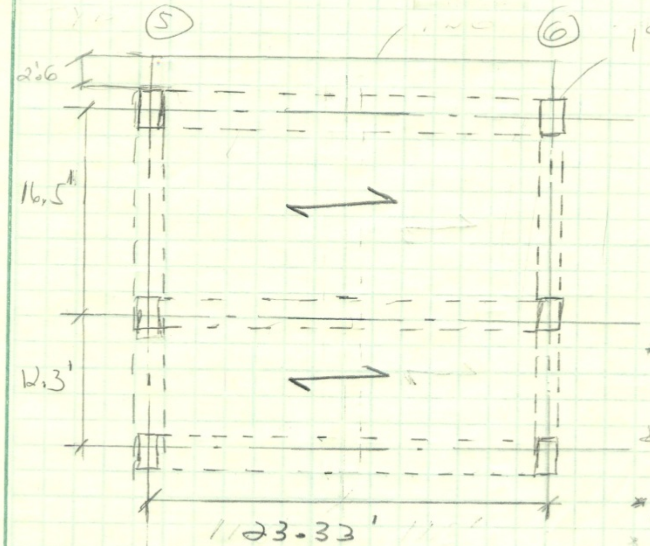
$$V_c = \left(\frac{(40)(7.25)}{117} + 2\right)\sqrt{5000}\,(117)(7.25) = 268.6^{k}$$

$$\phi V_c = .75(239.9) = 179.9 > 85.4^{k} \text{ ok}$$

∴ slab ok for punching shear

Appendix C: One Way Slab with Reinforced Concrete Beams

Dominick Lovallo | 1-WAY SLAB | Tech 2 pg 1/6



* span values rounded for simplicity of calculation

* TBC 9.5(a) → SLAB MIN. THICKNESS

* end span = $l/24$

* interior span = $l/28$

* $f'c = 5{,}000$ psi

$l/28 = \frac{23.33(12)}{28} = 9.99$ Try 10"

$W_{LL} = 40\,psf + 15\,psf = 55\,psf$

$W_{DL} = 10\,psf + (10/12)150 = 135\,psf$ * load combo control by inspection

- Factored load → $1.2(135\,psf) + 1.6(55\,psf) = 250\,psf$

* Try #4 Bars

$d = h - \text{clr cov.} - d_b/2 = 10'' - .75 - .5/2 = 9''$

- Beam design

$w = 250(23.33)/1000 = 5.83\ k/ft$

S.W ESTIMATE

$$M_u = \frac{(5.83)(16.5 - (14/12))^2}{8} = 171.3(1.1) = 188.5\ k\text{-}ft$$

DL | 1-WAY SLAB | Tech 2 pg 3/6

- Beam size → $bd^2 = 20 M_u$, let $b = 14''$

$(14)(d^2 = 20(188.5)) = d = 14.$

Try $h = 20$,

$W_{sw} = \frac{14''(20)}{144}(150pcf) = .2925$

$W_u = 5.83 + 1.2(.292) = 6.18$

$M_u = \frac{(6.18)(15.33)^2}{8} = 181.5$

$3811.5 > (20)(181.5) = 3630$ ok

- Required steel

$A_S = \frac{M_u}{4d} = \frac{181.5}{4(17.5)} \text{ k-ft} = 2.59 \text{ in}^2$

* Try #6 $2.59/.44 = 6$ BARS $A_S = (.44)(6) = 2.64 \text{ in}^2$

$d = 20 - 1.5''_{CLR} - .75/2 = 18.125''$

- Nominal Moment ($f'c = 5{,}000$, $F_Y = 60ksi$)

$a = \frac{A_s f_y}{.85 f'c\, b} = \frac{(2.64)(60)}{.85(5)(14)} = 2.66''$

$c = a/\beta_1 = 2.66/.8 = 3.325$

$\varepsilon_s = .003\left(\frac{18.125 - 3.325}{3.325}\right) = .0134 > .005$ ok

