Voorhees Replacement Hospital

Voorhees, New Jersey

Technical Report #2

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Structural Option

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October 28th, 2009

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Executive Summary

The Virtua Voorhees Replacement Hospital is located in Voorhees, New Jersey. It will be replacing the old Voorhees hospital due to its inability to be renovated. The building is split into two main buildings; a main bed tower and a services building. For this technical report the floor system of the hospital was analyzed and compared to alternate systems.

In this report the current floor system, composite steel beams with composite steel deck, was analyzed and compared to three separate floor systems: a two way flat slab with drop panels, a one way slab with beams, and a precast hollow core slab resting on steel beams. The new systems were designed and compared to the current system using different criteria including weight, depth, cost, vibration, fire proofing, constructability, lead time, framing layout changes, and foundation changes.

The current system, the steel composite system, was found to be the best system for the building because it stood out in many different ways. First, it was by far the lightest of the four systems having almost half the weight of any alternative. It was also one of the cheapest systems costing only $0.07 more than the cheapest alternative. The ease of construction was another selling point for this system.

The only other system that might be a viable option for the hospital would be a two way flat slab with drop panels. This is because the two way system is the cheapest of all the systems. It also has the smallest depth which would allow for more space for mechanical systems. However, the disadvantages of this system are the added weight and the fact that some of the bays would have to be redesigned in order to achieve square shapes.

The remaining two systems, the precast hollow core and the one way slab, were undesirable for a couple of reasons. First, the one way system was much heavier and more expensive than the other systems. Since weight is a serious consideration for this building, the one way slab was eliminated. As for the precast system, it was the most expensive system checked costing upwards of 30% more than the existing composite system. It also had the greatest depth of the systems which would cause the floor to floor heights to be increased.

Introduction

The Virtua Voorhees Replacement Hospital is located in Voorhees, New Jersey (Latitude: 39.84° Longitude: -74.93°), immediately off Rt. 73. It will be replacing the old Voorhees hospital because of its inability to be renovated. The new hospital will have 9 floors, starting with a Garden Level continuing up through Floor 8. The building is broken up into two main areas, the main bed tower (referred to as Building A, or Northern Building in this report), and a services building (referred to as Building B, or Southern Building in this report). The building is also broken up into 7 smaller zones, for ease of reference in the drawings. Figure 1 shows how the building is split up.

The main bed tower, zones 1-3, is 8 levels and holds 350 individual patient rooms. It is a curved building with a curtain wall facing the majority of the site. This curtain wall allows residents to get an excellent view of the site as well as the wetlands that were protected during construction. The majority of the 8 floors in the main tower have the same floor plan with minor differences.

The services building, which holds zones 4 through 7, is attached to the main bed tower via a thin corridor. The services building houses most of the labs, offices, and surgical rooms needed in the hospital. These services are located between the ground floor and the 5th floor. Above the 5th floor, the building narrows, to match the width of the corridor connecting the bed tower and the services building. Mechanical spaces start on the 6th floor and continue up to the 9th floor. The services building also allows for future growth, with the potential to add more space on top of zone 6.

For the second technical report of the Voorhees Replacement Hospital an existing bay's floor system was analyzed and compared to three other systems: a two way flat slab with drop panels, a one way flat slab with beams, and a precast hollow core slab on steel beams. Many factors of each system were compared ranging from its cost to constructability. The bay chosen for analysis is located in Zone 4 on the first floor. Figure 2 shows the location of the bay selected, while Figure 3 shows a close up of the bay. This bay was chosen because it is one of the most typical bays located in Building B. It was chosen as a representative of the entire building.

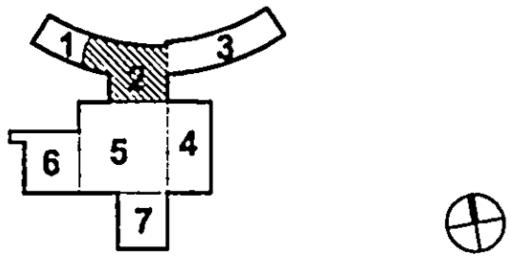


Figure 1 – Key Plan

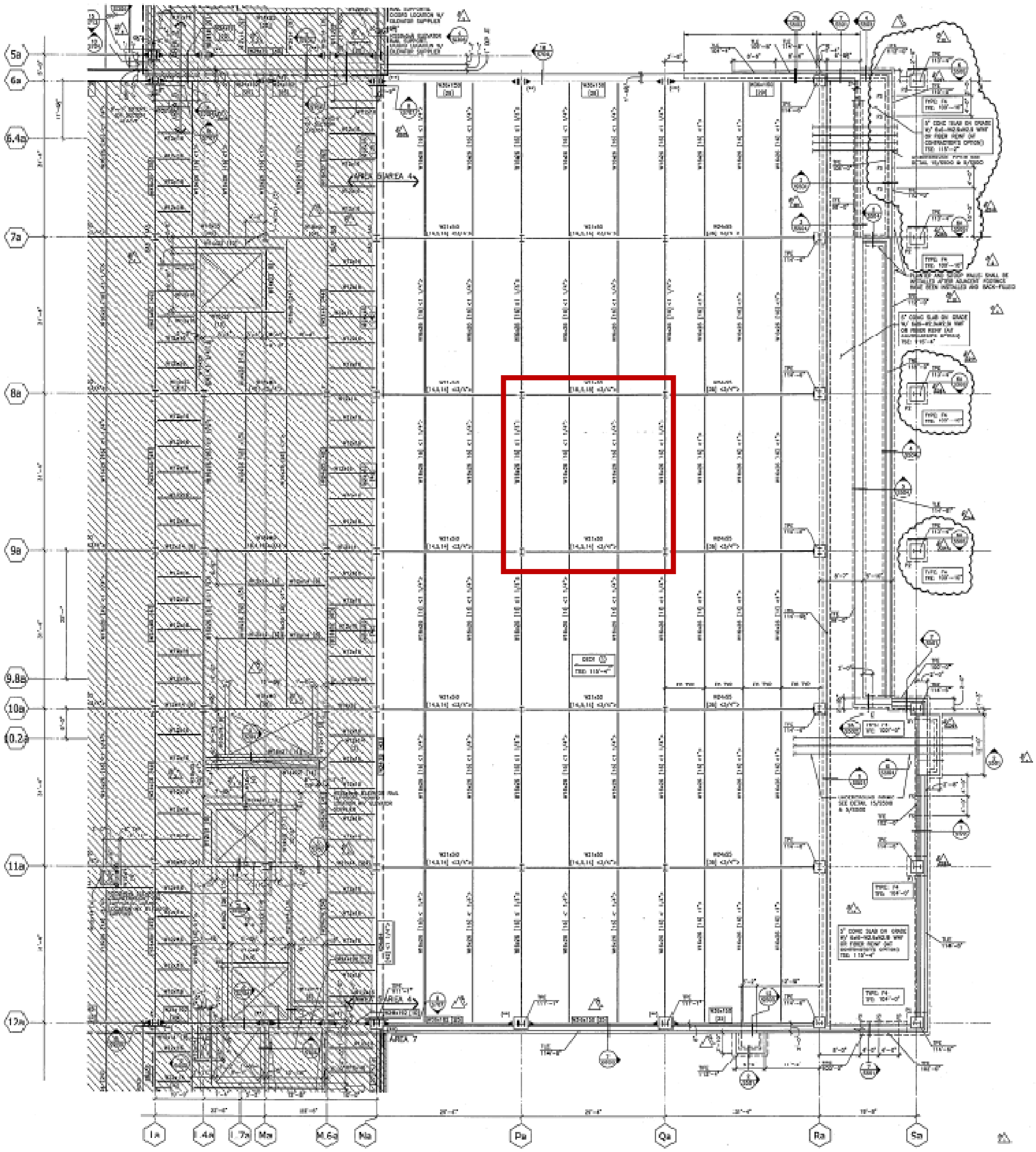


Figure 2 – Framing Plan Highlighting the Selected Frame

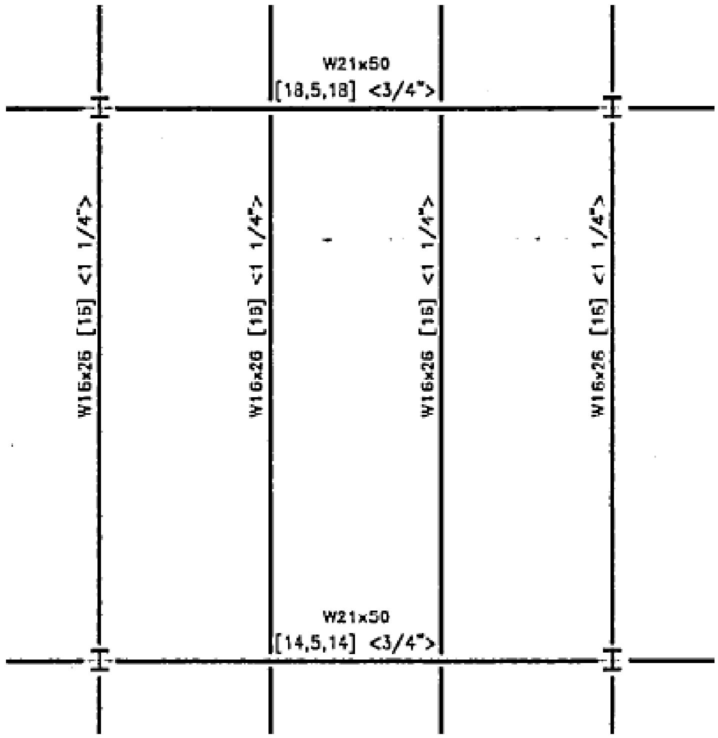


Figure 3 – Enlarged View of the Selected Frame

Structural System Overview

Foundations

The soil for the Voorhees Replacement Hospital is mainly a sandy soil. To prevent these loose soils from liquefaction, stone piers, or geopiers, were required to densify the soil. These geopiers were required to increase the bearing pressure of the soil to 6,000 psi for the soil below all the building's foundations, canopy foundations, and utility structures. For any soil below the site's retaining walls, the bearing pressure was required to be increased to 3,000 psi. The minimum required equivalent coefficient of friction equals 0.36 for sliding resistance across the entire footing bottom area for the retaining walls, and brace frame foundations.

The foundation system is a series of concrete footings either resting on concrete piers or resting on grade. The exterior columns are concrete footings with sizes ranging from 4' x 4' x 1' – 6" to 13' x 13' x 3' – 4" with rebar sizes ranging from #6 - #10 both ways. The columns that rest on concrete piers range in size from 2' – 4" x 2' – 4" to 3' x 4' – 6" with rebar sizes ranging from #9 - #11 for the vertical reinforcement, and #4's or #5's for the ties.

The garden level floor system is constructed, in most places, using a 5" concrete slab on grade, with 6 x 6 – W2.9xW2.9 WWF. In specified spots the size of the concrete slab is increased for specialized equipment, such as refrigerator equipment required for both the kitchen and the dietary section of the hospital. In zones 4 and 5, a grade beam travels along the perimeter.

