New York Police Academy

Architectural Engineering Senior Thesis 2010

Technical Report II
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Structural Option
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AE 481W

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EXECUTIVE SUMMARY

The *Structural Study of Alternative Floor Systems* report compares the advantages and disadvantages of four floor systems. The existing floor framing system designed in the New York Police Academy and three alternative floor framing systems are analyzed in this report. The New York Police Academy is an eight story - one million square foot mixed use office, educational and training facility located in Queens, New York. Floor heights, architectural bays and slab depth were manipulated as needed in this report.

The existing superstructure consists of primarily 30'-0" x 30'-0" bays with a 3.25" lightweight concrete slab on 3" composite metal deck. This rests on W18 shaped beams and W24 shaped girders. The three alternative structural slab systems analyzed in this report are:

- Pre-cast Hollow Core Plank on Steel Framing
- Two-Way Flat Slab with Column Capitals
- One-Way Wide-Module Joist System

The pre-cast hollow core plank on steel framing system was designed with the aid of Nitterhouse Concrete Products. This system altered the bay sizing to 32'-0" x 30'-0" because the planks come in 4'-0" increments. The total slab thickness of this system was 34" because the 10" hollow core plank rested on a W24 shaped girder. The two concrete superstructure systems maintained the square bay size. The two-way concrete slab with column capital system and one-way wide-module pan joist system had a total slab thickness of 12.25" and 20.5" respectively.

After calculations were performed, all of the framing systems were compared to one another with respect to architecture, lateral system, foundation, weight, slab and system depth, system cost, fire protection, and constructability. It was determined that the two-way flat slab with column capitals would be the most feasible design alternative for further investigation. However, the one-way pan joist system may also be a viable option for further investigation.

INTRODUCTION

The New York Police Academy is located in College Point, a neighborhood in Queens, New York. This building is an 8-story structure with a west and east campus. It is the first and largest phase of a multiphase project. The west campus houses a physical training facility and a central utility plant while the east campus houses an academic building. The east campus will be analyzed in this technical report. The physical training facility includes a 1/8 mile running track and special tactical gymnasiums. The academic building has a wide variety of classrooms ranging from a capacity of 30 to 300 cadets. Some classrooms create a mock environment for the cadets to experience immersion learning. This phase is expected to cost $656 million. Construction is to begin in October 2010 and culminate in December 2013.

FIGURE 1: THIS IMAGE SHOWS THE LOCATION OF THE NEW YORK POLICE ACADEMY IN ITS SURROUNDINGS. IMAGE COURTESY OF NEWYORK.CONSTRUCTION.COM

The purpose of Technical Report II, *the Structural Study of Alternative Floor Systems,* is to gain a better understanding of the current floor system and explore alternatives that meet the design of the New York Police Academy. These alternative floor systems will be analyzed and conclusions will be determined on the feasibility of a system to be investigated further.

ARCHITECTURAL OVERVIEW

This 8-story 1,000,000 SF structure is used as an academy to train New York Police Department recruits. The building was designed for LEED Silver Certification as designated by the United States Green Building Council (USGBC). This is accomplished by using numerous tactics to minimize its carbon footprint. This building utilizes green roofs and encourages environmentally friendly means of commuting among various other strategies to create a healthier environment.

FIGURE 2: THIS IMAGE SHOWS THE GLAZED ALUMINUM CURTAINWALLS WITH ALUMINUM PANELING. THIS RENDERING IS COURTESY OF TURNER CONSTRUCTION.

The façade of this building is embellished with glazed curtain walls and shimmering aluminum paneling. The aluminum panels act as louvers above the windows both to shade and channel natural light into the building (See Figure 2).

EXISTING SYSTEM STRUCTURAL OVERVIEW

The New York Police Academy's East Campus is 536 feet long and 95 feet wide. The floor to floor height ranges from 14 feet to 16 feet. A green roof system is utilized on the top of the building. The structure of the New York Police Academy consists predominantly of steel framing with a 14" concrete slab on grade on the first floor. All other floors have a lightweight concrete on metal deck floor system. All concrete is cast-in-place.

FOUNDATION SYSTEM

The geotechnical engineering study was conducted by the URS Corporation. The study showed a variety of soil composition, with bedrock reasonably close to the surface. The building foundations for the New York Police Academy bear on piles with a minimum bearing capacity of 100 tons as specified by the URS Corporation. All piles are driven to bedrock. All exterior pile caps are placed a minimum of 4'-0" below final grade. Please see Figure 3 for sample pile cap. Concrete piers, walls, structural slabs on grade, pile caps and grade beams are placed monolithically. Pile caps are 16" in diameter.

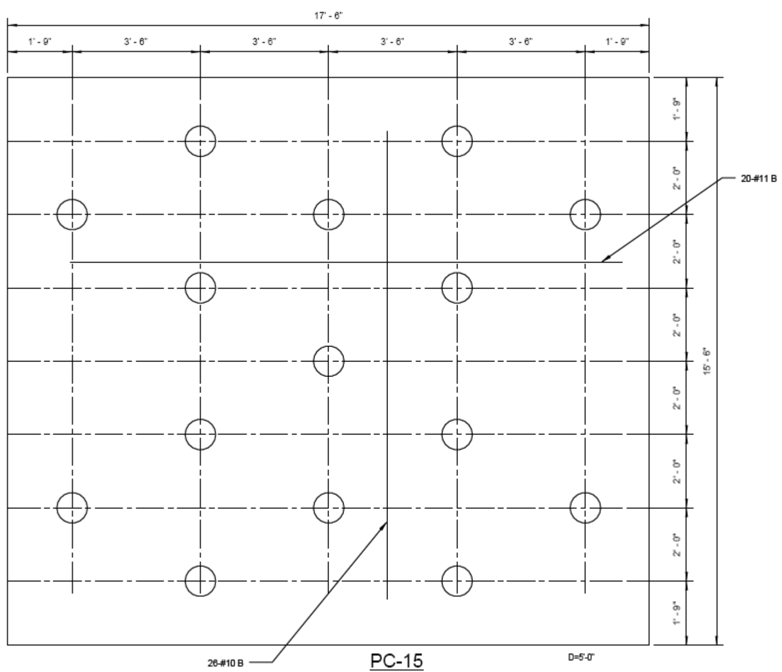


FIGURE 3: THIS IS PLAN OF A SAMPLE PILE CAP. DETAIL COURTESY OF TURNER CONSTRUCTION.

FLOOR SYSTEM

The floor system is made up of 3.25" lightweight concrete slab on 3" - 18 gage metal decking. This will form a one-way composite floor slab system. Units are continuous over three or more spans except where framing does not permit. Shear stud connectors are welded to steel beams or girders in accordance to required specifications. See Figure 4 for details.

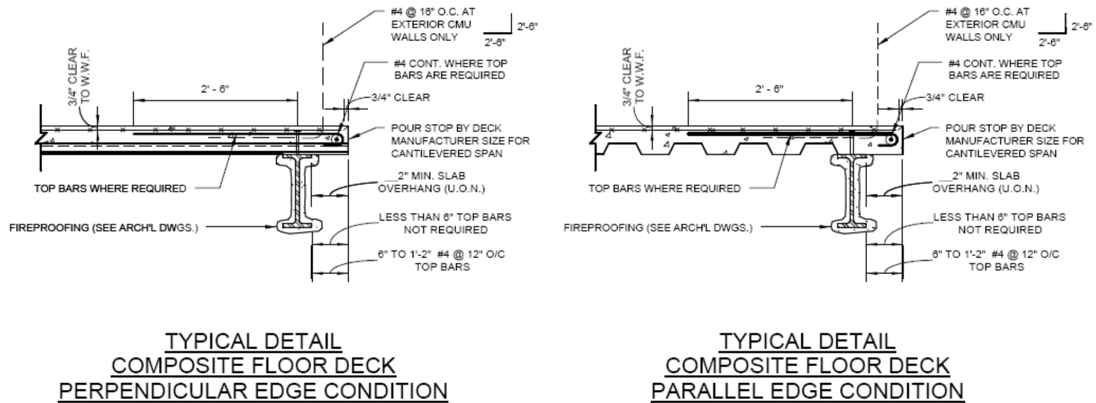
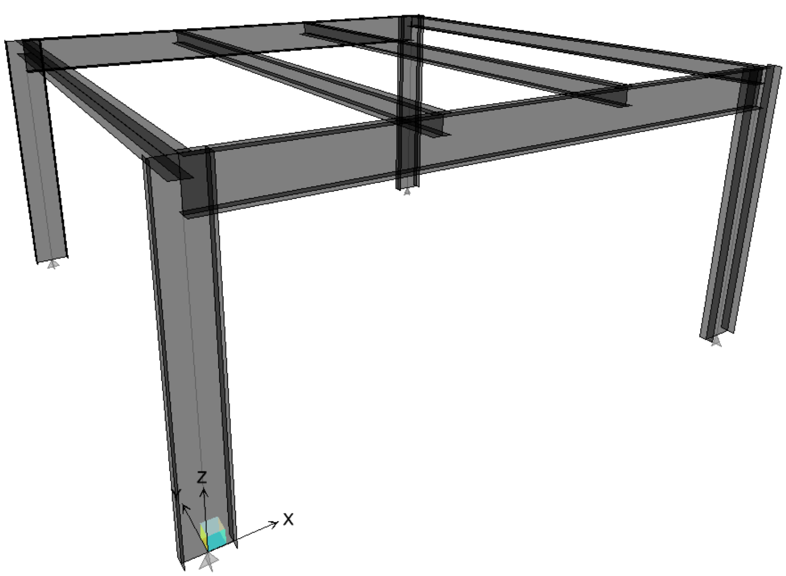


FIGURE 4: TYPICAL SLAB ON DECK FLOOR SECTIONS. DRAWINGS NOT TO SCALE. DETAIL COURTESY OF TURNER CONSTRUCTION.

FRAMING SYSTEM

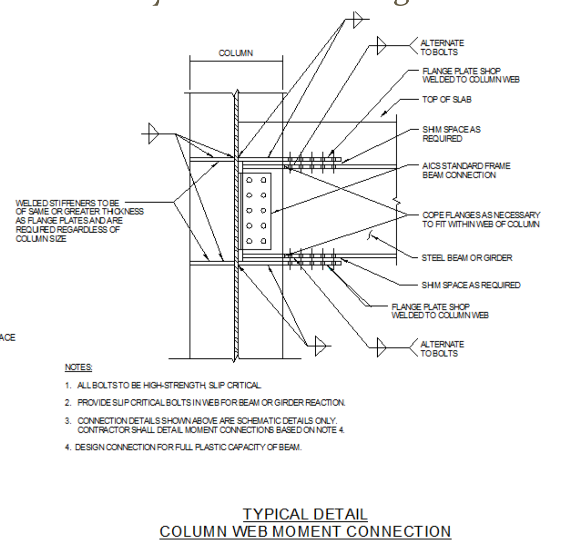


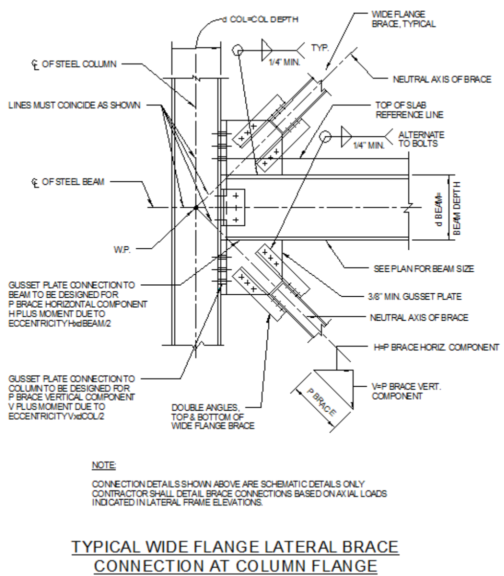
The superstructure is primarily composed to W18 beams, W24 girders and W24 columns. Beams are spaced at 10' increments while girders are spaced at 30' increments. Columns are on a 30'x30' grid. The columns are spliced at 4' above every other floor level and typically span from 30' to 34'. A typical bay is shown in Figure 5.