Use $\phi = .9$

$\phi M_n = .9(2.64)(60)(18.125 - 2.64/2) = 199.5$ k-ft

$\phi M_n > M_u$ ok

DL | 1-WAY SLAB | Tech 2 pg 3/6

- Min area of steel

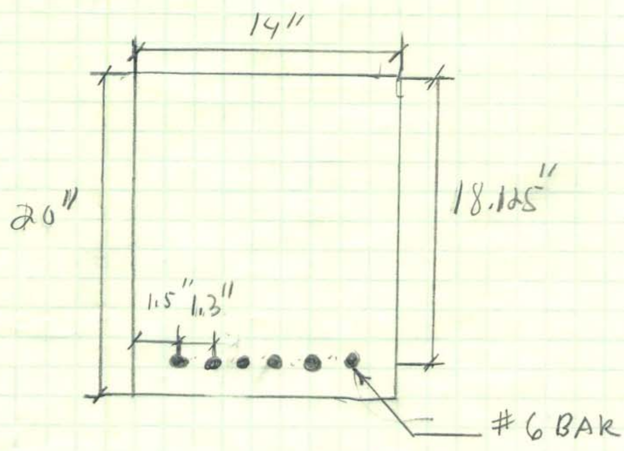
$A_{s\,min} = \frac{200}{f_y} bd = \frac{200}{60{,}000}(14")(18.125) = .846 in^2$

$A_s = 2.2 in^2 > A_{s\,min} = .846$ ok ✓

- Check Max. Reinforcing Ratio

$\rho_{max} = .85 \beta_1 \left(\frac{f'c}{f_y}\right)\left(\frac{\varepsilon_u}{\varepsilon_u + .005}\right) = .85(.8)\left(\frac{5}{60}\right)\left(\frac{.003}{.008}\right) = .0212$

$\rho = \frac{2.64}{(14)(18.125)} = .0104 < .0212$ ok ✓



* Use 14" x 20" beam w/ (6) #6 BARS

DL | 1-WAY SLAB | Tech 2 pg 4/6

- Slab Design

$t = 5.5''$, * Try #4 Bars

- using unit strip method → $As_{min} = .002\,bh$
- 1' section → $As_{min} = .002(12'')(10'') = .24$

$As/ft = A_b\left(\frac{12''}{SPACING}\right)$, $.24 = .2\left(\frac{12''}{SPACING}\right) = 10''$

SPACING ≤ 10'' , S_{MAX} min $\begin{cases} 3(t) = 3(18) = 30'' \\ 18'' \end{cases}$

crack control SPACING $\begin{cases} S = 15\left(\frac{40{,}000}{60{,}000}\right) - 2(.75) = 8.5'' \\ 12\left(\frac{40{,}000}{60{,}000}\right) = 8'' \rightarrow \text{controls} \end{cases}$

* Try #4 @ 8'' O.C. , $A_s = .3\,in^2/ft$

$\rho = \frac{A_s}{bd} = \frac{.3}{(12'')(10)} = .0025$, $\phi = .9$

$a = \frac{(.3)(60)}{.85(5)(12)} = .353''$, $d = 10'' - .75 - .5/2 = 9''$

$\phi M_N = (.9)(.3)(60)\left(9'' - .353/2\right) = 11.9^{k\text{-}ft}$

$W_u = 1.2\left[\left(\frac{10}{12}\right)(150)(1ft) + 10psf(1ft)\right] + 1.6\left[55psf(1ft)\right]$