Floor System

The floor system of the Voorhees hospital is a composite steel/concrete system. In Building A the typical bay sizes are around 30' x 30' or 30' x 10', depending on the area of the building they are located in. In Building B the bay sizes are typically 31' – 4" x 31' – 4" or 31' – 4" x 29' – 4". 3 – ½" light weight concrete sits on top of 3" x 18 Gage composite steel deck. The total thickness of the concrete is 6 – ½" with 6x6-W2.1xW2.1 WWF.

The steel deck is connected to the W-shape beams by ¾" diameter x 5" long shear studs allowing the two systems to work together in composite action. The beams then frame into larger W-shape girders via a single angle connection or a single plate connection. The beams are coped allowing them to connect to the girder's web so that the composite deck can sit on both the beams and the girders. The W-shape girders frame into W-shape columns by either double angle connections, or by moment connections.

Columns

Typical columns for the Voorhees Replacement Hospital were W14's. The gravity columns were much lighter than the lateral columns. This is due to the added lateral force that the lateral columns must take. The columns are spliced every two floors, 4'-0" above the floor with either a bolted column splice or a welded splice. The columns located in zone 6 were designed for future expansion to be built above.

Lateral System

The Voorhees Replacement Hospital uses a combination of braced framing and moment connections for its lateral system. Though, in both buildings the composite floor system and the roof deck acts as a diaphragm to transfer loads to either the braced frames, or the moment connections. In building A the braced frame supports the N-S lateral forces while the moment connections brace the E-W lateral system. The braced system consists of diagonal, square, HSS connected to W shapes. The braced frames are of two different styles, the bracing either frames from corner to corner, or from lower corner to the midpoint of the top beam. The moment frames in the Northern Building support the E-W lateral forces. The moment connections are located at the columns at the perimeter of the building

In Building B a combination of systems is used. In the N-S direction braced frames are used to resist the lateral forces. In the E-W direction, both braced frames, and moment connections resist the loads. The moment connections, again, are typically exterior columns running along the perimeter of the building. The diagonal braces are typically, like in Building A, diagonal HSS's connected to W shapes.

Roof System

The roof system is composed of 3" x 20 Gage steel roof deck topped with a concrete slab, vapor retarder, and insulation system. In certain areas the roof deck must support the green roof. To support the extra 100 psf of added weight from the green roofs, W shapes were added with a short beam to beam span.

Gravity Loads

Building live loads were determined by referencing ASCE 7-05. Table 1 below outlines the live loads.

Live Loads		
Load Description	**ASCE 07-05 Load (psf)**	**Assumed Partition Load (psf)**
Labs	60	20
Operating Rooms	60	20
Private Rooms/Wards	40	20
Offices	50	20
Corridors above the 1st floor	80	N/A
Lobbies/1st floor corridors	100	N/A
Stairs and Exits	100	N/A
Storage	125	N/A
Mechanical Room	125	N/A
Roof Garden	100	N/A
Roof	20	N/A

Table 1: Live Loads

For the calculations in this report, a superimposed dead load of 15 psf was used to account for mechanical and other specialty equipment. A live load of 80 psf was used because the selected bay contains both a corridor that is above the 1st floor and operating rooms. Since the corridor has a higher live load, 80 psf, it was selected to control the entire bay.

Codes & Design Standards

Design Codes and Reference Manuals:

International Building Code (IBC) 2006

American Society of Civil Engineers (ASCE 7-05), Minimum Design Loads for Buildings and Other Structures, 2005

American Concrete Institute (ACI 318), Building Code Requirements for Structural Concrete

American Institute of Steel Construction (AISC), Steel Construction Manual, 13th Edition

Vulcraft Steel Roof and Floor Deck, 1996

PCI Design Handbook, 6th Edition

ACI Manual of Concrete Practice, 2009

Existing Material Strength Requirement Summary:

Cast-in-place Concrete:

f'_c = 3,500 psi @ 28 days for all lightweight concrete on metal decking

f'_c = 4,000 psi @ 28 days for all other concrete types

Concrete Masonry:

Concrete Masonry Units: ASTM C90 Type “N-1”

Masonry Grout: f'_c = 3,000 psi @ 28 days

Masonry Mortar: ASTM C270 (Type S uno)

Steel Reinforcing:

Reinforcing Bars: ASTM A615 (Grade 60)

Welded Bars & Anchors: ASTM A706 (Grade 60)

Deformed Bar Anchors: ASTM A496

Epoxy-Coated Reinforcing Bars: ASTM A775 or ASTM A934

Welded Wire Fabric: ASTM A185

Structural Steel:

W & WT Shapes: ASTM A992, F_y = 50 ksi

Plates & Shapes Other Than W: ASTM A36, F_y = 36 ksi

Rectangular HSS: ASTM A500, Grade B, F_y = 46 ksi

Round HSS: ASTM A500, Grade B, F_y = 42 ksi

Pipes: ASTM A53, Type E or S, Grade B, F_y = 35 ksi

Bolts: ASTM F1554, F_y = 36 ksi

Expansion Bolts: Hilti, Rawl, Thunderstuds, or National Fasteners

Adhesive Anchors/Grout: Sika, Hilti, Epcon

Headed Studs/Shear Connectors: ASTM A108

Welds:

All Types: E70XX

<u>Designed Material Strength Requirement Summary:</u>

Concrete:

f'_c = 4,000 psi @ 28 days for 1-Way Slab and Precast Hollow core Slab

f'_c = 5,000 psi @ 28 days for the 2-Way Flat Slab with Drop Panels

Fire Protection and Fire Ratings

For the majority of the hospital, the required fire rating between floors is 2 hours. In specified spots a 3 hour rating for structural framing is required. In the bay analyzed in this report the required fire rating is 2 hours. Fireproofing methods of structural steel, such as fireproofing or intumescent paint, were not analyzed for this report.

The ACI Manual of Concrete Practice, 2009, was used to determine proper concrete thicknesses and covers for reinforcement, to provide the proper ratings. The tables used are located below in Tables 2, 3, and 4.

Aggregate type	Minimum equivalent thickness for fire-resistance rating, in.				
	1 hour	1-1/2 hours	2 hours	3 hours	4 hours
Siliceous	3.5	4.3	5.0	6.2	7.0
Carbonate	3.2	4.0	4.6	5.7	6.6
Semi-lightweight	2.7	3.3	3.8	4.6	5.4
Lightweight	2.5	3.1	3.6	4.4	5.1

Table 2 – Fire resistance of single-layer concrete walls, floors, and roofs

Aggregate type	Cover*† for corresponding fire resistance, in.					
	Restrained	Unrestrained				
	4 or less	1 hour	1-1/2 hours	2 hours	3 hours	4 hours
Nonprestressed						
Siliceous	3/4	3/4	3/4	1	1-1/4	1-5/8
Carbonate	3/4	3/4	3/4	3/4	1-1/4	1-1/4
Semi-lightweight	3/4	3/4	3/4	3/4	1-1/4	1-1/4
Lightweight	3/4	3/4	3/4	3/4	1-1/4	1-1/4
Prestressed						
Siliceous	3/4	1-1/8	1-1/2	1-3/4	2-3/8	2-3/4
Carbonate	3/4	1	1-3/8	1-5/8	2-1/8	2-1/4
Semi-lightweight	3/4	1	1-3/8	1-1/2	2	2-1/4
Lightweight	3/4	1	1-3/8	1-1/2	2	2-1/4

*Shall also meet minimum cover requirements of 2.3.1.
†Measured from concrete surface to surface of longitudinal reinforcement.

Table 3 – Minimum cover in concrete floors and roof slabs

Restraint	Beam width, in.	Cover for corresponding fire-resistance rating, in.				
		1 hour	1-1/2 hours	2 hours	3 hours	4 hours
Restrained	5	3/4	3/4	3/4	1	1-1/4
	7	3/4	3/4	3/4	3/4	3/4
	≥10	3/4	3/4	3/4	3/4	3/4
Unrestrained	5	3/4	1	1-1/4	NP*	NP
	7	3/4	3/4	3/4	1-3/4	3
	≥10	3/4	3/4	3/4	1	1-3/4

*Not permitted.

Table 4 – Minimum cover in nonprestressed beams

Existing Floor System

The existing composite bay was checked using AISC 13th ed. and the Vulcraft Steel Deck Design Guides, 1996. The bay chosen contains 3 ½" lightweight concrete on a 3" x 18 Ga. composite steel deck. The deck rests on a W16x26 composite beam with 16 shear studs, and a 1 ¼" camber. The composite steel deck was found to be adequate, holding 247 psf, which is more than the required 146 psf. The composite beam was checked using a superimposed dead load of 15 psf, and a dead load of 48 psf from the concrete and metal deck. The concrete and metal deck loads were found from the Vulcraft Design Guides. The W16x26 beam was also found to be adequate using 10 shear studs and a 1-¼" camber.

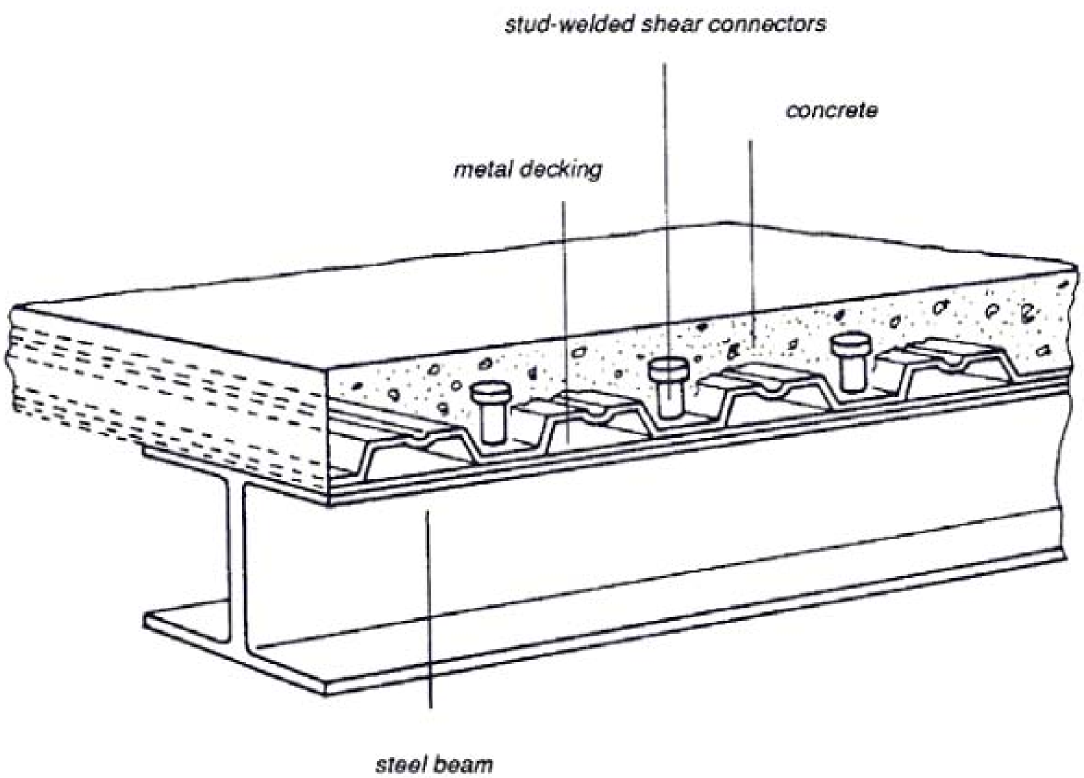


Figure 4 – Typical composite steel detail

Alternative Floor Systems

One Way Flat Slab with Beams

This system was designed by hand calculations per ACI 318-05 using an f'_c of 4,000 psi and an f_y of 60,000 psi. The calculated slab thickness was found to be 13" with cracking controlling the flexural rebar placement. #8 rebar was placed at 10" O.C. for flexural strength. A clear cover of ¾" was used as per ACI Manual of Concrete Practice to allow for a 2 hour fire rating between floors.