FIGURE 5: THIS IS AN ETABS MODEL OF THE TYPICAL BAY FRAMING.

LATERAL SYSTEM

The lateral resisting system consists of steel moment connections in addition to lateral HSS and wide flange bracing (see Figure 6). Lateral HSS bracing is found predominantly in the North/South direction to oppose seismic and wind forces. The HSS bracing ranges in size from HSS 6.625x0.375 to 16x0.625. The HSS bracing in the East/West direction is solely used in the bridge to connect two parts of the campus.





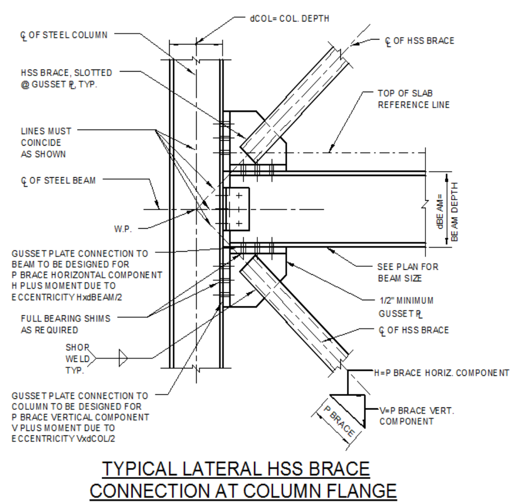


FIGURE 6: TYPICAL COLUMN WEB MOMENT CONNECTION (TOP RIGHT). TYPICAL LATERAL HSS BRACE CONNECTION (BOTTOM RIGHT). TYPICAL WIDE FLANGE LATERAL BRACE CONNECTION (LEFT). ALL DRAWINGS ARE NOT TO SCALE. DETAILS COURTESY OF TURNER CONSTRUCTION.

DESIGN CODES AND STANDARDS

DESIGN CODES:

Design Codes:

- American Concrete Institute (ACI) 318-08, Building Code Requirements for Structural Concrete
- American Concrete Institute (ACI) 315-08, Details and Detailing of Concrete Reinforcement
- American Institute of Steel Construction Manual, 13th Edition
- American Welding Society D1.1-08: Structural Welding Code

Model Codes:

- New York City Building Codes 2008

Structural Standards:

- American Society of Civil Engineers (ASCE) 7-98, Minimum Design Loads for Building and Other Structures

THESIS CODES:

Design Codes:

- American Concrete Institute (ACI) 318-05, Building Code Requirements for Structural Concrete
- AISC Steel Construction Manual, 13th Edition

Model Codes:

- 2006 International Building Code (IBC)

Structural Codes:

- American Society of Civil Engineers (ASCE) 7-08, Minimum Design Loads for Building and Other Structures

DESIGN CRITERIA

DEFLECTION

Horizontal Framing:

- Live Load
    - $< \frac{L}{600}$
- Total Load Excluding Self Weight
    - $< \frac{L}{480}$

Lateral Drift:

- Wind Loads
    - $< \frac{L}{400}$
- Seismic Loads
    - $< \frac{L}{76}$

Main Structural Elements Supporting Components and Cladding:

- At Screen Walls
    - $< \frac{L}{240}$
- At Floors Supporting Curtain Walls
    - $< \frac{L}{600}$
- At Roof Parapet Supporting Curtain Walls
    - $< \frac{L}{600}$
- At Non-Brittle Finishes
    - $< \frac{L}{240}$

MATERIAL PROPERTIES

STEEL

Wide Flanges, Tees	F_y = 50 ksi (A992)
Hollow Structural Sections	F_y = 50 ksi (A500 Grade B)
Structural Pipe Sections	F_y = 36 ksi (A36)
Channels and Angles	F_y = 36 ksi (A36)
Slabs	F_y = 50 ksi (A572 Grade 50)
Slabs	F_y = 42 ksi (A572 Grade 42 for t_{steel}>4")
Bolts	F_u = 105 ksi (A325)
	F_u = 150 ksi (A490)
Anchor Bolts	F_y = 36 ksi (F1554 Grade 36)
Metal Deck	F_y = 33 ksi (A653)
Weld Strength	F_y = 70 ksi (E70XX)

CONCRETE

Foundations, Int. Slab on Grade	NWC f'c = 4000 psi
Slab on Metal Deck	LWC f'c = 4000 psi

REINFORCING

Welded Wire Fabric	70 ksi
Bars to be Welded	60 ksi
Epoxy Coated Bars	60 ksi
All Other Bars (unless otherwise noted)	60 ksi

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

DESIGN LOADS

DEAD AND LIVE LOADS

Robert Silman Associates, the structural engineer of record on this project, used ASCE 7-98 and the BCNYC 2008 as the main reference for dead and live loads on this project. These loads are compared to the most recent applicable standards, ASCE 7-10, *Minimum Design Loads for Buildings and Other Structures*. The load differences per respective codes can be compared in Tables 1 and 2 below. Table 1 shows dead loads while Table 2 outlines the live loads for this building. The loads used for thesis analyses are from ASCE 7-10 unless not specified in the code.

SUPERIMPOSED DEAD LOADS			
DESCRIPTION	**LOCATION**	**NYCBC 2008**	**ASCE 7-10**
CEILING	FLOORS 2-8, ROOF, MEP	5 PSF	--
MEP	FLOORS 2-8, ROOF, MEP	5 PSF	5 PSF
FLOOR FINISHED	FLOORS G-8	5 PSF	--
ROOFING AND INSULATION	FLOORS 3, ROOF, MEP	8 PSF	15 PSF
PARTITIONS	FLOORS G-8	20 PSF	20 PSF
CURTAIN WALL	FLOORS G-ROOF	NOT SPECIFIED	15 PSF
GREEN ROOF	ROOF	NOT SPECIFIED	100 PSF

TABLE 1: THIS TABLE COMPARES SUPERIMPOSED DEAD LOADS BETWEEN NYCBC-08 AND ASCE 7-10.

LIVE LOADS			
DESCRIPTION	LOCATION	NYCBC 2008	ASCE 7-10
ARMORIES AND DRILL ROOMS	FLOOR G	150 PSF	150 PSF
FIXED ASSEMBLY AREA	FLOORS 2-5, 8	60 PSF	60 PSF
LOBBIES	FLOORS G-8	100 PSF	100 PSF
CORRIDORS (TYP.)	FLOORS 2-8	100 PSF	100 PSF
1ST FLOOR OFFICE CORRIDORS	FLOORS G	100 PSF	100 PSF
UPPER FLOOR OFFICE CORRIDORS	FLOORS 2-8	80 PSF	80 PSF
EQUIPMENT ROOMS	FLOORS G, 2, 7-8	75 PSF	75 PSF
LIBRARY READING ROOMS	FLOOR 8	60 PSF	60 PSF
LIBRARY STACKS	FLOOR 8	150 PSF	150 PSF
OFFICES	FLOOR 2-8	50 PSF	50 PSF
FILE AND COMPUTER ROOMS	FLOOR 7	150 PSF	100 PSF
CLASSROOMS	FLOORS 2-8	50 PSF	50 PSF
STAIRS AND EXITS	FLOORS G-MEP	100 PSF	100 PSF
LIGHT STORAGE	FLOORS G-7	125 PSF	125 PSF
HEAVY STORAGE	FLOORS 7, MEP	250 PSF	250 PSF
SNOW	FLOORS 3, MEP, ROOF	22 PSF	22 PSF
*LIVE LOADS REDUCED WHERE APPLICABLE **SNOW DRIFT INCLUDED WHERE APPLICABLE			

Table 2: This table compares live loads between NYCBC-08 and ASCE 7-10.

FLOOR SYSTEM ANALYSIS

Analyses were performed on four different floor systems. The existing system and three alternatives were evaluated to explore floor system options. Vibration calculations were not performed for this portion of the design process due to the complexity of the analyses. Qualitative analysis was completed in order to compare how different floor systems handled vibration. More in depth calculations will be completed once an alternative floor system is proposed. The effects of floor system changes on lateral systems was not analyzed in this report, but noticing that changes would need to be made were taken into account.

EXISTING LIGHT WEIGHT CONCRETE ON COMPOSITE DECK FLOOR

Description:

The superstructure consists of structural steel framing with 3.25" lightweight concrete slab on 3" - 18 gage metal decking. This will form a one-way composite floor slab system. Deck units are continuous over three or more spans except where framing does not permit. The lightweight concrete has a compressive strength of f'c = 4000 psi. Bay sizes are typically on a 30'-0" x 30'-0" square grid. The decking rests on W18X50 beams spaced 10'-0" apart. The beams rest on W24X76 girders, which are spaced 30'-0" apart. The girders frame into columns which are typically W14X145 shapes. Figure 7 illustrates a section of concrete slab on metal deck resting on beams and girders.

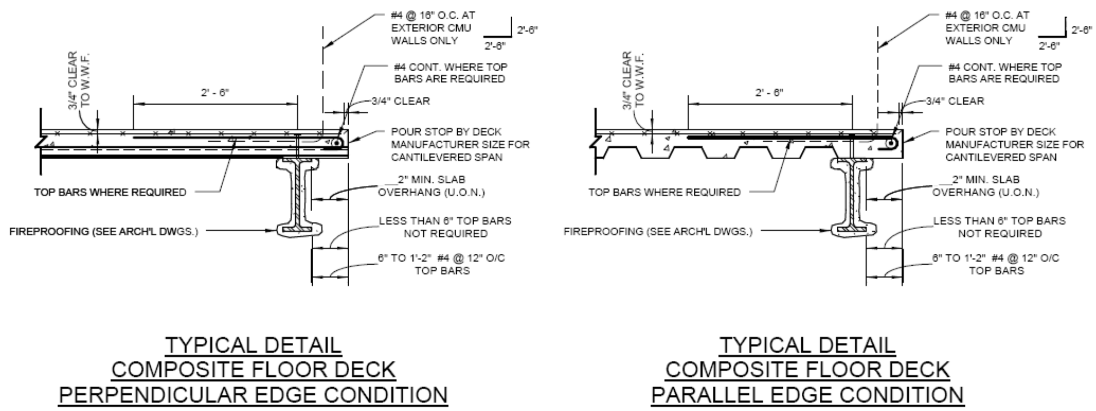


Figure 7: This figure shows the typical slab/deck on framing members. Details courtesy of Turner Construction.

Thesis calculations determined that the existing floor system is sufficient and economically designed. The slab thickness and gage chosen by Robert Silman Associates matched the thesis calculations and can be seen in Appendix B

Advantages:

The existing system has a very low self-weight and is very easy to construct. The relatively light weight reduces member sizing. The decking acts as formwork for the cast-in-place concrete so additional formwork is unnecessary. The shallow deck and slab thickness provides more room for mechanical, electrical and plumbing space. Though the net slab thickness is shallow the fire rating is still two hours which is large relative to the slab depth.