$W_u = .250\,k/f$

$M_u = \frac{.250\,(23.33)^2}{8} = 17^{k\text{-}ft}$

$\phi M_N < M_u \rightarrow NG$

$17^{k\text{-}ft} = .9(As)(60)\left(9'' - .353/2\right)$

Required $A_s = .43\,in^2/ft$

DY | 1-WAY SLAB | TECH 2 Pg 5/6

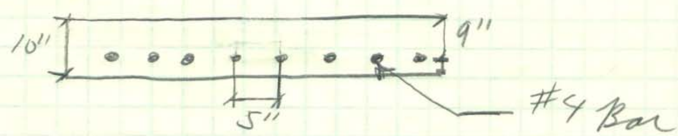
Try #4 @ 5" O.C., $.48\ \text{in}^2/\text{ft}$

$\rho = \frac{.48}{(12)(10)} = .004 \quad \phi = .9$

$a = \frac{.48(60)}{.85(5)(12)} = .564, \quad \phi M_n = (.9)(.48)(60)(9 - .564/2)$

$\phi M_n = 18.8^{k\text{-}ft} > M_u = 17^{k\text{-}ft}$ ok

$\boxed{\text{* use a 10" slab w/ \#4 BAR SPACED 5" O.C}}$



DL | 1-WAY SLAB | TECH 2 | pg 6/6

Deflection

- Beam $\longrightarrow \frac{(14'')(20'')^3}{12} = 9333.3$

$\Delta_{TL} = \frac{5w\ell^4}{384EI}$, $w_T = [10psf + (\frac{8}{12})150](23.33) + \frac{(14)(20)}{144}(150)$

$w_T = 3.44$ 125 291.7

$\Delta_{TL} = \frac{5(3.44)(16.5)^4(1728)}{384(4036.5)(9333.3)} = .15$

slab

$w_T = (10psf + (\frac{10}{12})(150)(1ft)$, $I = 1000 in^4$

$\Delta_{TL} = \frac{5(.135)(23.33)^4(1728)}{384(4030.5)(1000)} = .22$

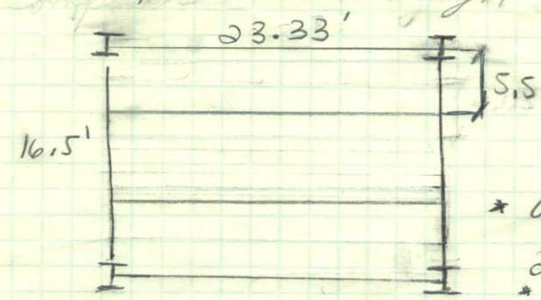
$\ell/480 = \frac{23.33(12)}{480} = .583 > .22$ ok ✓

$\ell/360 = \frac{23.33(12)}{360} = .778 > .15$ ok ✓

Appendix D: Composite Floor System on Steel Framing

Dominick Lovallo | Composite Floor System | Tech 2 pg 1/1

- Composite Floor Design $16.5/3 = 5.5'$



$W_{LL} = 40psf + 15psf = 55psf$

$W_{DL} = 10psf$

* Using VULCRAFT Manual

2VLI 22, span 5'6"

* 3 span condition

Deck span (5.5') < max. Construction span (9'4") ✓

Superimposed LL (274psf) > 55 + 10psf + 39 = 104psf

* assume use 5,000 psi concrete

- Beam Design

$w_u = [(1.2)(10+39) + 1.6(55)](5.5) = .807KLF$

$V_u = \frac{(.807)(23.33)}{2} = 9.4^k$

$M_u = \frac{wl^2}{8} = \frac{.807(23.33)^2}{8} = 55^{k\cdot ft}$

b_{eff} (min)

$\frac{23.33(12")}{8} = 35"$

$\frac{5.5'(12)}{2} = 33" \rightarrow$ controls

$b_{eff} = 66"$

* Due to height limitations chose member based on depth

DY | Composite Floor | Tech 2 pg 2/7

* assume $a = 2$, $d = 10, 12$

$$A_s = \frac{M_u}{\phi F_y (d/2 + t - a/2)} = \frac{55(12)}{(.9)(50)(10/2 + 4 - 2/2)} = 1.83\,in^2$$

$$= \frac{55(12)}{(.9)(50)(12/2 + 4 - 2/2)} = 1.62\,in^2$$

* Can W10x12, 12x14
* try W10x12, $A_s = 3.54\,in^2$, $d = 9.87''$, $t_w = .19''$, $t_f = .21''$

