Technical Report #2

John Jay College Expansion Project

New York, NY

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Structural Option

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Executive Summary

In the second technical report of the John Jay College Expansion Project alternative floor systems are investigated. A typical interior bay of 30'-0" x 25'-10" was analyzed and designed for four floor systems, including the existing, and were compared based on: self weight, total structural depth, constructability, impact on the existing architecture and steel structure, fire ratings, and cost. The existing floor system is composite steel and was chosen because of its light self weight and ability to span long distances. The three other systems that are studied in this report include:

- Two-Way Flat Slab with Drop Panels,
- Two-Way Post-Tensioned Slab, and
- Pre-Cast Hollow Core Planks on Steel Beams.

The design of a two-way flat slab floor system resulted in a 10" thick slab with 13" thick drop panels. This system is efficient for the typical bay analyzed in this report, but would not be economical for longer spans. Transferring gravity loads over Amtrak tracks beneath the building would be very complicated and expensive due to the self weight of this concrete floor system, and therefore it will not be studied beyond this report.

The goal of analyzing a post-tensioned two-way slab was to minimize the self weight of the floor system to reduce the gravity loads that must be transferred over the Amtrak tracks by maintaining a thin slab profile. After designing the post-tensioned concrete slab floor system, it was determined that a 7" slab was adequate to span 30'-0", but drop panels had to be incorporated to resist punching shear. This thin slab required a substantial amount of steel reinforcement at interior supports for ultimate strength requirements. Despite the uncertainty of transferring the heavy self weight of a concrete structure over the tracks, the post-tensioned system will be investigated further because it fits into the layout of the existing building.

30'-0" long pre-cast hollow core planks were sized according to Nitterhouse Concrete Products Hollow Core Plank Design Tables and were determined to be 10" thick. 2" of lightweight topping was added to the hollow core planks to ensure a level floor surface and to provide a rigid diaphragm for distributing lateral forces to the centralized braced frames. These planks are supported by W24x104 non-composite steel beams. Less structural steel is used in this floor system because there are no infill beams and less concrete is used as well, resulting in a lighter self weight of the structure. Efficient manufacturing and construction methods, as well as long span capabilities, make pre-cast hollow core planks on steel beams a viable option worth studying further.

Introduction

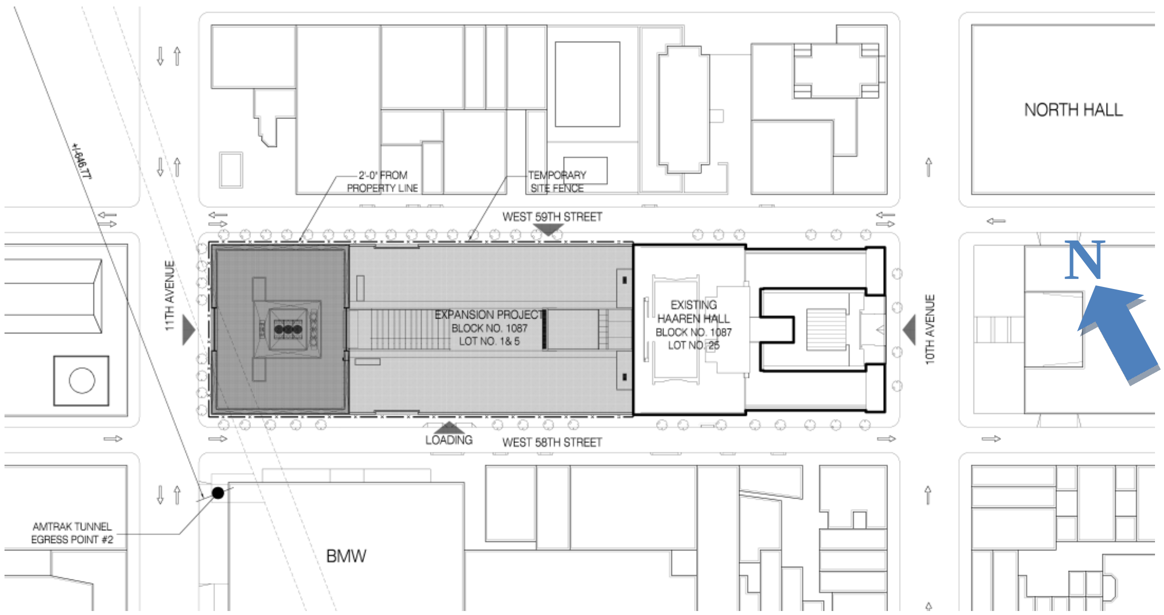


Figure 1 – Site plan

This major expansion project in Manhattan will unify the City University of New York's John Jay College of Criminal Justice into a one block campus that will "demonstrate the transparency of justice". The design includes a mid-rise tower situated on the west side of the site, which will contain classrooms, forensic laboratories, department offices, several student lounge spaces, a "moot" courtroom, a café, and a student bookstore.

A mid-rise structure connects the expansion to Haaren Hall (the existing building) and calls for a multi-level grand cascade, which also serves as a main lounge space for students (see picture 1 below). The connection also contains classrooms, a black box theater, and two cyber cafes. A landscaped roof accommodates outdoor lounge and dining areas, and an outdoor commons.

Picture 1 – Rendering of the Grand Cascade.

Amtrak tracks cross the south-west corner of the site, which is beneath the mid-rise tower. This restriction led to a unique structural solution to transfer over the tracks. Floors 1 through 5 are transferred over the tracks using built-up steel transfer girders and floors 6 through 14 are hanging from perimeter plate hangers supported at the penthouse level by transfer trusses that are one-story tall. These trusses then transfer the loads to a braced frame core.

The existing floor system of the John Jay College Expansion Project is a composite steel system with the most typical bay size being 30’-0”x37’-10”. This system was chosen to reduce the self weight of the structure to permit transferring gravity loads over the Amtrak tracks. Braced Frames wrap around a centralized service core in the 14 story tower and cascade.

The remainder of this report investigates the existing floor framing system, as well as three alternative solutions. All designs are considered schematic as the objective of this report is to study the various floor systems that can be applied to the expansion project. Several variables are taken into account when comparing floor systems such as: fire protection, weight, cost, overall structural depths, and constructability. All alternative floor systems will be designed and compared using a typical interior bay, as seen in figure 2.