The one way slab spans the short direction to a concrete beam on either side. The concrete beam was designed to have a depth of 36" and a width of 24". The beam contains 9 #9 bars for flexural strength and #4 stirrups spaced at 16" for shear reinforcement. Figures 5 and 6 show the rebar detailing of the slab and beam. Additional details can be found in Appendix C.

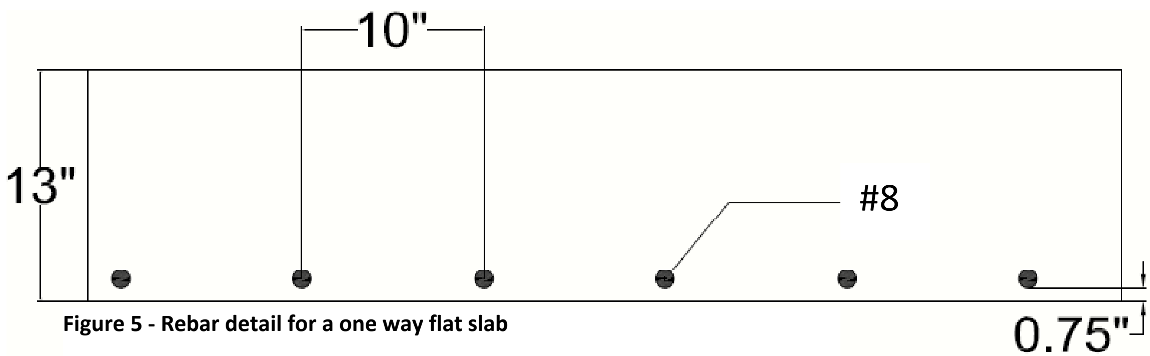


Figure 5 - Rebar detail for a one way flat slab

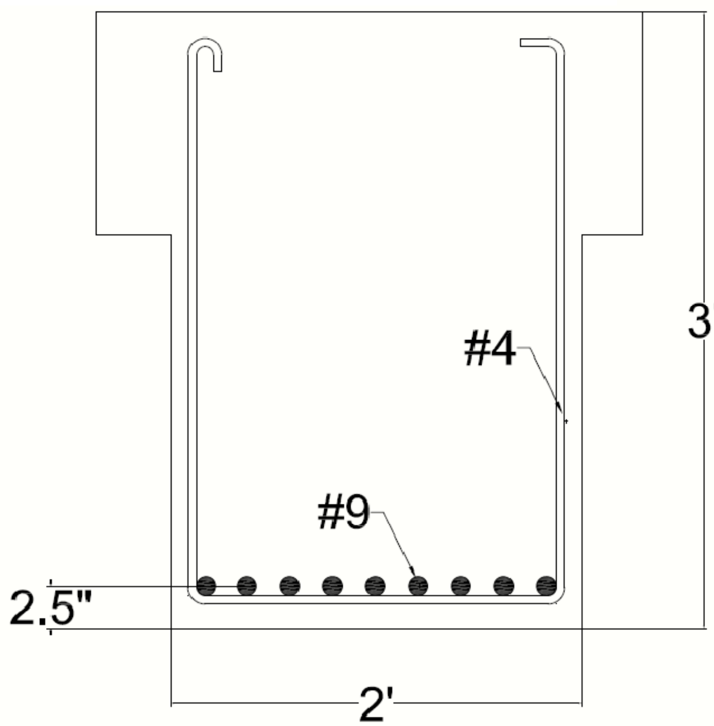


Figure 6 – Rebar detail for the concrete beam supporting the one way flat slab

Two Way Flat Slab with Drop Panels

This system was designed by hand calculations per ACI 318-05 using the direct design method. For this method an f'_c of 4,000 psi was originally tired, but found to be insufficient, so an f'_c of 5,000 psi was used instead. An f_y of 60,000 psi was used for the reinforcement steel. It was also assumed that a column of 24" x 24" was supporting the floor system.

After analysis, which can be found in Appendix D it was determined that a slab thickness of 9.5" was required. The drop panels were designed as 12" thick, and spanned 48" from the column. A varying amount of #8 rebar was used to support the flexural loads depending on what part of the slab it was placed in. The rebar was placed in accordance with ACI 318-05 Figure 13.3.8. A clear cover of ¾" was again used as per ACI Manual of Concrete Practice to allow for the required 2 hour fire rating between floors. Below in Figures 7, 8, 9, and 10 the placement of the rebar in the slab is shown.