$I_x = 53.8\,in^4$, $S_x = 10.9\,in^3$, $Z_x = 12.6\,in^3$

$h/t_w = 46.6$

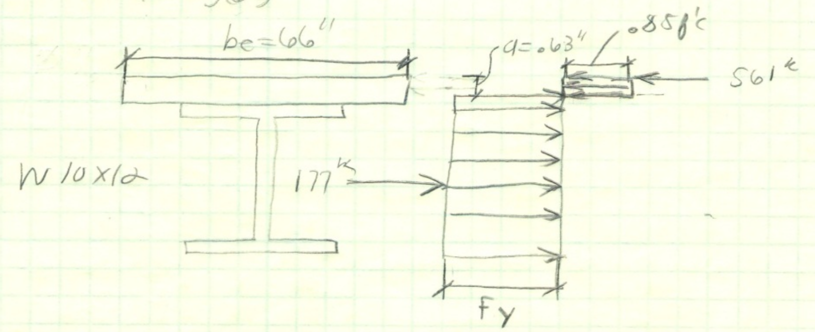
* assume ignore concrete in ribs of deck

$V'_c = .85(5ksi)(66'')(2'') = 561$

$V'_s = (3.54)(50\,ksi) = 177$

$V'_c > V'_s \rightarrow$ PNA is in the slab

$a = \frac{(3.54)(50ksi)}{.85(5ksi)(66)} = .63''$ * $a_{ACTUAL} < a_{ASSUMED}$ ok ✓



DL | Composite Floor | Tech 2 pg 3/7

- check WET Concrete Deflection

$W_{wc} = 39psf(5.5) + 12 plf = .2265^{k/FT}$ (12 plf = BEAM S.W.)

$\Delta_{LL} = \frac{5(.2265)(23.33)^4(1728)}{(384)(29000)(53.8)} = .967$

$l/240 = \frac{23.33(12)}{240} = 1.167 > .967$ ok ✓

- Check unshored Strength

$W_{LL} = C_{LL} = 20psf(5.5) = .110klf$

$W_{DL} = (39psf)(5.5') + 12plf = .2265klf$

$W_u = 1.2(.2265) + 1.6(.110) = .4478$

$M_u = \frac{.4478(23.33)^2}{8} = 36.5^{k\text{-}ft} < \phi M_p = 46.9^{k\text{-}ft}$

$\boxed{\text{* Use (4) W10x12}}$

DL | Composite Floor | Tech 2 4/7

$\phi M_N = \phi(A_s F_y)[(d/2) + h_r + t - (a/2)]$

$= (.9)(3.54)(50)[9.87/2 + 2'' + 2'' - .63/2]/12'' = 114.9^{k\text{-}ft}$

$\phi M_N > M_u$ ok ✓

- Check shear

$\phi V_N = (\phi)\, .6 F_y A_{WEB} = (.9)(.6)(50ksi)(.19)(9.87)$
$= 50.6^k$

$\phi V_N > V_u$ ok ✓

- Design Shear Studs

$\Sigma Q_n = \min(V_s', V_c') \to 177^k$

* Assume 3/4" DIA. STUDS, DECK Runs ⊥ to beams, 1 STUD/RIB, WEAK POSITION

$Q_n = 17.2$, $\#\,STUDS = \frac{\Sigma Q_n}{Q_n} = \frac{177}{17.2} = 10.3 \times 2 = 21$