Disadvantages:

Although formwork is not needed, for this particular design shoring is needed because the three-span limit is breached. This slows the speed of construction and increases the cost. Fire proofing is spray-on and adds to the duration as well as cost of construction. The overall thickness of the floor is shallow, but the depth of the beams and girders interfere with this shallow system. This system is also expensive because of the price of materials. However the costs associated with the columns and foundation should be lower due to the self-weight.

PRE-CAST HOLLOW CORE PLANKS ON STEEL FRAMING

Description:

Hollow Core slabs are precision-manufactured, pre-cast/pre-stressed concrete planks produced with normal-weight, high strength concrete. These planks were sized according to Nitterhouse Concrete Products. A 10" thick by 4'-0" wide plank spanning 30'-0" was needed in the New York Police Academy. At 30'-0" spans 7 – ½"ϕ strands must be used. It has a capacity of 162 pounds per square foot, which is sufficient to carry the service loads required. This value can be found in Table 3 below. A 2" lightweight concrete topping was assumed to create a more rigid floor system for lateral loads and to level floors from camber of planks. The strength of the concrete in this system is 6000 psi.

SAFE SUPERIMPOSED SERVICE LOADS														IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)						
Strand Pattern		SPAN (FEET)																		
		26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44
6 - 1/2"ø	LOAD (PSF)	202	181	161	144	128	114	101	90	79	69	60	52	45	38					
7 - 1/2"ø	LOAD (PSF)	246	222	200	180	162	146	131	118	105	94	84	74	66	58					

TABLE 3: THIS TABLE SHOWS WHERE AND HOW VALUES WERE OBTAINED FOR THE HOLLOW CORE PLANK. TABLE COURTESY OF NITTERHOUSE CONCRETE PRODUCTS.

A steel girder was designed to support the hollow core planks. The girder runs perpendicular to the planks and spans 32'-0". A W24X146 was the most economical shape that can support the necessary loads. Figure 8 shows a hollow core plank connection to a steel beam. See Appendix C for calculations.

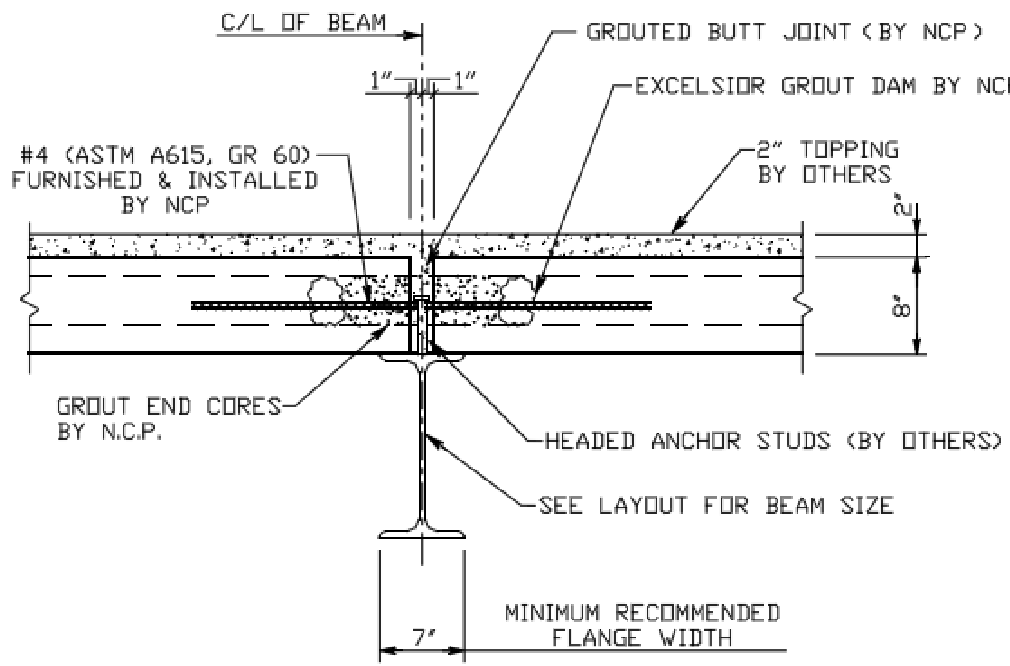


FIGURE 8: THIS FIGURE SHOWS A HOLLOW CORE PLANK CONNECTION BEARING ON A STEEL BEAM. DETAIL COURTESY OF NITTERHOUSE CONCRETE PRODUCTS

Advantages:

Hollow core floors and ceilings offer superior durability and natural sound attenuation. This floor system can be installed quickly in various weather conditions and is low maintenance. Because pre-cast members are constructed in a plant under controlled conditions the planks are at full strength during the time of erection, thus accelerating the construction process. This system achieves a 2 hour fire rating without any additional fireproofing.

Disadvantages:

Normal weight, high strength concrete with a minimum of 5000 psi must be used in a hollow core slab system. 6000 psi concrete was required for this particular design. The normal weight concrete increases the weight of the superstructure considerably. This affects the column, girder and foundation sizes driving up the cost. When 6000 psi concrete is specified by a designer in New York City special requirements apply. Inspectors must be hired to ensure the concrete on site is the required 6000 psi. If the concrete is not up to par then the inspector can halt the project until proper requirements are met. This can also interfere with scheduling and increase cost. Because the planks are prefabricated this process may be avoided, but more research must be done to confirm that. The depth of the hollow core system is the greatest of all systems analyzed. This would either lower floor to ceiling height, or the designer would have to increase the total height of the building to maintain the current floor to ceiling height. Because planks come in four foot increments, bay sizes for this particular building layout had to be changed from a 30'-0" x 30'-0" square grid to a 32'-0" x 30'-0" rectangular grid.

TWO-WAY FLAT SLAB WITH COLUMN CAPITALS

Description:

The flat Slab floor system with column capitals is a two-way concrete plate with reinforcing spanning orthogonally in two directions. When column capitals are incorporated for punching shear it is very similar to a flat slab system with drop panels. The capital thickens the slab at the columns in order to oppose punching shear. This can be seen in Figure 9.

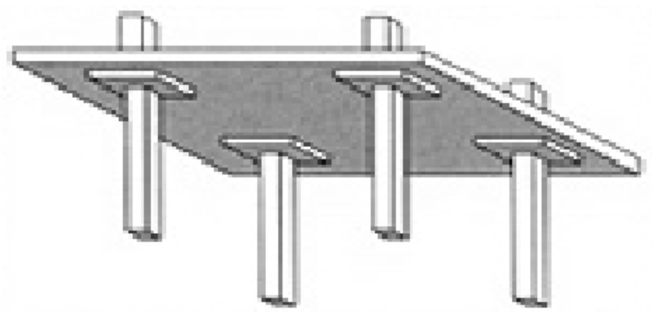
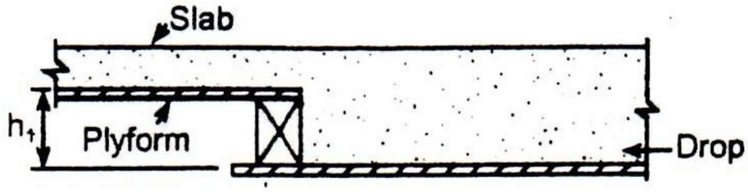


FIGURE 9: THIS FIGURE SHOWS THE LAYOUT OF A TWO-WAY FLAT PLATE SYSTEM WITH COLUMN CAPITALS. COURTESY OF THE CONCRETE REINFORCING STEEL INSTITUTE.

A 10" thick slab spanning 30'-0" was needed in this building. The slab was able to support all moments, but at longer spans the shear around the column was large despite the use of lightweight concrete (4000 psi). Column capitals have been used less in recent construction due to the inconvenience it causes during construction because it tends to use oddly shaped formwork. However, this was taken into account in this design by using 4'-0" square column capitals with a 2¼" thick drop. This is slightly overdesigned, but the advantages in construction explain this. The 4'-0" modules work well when placing plyform because plyform comes in 4'-0" increments. The 2¼" thick drop is used because it takes into account a wood 2x4 in between the two ¾" plyforms. This can be seen in Figure 10 below.



Nominal Lumber Size (in.)	Actual Lumber Size (in.)	Plyform Thickness (in.)	h_1 (in.)
2x	1 1/2	3/4	2 1/4
4x	3 1/2	3/4	4 1/4
6x	5 1/2	3/4	6 1/4
8x	7 1/4	3/4	8

FIGURE 10: THIS FIGURE SHOWS DIMENSIONS NEEDED TO ACCOMMODATE FORMWORK IN THE FIELD. DETAIL COURTESY OF THE PORTLAND CEMENT ASSOCIATION'S GUIDE TO ESTIMATING AND ECONOMIZING CONCRETE FLOOR SYSTEMS

Because the column capitals were used no additional rebar was needed for punching shear. Moments were calculated by using the *Direct Design Method*. #7 rebar, though conservative in some instances, was used for reinforcement throughout the analysis. This was done for simplicity during construction. The use of a standard bar size decreases the chance of that the wrong reinforcement is inserted and avoids potential failure because of this reason. See Appendix D for all calculations.

Advantages:

The flat Slab system provides low floor to floor heights in order to reduce the total height of the structure or fit more floors into the total height. Flat Slabs offer flat ceilings which reduce ceiling finishing costs because the architectural finish can be applied directly to the underside of the slab. The increased slab thickness around the columns increases relative stiffness of the system as well.

Although material cost is greater by utilizing a larger amount of concrete in the capital than is structurally necessary, the cost of labor decreases because of the simplicity of the formwork. Simple formwork yields simple and timely construction, which will help reduce the duration of construction as well as the cost of labor. This method also uses much less concrete and formwork than the standard two-way flat slab with drop panel system, while providing similar results.

Disadvantages:

The column capitals do require the use of more formwork and concrete than the typical flat Slab floor system. However, it is still cheaper than if the floor slab thickness was increased because of the additional concrete used is far greater than the amount used in the column capitals.

ONE-WAY WIDE-MODULE PAN JOISTS

Description:

The wide-module joist floor system (also known as the skip joist floor system) consists of regularly spaced concrete joists (or ribs) spanning in one direction, a reinforced concrete slab cast integrally with the joists, and beams that span between the columns, perpendicular to the joists. The joists are formed using pans that vary in size based on desired joist location. A picture of this system can be seen in Figure 10. 66" pans were chosen in this design because they have the cheapest cost index for the desired 30'0" spans. For this pan width the pan depth ranges from 14"-24". To maintain the 30' x 30' grid a 16" pan depth was chosen. The joists are 6" wide. The slab thickness is 4½" to obtain a two hour fire rating. The 16" pan depth is adequate for deflection control. The flexural reinforcement required in

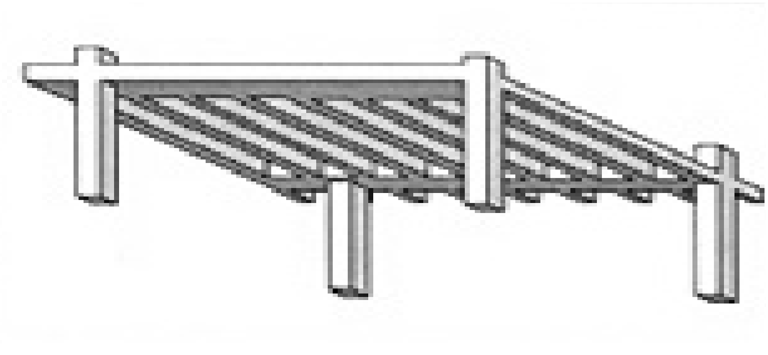


FIGURE 10: THIS FIGURE SHOWS THE LAYOUT OF A ONE-WAY JOIST SYSTEM. COURTESY OF THE CONCRETE REINFORCING STEEL INSTITUTE.

the slab is one #3 bar spaced at 12”. The flexural reinforcement for both positive moment reinforcing (bottom) and negative moment reinforcing (top) is two #9 bars. #3 stirrups spaced at 3” are used in the joists for shear.