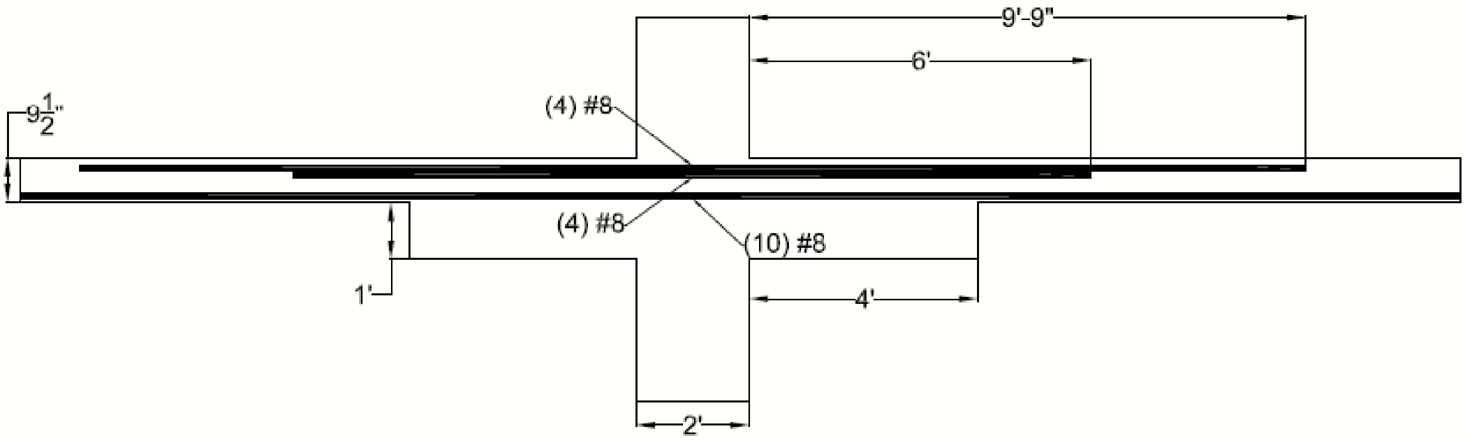


Figure 7 – Rebar detail for frame A column strip

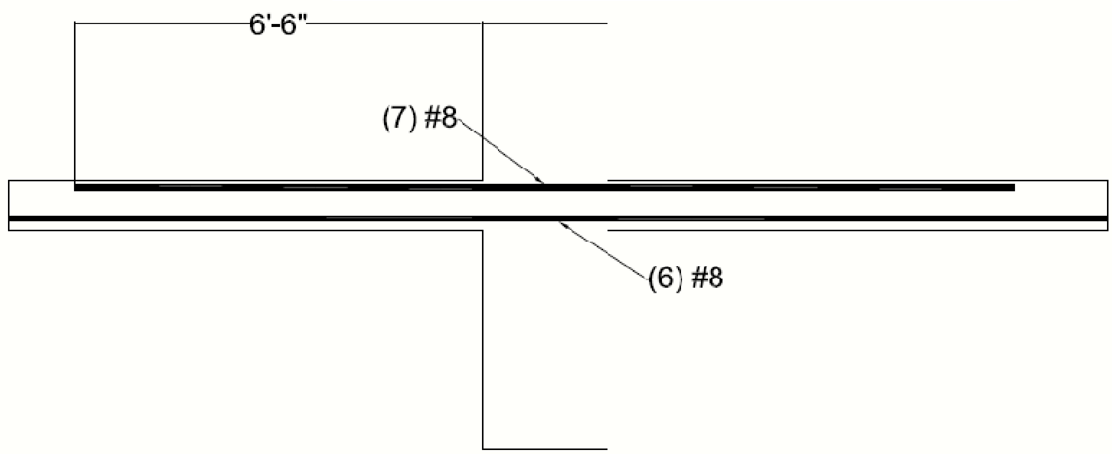


Figure 8 – Rebar detail for frame A middle strip

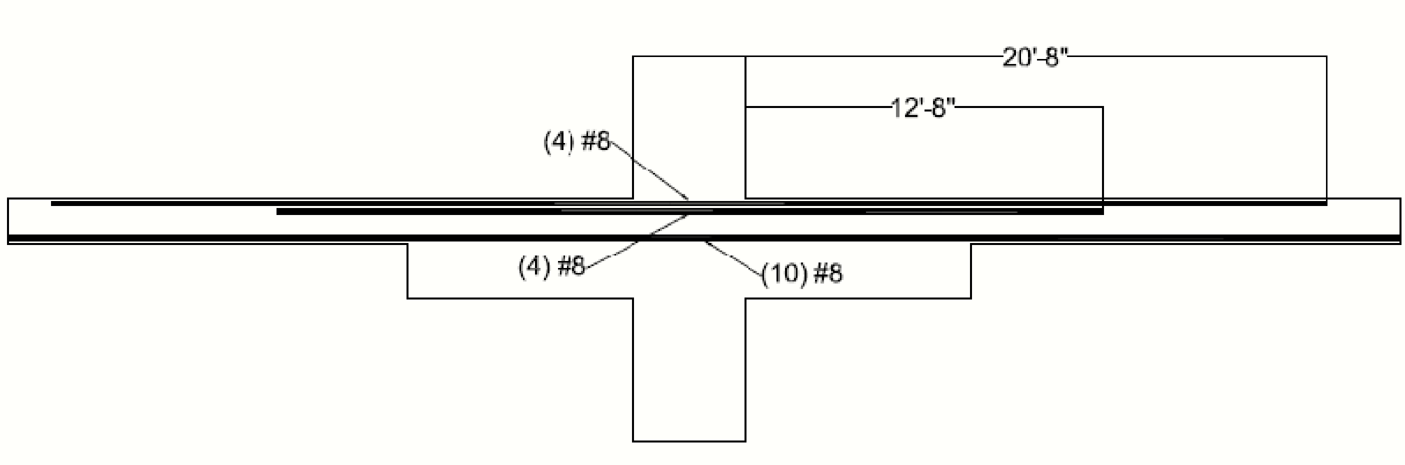


Figure 9 – Rebar detail for frame C column strip

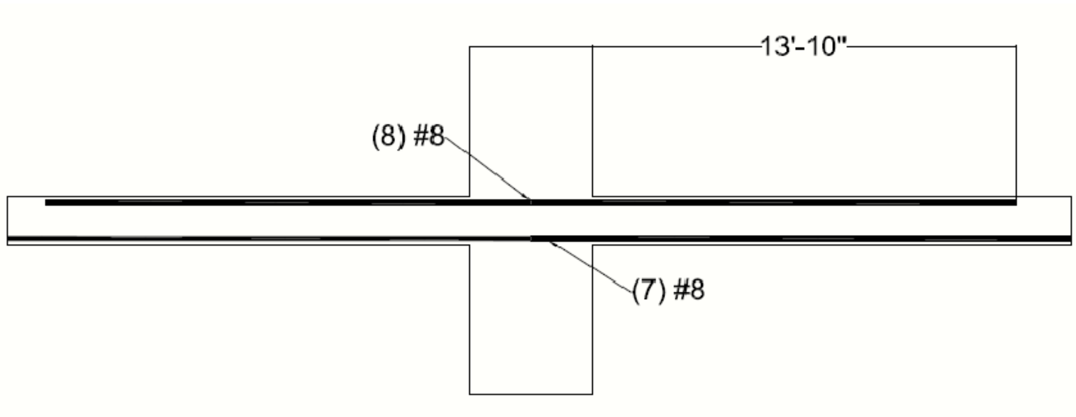


Figure 10 – Rebar detail for frame C middle strip

Precast Hollow core Slab on Steel Beams

This system was designed using PCI Ed. 6. Per the design guide it was found that a 58-S, 4'-0" x 10" slab, with 2" normal weight topping is appropriate. The superimposed service load in psf for that particular member is 148 psf, which is more than the 146 psf calculated load. This member also requires a 0.5" camber at erection to prevent deflection. The precast slab is supported by steel beams that frame to the columns. The steel beam was sized using AISC 13th ed. Table 3-10 and found to be a W24x146.

Figure 11 – A precast hollow core slab on steel beams during construction

Floor System Comparisons

A comparison of all of the systems was preformed for this report. The comparison consisted of many different factors including weight, depth, vibration, fireproofing, constructability, lead time, cost, framing layout changes, and foundation changes. Although not every factor played an equal factor in determining the best system, all were considered.

Weight

Since the seismic forces controlled over the wind forces, the overall weight of the building is very important. If the building's weight is much higher because of the floor system it will force the gravity system to be increased in size, and this in turn will force the foundation to be increased. All the increases added to the system's weight will create a higher seismic force which will create the need for a stronger lateral system. All of these added increases will create a higher cost in construction since larger members will have to be purchased. Since it is always best to keep the cost to a minimum, the desired building weight should be as low as possible. Below in Table 5 the weights of all four systems are compared.

	Framing /Drop Panel Wt. (lbs)	Slab/Deck Weight (psf)	Slab/Deck Weight (lbs)	# of Shear Studs	Equivalent Shear Weight (lbs)	Total Weight (kip)	Total Weight (psf)
Steel Composite	6391	48	44117	138	1380	51888	57
Two Way FS with DP	9000	118.75	109144	N/A	N/A	118144	129
One Way Flat Slab	56394	162.5	149356	N/A	N/A	205750	224
Precast HC Slab	9149	93	85477	N/A	N/A	94626	103

Table 5 – Weight comparisons

The weights of the different systems were found to be drastically different. The lightest system, the existing steel composite beams with composite steel deck, was almost half the weight of any other system. The one way flat slab was found to be the heaviest, which is expected because it has a 13" slab with 2' x 3' beams supporting it. When weights are compared, the composite steel deck is clearly the best choice.

Depth

The depths were compared in order to find the smallest possible floor to floor height. Since this is a hospital, it contains many different mechanical systems that are required to be run in the ceiling. If the floor system's depth were to be decreased, this would allow for a greater area for equipment to run, potentially eliminating any spacing conflicts during construction. The depths are listed in Table 6 below.

	Framing Depth (in.)	Slab/Deck Depth (in.)	Total Depth (in.)
Steel Composite	21	6.5	27.5
Two Way Flat Slab with Drop Panels	12	9.5	21.5
One Way Flat Slab with Concrete Beams*	36	13	36
Precast Hollow core Slab on Steel Beams	24	12	36

*The concrete slab frames into the beam

Table 6 – Depth comparisons

The depths of the systems were found to be similar with values around 2'-3'. The two way flat slab with drop panels was found to have the smallest depth while the precast and the one way flat slab had the most with 36". For this comparison the two way flat slab is the ideal system because it can eliminate space above the ceiling.

Vibration

Exact vibration calculations were not preformed in this report, but approximate, relative methods were used to comparatively determine how each system would act. Vibration is proportional to the mass and the stiffness of each element. It can therefore be assumed that heavier and more rigid elements will vibrate less than lighter more flexible elements. Since this is the case, it is believed that the heavier concrete structures will vibrate less than that of the steel composite system. However, if any system is to be considered in the future, then vibration should be analyzed.

Fireproofing

For this particular bay a two hour fire rating was required between floors regardless of the system. For the concrete systems, according to ACI Manual of Concrete Practice, 2009, in order to achieve the two hour rating they must have a minimum thickness of 4.6" and a minimum clear cover of ¾". For the steel composite and the precast system, the steel beams must be sprayed with fireproofing, painted with intumescent paint, or encased in gyp board per a UL Design. Since any of those tasks would likely be given to a subcontractor, it would increase man hours and cost of materials. Considering that concrete is its own fireproofing, it has the clear advantage over the steel system.

Constructability

Constructability is an important factor when considering floor systems. Since a major cost of construction is the labor, if a system is very difficult to install, it will add to the total cost of the building. The steel composite and the precast slab would be the easiest and most time efficient to install. The steel composite system is a common system and would be easy to install since it does not require any special construction considerations. The precast would also be very constructible. Since the site is so large it is possible to preorder shapes and have them waiting on site while the steel is being erected. The one consideration for the precast is that due to some of the irregular bays shapes might have to be special ordered or cut on site in order to fit the bays. This has the possibility of adding to the cost. The two way and the one way slabs are not as constructible since they require formwork to be installed before each pour. Also, since the concrete would have to harden before moving up a floor, it would take much more time to finish the structural frame and move on to the interior of the building.

Lead Time

For this project, lead time was important but not one of the contributing factors. For every project the faster a building can be built, the better it is for all parties involved. One way to cut down on construction time is to get materials that have a short lead time. Both the two way and the one way slabs have virtually no lead time because there are no prefabricated members to order prior to installation. However, for the steel composite and the precast systems, there is a lead time associated with them. There is also potential for a long lead time for the precast system if specialized bay sizes are ordered.

Cost

Cost is generally one of the most important factors when it comes to selecting a floor system. It is very important to find the least expensive system possible. The costs for each floor system were found using R.S. Means Assemblies Cost Data, 2000. The location and date multipliers were not considered because the numbers were used as a comparison only. It should also be noted that these costs do not take into account any possible problems during construction; for example, protecting the concrete during the winter, or cutting/special ordering the precast shapes.

	Material Cost Per S.F.	Installation Cost Per S.F.	Total Cost Per S.F.	Cost Per Bay
Steel Composite	$7.70	$4.67	$12.37	$11369
Two Way Flat Slab with Drop Panels	$5.35	$6.95	$12.30	$11305
One Way Flat Slab with Concrete Beams	$4.68	$8.70	$13.38	$12298
Precast Hollow core Slab on Steel Beams	$13.65	$3.64	$17.29	$15892

Table 7 – Cost comparisons

The costs were found to be somewhat similar with the exception of the precast slab. The two way flat slab and the composite steel were found to be very similar and the cheapest systems. The one way slab was similarly cheap as well, with its cost per square foot only a little over $1.00 more than the composite, or two way. The precast was found to be much more expensive than any of the other systems. Because the precast is so expensive and the fact that it may need to be preordered or cut into shape adding addition costs makes this system the least favorable.

Framing Layout Changes

For this report a rectangular bay that is very similar to a square bay was analyzed. The Voorhees hospital not only features square bays, but it also features long narrow rectangular bays. Since not all of the systems checked would work well with a rectangular bay, there may be potential changes to the bay sizes in order to make them more or less square. The steel composite and the one way flat slab would both do well with a rectangular bay and would not require many changes, if any, to the bay sizes. However, the two way would work well if the bays were square. Since that is the case, that system would have to rearrange the bays. The precast would also have to change the framing layout, but not necessarily to a square bay. The precast shapes are manufactured in 4' strips, which means that the bay sizes should be in multiples of 4' so that cutting of the shapes is not required. This would require movement and rearrangement of most if not all of the bays.

Foundation Changes

The foundation changes would be directly related to the changes in weight. If the building's weight were to increase, then the foundation would also have to be increased in order to hold the added weight. Since that is the case, if any of the alternative structures are selected, the foundation capacity must be increased respectively. This is a problem, especially for this building, because the site consists of a sandy soil and soil densification was required to densify the soil. If the weight of the building is increased then special measures will have to be taken to make sure that the soil capacity is not exceeded. This makes the steel composite system the most desirable since the weight of the building will not change.

Summary

A summary of the comparisons are made in Table 8. Strong advantages for a system were highlighted in green, while the strong disadvantages were listed in red. Values that were neither strong advantages nor strong disadvantages were left blank

Comparison Summary				
	Steel Composite	**Two Way Flat Slab with Drop Panels**	**One Way Flat Slab**	**Precast Hollow Core Slab on Steel Beams**
Weight (psf)	57	129	224	103
Depth (in.)	27.5	21.5	36	36
Vibration	Average	Low	Low	Low
Fire Proofing	Easy	None	None	Easy
Constructability	Easy	Average	Average	Average
Lead Time	Average	Low	Low	Average
Cost	$12.37/ sq. ft.	$12.30/ sq. ft.	$13.38/ sq. ft.	$17.29/ sq. ft.
Framing Layout Changes	None	Multiple	Minimal	Multiple
Foundation Changes	No	Yes	Yes	Yes