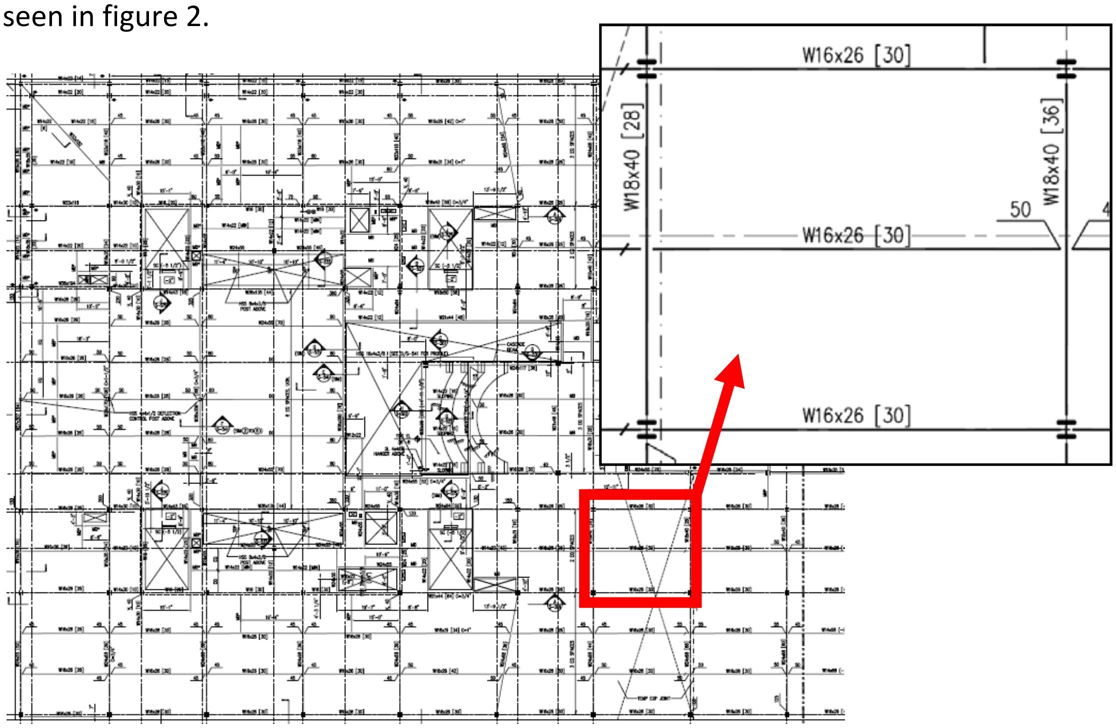


Figure 2 – Typical interior bay used to design the floor systems in this report.

Gravity Loads:

Dead loads used to design the floor systems in this report included the self weight of the floor system and the superimposed dead load of 37 psf determined in technical report one. This is a typical superimposed dead load for the building.

Typical live loads for the John Jay College Expansion Project are:

Assembly Areas: 60 psf
Office Spaces: 50 psf
Public Spaces: 100 psf

Live loads used to design the floor systems in this report were taken as 80 psf. This is an average of assembly areas and public areas. There are also several areas such as laboratories, cafeterias, and large corridors where live loads are not permitted to be reduced. Therefore, live loads will not be reduced in this report.

Design Codes and Reference Manuals:

Steel Construction Manual 13th edition, American Institute of Steel Construction

ACI 318-05, Building Code Requirements for Structural Concrete, American Concrete Institute

PCI design example provided by Dr. Ali Memari

Nitterhouse Concrete Products – Hollow core Plank Design Tables

Deflection Criteria:

Construction Dead Load deflection of beams and girders are limited to L/240

Live load deflections of beams < 60’ are limited to L/500 or ¾”, whichever is smaller

Live load deflections of beams ≥ 60’ are limited to L/500 or 1-3/8”, whichever is smaller

Live load deflections of beams supporting elevator sheave beams are limited to L/1666

Fire Protection and Fire Ratings:

The following table was taken from ACI 216.1 – 97 and was used to provide adequate clear cover for reinforced concrete slabs and post-tensioned/pre-stressed concrete slabs to meet a 1.5 to 2 hour fire rating.

Aggregate type	Cover[A,B] for corresponding fire resistance, in.					
	Restrained	Unrestrained				
	4 or less	1 hr	1 1/2 hr	2 hr	3 hr	4 hr
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

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

Fire ratings of the pre-cast hollow core planks were provided by the manufacturer, Nitterhouse Concrete Products. See appendix D for information regarding the pre-cast members.

It is also worth noting that all exposed structural steel members must be protected against fire and must meet the minimum requirements of Underwriters Laboratories. Fireproofing methods for protecting structural steel - such as spray on fireproofing, intumescent paint, and encasing members in gypsum board – will not be analyzed when comparing floor systems.

Existing Floor Framing System

The existing floor system for the John Jay College Expansion Project is a composite steel system (see figure 3) with a typical exterior bay of 30’-0”x 37’-10” and a typical interior bay of 25’-10”x30’-0”. 3” deep metal decking with a 3 ¼” thick lightweight concrete slab is used to span approximately 12 feet to typical W16x26 infill beams. Infill beams span into W24x68 girders or two back-to-back MC shapes. Previous calculations determined that the existing floor system has adequate capacity for gravity loads. See appendix E for the existing floor framing design of a typical interior bay.

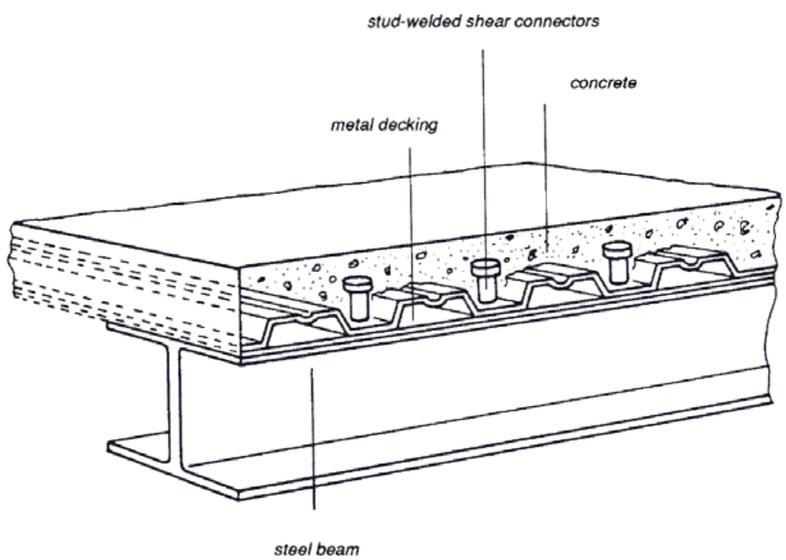


Figure 3 – Typical composite construction (www.epitech.com)

Pro-Con Analysis: Existing Composite Steel Floor System

After designing a typical interior bay, it was determined that the existing floor framing system is adequate to handle the heavy loading and long span requirements of the project. This system’s self weight is less than the other floor systems compared in the remainder of this report, which reduces the gravity loads transferred over the Amtrak tracks. A 2 hour fire rating is attained by providing a 3 ¼” lightweight concrete slab on 3” metal decking. 12 foot spans are achieved without using shoring, which simplifies construction. Steel erection is faster than forming, placing, and curing concrete and the metal decking acts as formwork for the concrete slab.

Although this system is light and efficient to construct, it reaches a depth of up to approximately 42 inches for a 68’-4” span. Steel beams must also be protected from fire with spray-on fireproofing or intumescent paint. This system is the most expensive floor system in this report due to the high material costs, but it is also lighter which reduces the costs of columns and foundations.

Overall, this is an excellent floor system for the John Jay College Expansion Project. It has enough capacity to resist heavy laboratory loads, the ability to span great distances, and it keeps the self weight of the structure relatively light to allow loads to be transferred over the train tracks under the building.