A girder spanning 30’-0” was designed to be 20.5” deep by 36” wide. Calculations were performed for a 20.5”x24” girder, however it was insufficient. Eleven #9 bars were needed for top reinforcing and eight #9 bars were needed for bottom reinforcing. Please see Appendix E for all calculations.

Advantages:

The 66” pan with the 6” rib width created 6-’0” intervals for each pan. This fit into the existing 30’-0” x 30’-0” bay size so spans did not need to be changed. One-way joist systems are economical for long spans and heavy loads. The 100 pound-per-square-foot live load is considered a heavy load. The large pan voids reduce the dead load. Electrical and mechanical equipment can be placed between joists so that the overall floor depth need not be increased to accommodate this equipment. This system is good for office buildings and schools. The New York Police Academy incorporates both offices and educational spaces within its design so this system fits the function well.

Disadvantages:

Although the dead load is decreased because of the large pan voids, the self-weight is still much greater than the existing system. This weight difference impacts the foundation and must be taken into account. The cost of formwork in the pan joist system is high because ribs must be placed individually. This is both time consuming, labor intensive and the cost of formwork increases as well.

FLOOR SYSTEM COMPARISON

	EXISTING LWC SLAB ON METAL DECK	OPTION #1 PRECAST CONCRETE HOLLOW CORE PLANKS	OPTION #2 TWO–WAY CONCRETE FLAT SLAB WITH COLUMN CAPITALS	OPTION #3 ONE-WAY WIDE-MODULE PAN JOIST
ARCHITERCTURAL ALTERATION (BAY SIZES)	NO (30'x30')	YES (30'x32')	**NO (30'x30')**	**NO (30'x30')**
LATERAL SYSTEM ALTERATIONS	NO	**MINIMAL**	YES	YES
FOUNDATION IMPACT	N/A	HIGH	HIGH	**MEDIUM**
WEIGHT	51 PSF	**125 PSF**	318PSF	277PSF
SLAB DEPTH	3.25"	10"	10"	**4.5"**
SYSTEM DEPTH	25"	34"	**12.25"**	20.5"
SYSTEM COST	$26.03/SF	$32.36/SF	**$22.70/SF**	$25.13/SF
FIRE PROTECTION METHOD	SPRAY ON	SPRAY ON	**N/A**	**N/A**
FIRE RATING	2 HOUR	2 HOUR	**> 2 HOUR**	2 HOUR
FORMWORK	NO	**NO**	YES	YES
VIBRATION CONTROL	AVERAGE	BELOW AVERAGE	AVERAGE	**ABOVE AVERAGE**
CONSTRUCTABILITY	GOOD	**GOOD**	AVERAGE	BELOW AVERAGE
FEASABLE AS ALTERNATIVE SYSTEM	N/A	NO	**YES**	**YES**

TABLE 4: THIS TABLE COMPARES THE DIFFERENT FLOOR SYSTEM AMONG VARIOUS CRITERIA. FURTHER DISCUSSION IS LISTED BELOW.

ARCHITECTURAL ALTERATIONS

All of the systems analyzed maintain the typical 30"-0" x 30'-0" bay size except for the hollow core plank system. This system requires a 2'-0" increase in one direction creating a 32'-0" x 30'-0" bay size. This will either increase the total size of the building or other bay sizes will need to be altered to fit these dimensions. Nitterhouse planks only come in 4'-0" increments, which results in the change of bay size. The curtain walls are hung on the steel framing in both the existing system and the hollow core system. This is more difficult to accomplish with the concrete framing systems. Façade alterations would most likely need to be performed to better suit the concrete superstructure.

LATERAL SYSTEM ALTERATIONS

The existing lateral system consists of moment frames and lateral steel bracing. This system would need to be altered slightly in order to accommodate the difference in bay sizes for the hollow core system, but would remain very similar. The concrete systems cannot use steel moment connections or steel lateral bracing. The concrete lateral bracing systems require steel reinforcing bars to oppose wind and seismic loads.

FOUNDATION ALTERATIONS

The existing superstructure is very light in weight when compared to the other systems with a square foot unit weight of 52 pounds. The pile caps currently need to support 100 tons. The hollow core plank is almost two and a half times as heavy as the existing system weighing 125 pounds per square foot. This could be due to the change in system, but also can be attributed to the use of normal weight concrete in the precast hollow core plank system. If the foundation needed to support two and a half times more weight the pile caps would need to accommodate this. The concrete structures are much heavier than the steel structures. The two-way flat slab and one-way joists systems weigh 318 pounds per square foot and 277 pounds per square foot respectively. This is much heavier than the existing system especially considering lightweight concrete is utilized in these systems. The foundations would need to be completely redesigned to accommodate the additional load on this structure.

WEIGHT

The systems requiring a concrete superstructure are much heavier than those utilizing a steel superstructure. The existing system is by far the lightest because the least amount of concrete is used, and the concrete that is used is lightweight. The precast hollow core plank system is heavier than the existing system because it requires normal weight concrete with a similar framing system. The concrete systems are much heavier and will have a large impact on the foundation as stated above.

SLAB AND SYSTEM DEPTH

While the existing system has the smallest slab thickness the total system depth is rather large because W24 girders that are needed to support the concrete on metal deck. The one-way pan joist system has the smallest net slab thickness with 4.5", but the 16" deep joists add to the overall depth of the system. The hollow core plank and two-way concrete slab thicknesses are both 10" thick, but the total system depth is quite different. The 10" thick hollow core slab rests on W24 girders creating a system depth of almost three feet. The two-way concrete flat slab has 2.25" drops giving the system a total thickness of 12.25". This is the thinnest system by over 8".

SYSTEM COST

The costs for this project varied from $22.07 per square foot using the two-way flat slab with column capital system to $32.36 per square foot for the precast hollow core plank system. This is a wide range of costs for these floor systems. The existing system ranks as the second highest cost. This could result from the price of materials and the need for spray-on fireproofing. The systems with concrete superstructure are both cheaper than those with steel superstructure.

FIRE PROTECTION

The steel systems need additional spray-on fireproofing. The concrete systems need no additional fireproofing. All of the systems achieve the desired 2 hour rating.

CONSTRUCTABILITY

The existing system and the precast concrete hollow core plank system are both steel in superstructure and highly constructible. The existing system is not as efficient because shoring is needed. The planks are pre-fabricated and can be transported and easily placed on site. The transportation from factory to site could be an issue depending on building location. With the New York Police Academy being in New York City, driving truck-loads of hollow core planks to this one million square foot project could be a cause for concern. Both of these systems require spray-on fireproofing to maintain the necessary 2-hours fire rating. This increases the cost and duration of construction.
The two-way flat slab with column capital system and the one-way joist system require formwork to place the concrete superstructure. The one-way joist system require more formwork because 16” x 6” ribs are located every 6’-0”. This increases the duration of construction and increases labor costs.

CONCLUSION

Upon completion of the floor system study, the ultimate goal was to select a reasonable alternate floor system for the New York Police Academy. The analysis showed that the two-way flat slab with column capitals is the most feasible alternative floor system design. In this design the bay size remains the same; the overall system depth is reduced by almost 50%, and it has the lowest cost of all systems analyzed. However, the weight increases dramatically, which will have a large impact on the foundation. The curtain wall façade would need to be altered to better suit the concrete superstructure as well.

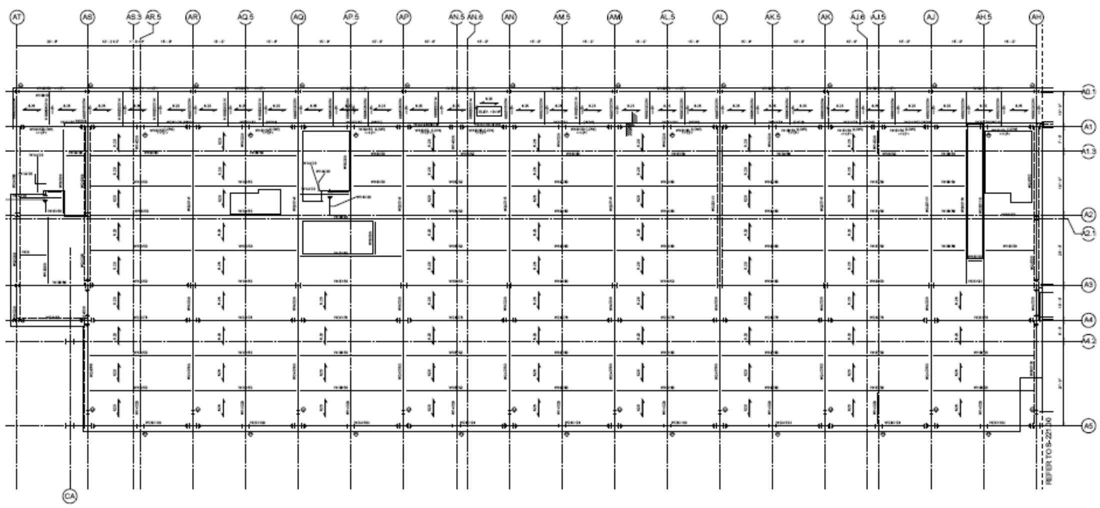
The New York Police Academy is a government project so the costs must be minimized as much as possible. This building will be paid in tax dollars so the owner (the New York Department of Design and Construction) does not have an unlimited budget. The cost of floor systems alone does not reflect the net cost of the project. The effect on architecture, lateral system and foundation must also be considered. Heavier buildings require stronger foundations. This would increase cost of the project. If shear walls are needed to resists lateral loads (upon further analysis) then this may drive up costs as well. Despite these factors the two-way flat slab floor system seems to be the most viable option that can maintain a low cost, preserve the designed bay sizes, while increasing ceiling height.

The two other systems analyzed were less practical than the two-way flat slab with column capital system. The precast hollow core plank on steel framing system although highly constructible is just too much money for the scope of this project. The one-way pan joist system, although maintaining bay size and having an inherent vibration resisting system is more expensive than the two-way flat slab with column capital system and not much lighter in weight. This system is not to be ruled out completely as further research may indicate that it is more useful.

From the information gathered in this floor system comparison report, it has been determined that the two-way flat slab with column capital and one-way pan joist systems shall be further investigated as possible alternate floor systems.