Table 8 – Summary of comparisons

Conclusions

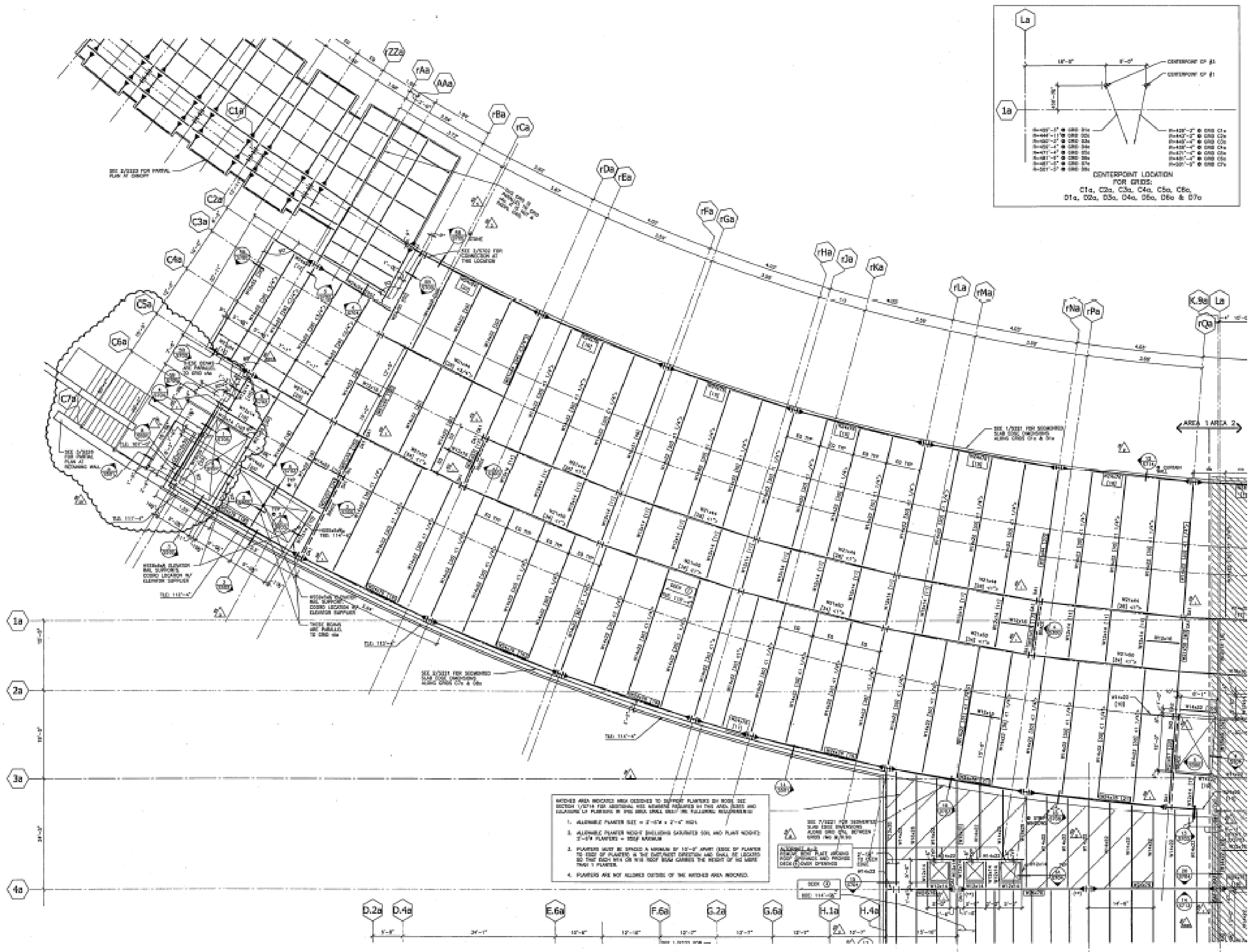
After analyzing the different floor systems, it was found that the current steel composite floor is the best for this building. The only other option that has potential to be considered is the two way flat slab with drop panels. The steel composite system seems to be the best fit for this building for many reasons. The main reason the system seems viable is that it is the lightest by far. Since the soil conditions are a major obstacle for this building, taking away the most amount of weight will reduce any problems that might occur from the soil. This method also allows for the current framing layout to stay intact. It is also a very easily constructed system, allowing for the building to be built at a very fast rate compared to the concrete options. The steel composite system is one of the cheapest options as well with the two way slab only $0.07 cheaper. The only disadvantage of the steel composite system is that it does not allow for the shallowest depths, which would create crowded mechanical areas, or a tall floor to floor height.

The two way flat slab with drop panels is also a viable option that might want to be looked at in the future. Since this system has the smallest depth it would help with coordination of mechanical systems and could possibly reduce the floor to floor height, reducing the building's weight. This system is also the cheapest, costing only an estimated $12.30 / square foot and provides virtually no lead time. However, the major down side of this method, is the extra weight that is added to the building. If this system is chosen, it would more than double the current weight of the floor system. This would require a closer look at the foundation and the soil capacity since these were both major concerns for the hospital. This system would also require a redesign of the existing bays in order to achieve square bays so that the system would be efficient.

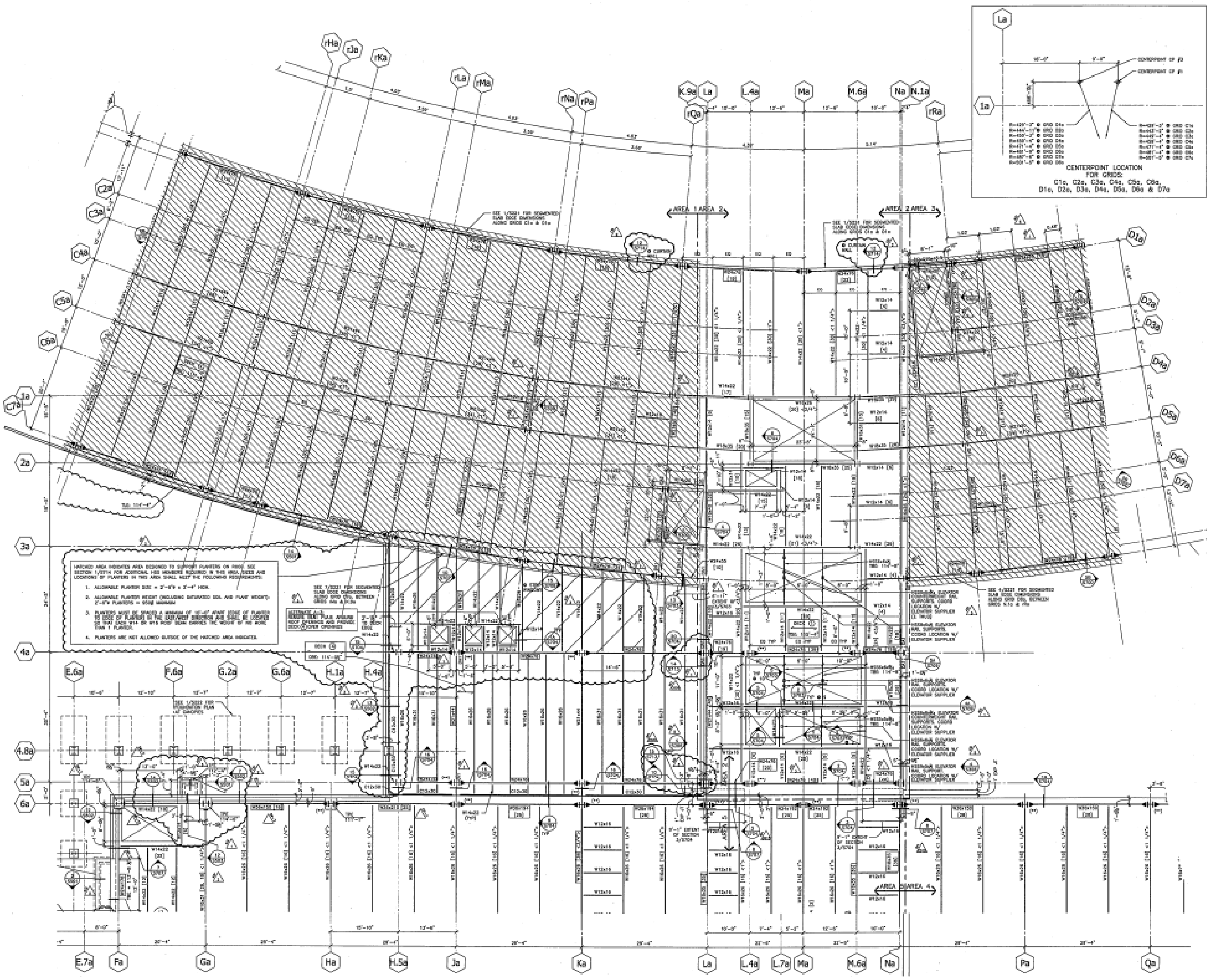
The one way flat slab with concrete beams is not a viable option even though it has a low lead time and has no additional fire proofing. Since this system is almost four times heavier than the existing system, too much weight would be added to the foundation and gravity system, so that it would not be economical. It is also not as cheap as the two way or steel composite to begin with, so adding cost for the floor, and the gravity system eliminates this system from further consideration.

The precast hollow core slab on steel beams is also not a viable option. One of the reasons that this system is eliminated is because it is again, too heavy. The added weight will add cost to the gravity and foundation systems requiring more analysis of the foundation. It also is the most expensive system costing 30% more than the existing system. Since there is added cost to the gravity and foundation systems, adding more money for the floor system should eliminate this option due to cost.