* use 24 STUDS/BEAM

- Check W_{LL} Deflection

$W_{LL} = 55 psf (5.5') = 302.5$ * use $I_{LB} = 180$ @ $Y_2 = 3.5$

* conservative assumption actual $Y_2 = 3.68''$

$\Delta_{LL} = \frac{5(.3025)(23.33)^4(1728)}{384(29{,}000)(180)} = .386$

$l/360 = \frac{23.33(12)}{360} = .778 > .386$ ok ✓

DL | Composite Floor | Tech 2 pg 5/7

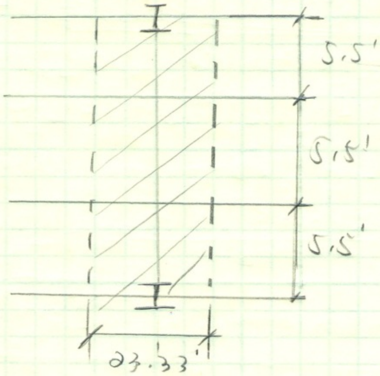
<u>Girder Design</u>

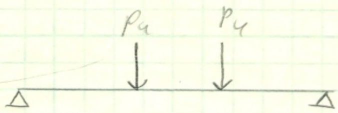
$W_{T(DL)} = (39psf + 10psf)(5.5') = .2695$

$W_{T(LL)} = .3025$

$W_{T(DL,GIRDER)} = .2695(16.5) = 4.45^k$

$W_{T(LL,GIRDER)} = .3025(16.5) = 4.99^k$





$3 @ 5.5' = 16.5'$

$P_u = 1.2(4.45) + 1.6(4.99)$

$P_u = 13.3^k$, $M_u = 13.3(5.5')$
$M_u = 73.2^{k\text{-}ft}$

$b_{eff} = \begin{cases} \frac{16.5(12)}{8} = 24.75 \rightarrow \text{controls} \\ \frac{23.33(12)}{2} = 140'' \end{cases}$, $b_{eff} = (2)(24.75) = 49.5$

* assume a = 2 *

$A_s = \frac{73.3(12)}{(.9)(50)(10/2) + 4 - 2/2} = 2.44 in^2$

* Try W10 x 12

DL | Composite Floor | Tech 2 pg 6/7

$V'_c = .85(5ksi)(49.5)(2) = 420.8^k$

$V'_s = (3.54)(50ksi) = 177^k$

$V'_c > V'_s \rightarrow$ PNA in slab

$a = \frac{(3.54)(50ksi)}{.85(5ksi)(49.5)} = .84$ $a_{ACTUAL} < a_{ASSUMED}$ ok ✓

$\phi M_N = (.9)(3.54)(50)\left[\frac{9.87}{2} + 2'' + 2'' - \frac{.84}{2}\right]/12 = 113^{k\text{-}ft}$

$\phi M_N > M_u$ ok ✓

- check Shear

$\phi V_N = 50.6^k > V_u = 13.3^k$ ok ✓

- check W_{LL} deflection

*use $I_{LB} = 180 @ Y_2 = 3.5$ * actual $Y_2 = 3.58$

$\Delta_{LL} = \frac{Pl}{28EI} = \frac{4.99(16.5)^3(1728)}{28(29{,}000)(180)} = .265$

$l/360 = \frac{16.5(12)}{360} = .55 > .265$ ok ✓

- check unshored Strength

$W_{DL} = (39)(5.5')(23.33) + 12(23.33) = 5.28^k$

$W_{LL} = (20)(5.5)(23.33) = 2.57^k$

$1.2(5.28) + 1.6(2.57) = 10.4$

$\frac{PL}{4} = \frac{10.4(16.5)}{4} = 43.1^{k\text{-}ft} < \phi M_p = 46.9^{k\text{-}ft}$ ok ✓