APPENDIX A: FRAMING PLANS AND ELEVATIONS

FRAMING PLAN PART 1 (WEST END)



FRAMING PLAN PART 2 (EAST END)

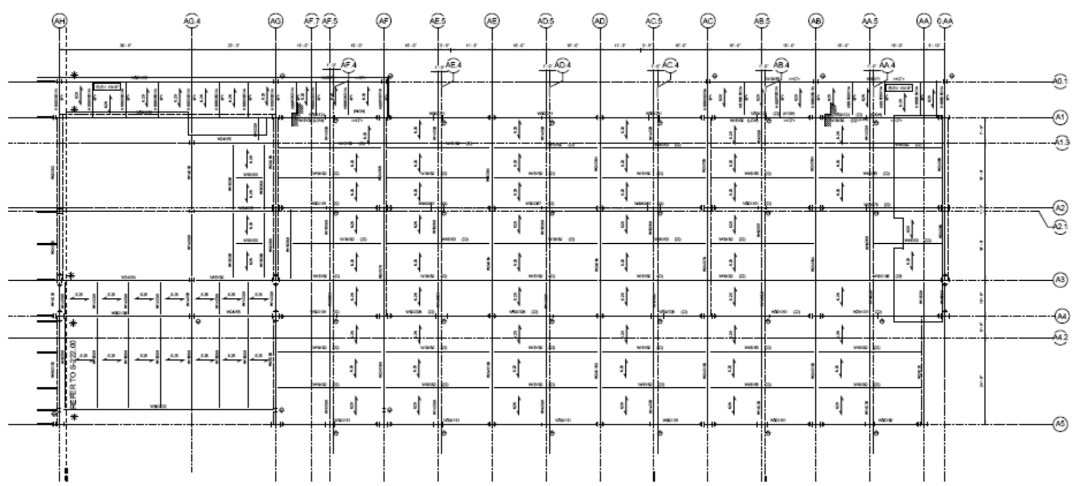


FIGURE 11: THIS IS THE TYPICAL FRAMING PLAN OF ONE FLOOR OF THE NEW YORK POLICE ACADEMY. PLEASE NOTE THAT THE BUILDING IS SO OBLONG THAT EACH FLOOR PLAN IS SPLIT INTO TWO SHEETS WITH PART 1 (THE WEST END) AND PART 2 (THE EAST END).

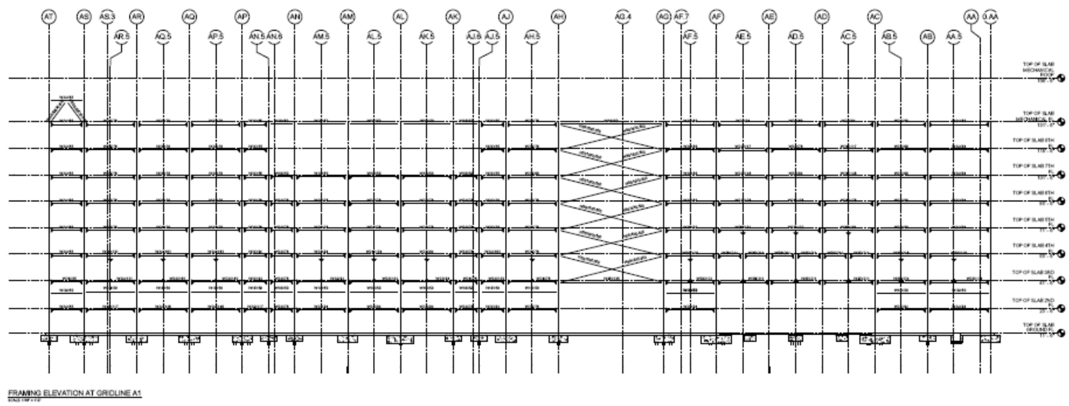
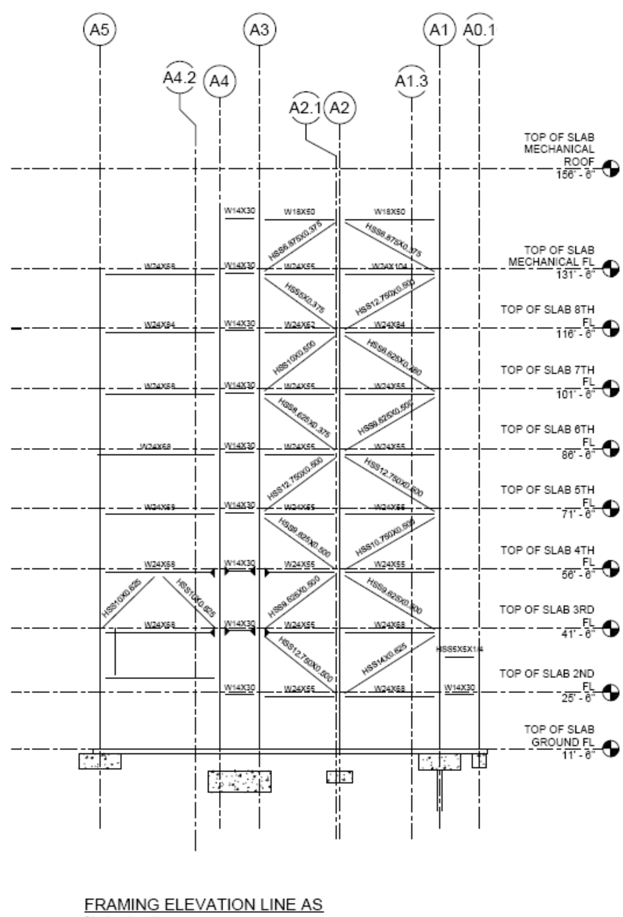


Figure 12: Above is an elevation of the framing system looking in the North/South direction. Notice only moment connections except for the cross bracing on the bridge. Below is an elevation of the framing system looking in the East/West dirction. Notice the majority of cross bracing in this direction compared to few moment connections.



APPENDIX B: EXISTING LIGHTWEIGHT CONCRETE ON METAL DECK SYSTEM CALCULATIONS

JAKE POLLACK | TECH #2 | TYPICAL SLAB / EXISTING SYSTEM CALCULATIONS | 1/7

TYPICAL SLAB/METAL DECK

3.25" LIGHTWEIGHT CONCRETE FILL ON 3",
18 GAGE COMPOSITE METAL DECK:
- MATCHES 1.5 VL 18 IN VULCRAFT STEEL ROOF AND FLOOR DECK CATALOG

MAXIMUM UNSHORED SPAN:

9'-9", 3 SPAN ⇒ BEAMS TYPICALLY SPACED 10'-0" O.C. ⇒ CLEAR SPAN > 9'-9"

∴ SHORING IS REQUIRED

UNFORM LIVE SERVICE LOAD:

STUDS SPACED 1 PER FOOT, 4" SLAB, 10" SPAN
⇒ $\boxed{184 \text{ PSF}}$ = CAPACITY

USE UNFACTORED LOADS TO CHECK:

DL = 46 PSF
SDL = 40 PSF
LL = 100 PSF*
*REDUCED TO 60 PSF

TOTAL REDUCED LOAD = $\boxed{146 \text{ PSF}}$

$100 \times \left[0.25 + \frac{15}{\sqrt{2(30)(30)}}\right] = 60 \text{ PSF}$

$146^{PSF} < 184^{PSF}$ ∴ OK

JAKE POLLACK | TECH #2 | EXISTING SYSTEM | 2/7

COMPOSITE BEAM

TRIBUTARY AREA:

$A_T = 10' \times 30' = 300$ SF

$A_T < 400$ SF ∴ NOT REDUCIBLE

LOADS:

SDL = 40 PSF

FROM VULCRAFT CATALOG

DL = SLAB + BM = 46 + 5 = 51 PSF

ASSUMED

LL = 100 PSF

$W_u = [1.2(40+51) + 1.6(100)]10' = \boxed{2.69 \text{ KLF}}$

MAX $M_u^- = \frac{W_u l_n^2}{8} = \frac{2.69(30^2)}{8} = \boxed{302.6^{\text{'K}}}$

FIND b_{eff}

$b_{eff} = $ MIN
- $l(12)/8 \times 2 = \boxed{90''}$ ⇐ CONTROLS
- EDGE DISTANCE ⇒ N/A
- 1/2 DISTANCE TO NEXT BEAM = 120''

FIND Y_2

ASSUME $a \approx 1$

GIVEN $t_{DECK} = 3''$

$t_{SLAB} = 3.25''$

$Y_2 = t_{DECK} + t_{SLAB} - a/2 = 5.75'' \approx \boxed{5.5''}$

USING TYPICAL W18X50

USE $Y_1 = 0.428$ FOR PNA = 4 ⇒ $\boxed{\phi M_n = 707^{\text{'K}}} > 302.6^{\text{'K}}$ ∴ OK

$\boxed{\Sigma Q_n = 413^{\text{K}}}$

JAKE POLLACK | TECH #2 | EXISTING SYSTEM | 3/7

USING 3/4" STUDS ⇒ FIND # STUDS, a, Y_2, WEIGHT

$$\frac{\Sigma Q_n}{Q_n} = \frac{413}{21.2} = 20 \times 2 = \boxed{40 \text{ STUDS}}$$

AISC TABLE 3-21

$$a = \frac{\Sigma Q_n}{0.85(f'_c)(b_{eff})} = \frac{413}{0.85(4)(90)} = \boxed{1.35" = a}$$

$$Y_2 = t_{SLAB} + t_{DECK} - a/2$$
$$= 3.25 + 3 - 1.35/2 = \boxed{5.58"} > 5.5" \therefore OK$$

BEAM WEIGHT × l + # STUDS × $STUD_{WT}$

$$50 \times 30 + 40 \times 10 = \boxed{1.9^k}$$

CHECK UNSHORED STRENGTH

TABLE 3-2 $\boxed{\phi_b M_p = 413^{1k}}$

$$W_u = 1.2((46+40)\times10 + 50) + 1.6(100\times10) = \boxed{2.69 \text{ KLF}}$$

$$M_u = \frac{W_u l_n^2}{8} = \frac{2.69(30^2)}{8} = 302.6^{1k} < 413^{1k} \therefore OK$$

$$V_u = \frac{W_u l_n}{2} = \frac{2.69(30)}{2} = 40.4^k < 192^k \therefore OK$$

CHECK Δ_{LL}

FIND W_{LL} = 100 × 10' = 1 KLF

AISC TABLE 3-20 ⇒ I_{LB} = 2120

$$\Delta_{LL} = \frac{5W_{LL}(l_n^4)(1728)}{384\,E I_{LB}} = \frac{5(1.0)(30^4)(1728)}{384(29000)(2120)} = \underline{0.30" < 1" \therefore OK}$$

$$\frac{L}{360} = \frac{l(12)}{360} = 1"$$

CHECK $\Delta_{WET\ CONCRETE}$

$W_{WC} = (46 \times 10) + 50 = \boxed{0.51\ KLF}$

$\Delta_{WC} = \frac{5 w_c l_n^4 (1728)}{384 EI} = \frac{5(0.51)(30^4)(1728)}{384(29000)(800)} = \boxed{0.40''}$

TABLE 3-3

$L/240 = \frac{l(12)}{240} = 1.5'' > 0.4'' \therefore OK$

$\boxed{W18\times50 \text{ w/ } 40\ 3/4''\ \phi \text{ STUDS}}$

JAKE POLLACK TECH # 2 EXISTING SYSTEM 5/7

COMPOSITE GIRDER

TRIBUTARY AREA:

$A_T = 30' \times 30' = 900\ SF > 400 \therefore$ REDUCIBLE

LOADS

SDL = 40 PSF

DL = SLABS + GIRDER = 46 + GIRDER SIZE

$LL = 100\ PSF \times \left[0.25 + \frac{15}{\sqrt{2(30)(30)}}\right] = 60\ PSF$

$W_u = 1.2((46+40)30 + 76) + 1.6(60 \times 30) = 6.07\ KLF$

← W24x76

$M_u\ M_{MAX} = \frac{w_u l_n^2}{8} = \frac{6.07(30^2)}{8} = 682.9^{\prime k}$

USING W24x76 ASSUME $Y_2 = 7''$

$Y_1 = 0.51''$, $\phi M_n = 1340^{\prime k}$, $\Sigma Q_n = 660^k$, PNA = 4"

OF STUDS (3/4" ϕ STUDS)

$\frac{\Sigma Q_n}{Q_n} = \frac{660}{21.2} = 31.1 \times 2 = 64$ STUDS

← DECK // LWC

FIND WEIGHT

$76 \times 30' + 64 \times 10^{\#} = 2.92^k$

CHECK UNSHORED STRENGTH (AISC TABLE 3-2)

$\boxed{\phi_b M_p = 750^{\prime k}} > 682.9^{\prime k} \therefore$ OK

$V_u = \frac{w_u l_n}{2} = \frac{6.07(30)}{2} = \boxed{91.0^k} < 316^k \therefore$ OK

JAKE POLLACK | TECH #2 | EXISTING SYSTEM | 6/7

CHECK Δ_{LL}

$$\Delta_{LL} = \frac{5 w_L l_n^4 (1728)}{384 E I_{LB}} = \frac{5(60\times30)(30^4)(1728)}{384(29000)(5080)} = \boxed{0.22''}$$

$$\Delta_{LL} = \frac{l_n(12)}{360} = 1'' > 0.22'' \therefore OK$$

CHECK Δ_{WC}

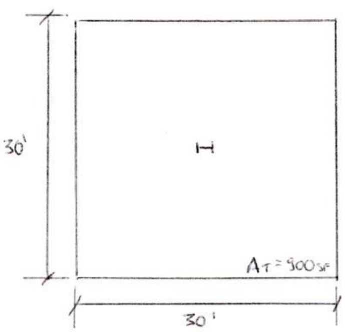
$$\frac{46\times30}{1000} = 1.38\ KLF$$

$$\Delta_{WC} = \frac{5 W_{WC} l_n^4 (1728)}{384 E I_x} = \frac{5(1.38)(30^4)(1728)}{384(29000)(2100)} = \boxed{0.41''}$$

$$L/240 = \frac{30(12)}{240} = 1.5'' > 0.41'' \therefore OK$$

JAKE POLLACK | TECH #2 | EXISTING SYSTEM | 7/7

COLUMNS EXTEND 8 FLOORS AND IS SPLICED AT FLOORS 3, 5 & 7.



$$LL_{RED} = 100 \text{ PSF} \times \left[0.25 + \frac{15}{\sqrt{4(30^2)}}\right]$$

$$= 50 \text{ PSF}$$

$$P_{DL} = (\overset{SDL}{46} + \overset{DL}{40})(8)(900) = 619.2^k$$

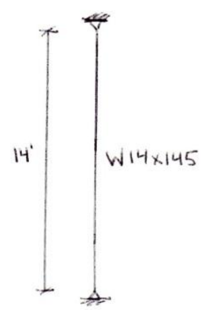
$$P_{SNOW} = 22\ (900) = 19.8^k$$

OBTAINED IN TECH 1

$$P_L = (50)(900)(8) = 360^k$$

$$P_{TOT} = 1.2(619.2) + 1.6(360) + 0.5(19.8)$$

$$= 1329^k$$



$$\phi P_{n\ W14x145 @ 14'} = \boxed{1690^k} > 1329^k \therefore OK$$

* NOTE W14x145 TAKEN FROM STRUCTURAL DESIGN DRAWINGS

APPENDIX C: HOLLOW CORE PLANK ON STEEL FRAMING SYSTEM CALCULATIONS

PRE-CAST HOLLOW CORE PLANKS ON STEEL BEAMS

MAXIMUM BAY SIZE = 32' x 30'

* NOTE CHANGE IN BAY SIZE (TYP. 30' x 30')

LOADING FOR PLANKS:

SAFE SUPERIMPOSED SERVICE LOAD:

LL + SDL = 100 + 40 = $\boxed{140\ \text{PSF}}$

* ASSUMPTION: PLANK & TOPPING SELF-WEIGHT TAKEN INTO ACCOUNT IN TABLE.

⇒ USE NITTERHOUSE PRESTRETSED CONCRETE $\boxed{10'' \times 4'\text{-}0''}$ HOLLOW CORE PLANK W/ $\boxed{2''\ \text{TOPPING}}$ ← LWC

AT 30' SPAN USE $\boxed{7 - \tfrac{1}{2}''\ \phi\ \text{STRANDS}}$

⇒ CAN CARRY $\boxed{162\ \text{PSF}}$

NOTE: $\boxed{f'_c = 6000\ \text{PSI} \quad NWC = 150\ \text{PCF}}$

PLANK & TOPPING SELF-WEIGHT

= 68 + 25 PSF = $\boxed{93\ \text{PSF}}$

DESIGN GIRDER B/W COLUMNS

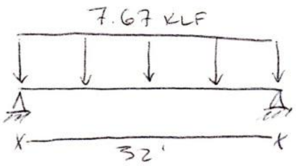
RUNNING PERPENDICULAR TO PLANKS:

SDL = 40 PSF

DL = 93 PSF

LL = 100 PSF ← REDUCE TO $\left\{ LL = 100\left(0.25 + \frac{15}{\sqrt{2(32)(30)}}\right) = 60 \text{ PSF} \right.$

$W_u = 1.2(40+93)(30) + 1.6(60)(30) = \boxed{7.67 \text{ KLF}}$



$M_u = \frac{W_u l^2}{8} = \frac{7.67(32)^2}{8} = \boxed{981.8^{\text{IK}}}$

$V_u = \frac{W_u l}{2} = \frac{7.67(32)}{8} = \boxed{122.7^{\text{K}}}$

FROM TABLE 3-10: USE A $\boxed{\text{W24x146}}$

$\boxed{\phi M_n = 1017^{\text{IK}}} > 981.8^{\text{K}} \therefore$ OK

TABLE 3-2:

$\boxed{\phi V_n = 482^{\text{K}}} > 122.7^{\text{K}} \therefore$ OK

JAKE POLLACK | TECH II | PRE-CAST CORE PLANKS ON STEEL FRAMING | 3/3

CHECK DEFLECTION

CONSTRUCTION Δ: $DL = 93(32) + (146) = \boxed{3.12 \text{ KLF}}$

$$\Delta = \frac{5}{384}\left[\frac{3.12(32)^4(1728)}{29000(4580)}\right] = \boxed{0.55''}$$

$$L/360 = \frac{32(12)}{360} = 1.07'' > 0.55'' \therefore OK$$

LIVE Δ:

$$\Delta = \frac{5}{384}\left[\frac{(0.06)(30)(32)^4(1728)}{(29000)(4580)}\right] = \boxed{0.32''}$$

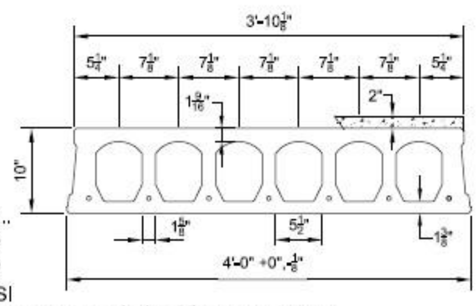
$$L/360 = 1.07'' > 0.32'' \therefore OK$$

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

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES Composite Section	
A_c = 327 in.^{2}	Precast b_w = 13.13 in.
I_c = 5102 in.^{4}	Precast S_{bcp} = 824 in.^{3}
Y_{bcp} = 6.19 in.	Topping S_{tct} = 1242 in.^{3}
Y_{tcp} = 3.81 in.	Precast S_{tcp} = 1340 in.^{3}
Y_{tcp} = 5.81 in.	Precast Wt. = 272 PLF
	Precast Wt. = 68.00 PSF

DESIGN DATA



1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø and 0.6"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   6-1/2"Ø, 270K = 168.1 k-ft at 60% jacking force
   7-1/2"Ø, 270K = 191.7 k-ft at 60% jacking force
7. Maximum bottom tensile stress is $10\sqrt{f'c}$ = 775 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS		IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)																		
Strand Pattern		SPAN (FEET)																		
		26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44
6 - 1/2"ø	LOAD (PSF)	202	181	161	144	128	114	101	90	79	69	60	52	45	38					
7 - 1/2"ø	LOAD (PSF)	246	222	200	180	162	146	131	118	105	94	84	74	66	58					

NITTERHOUSE
CONCRETE PRODUCTS

2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17202-9203
717-267-4505 Fax 717-267-4518

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

11/03/08

10F2.0T

APPENDIX D: TWO-WAY FLAT SLAB WITH COLUMN CAPITALS

DESIGN A TWO-WAY FLAT PLATE

BAY: 30' x 30'

ASSUME 24" SQUARE COLUMNS

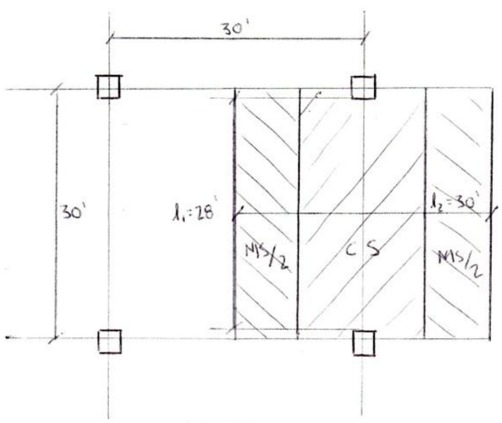
USE LIGHTWEIGHT CONCRETE (110PCF)

$f'_c = 4000$ PSI

$f_y = 60$ KSI

LL = 100 PSI

NOTE: BECAUSE SQUARE BAYS ARE UTILIZED DIMENSIONS IN BOTH DIRECTIONS WILL BE IDENTICAL.



$$l_n = 30' - \underbrace{1' - 1'}_{\text{FROM COLUMN}} = 28'$$

ACI 9.5.3.2 - DETERMINE SLAB THICKNESS

- SLABS W/OUT INTERIOR BEAMS

$$t_{SLAB} = \max \begin{cases} 4" \\ \dfrac{l_n}{} = \dfrac{28'(12"/FT)}{} = \end{cases} \Rightarrow \text{USE } 10"$$

METHOD FOR COMPUTING DESIGN MOMENTS

DIRECT DESIGN METHOD CAN BE USED:

- MORE THAN 3 CONTINUOUS SPANS IN EACH DIRECTION ✓
- $\dfrac{\text{LONG SPAN}}{\text{SHORT SPAN}} = 1 < 2$ ✓
- SUCCESSIVE SPAN LENGTHS NO MORE THAN 1/3 LONGER SPAN ✓
- COLUMN OFFSETS ✓