Alternative Floor Framing Systems

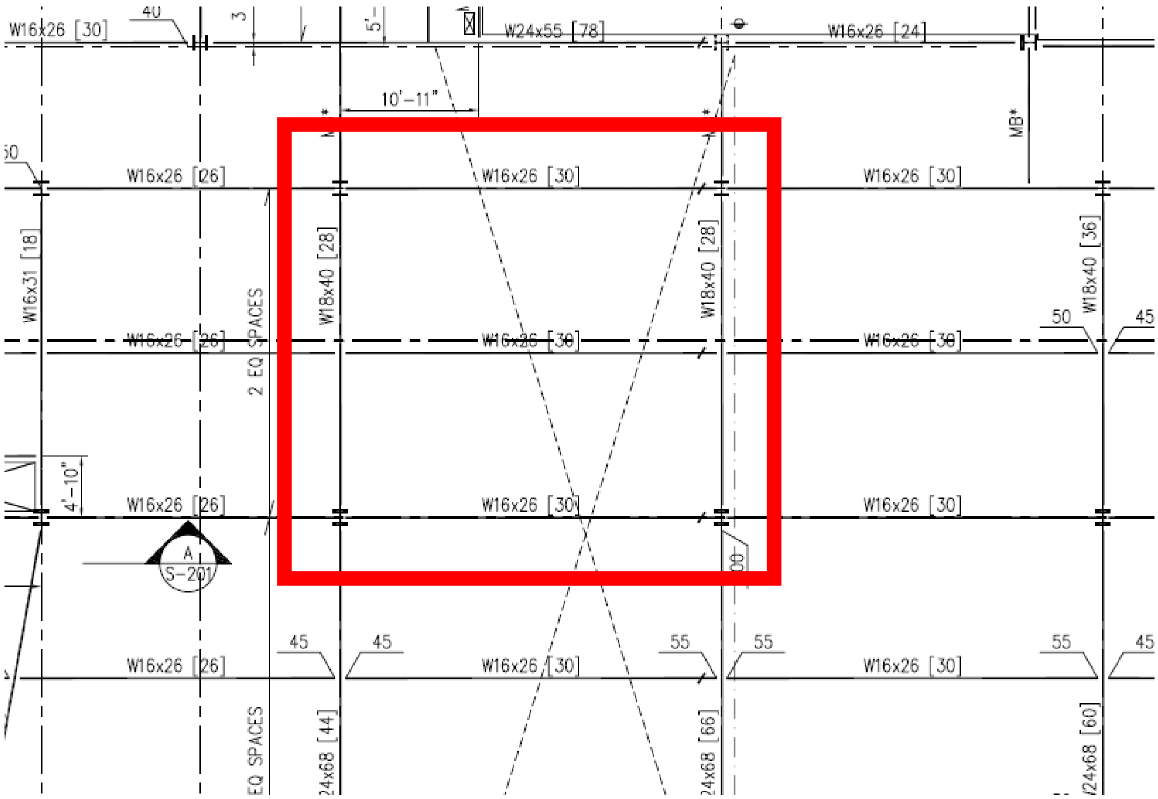


Figure 4 – Typical 30'-0" by 25'-10" interior bay used to design the alternative floor systems

Three additional alternative floor systems were designed and compared for the typical interior bay above in figure 4. The following floor systems were selected based on their span capabilities, structural depth, and effect on the existing building, both structurally and architecturally:

- Two-way reinforced concrete slab with drop panels,
- Two-way post-tensioned concrete slab, and
- Pre-cast hollow core plank on steel beams.

Two-Way Flat Slab System

This system uses a two-way reinforced concrete slab to transfer gravity loads directly to columns. A typical interior bay of 25'-10"x30'-0" was used to design the floor system. To keep the slab thickness economical, it is assumed that all spans in the building will be similar to the typical interior bay (the feasibility of this assumption will be investigated at a later date if the system is still under consideration). A 2 hour fire rating was attained by providing a minimum clear cover of ¾" with carbonate aggregate.

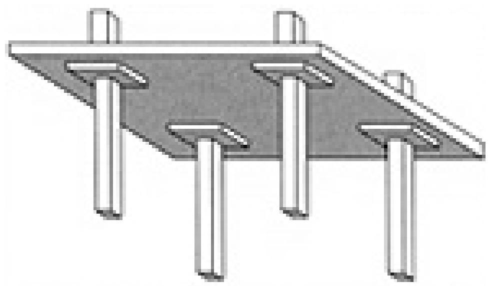


Figure 5 – Two-way flat slab with drop panels (www.crsi.com)

The original intent of this floor system was to avoid the use of drop panels. After using the direct design method it was determined that an 11 inch thick slab could be used, which unfortunately did not meet punching shear requirements. Therefore, drop panels were used to eliminate punching shear, which permitted the slab thickness to be reduced to 10". Total structural depths of this system are 13 inches at drop panels and 10 inches at mid-span. This does not include space for electrical equipment or mechanical ductwork.

Pro-Con Analysis: Two-Way Flat Slab Floor System

A two-way flat slab floor system works very well for the typical interior bay analyzed in this report. Even with drop panels added to prevent punching shear, the total structural depth is nearly half of the existing composite steel floor system. By incorporating drop panels, the slab thickness was slightly reduced, which also reduces required floor to floor heights.

Although this system is efficient for a typical interior bay of the John Jay College Expansion Project, complications arise when the entire structure is considered. A concrete floor system would need a different lateral force resistance system than the existing steel braced frames. The additional weight of the concrete system would also call for a massive transfer system over the Amtrak tracks beneath the building, which could be very expensive and difficult to incorporate into the architecture of the building. Labor costs are also high compared to the other systems analyzed in this report due to the heavy use of formwork and placing large quantities of concrete. Another area of concern is the bay spacing: a span of approximately 68 feet is required in the grand cascade and a flat slab could not efficiently be used. To incorporate this floor system, the architecture would have to be altered to incorporate more columns in long span areas creating similar bays to the one analyzed in this report.

Due to the increased self weight of the structure and the long span requirements for the John Jay College Expansion Project, a two-way flat slab floor system is not a very efficient solution.

Two-Way Post-Tensioned Floor System

This floor system utilizes a thin post-tensioned concrete slab. A typical interior bay was analyzed and designed for this section resulting in a 7 inch thick slab with (24) ½" diameter 270 ksi 7-wire strands in each direction. Minimum reinforcement was provided at midspan, while negative moment reinforcement at the supports was determined by strength requirements. This thin slab did not meet punching shear requirements due to the heavy loadings and therefore required 11" deep drop panels at the columns. Even with the thin floor slab, a 1.5 hour fire rating is still achieved by providing a 1 ½" clear cover at the bottom of unrestrained slabs. See appendix C for design assumptions and calculations.

Figure 6 – Two-way post-tensioned floor system (www.suncoast-pt.com)

Pro-Con Analysis: Two-Way Post-Tensioned Floor System

This system is very efficient when spanning great distances and carrying heavy loads. Thin floor slabs and minimal columns create open spaces which are attractive to the buildings tenants. Similar to the flat-slab floor system, the thin slabs allow smaller floor to floor heights which may lead to an additional floor without increasing the total building height. The self weight of this floor system is still greater than the existing system, but is lighter than the two-way flat slab system.

If this floor system would be implemented into the design of the John Jay College Expansion project, the lateral systems would need to be changed from the existing systems. Alternative transfer systems would also need to be studied to avoid placing columns near the Amtrak tracks. Construction for this system is very difficult and requires an experienced construction team. Most penetrations must be planned prior to construction to avoid coring through post-tensioning strands. This system is also expensive compared to other concrete floor systems and takes more time to construct, but with increased spans it has the ability to be more economical and efficient.

Overall, this system is a viable option due to the long span requirements of the John Jay College Expansion Project. A more comprehensive study of transfer systems over the Amtrak tracks must be performed to see if an efficient and economical transfer method can be achieved with the increased self weight of a concrete structure.

Pre-Cast Hollow Core Planks on Steel Beams

Pre-cast hollow core planks were studied for their ability to span long distances, while maintaining a light self weight. Hollow core planks were sized according to Nitterhouse Concrete Products (see appendix D for calculations). A 10" thick x 4' wide hollow core plank spanning 30'-0" was determined to be adequate for the heavy loading requirements of the public spaces and laboratories. 2" of lightweight concrete topping was assumed to be added to level floors from hollow core plank cambers, and also to create a rigid diaphragm for lateral loading. These planks also achieve a 2 hour fire rating without the need of additional fire proofing.