DY | Composite Ribbon | Tech 2 pg 7/7

- Design Shear studs

$\leq Q_n = 177$ * assume 3/4" DIA STUDS

1 STUD/RIB , $Q_n = 18.3$ Deck // to girder

$\#STUD = \frac{177}{18.3} = 9.72 \cong 10$, * use 20 STUDS/GIRDER

$\boxed{\text{* Use W10X12}}$

Appendix E: Hollow Core Planks n Steel framing

3.6 Hollow-Core Load Tables (cont.)

3

Strand Pattern Designation
76-S

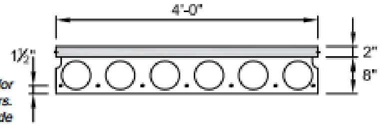
S = straight
Diameter of strand in 16ths
Number of strand (7)

Safe loads shown include dead load of 10 lb/ft² for untopped members and 15 lb/ft² for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

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

Key
385– Safe superimposed service load, lb/ft²
0.1 – Estimated camber at erection, in.
0.2 – Estimated long-time camber, in.

4'-0" × 8"
Normalweight Concrete



f'_c = 5000 psi

f_{pu} = 270,000 psi

Section Properties

	No Topping	2 in. topping
A =	215 in.²	-
I =	1666 in.⁴	3071 in.⁴
y_b =	4.00 in.	5.29 in.
y_t =	4.00 in.	4.71 in.
S_b =	417 in.³	581 in.³
S_t =	417 in.³	652 in.³
wt =	224 lb/ft	324 lb/ft
DL =	56 lb/ft²	81 lb/ft²
V/S =	1.92 in.	

4HC8

Table of safe superimposed service load, lb/ft², and cambers, in. — **No Topping**

Strand designation code	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
66-S	385	345	313	283	260	240	223	204	179	158	140	124	110	98	87	77	69	61	54	48	43	38	33	29						
	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.2	0.1	0.0	0.0	−0.1	−0.2	−0.3	−0.5	−0.6						
	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.1	0.0	−0.1	−0.2	−0.3	−0.5	−0.7	−0.9		−1.4						
76-S	449	407	367	334	309	285	263	242	213	188	167	149	133	119	106	95	86	77	69	62	55	50	44	39	35	31	26			
	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.3	0.3	0.3	0.3	0.2	0.2	0.1	0.0	−0.1	−0.2	−0.4	−0.5	−0.7	−0.9			
	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	0.0	−0.1	−0.2	−0.4	−0.6	−0.8	−1.1	−1.4	−1.7	−2.0			
58-S	422	380	346	316	290	267	247	231	216	202	190	179	169	160	144	130	118	107	97	88	80	72	66	60	54	48	42	37	32	28
	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.5	0.4	0.3	0.2	0.1	0.0	−0.4	−0.3	−0.5	−0.7	−0.9
	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.4	0.3	0.2	0.0	−0.2	−0.4	−0.6	−0.9	−1.2	−1.6	−2.0	−2.4
68-S	476	430	393	361	332	309	286	269	253	235	223	209	200	180	165	153	142	132	121	110	101	92	84	77	70	63	56	51	45	40
	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.2	0.1	−0.1	−0.3
	0.3	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.0	0.9	0.9	0.9	0.8	0.7	0.6	0.4	0.2	0.0	−0.2	−0.5	−0.8	−1.1	−1.5
78-S	488	442	402	370	341	318	295	275	259	241	229	215	203	195	180	168	157	144	135	126	118	110	101	92	84	77	70	64	58	52
	0.3	0.3	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.0	0.9	0.8	0.7	0.6	0.5	0.3
	0.4	0.5	0.5	0.6	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.2	1.3	1.3	1.3	1.3	1.3	1.2	1.2	1.1	1.0	0.8	0.7	0.5	0.3	0.0	−0.3	−0.7

Span, ft

4HC8 + 2

Table of safe superimposed service load, lb/ft², and cambers, in. — **2 in. Normalweight Topping**