- GRAVITY LOADS & UNIFORMLY DISTRIBUTED ✓
- UNFACTORED LL ≤ 2 × UNFACTORED DL

LL = 100 PSF
SDL = 40 PSF
SLAB = 10"/12"/FT (110 PCF) = 91.7 PSF

$100 \leq 2(40 + 91.7)$ ✓

COMPUTE POSITIVE & NEGATIVE MOMENTS IN SLAB

$$M_o = \frac{q_u \, l_n^2 \, l_2}{8}$$

$$q_u = 1.2\left[\frac{10''}{12''/\text{FT}}(110\text{ PCF}) + 40\text{ PSF}\right] + 1.6(100\text{ PSF}) = \boxed{0.318\text{ KSF}}$$

$l_n = 30' - 1' - 1' = 28'$

$l_2 = 2\left(\frac{30'}{2}\right) = 30'$

$l_1 = 30'$ (CONSERVATIVE)

$$M_o = \frac{(0.318)(30)(28)^2}{8} = \boxed{934.9^{\text{'K}}}$$

$$M_u^- = -0.65M_o = -0.65(934.9) = \boxed{-607.7^{\text{'K}}}$$

$$M_u^+ = 0.35M_o = 0.35(934.9) = \boxed{327.2^{\text{'K}}}$$

DISTRIBUTE TO COLUMN STRIP & MIDDLE STRIP

$\alpha_{f1} = 0$ (NO BEAMS) , $l_2/l_1 = 1$

NEGATIVE MOMENTS:

ACI 13.6.4.1

COLUMN STRIP $M^- = 0.75(-607.7) = -455.8^{\text{'K}}/15' = -30.4^{\text{'K}}/\text{FT}$

MIDDLE STRIP $M^- = (1-0.75)(-607.7) = -151.9^{\text{'K}}/15' = -10.1^{\text{'K}}/\text{FT}$

JAKE POLLACK TECH #2 TWO-WAY FLAT PLATE WITH COLUMN CAPITALS 3/6

POSITIVE MOMENTS:

ACI 13.6.4.4

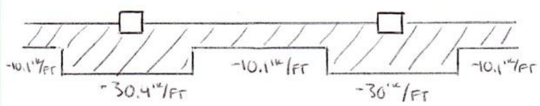
COLUMN STRIP $M^+ = 0.60(327.2) = 196.3^{'k}/15' = 13.1^{'k}/FT$

MIDDLE STRIP $M^+ = (1-0.60)(327.2) = 130.9^{'k}/15' = 8.7^{'k}/FT$

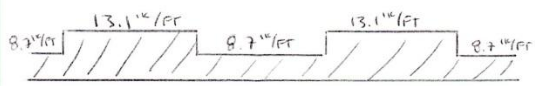
COLUMN STRIP SIZE

$$\ell_{MIN}/4 = \min \begin{cases} \ell_1/4 = 30'/4 = 7.5' \\ \ell_2/4 = 30'/4 = 7.5' \end{cases}$$

NEGATIVE MOMENTS



POSITIVE MOMENTS



COLUMN STRIP NEGATIVE REINFORCING

$A_s = \dfrac{M_u}{\phi f_y jd} \Rightarrow jd = d - a/2 \Rightarrow$ ASSUME $jd = 0.9d$ WHERE $d_{\ell_1} = h_{SLAB} - 1.9" = 8.1"$

$d_{\ell_2} = h_{SLAB} - 1.15" = 8.85"$

0.95d IS NOT SUFFICIENT USE 0.9d TO BE CONSERVATIVE

* BECAUSE SPANS ARE EQUAL USE d_{ℓ_1}, IT WILL YIELD A MORE CONSERVATIVE ANSWER

$$A_s = \frac{-30.4^{'k/FT}(12")}{0.9^2(60)(8.1)} = 0.93\ IN^2$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.93(60)}{0.85(4)(12)} = 1.37" \qquad c = \frac{a}{\beta_1} = \frac{1.37}{0.85} = 1.61"$$

$$\varepsilon_s = \frac{0.003}{c}(d-c) = \frac{0.003}{1.61}(8.1-1.61) = 0.012 > 0.005 \therefore \text{ TENSION CONTROLLED}$$

$$jd = d - a/2 = 8.1 - 1.37/2 = 7.42'' > 0.9d \therefore \text{ CONSERVATIVE}$$

ACI 13.3.1

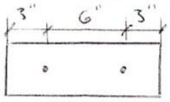
$$A_{s\,MIN} = 0.0018bh = 0.0018(1')(12)(10) = 0.216 < 0.93_{in^2} \therefore 0.93 \text{ CONTROLS}$$

ACI 13.3.2

$$S_{MAX} = 2h = 2(10) = 20'' > 18'' \Rightarrow \text{CONTROLS}$$

ACI 7.12.2.2

$$\frac{A_s}{A_{BAR}} = \frac{0.93}{0.60} = 1.55 \therefore \text{ PICK } \boxed{2\ \#7 \text{ SPACED @ } 6'' \text{ O.C.}}$$



$6'' < 18'' \therefore$ OK

NOTE: REINFORCING WORKS IN BOTH DIRECTIONS

$$\rho = \frac{A_s}{bd} = \frac{1.20}{12(8.1)} = 1.2\% < 4\% \therefore \text{ OK}$$

$$\phi M_n = \phi A_s f_y (d - a/2) = 0.9(1.20)(60)(8.1 - \tfrac{1.37}{2}) = \boxed{40.0^{1k}/FT} > 30.4^{1k}/FT \therefore \text{ OK}$$

MIDDLE STRIP NEGATIVE REINFORCEMENT

$A_s = 0.308_{in^2}$

$a = 0.453''$

$c = 0.533''$

$\varepsilon_s = 0.043 > 0.005 \therefore$ TENSION CONTROLLED

$jd = 7.87 > 0.9d \therefore$ OK

$A_{s\,MIN} = 0.216 < 0.308_{in^2}$ E CONTROLS

$S_{MAX} = 18''$

PICK $\boxed{1\ \#7 \text{ BAR SPACED @ } 12'' \text{ O.C.}}$

$12'' < 18'' \therefore$ OK

$\rho = .6\% < 4\% \therefore$ OK

$\boxed{\phi M_n = 21.3^{1k}/FT} > 10.1^{1k}/FT \therefore$ OK

JAKE POLLACK TECH #2 TWO-WAY FLAT PLATE WITH COLUMN CAPITAL 5/6

POSITIVE MOMENTS

COLUMN STRIP

$A_s = 0.399\,in^2$

$a = 0.588''$

$c = 0.691''$

$\varepsilon_s = 0.032 > 0.005$ ∴ TENSION CONTROLLED

$jd = 7.81 > 0.9d$ ∴ CONSERVATIVE

$A_{s\,MIN} = 0.216\,in^2 < \underline{0.399\,in^2}$ CONTROLS

$S_{MAX} = 18''$

$\boxed{\text{USE } 1\ \#7 @ 12'' \text{ O.C.}}$

$\rho = 0.6\%$ ∴ OK

$\boxed{\phi M_n = 21.3\ ^{'k}/_{FT}} > 13.1\ ^{'k}/_{FT}$ ∴ OK

MIDDLE STRIP

$A_s = 0.265\,in^2$

$a = 0.390\,in$

$c = 0.459''$

$\varepsilon_s = 0.050 > 0.005$ ∴ TENSION CONTROLLED

$jd = 7.51 > 0.9d$ ∴ CONSERVATIVE

$A_{s\,MIN} = 0.216\,in^2 < \underline{0.265\,in^2}$ CONTROLS

$S_{MAX} = 18''$

$\boxed{\text{USE } 1\ \#7 @ 12'' \text{ O.C.}}$

$\rho = 0.6\%$ ∴ OK

$\boxed{\phi M_n = 21.3\ ^{'k}/_{FT}} > 8.7\ ^{'k}/_{FT}$ ∴ OK

NOTE: #7 BARS ARE MORE THAN SUFFICIENT, HOWEVER USING ONE BAR SIZE FOR ALL ELIMINATES CONFUSION ON JOBSITE DURING CONSTRUCTION, IT IS EASIER FOR FABRICATION AND IT IS CONSERVATIVE.

DESIGN FOR SHEAR

ONE-WAY PUNCHING SHEAR

$d = 10 - 1.5" = 8.5"$

$V_u = q_u \cdot A = 0.318\left(\frac{30}{2} - \underbrace{\frac{24/2 + 8.5}{12"/\text{FT}}}_{c+d}\right)(30) = \boxed{126.8^k}$

$\phi V_n = \phi V_c = 0.75\,(2\lambda\sqrt{f'_c}\,b_o d)$

$= 0.75(2)(\sqrt{4000})(30 \times 12"/\text{FT})(8.5) = \boxed{290.3^k} > 126.8^k \therefore OK$

TWO-WAY PUNCHING SHEAR

$V_u = 0.318\left[30^2 - \left(\frac{24 + 2(8.5/2)}{12}\right)^2\right] = \underline{283.9^k}$

$\phi V_n = \phi V_c = 0.75(4\lambda\sqrt{f'_c}\,b_o d)$

$= 0.75(4)\sqrt{4000}(130)(8.5) = \underline{209.7^k}$

$\therefore$ NEED ADDITIONAL SHEAR STRENGTH ⇒ TRY COLUMN CAPITAL

$b_{o\,REQD} = \frac{V_u}{\phi 4\sqrt{f'_c}\,d} = \frac{283.9}{0.75(4)(\sqrt{4000})(8.5)} = 176"/4 = 44"$

$\boxed{\text{TRY } 48" \times 48" \times 2.25"}$ ⇒ NOTE: DIMENSIONS FOR EASIER FORMWORK, SAVINGS IN LABOR COST > LOSS IN MATERIAL COST

$d = 10 + 2.25 - 1.5 = 10.75"$

$V_u = 0.318\left[30^2 - \left(\frac{24 + 10.75}{12}\right)^2\right] = \boxed{283.5^k}$

$b_o = 4(10.75 + 24) = 139$

$\phi V_c = 0.75(4)(139)(10.75)\sqrt{4000}$

$= \boxed{284.0^k} > 283.5^k \therefore OK$

DEFLECTION

CONSTRUCTION Δ

$$W_u = \left(\frac{10}{12}\right)(30)(110) + 4\left(\frac{2.25}{12}\right)(110) = 2.83 \text{ KLF}$$

$$\Delta = \frac{5 w_u l_n^4 (1728)}{384 EI} = \frac{5(2.83)(30^4)(1728)}{384(2408)(30,000)} = 0.71'' < 1.0'' \therefore \text{OK}$$

$l/360$

$E_c = 33 W_c^{1.5}\sqrt{f'_c} = 2408$ ksi

$I = \frac{1}{12}bh^3 = 30,000 \text{ in}^4$ (CONSERVATIVE B/C CAPITALS NOT TAKEN INTO ACCOUNT)

LIVE Δ

$$\Delta = \frac{5(0.06(30))(30^4)(1728)}{384(2408)(30,000)} = 0.68'' < 1'' \therefore \text{OK}$$

$l/360$

APPENDIX E: ONE-WAY WIDE-MODULE JOIST SYSTEM CALCULATIONS

DESIGN DATA:

LIGHTWEIGHT CONCRETE (110 PCF)

3/4" MAX AGGREGATE

$f'_c = 4000$ PSI

REINFORCING STEEL:

GRADE 60 ($f_y = 60$ KSI)

USE 66"/6" WIDE-MODULE PAN SIZE

4 1/2" SLAB (2 HR FIRE RATING)