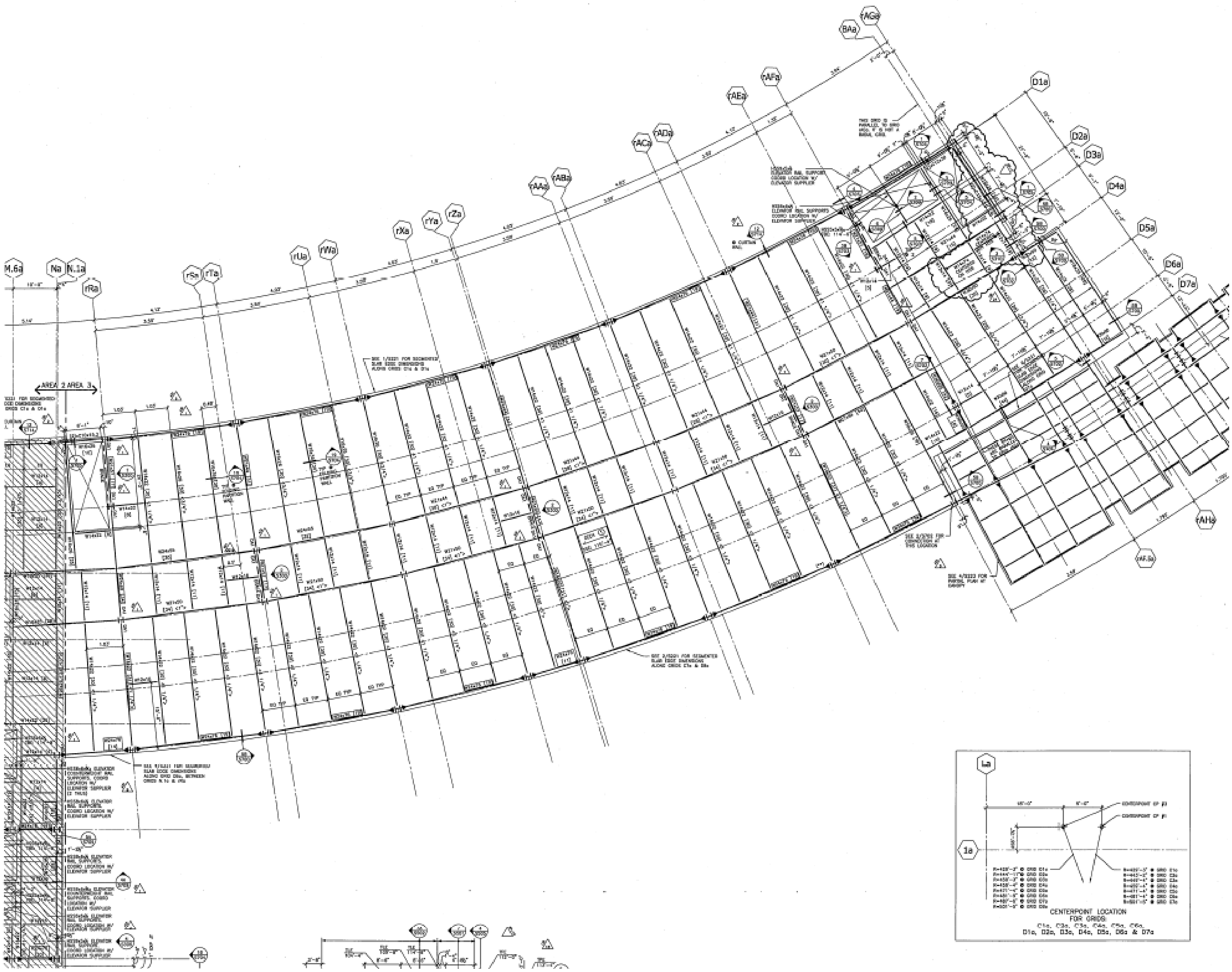
Appendix A



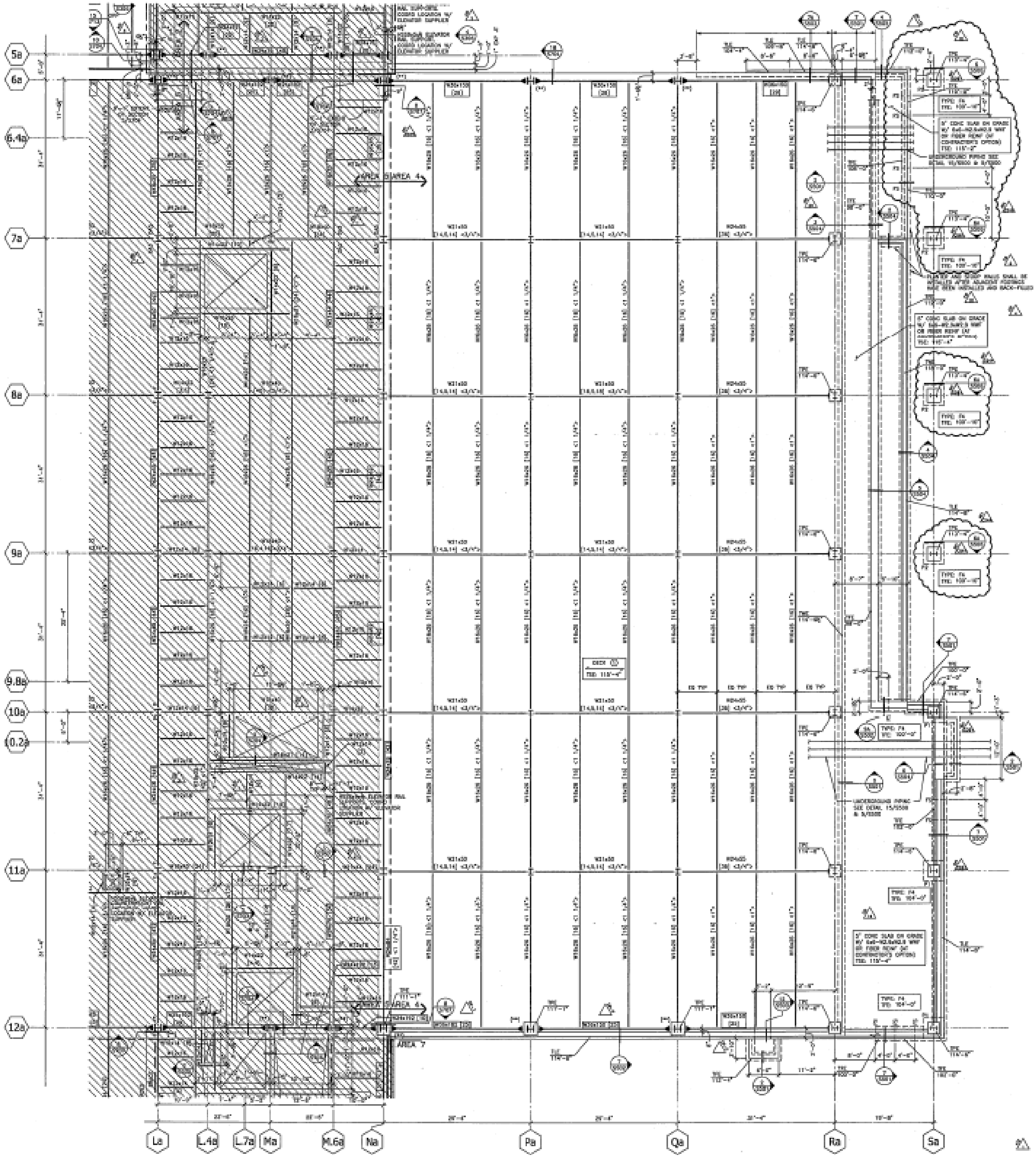
Level 1, Zone 1 Framing Plan



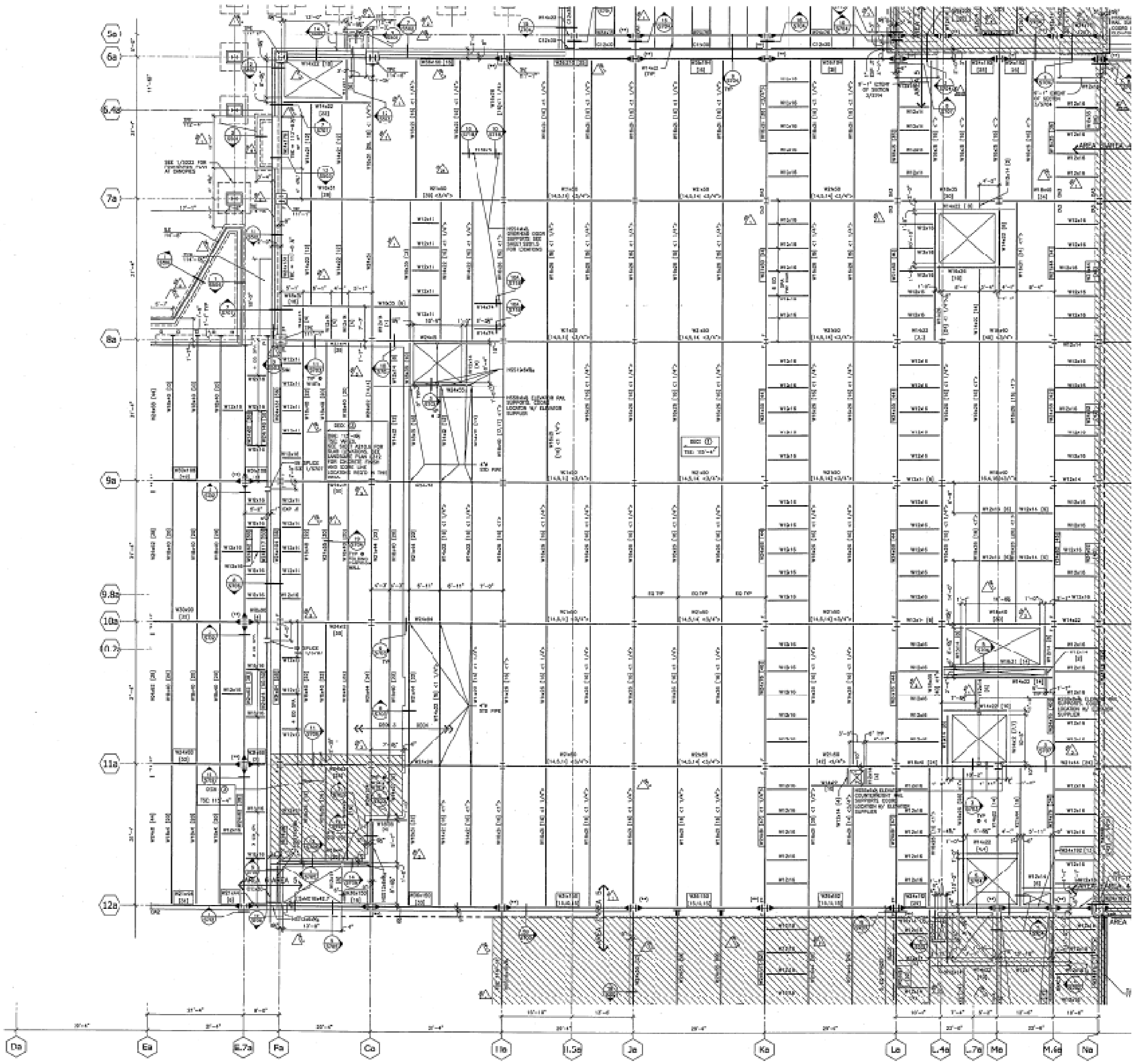
Level 1, Zone 2 Framing Plan



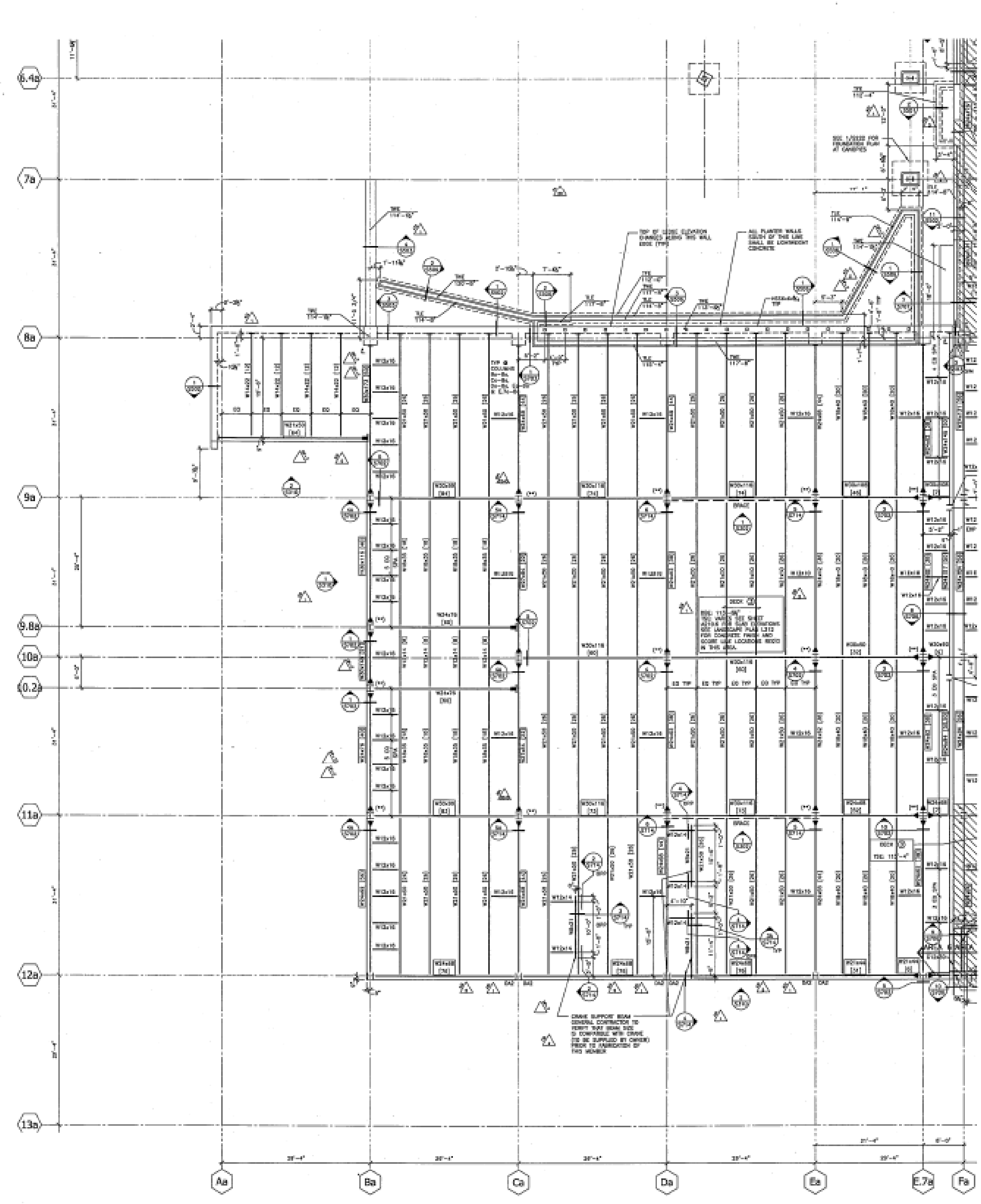
Level 1, Zone 3 Framing Plan



Level 1, Zone 4 Framing Plan



Level 1, Zone 5 Framing Plan



Level 1, Zone 6 Framing Plan

Level 1, Zone 7 Framing Plan

Appendix B

Tech 2 | Existing System | 1

Composite Metal Deck:

3 1/2" lightweight concrete slab on 3"×18 Ga composite steel deck w/ 6×6 W2.1×W2.1 WWF

$f'_c = 3.5ksi$ $f_y = 60ksi$ 115pcf weight

Span $= \frac{29.33'}{3 \text{ bays}} = 9.77'$

$w_u = 1.6(80) + 1.2(15) = 146psf$

According to Vulcraft Steel Deck 1996

Superimposed LL $= 247psf > 146psf$ ok ✓

Composite beam:

DL $= 15psf + 48psf = 63psf$ (from Vulcraft)

$w_u = 1.2(63) + 1.6(80) = 203.6psf(9.77) = 1.99 klf$

$M_u = \frac{1.99(31.33)^2}{8} = 244.17^{k-ft}$ $V_u = \frac{1.99(31.33)}{2} = 31.17^k$

Assume $a = 1"$

$Y_2 = 6.5 - 1/2 = 6$

use a W16×26 $M_u = 252$ @ PNA-7

$b_{eff} = 9.77 = 117.24"$
$= 31.33/4 = 94"$ ← controls

$a = \frac{96}{.85(3.5)(94)} = .34 \leq 1"$ ∴ ok

$Y_2 = 6.5 - .34/2 = 6.33" > 6.0"$ ∴ ok

studs $= \frac{2(96)}{21} = 9.14 = 10$ studs

Self weight:
$1.99 + 1.2(.026) = 2.02$

$M_u = \frac{2.02(31.33)^2}{8} = 247.85 ft^k < M_u = 252 ft\text{-}k$

Tech 2 | Existing System | 2

Construction loads:

$48(9.77) + 26 = .495 \text{ klf}$ $\quad LL_{const} = 22 \text{ psf}$

$w_u = 1.2(.495) + 1.6(0.022 \times 9.77) = 0.94 \text{ k-ft}$

$M_u = \frac{.94(31.33)^2}{8} = 115 \text{ ft-k} < \phi M_p = 166 \text{ ft-k}$

deflection:

$\Delta_{const} = \frac{5(.495)(31.33)^4(1728)}{29000(301)(384)} = 1.23" > 1"$ ∴ no good

use 1-¼" camber $\quad \Delta_{const} = 1.23 - 1.25 = -0.02"$ ∴ ok

$E_s = 29000 \text{ ksi}$ $\quad E_c = (115)^{1.5}\sqrt{3.5} = 2307.1 \text{ ksi}$ $\quad c = 34"$

$n = \frac{29000}{2307.2} = 12.57$ $\quad b_{eff} = 94"$ $\quad w_{LL} = 80(31.33) = 2.51 \text{ klf}$

$\frac{b_{eff}}{n} = 7.48"$ $\quad I_s = 301 \text{ in}^4$ $\quad I_t = \frac{7.48(.34)^3}{12} = 0.245 \text{ in}^4$

$A_t = 7.48(.34) = 2.54 \text{ in}^2$ $\quad A_s = 7.68 \text{ in}^2$

$\bar{y} = 7.68(15.7/2) + 2.54(15.7 + 1.5 - .34/2) \,/\, (7.68 + 2.54) = 11.27 \text{ in}$

$I_{tr} = 1662.7 \text{ in}^4$

$\Delta_{LL} = \frac{5(2.51)(31.33)^4(1728)}{384(29000)(1662.7)} = 1.128" - 1.25" = -0.122" < L/360 = 1.044"$ ✓

W16×26 [10] <1-¼"> ok

Appendix C

Tech 2 | 1 way slab | 1

$L = 29'\text{-}4" = 29.333'$

use $f_y = 60$ ksi

$h_{min} = \frac{L}{28}$ per ACI 318-05 Table 9(c)

$f'_c = 4$ ksi

$h_{min} = 29.333(12)/28 = 12.57" \Rightarrow 13"$ use $d = 12"$

$w_L = 80$ psf

$w_{DS} = 13/12(150) = 162.5$ psf

$w_{SIDL} = 5$ psf

$w_u = 1.2(162.5) + 1.2(15) + 1.6(80) = .341$ ksf

$M_u = \frac{.341(29.33)^2}{8} = 36.67^{k\text{-}ft}$

$A_s = M_u/4d = 36.67/4(12) = 0.764$ in²/ft try #8 @ 12" $A_s = 0.79$

$d_{actual} = 13" - .75" - 8/16" = 11.75$ use $d = 11.75"$

Assume $f_s = f_y$ $a = \frac{A_s f_y}{.85 f'_c b} = \frac{0.79(60)}{.85(4)(12)} = 1.162"$ $c = \frac{a}{\beta} = \frac{1.162}{.85} = 1.367"$

$\epsilon_s = \frac{\epsilon_u}{c}(d-c) = \frac{0.003}{1.367"}(11.75 - 1.367) = 0.0228 > .005 \rightarrow \phi = .9$

$\phi M_n = \phi A_s f_y (d - a/2) = \frac{.9(.79)(60)(11.75 - 1.162/2)}{12} = 39.7^{k\text{-}ft} \geq 36.67^{k\text{-}ft}$ ✓

Crack control:

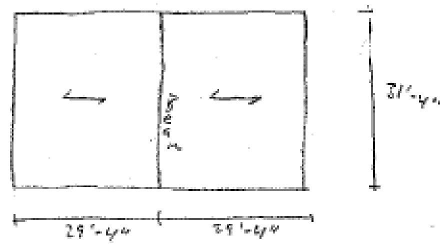
$S \leq 15 - 2.5c_c = 15 - 2.5(13 - (11.75 + 8/16)) = 10.625" < 12"$ not ok X

use 13" slab w/ #8 @ 10" o.c.

Tech 2 | 1 way Slab | 2

Design Beam holding slab:

$f'_c = 4ksi$
$f_y = 60 ksi$
$DL = 162.5 + 15 = 177.5 psf$
$LL = 80 psf$



$w_u = 1.2(177.5) + 1.6(80) = 341 psf\left(\frac{29'4" + 29'4"}{2}\right) = 10.0 klf$

$M_u = \frac{w_u L_n^2}{8} = \frac{10(31.33' - 2')^2}{8} = 1075.55 \times 1.1 = 1183.1 k\text{-}ft$

assume $b_{girder} = 24"$; self weight

$bd^2 = 20 M_u$ try $b = 4/5 d$

$d^3 = 20(1183.1)(5/4)$
$d = 30.9"$
$h = d + 2.5 = 33.425 = 34"$

$b = 4/5(34 - 2.5) = 25.2" = 25"$
use $h = 36"$ $b = 24"$
$bd^2 = 24(36 - 2.5)^2 = 26934 in^3$

Self weight:

$w_{sw} = \frac{36(24)}{144} \times 150 = 900 plf$

$w_u = 10 + 1.2(.9) = 11.08 klf$

$M_u = \frac{11.08(29.33)^2}{8} = 1191.4 k\text{-}ft$ $1191.4(20) = 23828 < 26934$:. ok

Steel:

$A_s = \frac{M_u}{4d} = \frac{1191.4}{4(33.5)} = 8.89$ use (9) #9 $9(1) = 9 in^2$

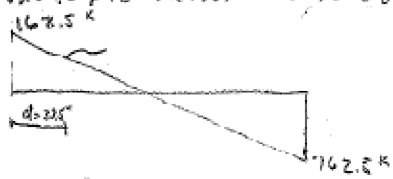
$\phi M_n = \phi A_s f_y d\left(1 - \frac{A_s f_y}{1.7 f'_c d b}\right) = .9(9)(60)(33.5)\left(1 - \frac{9(60)}{1.7(4)(33.5)(24)}\right)$

$= 1222.74 k\text{-}ft > 1191.4 k\text{-}ft$:. ok ✓

Tech 2 | 1 way slab | 3

shear: $V_c = 2\sqrt{4000}(24)(33.5) = 101.69^k$

$\phi V_n = .5\phi V_c = .5(.75)(101.69) = 38.14^k$



$\frac{162.5}{14.66'} = \frac{x}{11.875}$

$x = 131.67^k$

$V_s = \frac{131.6}{.75} - 101.69 = 73.8^k$

$\frac{162.5}{14.66} = \frac{38.14}{x}$

$x = 3.44' = 42''$

$14'\text{-}8'' - 42'' = 134'' = 11'\text{-}2''$

$V_s \le 8\sqrt{f'_c}\, b_w d = 8\sqrt{4000}(24)(33.5) = 406.8^k$

$4\sqrt{4000}(33.5)(24) = 203.3 > V_s$ ∴ $s_{max} = d/2 = 16.75''$ ← ✱
$= 24''$

$s_{max} = 16''$

$A_{v_{min}} = .75\sqrt{4000}(24)(16)/60000 = .303$

$= \frac{50(24)(16)}{60000} = .32 = .32 in^2$

use #4 stirrups @ 16"