Strand designation code	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
66-S	400	365	333	308	282	256	224	197	173	153	135	119	105	93	82	68	56	45	36	26								
	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.2	0.1	0.0	−0.0	−0.1	−0.2	−0.3								
	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	−0.1	−0.2	−0.3	−0.4	−0.6	−0.7	−0.9	−1.2	−1.4								
76-S	474	435	396	366	340	304	267	235	208	184	164	146	130	116	103	88	74	62	51	41	31							
	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.3	0.3	0.3	0.3	0.2	0.2	0.1	−0.0	−0.1	−0.2							
	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.1	0.0	−0.1	−0.2	−0.4	−0.5	−0.7	−0.9	−1.2	−1.4							
58-S	445	405	374	342	318	298	275	260	243	228	217	196	177	159	143	126	110	95	82	70	59	49	40	32				
	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.5	0.1	0.3	0.2	0.1	0.0	−0.1				
	0.3	0.3	0.4	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.3	0.2	0.1	−0.1	−0.2	−0.4	−0.6	−0.9	−1.2	−1.5	−1.8				
68-S		463	426	393	366	342	319	299	282	267	251	239	216	195	177	158	140	124	110	97	84	73	62	53	44	36	28	
		0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.2	0.1	−0.1	
		0.4	0.5	0.5	0.6	0.6	0.6	0.6	0.7	0.7	0.7	0.6	0.6	0.6	0.5	0.4	0.3	0.2	0.0	−0.2	−0.4	−0.6	−0.9	−1.2	−1.6	−2.0	−2.4	
78-S		472	435	402	375	348	325	305	288	273	257	245	232	220	207	186	167	149	133	119	106	94	83	73	64	55	46	38
		0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.0	0.9	0.9	0.7	0.6	0.5	0.3
		0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.8	0.9	0.9	0.9	0.9	0.8	0.8	0.7	0.7	0.6	0.4	0.3	0.1	−0.1	−0.3	−0.6	−0.9	−1.3	−1.7	−2.2

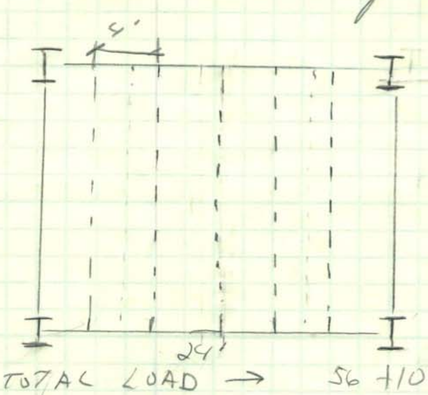
Span, ft

Strength is based on strain compatibility; bottom tension is limited to $7.5\sqrt{f'_c}$; see pages 3–8 through 3–11 for explanation.
See item 3, note 4, Section 3.3.2 for explanation of vertical line.

Dominick Lovallo | Hollow-Core-Floor | Tech 2 | pg 1/2

Hollow Core Floor System w/ Steel Framing

* increase span to 24'



* using PCI handbook

° Try 4'x8" – NO TOPPING

W_{DL} = 56 psf + 10 psf
(S.W of SLAB) (MEP)

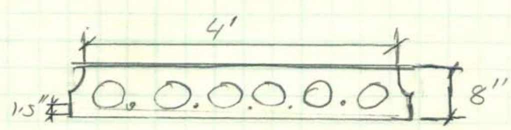
W_{LL} = 40 psf + 15 psf
(FLUR) (PARTITION)

TOTAL LOAD → 56 + 10 + 40 + 15 = 121

* for a span of 24" 58-S strand is selected

160 psf > 121 psf ok ✓

* use 4'x8" (4HC8) @ 24' span w/ 58-S strand hollow core plank



– Beam Design

$W_u = 1.2(56+10) + 1.6(55) = .1672$ ksf

$M_u = \frac{.1672(16.5')(24')^2}{8} = 198.6^{k\text{-}ft}$

* Due to H.T. limitations, choose W10x45
(AISC TBL. 3-2)