16" DEEP JOISTS (ACI TABLE 9.5a : $\ell/21 = 16"$

24" SQUARE COLUMNS

20.5" x 36" WIDE GIRDERS

LOADS: $LL = 100_{PSF} \times 6' =$ 0.600 KLF

$SDL = 40_{PSF} \times 6' =$ 0.240 KLF

$DL = \left[\dfrac{20.5 \times 6 + 4.5 \times 72}{144}\right] 110_{PCF} =$ 0.341 KLF

$W_u = 1.2(0.24 + 0.341) + 1.6(0.6)$

$= \boxed{1.66 \text{ KLF}}$

SLAB REINFORCING (NORMAL TO RIB)

$DL = \frac{4.5}{12}(110\,PCF) = 41.25\ PSF$

$W_u = 1.2(41.25 + 40) + 1.6(100) = 0.258\ KSF \times 1'$

$= 0.258\ KLF/FT$ OF SLAB

$M_{u\,MAX}^{-} = \frac{W_u l_n^2}{10} = \frac{0.258(66/12)^2}{10} = \boxed{0.78\ ^{1K}/FT \text{ OF SLAB}}$

$A_s = 0.0018(4.5)(12) = 0.097\ IN^2/FT$ OF SLAB

MINIMUM REINFORCING $\boxed{\text{USE \#3 @ 12" SPACING}}$

$A_s = 0.11\ IN^2/FT$ OF SLAB

$a = \frac{A_s f_y}{0.85 f'_c (b)} = \frac{0.11(60)}{0.85(4)(6)} = 0.32\ IN/FT$

$\phi M_n = 0.11(60)(2.25 - \frac{0.32}{2}) = \boxed{1.15\ ^{1K}/FT \text{ OF SLAB}}$

$> 0.78\ ^{1K}/FT \therefore OK$

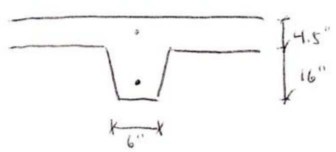
JAKE POLLACK TECH #2 ONE-WAY JOIST SYSTEM 3/6

USE ACI 8.3.3:

MAXIMUM PAN JOIST MOMENTS:

$$M^{+} = \frac{w_u l_n^2}{14} = \frac{1.66\,(\overbrace{27}^{30'-18''-18''})^2}{14} = 86.4^{\text{'k}}$$

$$M^{-} = \frac{w_u l_n^2}{10} = \frac{1.66\,(27)^2}{10} = 121.0^{\text{'k}}$$



$d = 20.5'' - 2.25''$
$= 18.25''$

TOP JOIST REINFORCING

$A_s = \frac{M_u}{4d} = \frac{121}{4(18.25)} = 1.66\,\text{in}^2 \Rightarrow$ TRY $\boxed{2\ \#9 \quad A_s = 2.00\,\text{in}^2}$

$\rho = \frac{A_s}{bd} = \frac{2.0}{6\,(18.25)} = 1.8\% < 4\%$ ∴ OK

$a = \frac{A_s f_y}{f'_c(b)(0.85)} = \frac{2.0(60)}{0.85(4)(6)} = 5.88'' \Rightarrow c = \frac{a}{\beta} = \frac{5.88}{0.85} = 6.92''$

$\varepsilon_t = \frac{0.003}{c}(d_t - c) = \frac{0.003}{6.92}(18.25 - 6.92) = 0.005$ ∴ TENSION CONTROLLED

$\phi M_n = \phi A_s f_y (d - a/2) = 0.9(2.0)(60)(18.25 - \frac{5.88}{2})$

$= \boxed{137.8^{\text{'k}}} > 121.0^{\text{'k}}$ ∴ OK

BOTTOM JOIST REINFORCING

$$A_s = \frac{M_u}{4d} = \frac{86.4}{4(18.25)} = 1.18\ in^2 \quad \boxed{\text{USE } 2\ \#9} \quad A_s = 2.0\ in^2$$

* NOTE 2 #8 BARS ARE SUFFICIENT, HOWEVER TO AVOID CONFUSION ON JOB SITE (#8 AND #9 BARS ARE SIMILAR IN APPEARENCE) #9 BARS ARE USED. ALSO EASIER FOR FABRICATION.

$$\boxed{\phi M_n = 137.8^{'k}} > 86.4^{'k} \therefore OK$$

SHEAR DESIGN

CONSERVATIVE

$$MAX\ V_u = \frac{1.15 w_u l_n}{2} = \frac{1.15(1.66)(30)}{2} = \boxed{28.6^k}$$

$$\phi V_c = \phi 2\sqrt{f'_c}\, b_w d = 0.75(2)\sqrt{4000}(6)(18.25) = \boxed{10.4^k}$$

$$\phi V_s = V_u - \phi V_c = 28.6 - 10.4 = \boxed{18.2^k}$$

$$\phi V_s = \frac{\phi A_s f_y d}{s_{max}} \Rightarrow S_{max} = d/2 = \frac{18.25}{2} = \boxed{9.1''\ \text{SPACING}}$$

$$= \frac{0.75(A_{s_v})(60)(18.25)}{9.1} = 0.20\ in^2 = A_{s_v}$$

$$\boxed{\text{USE } \#3 \text{ STIRRUPS @ } 3'' \text{ SPACING}}$$

$$\phi V_c + \phi V_s = 10.4 + \frac{0.75(0.11)(60)(18.25)}{3} = \boxed{40.5^k}$$

$$40.5^k > 28.6^k \therefore OK$$

GIRDER DESIGN

TRY 20.5" x 36" BEAM

LOADS: $W_{u\,\text{FROM FLOOR}} = 1.66\text{ KLF}/6' = 0.277\text{ KSF}$

$W_{u\,@\,\text{GIRDER}} = 3'(1.2\times 40 + 1.6\times 100) = 0.624\text{ KLF}$

$W_{u\,\text{GIRDER SELF}} = \dfrac{1.2(20.5\times 36)(110)}{144} = 0.677\text{ KLF}$

$\text{TOTAL} = 0.277\underbrace{(30)}_{\text{TRIBUTARY WIDTH}} + 0.624 + 0.554 = \boxed{9.53\text{ KLF}}$

TOP REINFORCING

$M_{u\,\text{MAX}}^{-} = \dfrac{W_u l_n^2}{10} = \dfrac{9.53(28)^2}{10} = \boxed{747.2^{\text{1k}}}$

$A_s = \dfrac{M_u}{4d} = \dfrac{747.2}{4(18.25)} = 10.24\text{ in}^2 \Rightarrow \boxed{\text{TRY } 11\ \#9}\ A_s = 11.0\text{ in}^2$

$a = \dfrac{A_s f_y}{0.85 f'_c b} = \dfrac{(11.0)(60)}{(0.85)(4)(36)} = 5.39'' \Rightarrow c = \dfrac{a}{\beta} = \dfrac{5.39}{0.85} = 6.34\text{ in}$

$\varepsilon_t = \dfrac{0.003}{c}(d_t - c) = \dfrac{0.003}{6.34}(18.25 - 6.34) = 0.0056 > 0.005 \therefore$ TENSION CONTROLLED

$\rho = \dfrac{A_s}{bd} = \dfrac{11.0}{(20.5)(36)} = 1.5\% < 4\% \therefore \text{OK}$

$\phi M_n = \phi A_s f_y (d - a/2) = 0.9(11.0)(60)(18.25 - \frac{5.39}{2})$

$= \boxed{770.0^{\text{1k}}} > 747.2^{\text{1k}} \therefore \text{OK}$

BOTTOM GIRDER REINFORCING

$$M_{u\,MAX}^{+} = \frac{w_u l_n^2}{14} = \frac{9.53(28^2)}{14} = \boxed{533.7'^k}$$

$$A_s = \frac{M_u}{4d} = \frac{533.7}{4(18.25)} = 7.31\text{ in}^2 \quad \boxed{\text{TRY } 8\ \#9} \quad A_s = 8.0\text{ in}^2$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{8.0(60)}{0.85(4)(36)} = 3.92\text{ in} \Rightarrow c = \frac{a}{\beta_1} = \frac{3.92}{0.85} = 4.61''$$

$$\varepsilon_t = \frac{0.003}{c}(d_t - c) = \frac{0.003}{4.61}(18.25 - 4.61) = 0.009 > 0.005 \therefore \text{TENSION CONTROLLED}$$

$$\rho = \frac{A_s}{bd} = \frac{8.0}{20.5(36)} = 1.1\% < 4\% \therefore \text{OK}$$

$$\phi M_n = \phi A_s f_y (d - a/2) = 0.9(8.0)(60)(18.25 - \tfrac{3.92}{2})$$
$$= \boxed{586.4'^k} > 533.7'^k \therefore \text{OK}$$

DEFLECTION

CONSTRUCTION Δ

$$w_u = \frac{4.5}{12}(110)(30) + \frac{6(20.5)}{144}(110)(5) + \frac{20.5 \times 36}{144}(110) = \boxed{2.27\text{ KLF}}$$

$$\Delta = \frac{5 w_u l_n^4 (1728)}{384\, E_c I} = \frac{5(2.27)(30^4)(1728)}{384(2408)(25845)} = 0.66'' < \frac{l}{360} \therefore \text{OK}$$

1''

$E = 33(110)^{1.5}\sqrt{4000} = 2408$ $I = \frac{1}{12}bh^3$

LIVE Δ

REDUCED LL

$$\Delta = \frac{5(0.06(30))(30^4)(1728)}{384(2408)(25845)} = 0.53'' < l/360 \therefore \text{OK}$$

APPENDIX F: COST ANALYSIS CALCULATIONS

JAKE POLLACK | TECH #2 | COST ANALYSIS

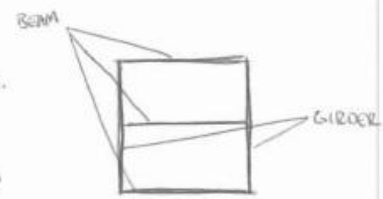
COST ANALYSIS USING RS MEANS 2011

LOCATION FACTOR: QUEENS, NY = 127.9

EXISTING SYSTEM: COMPOSITE BEAM, DECK & SLAB

p.94: $\$20.35/SF \times \frac{127.9}{100} = \boxed{\$26.03/SF}$

HOLLOW CORE SYSTEM:



$\left[\text{HOLLOW CORE} + \frac{\text{BM/GIRDER}}{2}\right] \times \text{LOC.}$

p.70 & 80: $\left[\$13.48/SF + \frac{\$23.70/SF}{2}\right] \times \frac{127.9}{100}$

$= \boxed{\$32.36/SF}$

NOTE: BM/GIRDER PRICE IS DIVIDED BY 2 BECAUSE BEAMS AND GIRDERS ARE TAKEN INTO ACCOUNT IN THIS COST WHILE ONLY THE COST OF GIRDERS IS DESIRED. TO BE CONSERVATIVE ASSUME GIRDERS ACCOUNT FOR 50% OF COST.

TWO-WAY FLAT SLAB SYSTEM:

p.63 $\$17.75/SF \times \frac{127.9}{100} = \boxed{\$22.70/SF}$ ⇐ CHEAPEST

NOTE: THE COST OF DROP PANELS WAS USED BECAUSE COLUMN CAPITALS WERE NOT LISTED. THIS IS CONSERVATIVE BECAUSE LESS CONCRETE/FORMWORK WILL ACTUALLY BE USED.

ONE-WAY JOIST SYSTEM

p.66 $\$19.65/SF \times \frac{127.9}{100} = \boxed{\$25.13/SF}$