Steel beams were chosen because they are lighter than pre-cast concrete inverted tee or rectangular beams. They also allow the existing steel braced frame to still be utilized, as well as the hanging structure and transfer trusses. After designing the non-composite steel beams, it was found that a W24x104 was required for strength.

Figure 7 – Pre-cast hollow core planks on steel beams (www.spancrete.com)

Pro-Con Analysis: Pre-Cast Hollow Core Plank on Steel Beam Floor System

The main advantage of using the pre-cast hollow core plank system is that it is very efficient. Members are easily prefabricated in a pre-cast plant, which results in higher quality members and reduce on site construction time. Therefore, construction is simple any time of the year and under any weather conditions. Pre-cast planks already meet the required fire ratings for the job, and there is no need for additional fireproofing materials. Hollow core planks contain less material than traditional concrete slab floor systems, which is not only cheaper, but also environmentally friendly. By using steel beams to support the planks, the existing braced frames can still be used to resist lateral forces. Less steel is also needed, as there is no need for infill beams.

With a W24 beam, a 10” thick hollow core plank, and 2” of light weight concrete topping, the total floor depth is approximately 36” for spans of 30’. Not only is depth an issue with this system, but pre-cast planks must be ordered long in advance for a large project. Design consultants must also have excellent communication throughout the project to account for penetrations in the floor system. Steel beams need to be protected from fire with spray on fireproofing or intumescent paint, which can be expensive.

In conclusion, this floor system is a practical option due to its self weight, constructability, sustainability, and long span capabilities. Erection time is minimal, which is attractive for a project which is behind schedule, such as the John Jay College Expansion Project. It is also the cheapest floor system because it uses less material and construction is simple.

Conclusion

Criterion	Floor System Comparison - Typical Interior Bay			
	Existing Composite Steel	Two-Way Flat Slab	Two-Way Post Tensioned Slab	Pre-cast Hollow Core Planks on Steel Beams
Self Weight (psf)	55	130	94	94
Slab Depth (in)	6.25	10	7	12
Total Depth (in)	24.15	13	11	36.1
Constructibility	Medium	Medium	Hard	Easy
Foundation Impact	--	Major	Yes	Yes
Architectural Impact	--	Major	No	No
Transfer System Impact	--	Major	Major	Yes
Lateral System Impact	--	Yes	Yes	No
Vibration	Average	Best	Above Average	Average
Fire Rating (hr)	2	2	1.5	2
Total Cost per ft^2 ($)	19.00	16.50	18.53	13.08
Possible Alternative	--	No	Yes	Yes
Additional Study	--	No	Yes	Yes

Table 1 – Comparison of floor systems analyzed.

Note: Costs are not a direct indication of what each floor system would cost to construct in New York City, but are used to make general cost comparisons between floor systems.

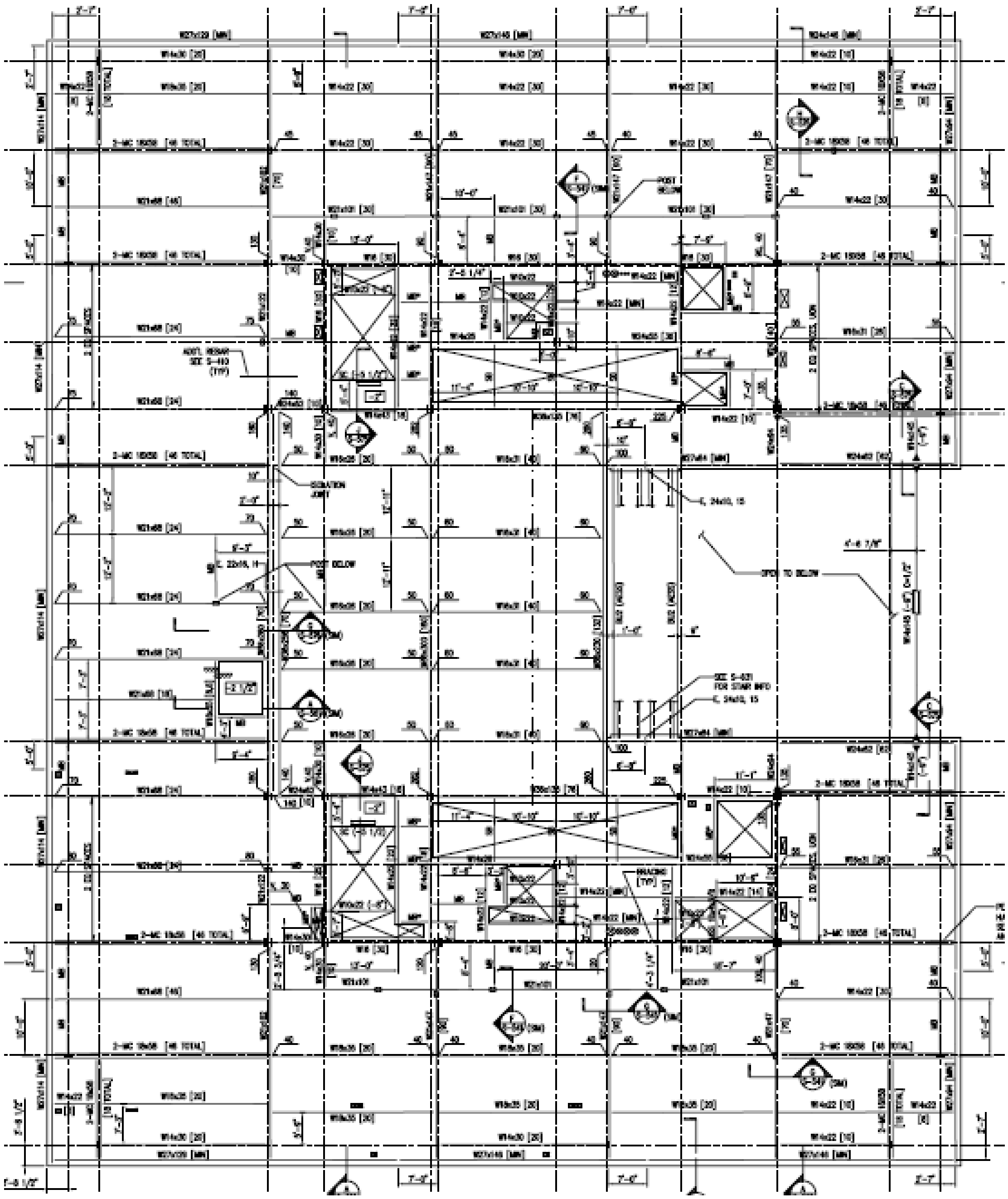
In the second technical report of the John Jay College Expansion Project, alternative floor systems were studied through schematic design of a typical interior bay. Self weight and long span capabilities were major factors when determining if a floor system was a viable option. The expansion project has a large middle bay of approximately 68 feet, which creates an open plan for the cascade. Not only must the floor system span great distances, but it also must have a light self weight to permit loads to transfer over the Amtrak tracks underneath the first level.

Due to these two main requirements, a two-way flat slab floor system cannot be used effectively. At 130 psf, it is by far the heaviest system analyzed and would need a massive transfer system that would not fit into the architecture of the building. Decreased bay sizes would need more interior columns, which affects the foundation and neglects the need for open space in the grand cascade. Despite the thin slab profile of this system, a two-way flat slab could not be implemented into the existing building.