* will be less economical due to increased weight

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$\phi M_n = 206^{k\text{-}ft} > 198.6^{k\text{-}ft}$ ok ✓

– check deflection

$\Delta_{LL} = l/360 = \frac{24(12)}{360} = .8$, $\Delta_{TL} = \frac{24(12)}{240} = 1.2$

S.W. GUESS

$1.2 = \frac{5wl^4}{384EI}$, $1.2 = \frac{5(1.997\,klf + .050)(24)^4\,1728}{384(29{,}000)\,I_x}$

$I_x = 439.1\,in^4$, $439.1 > 248\,in^4$ (W10x45) → NG

*TRY W12x58 , $I_x = 475\,in^4$

$\frac{5(.055)(16.5)(24)^4(1728)}{384(29{,}000)(475\,in^4)} = .49$

$\frac{5(1.997\,klf + .058)(24)^4(1728)}{384(29{,}000)(475\,in^4)} = 1.11$

$\Delta_{TL} = l/240 = 1.2 > 1.11$ ok ✓

$\Delta_{LL} = l/360 = .8 > .49$ ok ✓

$\boxed{\text{* Use W12x58}}$

Appendix F: Analysis Comparisons

Dominick Lovallo | Weight Analysis | pg 1/1

Weight Analysis of Floor System
(per 1 bay, loads in psf)

2-WAY-Slab

slab → $(8/12)(150) = 100$ psf

drop panel → $(3.5/12)(150)(2) = 87.5$ psf → 187.5 psf

(2/bay)

1-WAY-Slab w/ Beam

slab → $(10/12)(150) = 125$ psf

Beam → $(20''/12)(150)$(4 Beam/BAY) $= 1000$ psf → 1125 psf

Composite Deck w/ Steel Beam

slab + Deck → 34 psf

Beams → (12 plf/5.5')(4 Beams/bay) = 8.7 psf → 48.7 psf

Girders → (12 plf/23.33')(2 Girders/bay) = 1 psf

Hollow Core Planks w/ Steel framing

Planks → 56 psf

Beams → (58 plf/16.5)(2 Beams/BAY) = 7.03 psf

Girders → (58 plf/24)(2 Girders/bay) = 4.83 psf → 67.9 psf

Dominick Lovallo | Cost Analysis | Tech 2 pg 1

Comparison of Cost of Systems

* cost per square ft. data gathered using RS MEANS 2011
* values were approximated due to irregular

2-Way Slab w/ drop panels
total load → 165 psf + (3.5/12)(150) = 208.75
cost per square foot → 17.35/ft² (total)

1-WAY-Slab w/ beams
total load → 55 psf + 10 psf + 125 psf + 175 psf = 360 psf
Cost per square ft → $22.10/ft²

Composite Deck w/ steel framing
total load → 55 psf + 10 psf + 39 psf + (4)(12 plf/3.5) + (2)(12 plf/16.5) = 114 psf

Costs per/sq ft → $16.80/ft²

Hollow core Plank in steel framing
total load → 55 psf + 56 psf + 10 psf = 121 psf
(PLANKS)

total load → 135
(Beams)

Costs per ft² → $10.39/ft²
(PLANK)

→ $22.27/ft²

Cost per ft² → $11.88/ft²
(BEAM)

Dominick Lovallo | Depth Analysis | Tech 2 pg 1/1

Analyses of System Depths

1- Way Slab system
- Slab → 10"
- Beam → 20"

controlling depth = 20" + 10" = 30"

2- Way Slab System (existing)
- Slab → 8"
- Drop Panel section → 11.5"

controlling depth = 11.5"

Composite Deck w/ Steel framing
- Slab + Deck → 4"
- Beam → 9.87"
- Girder → 9.87"

controlling depth = 4" + 9.87 = 13.87"

- Hollow Core Planks on Steel framing
- Plank → 8"
- Beam → 12.2"

controlling depth = 12.2" + 8" = 20.2"