$2(.2) = .4 > .32$

Appendix D

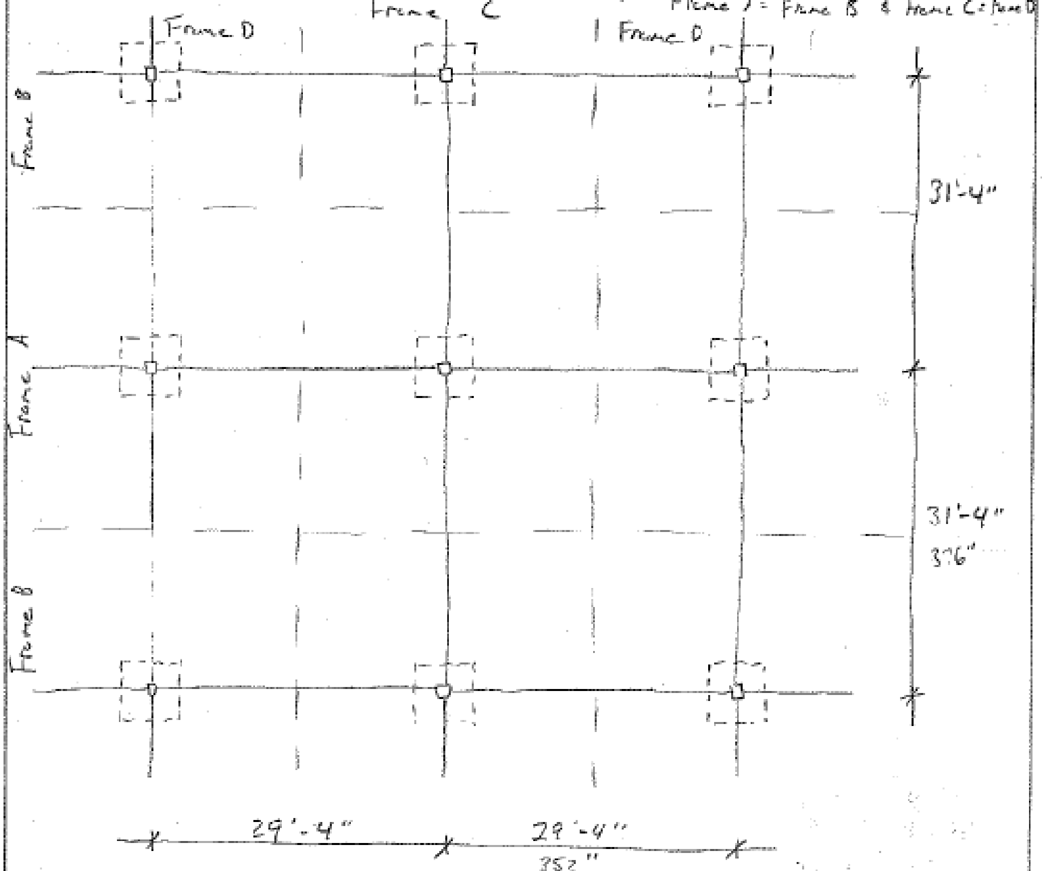
Tech 2 | 2 way flat slab w/ drop panels | 1

Design a 2-way flat slab with drop panels:

The frame combines a corridor (80psf) w/ operating rooms (60psf + 20psf for partitions) so will use 80psf LL

Frame size 31'-4" x 29'-4"

Design Frame A & C
Frame A = Frame B & Frame C = Frame D



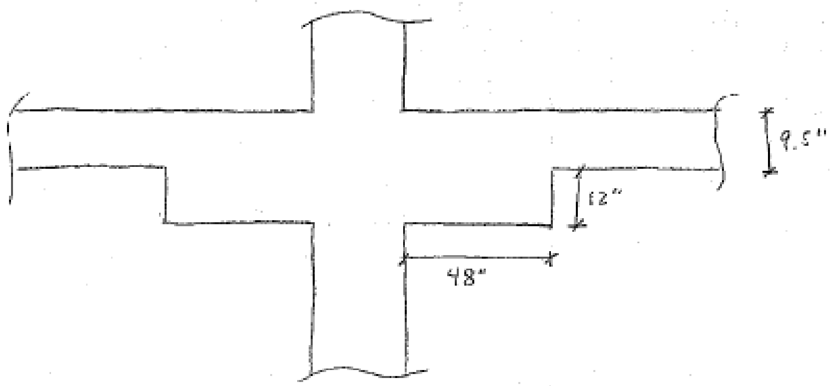
	Tech 2	2way FP w/ DP	2

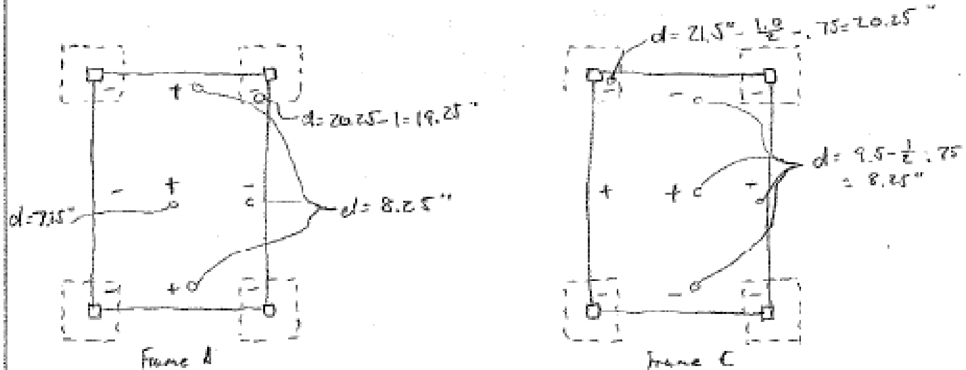
$L_n = 31'\text{-}4" - (2 \times 12"/2) = 30'\text{-}4" = 30.33'$

use 24×24" column
$f'_c = 4ksi$
$f_y = 60ksi$
#8 rebar

$t_{min} = \frac{30.33(12)}{40} = 9.09 \approx 9.5"$

use following set up:





check shear:

wide beam action:

$width = (29.33 \times 12) - 12 - d_{avg} = 352 - 12 - 19.75 = 320.25"$

$d_{avg} = \frac{19.25 + 20.25}{2} = 19.75$

$Area = 320.25"(31.33 \times 12) = 120414 in^2$

$w_L = 80psf$

$w_{t,s} = \frac{9.5}{12}(150) = 118.75psf$

$w_{t,DP} = \frac{21.5}{12}(150) = 268.75psf$

$w_{SDL} = 15psf$

	Tech 2	2 way FP w DP	3

$w_S = 118.75\,psf\,(31.33 - (4+4+2)) = 2.53\,klf$

$w_{DP} = 268.15\,psf\,(4' \times 2) = 2.15\,klf$ $\quad w_D = 2.53 + 2.15 + .439 = 5.12\,klf$

$w_{SI} = 15\,psf\,(31.33 - 2) = .439\,klf$ $\quad w_L = 80(29.33) = 2.35\,klf$

$w_u = 1.2(5.12) + 1.6(2.35) = 9.904\,klf$

$V_u = w_u A = \frac{9.9(320.25)}{12} = 264.2^k$

$V_n = 2\sqrt{f'_c}\, b_w d = 2\sqrt{4000}\,(29.33 \times 12)(7.75) = 345^k > 264.2^k \therefore ok$

Punching Shear:

$V_u = w_u A = 9.9(27.33) = 270.6^k$

$b_o = 48(4) = 192''$

$V_c = 4\sqrt{f'_c}\, b_o d = 4\sqrt{4000}\, 192(7.75) = 376.43^k$

$V_c = (\alpha d / b_o + 2)\sqrt{f'_c}\, b_o d = (40(7.75)/192 + 2)\sqrt{4000}\,(192)(7.75) = 340.2^k$ *

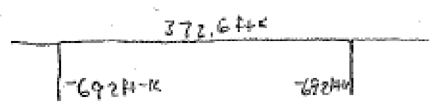
$\phi V_c = .75(340.2) = 255.1^k$ no good X

try $f'_c = 5000\,psi$

$\phi V_c = .75(40(7.75)/192 + 2)\sqrt{5000}\,(192)(7.75) = 285.24^k \geq 270.6^k \therefore ok$ ✓

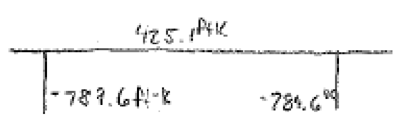
Frame A Design:

$M_u = 1/8(9.9)(29.33)^2 = 1065\,ft\text{-}k$



Frame C Design:

$M_u = 1/8(9.9)(31.33)^2 = 1214.7\,ft\text{-}k$



Tech 2 | 2 way FP w/ DP | 4

	Frame A	Frame C
1) Total transverse width	352"	376"
2) Column Strip width	176"	188"
3) Middle Strip width	2 @ 88"	2 @ 94"
4) $\beta_t = C/2I_s$	0	0
5) $\alpha_1 = I_b/I_s$	0	0
6) l_2/l_1	1.07	1.07
7) $\alpha_1 l_2/l_1$	0	0
9) M^-% to CS	75%	75%
10) M^+% to CS	60%	60%

Frame A

	M^-	M^+	M^-
Total M	-692	377.6	-692
% to CS	.75	.6	.75
M in CS	-519	226.6	-519
M in MS	-173	151	-173

Frame C

	M^-	M^+	M^-
Total M	-789.6	425.1	-789.6
% to CS	.75	.6	.75
M in CS	-592.2	255.1	-592.2
M in MS	-197.4	170	-197.4

	Tech 2	2way FP w/ DP	5

Design slab reinforcement for Frames A & C: Column Strip

	Frame A M^-	Frame A M^+	Frame C M^-	Frame C M^+
1) M_u (ft-k)	-519	226.6	-592.2	255.1
2) width of column strip w/ drop panel, b	128"	176"	140"	188"
3) Effective depth, d	19.25"	8.25"	20.25"	8.25"
4) $M_n = M_u/\phi$	-577 ft-k	251.8	-658	283.4
5) $R = M_n/bd^2$	146	227	124	289
6) ρ_{req}	0.00248	0.00389	0.002098	0.004103
7) $A_{s,req} = \rho bd$	6.11 in²	5.65	5.95	6.36
8) A_{min} = .0018bt	5.5 in²	3.34	6.02	3.57
9) N = 7.16/.79	8	8	8	9
10) N_{min} b/24	3	10	4	10

Design slab reinforcement for Frames A & C: Middle Strip

	Frame A M^-	Frame A M^+	Frame C M^-	Frame C M^+
1) M_u	-173	146	-197.4	170
2) b	88"	88"	94"	94"
3) d	8.25"	7.25"	8.25"	8.25"
4) M_n	-192.2 k-ft	162.2	-219.3	189
5) R	385	421	411	354.5
6) ρ_{req}	0.00674	0.007405	0.00722	0.00618
7) $A_{s,req}$	4.89 in²	4.72	5.6	4.79
8) $A_{s,min}$	1.67 in²	1.67	1.79	1.79
9) N_{req}	7	6	8	7
10) N_{min}	5	5	5	5

Appendix E

Tech 2 | Precast | 1

Precast Hollowcore roof & floor systems

LL = 80psf Assume DL = 15psf

span = 29.33' ≈ 30'

LL = 1.6(80) = 128psf + 1.2(15) = 146psf

PCI Ed 06 page 2-33

use 4'-0" x 10" Hollow core w/ 2" topping 58-S pdf page 73

safe load = 148psf > 146psf

w/ 0.5" camber at erection & 0.1" camber long-time

w_{sw} 93psf

Beams supporting deck:

w_{LL} = 80psf

w_{DL} = 15psf + 93psf = 108psf

w_u = 1.2(108) + 1.6(80) = 257.6psf ÷ (29.33') = 7.56 klf

$M_u = \frac{7.56(31.33)^2}{8} = 927.6$ K-ft

use W24 x 146 per AISC Table 3-10