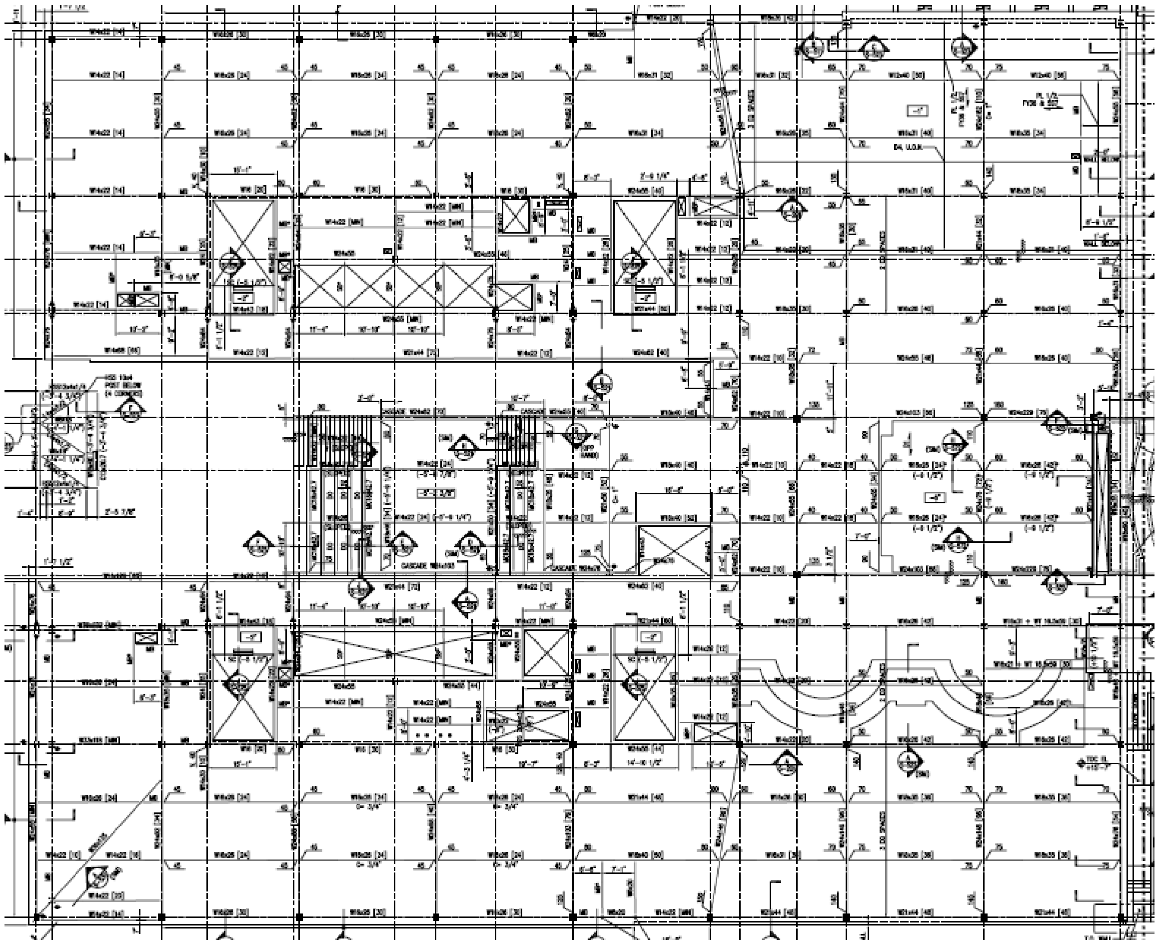
Although a post-tensioned two-way slab would require a different lateral force resistance system and is complicated to construct, it is still worth investigating because of its long span capabilities. By incorporating longer spans, this system has the capability to become more efficient and economical. An alternative transfer system would also need to be investigated, but it would be less massive than a system required for a non-post-tensioned concrete system due to a floor system self weight of 94 psf.

The most economical and constructible system in this study is the pre-cast hollow core planks on steel beams. This system is the cheapest because of the low labor costs associated with erecting the hollow core planks and steel beams. All structural members are fabricated off site, which allows for less material used and minimal construction time. A self weight of 94 psf would lead to increasing member sizes for the transfer systems, but this may still be economically feasible due to less steel members being used (no infill beams). This floor system also has the ability to utilize a braced frame to resist lateral forces and can span great distances. Therefore, hollow core planks on steel beams are worth future consideration.

Appendix A – Typical Framing Plans



Typical Tower Framing Plan



Typical Cascade Area Framing Plan

Appendix B – Two-Way Flat Slab with Drop Panels

TWO-WAY FLAT SLAB

ASSUMPTIONS:

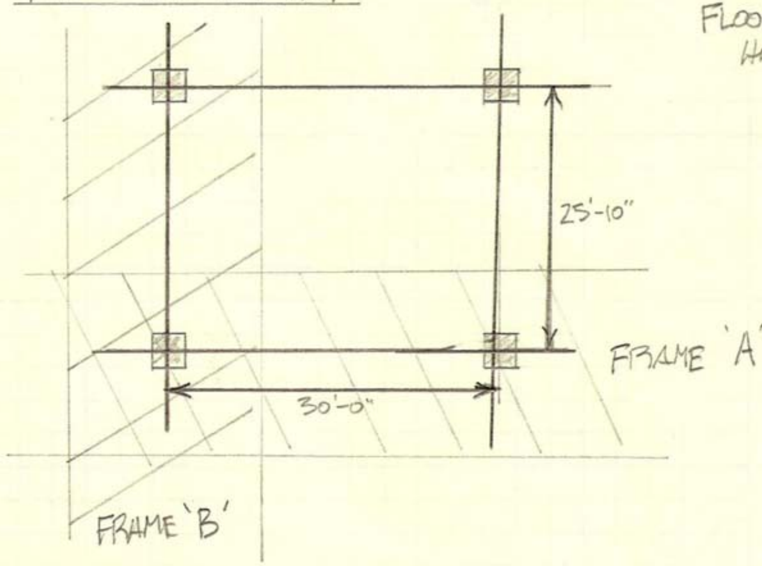
$f'_c = 5000$ PSI $f_y = 60{,}000$ PSI

LL = 80 PSF
SDL = 37 PSF

18"x18" COLUMNS

TYPICAL INTERIOR BAY

FLOOR TO FLOOR HEIGHTS: 15'-0"



DETERMINE SLAB THICKNESS

MIN. SLAB THICKNESS: $\frac{\ell_n}{33} = \frac{(30 \cdot 12 - 18)}{33} = 10.36" \Rightarrow$ TRY 11"

TABLE 9.5(c)

$W_U = 1.2\left(\frac{11}{12} \cdot 150 + 37\right) + 1.6(80) = 337$ PSF

$V_U = w_U \cdot AREA = 337\left[25.83' \times 30' - 1.5' \times 1.5'\right]$

$V_U = 260^K$

CHECK PUNCHING SHEAR:
ASSUME #5 BARS

$d_{AVG} = 11" - 0.75" - 0.625" = 9.625"$

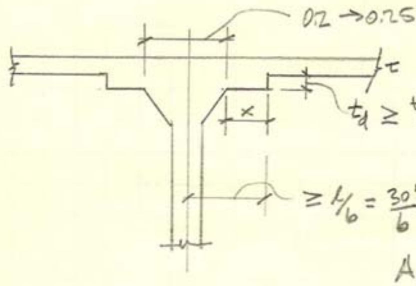
$b_o = (18" + 9.625")4 = 110.5"$

$$V_c = \min \begin{cases} 4\sqrt{f'_c}\, b_o d = 4\sqrt{5000}(110.5)9.625 = 300.8^K \\ \left(2+\frac{4}{\beta_c}\right)\sqrt{f'_c}\, b_o d = \left(2+\frac{4}{1}\right)\sqrt{5000}(110.5)9.625 = 451^K \\ \left(\frac{\alpha_s d}{b_o}+2\right)\sqrt{f'_c}\, b_o d = \left(\frac{40\cdot 9.625}{110.5}+2\right)\sqrt{5000}\cdot 110.5\cdot 9.625 = 412^K \end{cases}$$

$V_c = 300.8^K$ $\quad \phi V_c = 0.75 \times 300.8 = 225.6^K$ NO GOOD!

USE DROP PANELS $\quad t_{SLAB\,MIN} = \frac{30 \times 12''}{36} = 10''$ TABLE 9.5(c)

SIZE DROP PANEL



$x = 5' - 5.583'/2 = 2.20'$
$= 26.5''$

$t_d = 3''$

$A_{DROP} = 10' \times 10' = 100\ FT^2$

RECHECK PUNCH SHEAR AFTER DETERMINING REINF.

USE DIRECT DESIGN METHOD TO DISTRIBUTE MOMENTS

CALCULATION OF M_o: $\quad M_o = \frac{q_u}{8} l_2 l_n^2$

$q_u = 1.2\left(\frac{10}{12}\cdot 150 + 37\right) + 1.6(80) = 323$ PSF

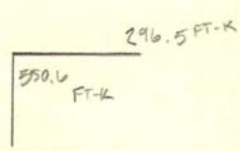
FRAME A: $\frac{0.323}{8}(25.83)(30-1.5)^2 = 847$ FT-K

FRAME B: $\frac{0.323}{8}(30)(25.83-1.5)^2 = 717$ FT-K

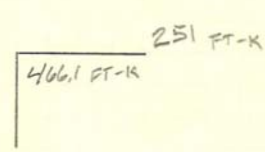
DISTRIBUTION OF M_o:

$-M = 0.65 M_o \qquad +M = 0.35 M_o \qquad$ 13.6.3.2

FRAME A:



FRAME B:



DISTRIBUTION TO COLUMN STRIP

FRAME A:

$-M_{INT} = 0.75(M) = 0.75 \cdot 550.6 =$ 413 FT-K TO C.S.

138 FT-K TO M.S.

$+M_{INT} = 0.60M = 0.60 \cdot 296.5 =$ 178 FT-K TO C.S.

119 FT-K TO M.S.

FRAME B:

$-M_{INT} = 0.75(M) = 0.75 \cdot 466.1 =$ 350 FT-K TO C.S.

117 FT-K TO M.S.

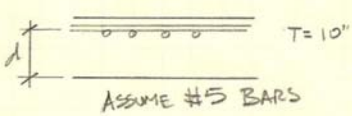
$+M_{INT} = 0.60(M) = 0.60 \cdot 251 =$ 151 FT-K TO C.S.

101 FT-K TO M.S.

SUMMARY OF MOMENTS

	FRAME A:		FRAME B:	
TOTAL M	550.6	296.5	466.1	251
CS M	413	178	350	151
MS M	138	119	117	101

ASSUME d



FRAME A: $d = 10" - 0.75" - 0.625"/2 = 8.93"$

FRAME B: $d = 8.93" - 0.625" = 8.3"$

DESIGN REINFORCEMENT FRAME A

DESCRIPTION	$-M_{CS}$	$-M_{MS}$	$+M_{CS}$	$+M_{MS}$	
MOMENT	-413	-138	178	119	
WIDTH, b	155"	155"	155"	155"	
EFF. d	8.93"	8.93"	8.93"	8.93"	
$M_n = M_u/\phi$	-459	-154	198	133	
$R = M_n/bd^2 \times 12000$	446	150	193	130	
TABLE A.5a $\rho =$	0.0079	0.0025	0.0033	0.0023	(FROM NILSON T.B.)
$A_s = \rho bd$	10.94	3.46	4.57	3.18	
$A_{S\,MIN} = 0.002bt$	3.1 ✓	3.1 ✓	3.1 ✓	3.1 ✓	
$N = A_s/A_{S\,BAR}$	25 (#6s)	12	15	11	#5 BARS
$N_{MIN} = \frac{W}{2t}$	7.5 ✓	7.5 ✓	7.5 ✓	7.5 ✓	

FRAME B

DESCRIPTION	$-M_{CS}$	$-M_{MS}$	$+M_{CS}$	$+M_{MS}$	
MOMENT	-350	-117	151	101	
WIDTH b	180"	180"	180"	180"	
EFF. d	8.3"	8.3"	8.3"	8.3"	
$M_n = M_u/\phi$	-389	-130	168	113	
$R = M_n/bd^2 \times 12{,}000$	378	126	163	110	
TABLE A.5a ρ =	0.0066	0.0021	0.0028	0.0019	(NILSON T.B.)
$A_s = \rho bd$	9.86	3.14	4.18	2.84	
$A_{s,MIN} = 0.002\, bt$	3.6 ✓	3.6	3.6 ✓	3.6	
$N = A_s / A_{s,BAR}$	23 (~~#6~~'s)	12	14	12	#5 BARS
$N_{MIN} = W/2t$	9 ✓	9 ✓	9 ✓	9 ✓	

CHECK PUNCH SHEAR @ DROP PANEL

$V_u = w_u A$

$V_u = 0.323(25.83' \times 30' - 2.47^2) = 248\ k$

ASSUMING #5 BARS

$d_{AVG} = 13'' - 0.75'' - 0.625'' = 11.625''$ $d/2 = 5.82$

$b_o = 4(18 + 11.625) = 118.5''$

① CONTROLS $V_c = 4\sqrt{5000}\,(118.5)\,11.625'' = 389^k$

$\phi V_c = 0.75(389^k) = 292^k$ $\phi V_c > V_u$ OK ✓

@ SLAB

$d = 10'' - 0.75'' - 0.625'' = 8.625''$

$V_u = w_u A$

$V_u = 0.323\left(25.83' \times 30' - \left(10' + \frac{8.625''}{12}\right)^2\right)$

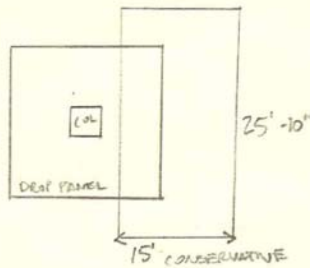
$V_u = 213^k$

$b_o = 4\left(10' + 8.625''/12''\right) = 42.875' = 514.5''$

③ CONTROLS $V_c = \left(\frac{40 \cdot 8.625}{514.5} + 2\right)\sqrt{5000} \cdot 514.5 \cdot 8.625 = 838^k$

$\phi V_c = 0.75(838) = 628^k$ $\phi V_c > V_u$ OK ✓

CHECK BEAM SHEAR



@ PANEL

$V_u = 0.323(25.83' \times 15') = 125.2^k$

$\phi V_c = 0.75(2)\sqrt{5000}\,(10\times12)\,11.625$

$\phi V_c = 148^k \qquad \phi V_c > V_u$ OK ✓

@ SLAB

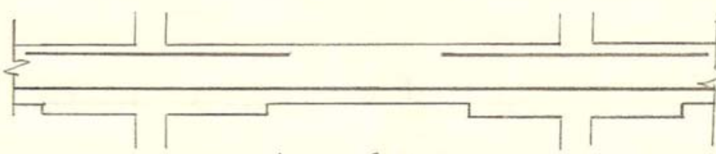
$V_u = 0.323(25.83' \times 10') = 83.5^k$

$\phi V_c = 0.75(2)\sqrt{5000}\,(25.83)12 \cdot 8.6 = 282^k$

$\phi V_c > V_u$ OK ✓

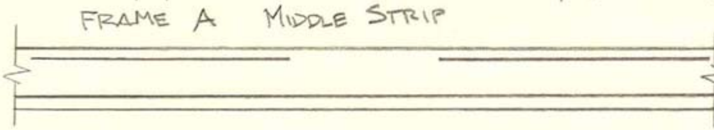
DESIGN SUMMARY

FRAME A COLUMN STRIP



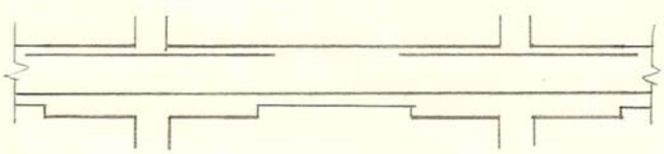
(-M) #6@6" o.c.
(+M) #5@10" o.c.

FRAME A MIDDLE STRIP



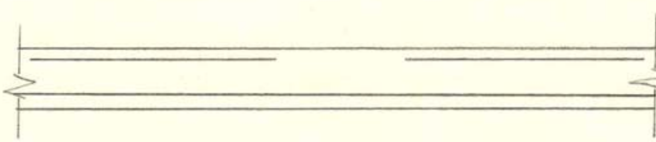
(-M) #5@12" o.c.
(+M) #5@12" o.c.

FRAME B COLUMN STRIP



(-M) #6@6" o.c.
(+M) #5@12" o.c.

FRAME B MIDDLE STRIP



(-M) #5@15" o.c. <18" OK ✓
(+M) #5@15" o.c. <18" OK ✓

MAX SPACING = 18" OR 3H = 30"

SYSTEM WEIGHT

$$S.W. = \frac{\frac{10}{12} \cdot 150 \cdot 30' \cdot 25.83 + \frac{5''}{12} \cdot 150 \cdot 10' \cdot 10'}{30' \cdot 25.83'}$$

$$S.W = 130 \text{ PSF}$$

Appendix C – Two-Way Post-Tensioned Floor System

Two-Way Post-Tensioned Design

Long Span:

L_1=	30	ft
L_2=	25.83	ft
f'c=	5000	psi
$f'c_i$=	3500	psi
fy=	60000	psi
SW=	87.5	psf
DL=	37	psf
LL=	80	psf
f_{pu}=	270	ksi
$A_{tendons}$=	0.153	in^2
Losses=	15	ksi
$f_{se}=0.7f_{pu}$-Losses=	174	ksi
$P_{eff}=Af_{se}$=	26.62	kips/tendon

Slab Thickness

$h=L_{avg}/45$=	7	in

Section Properties

A=bh=	2170	in^2
$S=bh^2/6$=	2531	in^3

Design Parameters

Allowable Stress: Class U

At time of jacking:

$f'c_i$=	3500	psi
$C=.60f'c_i$=	2100	psi
$T=3\text{sqrt}(f'c_i)$=	177.48	psi

At service loads:

f'c=	5000	psi
C=0.45f'c=	2250	psi

T=6sqrt(f'c)=	424.26	psi

Average Precompression Limits:

P/A=	125	psi min
	300	psi max

Target Load Balances:

0.75 SW=	65.63	psf

2 Hour Fire Rating:

Restrained Slabs:	0.75	in bottom
Unrestrained Slabs:	1.5	in bottom
	0.75	in top

Tendon Profile:

a_{int}=	5	in
a_{end}=	3	in

Pre-stress Force Required to Balance 75% of S.W.

$w_b=.75w_{DL}$=	1.70	klf

Force needed to counteract load in end bay:

$P=w_bL^2/8a_{end}$=	762.8	kips

Check Precompression Allowance:

# tendons=	24	tendons	(adjusted for allowable stresses)

Actual Force for Banded Tendons:

P_{act}=	638.9	kips

Adjust End Span Balanced Load:

w_b=	1.42	klf

Determine Actual Precompression Stress:

P_{act}/A=	294.5	psi	> 125 psi min	OK
			< 300 psi max	OK

Check Interior Span Force:

$P=w_bL^2/8a_{int}=$	457.7	k	< ext. span
$w_b=$	2.37	klf	
$w_b/w_{DL}=$	0.736		OK

Effective Prestress Force:

$P_{eff}=$ 638.9 kips

Check Slab Stresses:

Dead Load Moments:

$w_{DL}=$	3.22	klf
$M-=$	289.4	ft-k
$M+_{ext}=$	231.5	ft-k
$M+_{int}=$	72.4	ft-k

Live Load Moments:

$w_{LL}=$	2.07	klf
$M-=$	186.0	ft-k
$M+_{ext}=$	148.8	ft-k
$M+_{int}=$	46.5	ft-k

Total Balancing Moments:

$w_b=$	1.89	klf	(avg)
$M-=$	170.4	ft-k	
$M+_{ext}=$	136.3	ft-k	
$M+_{int}=$	42.6	ft-k	

Stage 1: Stresses Immediately after Jacking

Midpsan Stresses:

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

$f_{bot}=(M_{DL}-M_{BAL})/S - P/A$

Interior Span:

f_{top}=	-435.6	psi	OK	
f_{bot}=	-153.4	psi	OK	

End Span:

f_{top}=	-745.9	psi	OK	
f_{bot}=	156.9	psi	OK	

Support Stresses:

$f_{top}=(M_{DL}-M_{BAL})/S - P/A$

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

f_{top}=	269.9	psi	N.G.	NEED REINFORCEMENT
f_{bot}=	-858.8	psi	OK	

Stage 2: Stresses at Service Load

Midspan Stresses:

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

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

Interior Span:

f_{top}=	-656.0	psi	OK	
f_{bot}=	67.0	psi	OK	

End Span:

f_{top}=	-1451	psi	OK	
f_{bot}=	862.3	psi	N.G.	NEED REINFORCEMENT

Support Stresses:

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

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

f_{top}=	1151.5	psi	N.G.	NEED REINFORCEMENT
f_{bot}=	-1740	psi	OK	

Ultimate Strength:

M_1=Pe= 133.11 ft-k

M_{sec}=M_{BAL}-M_1= 37.3 ft-k at interior supports

M_u=1.2M_{DL}+1.6M_{LL}+1.0M_{SEC}

M_u= 534.5 ft-k midspan

M_u= -607.6 ft-k support

Minimum Bonded Reinforcement:

Positive Moment:

Exterior Span: Minimum positive moment reinforcement required

y=f_t/(f_t+f_c)h= 2.61 in

N_c=M_{DL+LL}/S*.5*y*L_2= 729 kips

As_{min}=N_c/.5fy= 24.30 in^2

Distribute reinforcement evenly across the width of the slab

As_{min}= 0.94 in^2/ft

Use #7 @ 6" O.C. Bottom = 1.2 in^2/ft OK

Minimum length 1/3 clear span

Negative Moment:

Interior Supports

A_{cf}= 2520 in^2

As_{min}=0.00075A_{cf}= 1.89 in^2

Use 10 #4 Top 2 in^2

Exterior Supports

A_{cf}= 2170 in^2

As_{min}=0.00075A_{cf}= 1.63 in^2

Use 9 #4 Top 1.8 in^2

Bars span minimum of 1/6 clear span each side of support

At least 4 bars in each direction

Max Bar Spacing= 10.5 in

Check if minimum reinforcement is sufficient for ultimate strength:

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

$d_{support}$= 6 in
$d_{midspan}$= 5.25 in

A_{ps}= 3.672 in^2
L/h= 51.4

At Supports:

$f_{ps}=f_{se}+10000+(f'cbd)/(300A_{ps})$= 192441 psi
$a=(Asfy+A_{ps}f_{ps})/(0.85f'cb)$= 0.628 in
ϕMn= 352.5 ft-k N.G.

As_{req}= 13.1 in^2

Provide #7 @ 12" OC As= 15.498 in^2

At Midspan:

$f_{ps}=f_{se}+10000+(f'cbd)/(300A_{ps})$= 191386 psi
$a=(Asfy+A_{ps}f_{ps})/(0.85f'cb)$= 1.640 in
ϕMn= 851.4 ft-k OK

Use Minimum Reinforcement

Short Span:

L_1=	25.83	ft
L_2=	30	ft
f'c=	5000	psi
$f'c_i$=	3500	psi
fy=	60000	psi
SW=	87.5	psf
DL=	37	psf
LL=	80	psf
f_{pu}=	270	ksi
$A_{tendons}$=	0.153	in^2
Losses=	15	ksi
f_{se}=0.7f_{pu}-Losses=	174	ksi
P_{eff}=Af_{se}=	26.62	kips/tendon

Slab Thickness

h=L_{avg}/45=	7	in

Section Properties

A=bh=	2520	in^2
S=bh^2/6=	2940	in^3

Design Parameters

Allowable Stress: Class U

At time of jacking:

$f'c_i$=	3500	psi
C=.60$f'c_i$=	2100	psi
T=3sqrt($f'c_i$)=	177.48	psi

At service loads:

f'c=	5000	psi
C=0.45f'c=	2250	psi
T=6sqrt(f'c)=	424.26	psi

Average Precompression Limits:

		psi min
P/A=	125	
	300	psi max

Target Load Balances:

0.75 SW=	65.63	psf

2 Hour Fire Rating:

Restrained Slabs:	0.75	in bottom
Unrestrained Slabs:	1.5	in bottom
	0.75	in top

Tendon Profile:

a_{int}=	5	in
a_{end}=	3	in

Pre-stress Force Required to Balance 75% of S.W.

$w_b=.75w_{DL}$=	1.97	klf

Force needed to counteract load in end bay:

$P=w_bL^2/8a_{end}$=	656.8	kips

Check Precompression Allowance:

# tendons=	24	tendons

Actual Force for Banded Tendons:

P_{act}=	638.9	kips

Adjust End Span Balanced Load:

w_b=	1.92	klf

Determine Actual Precompression Stress:

P_{act}/A=	253.5	psi	> 125 psi min	OK
			< 300 psi max	OK

Check Interior Span Force:

$P=w_bL^2/8a_{int}$=	394.1	k	< ext. span

w_b=	3.19	klf	
w_b/w_{DL}=	0.855		OK

Effective Prestress Force:

P_{eff}=	638.9	kips

Check Slab Stresses:

Dead Load Moments:

w_{DL}=	3.74	klf
M-=	249.2	ft-k
$M+_{ext}$=	199.4	ft-k
$M+_{int}$=	62.3	ft-k

Live Load Moments:

w_{LL}=	2.40	klf
M-=	160.1	ft-k
$M+_{ext}$=	128.1	ft-k
$M+_{int}$=	40.0	ft-k

Total Balancing Moments:

w_b=	2.55	klf	(avg)
M-=	170.4	ft-k	
$M+_{ext}$=	136.3	ft-k	
$M+_{int}$=	42.6	ft-k	

Stage 1: Stresses Immediately after Jacking

Midpsan Stresses:

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

$f_{bot}=(M_{DL}-M_{BAL})/S - P/A$

Interior Span:

f_{top}=	-334.0	psi	OK
f_{bot}=	-173.1	psi	OK

End Span:

f_{top}=	-510.89	psi	OK	
f_{bot}=	3.81	psi	OK	

Support Stresses:

f_{top}=(M_{DL}-M_{BAL})/S - P/A

f_{bot}=(-M_{DL}+M_{BAL})/S - P/A

f_{top}=	68.1	psi	OK	
f_{bot}=	-575.2	psi	OK	

Stage 2: Stresses at Service Load

Midspan Stresses:

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

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

Interior Span:

f_{top}=	-497.4	psi	OK	
f_{bot}=	-9.7	psi	OK	

End Span:

f_{top}=	-1033.8	psi	OK	
f_{bot}=	526.7	psi	N.G.	NEED REINFORCEMENT

Support Stresses:

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

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

f_{top}=	721.7	psi	N.G.	NEED REINFORCEMENT
f_{bot}=	-1228.8	psi	OK	

Ultimate Strength:

M_1=Pe=	133.11	ft-k
$M_{sec}=M_{BAL}-M_1$=	37.27	ft-k at interior supports

$M_u=1.2M_{DL}+1.6M_{LL}+1.0M_{SEC}$

M_u=	462.8	ft-k	midspan
M_u=	-518.0	ft-k	support

Minimum Bonded Reinforcement:

Positive Moment:

Exterior Span: Minimum positive moment reinforcement required

$y=f_t/(f_t+f_c)h$=	2.36	in
$N_c=M_{DL+LL}/S*.5*y*L_2$=	568	kips
$As_{min}=N_c/.5fy$=	18.95	in^2

Distribute reinforcement evenly across the width of the slab

As_{min}=	0.63	in^2/ft

Use #7 @ 10" O.C. Bottom = 0.72 in^2/ft OK
Minimum length 1/3 clear span

Negative Moment:

Interior Supports

A_{cf}=	2169.72	in^2
$As_{min}=0.00075A_{cf}$=	1.63	in^2

Use 10 #4 Top 2 in^2

Exterior Supports

A_{cf}=	2520	in^2
$As_{min}=0.00075A_{cf}$=	1.89	in^2

Use 10 #4 Top 2 in^2

Bars span minimum of 1/6 clear span each side of support
At least 4 bars in each direction

Max Bar Spacing= 10.5 in

Check if minimum reinforcement is sufficient for ultimate strength:

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

$d_{support}=$ 6 in
$d_{midspan}=$ 5.25 in

$A_{ps}=$ 3.672 in^2
L/h= 44.3

At Supports:

$f_{ps}=f_{se}+10000+(f'cbd)/(300A_{ps})=$	193804	psi	
$a=(Asfy+A_{ps}f_{ps})/(0.85f'cb)=$	0.544	in	
$\phi Mn=$	357.3	ft-k	N.G.

Asreq= 8.8 in^2

Provide #5 @ 12” As= 9.3 in^2

At Midspan:

$f_{ps}=f_{se}+10000+(f'cbd)/(300A_{ps})=$	192578	psi	
$a=(Asfy+A_{ps}f_{ps})/(0.85f'cb)=$	1.205	in	
$\phi Mn=$	698.2	ft-k	OK

Use Minimum Reinforcement

PUNCH SHEAR

$W_u = 1.2\left(\frac{7}{12} \cdot 150 + 37\right) + 1.6(80) = 277.4$

$V_u = W_u A = 277.4(25.83' \times 30' - 1.5^2)$

$V_u = 214.3^k$

$d = 5.75''$

$b_o = (18'' + 5.75'')4 = 95''$ $\quad b_o/d = 16.52 \quad \alpha_s = 40$ INT. COL

$V_c = 4\sqrt{5000} \cdot 95 \cdot 5.75 = 154.5^k$

$\phi V_c = 0.75 \cdot 154.5^k = 115.8^k$ NOT GOOD!

NEED DROP PANELS.

$V_u \leq \phi V_c$

$214.3^k = 0.75 \cdot 4\sqrt{5000} \cdot [4(18 + d)] \cdot d$

$d = 9.5'' \quad \phi V_c = 221^k$

NEED 10.75'' THICK DROP PANEL

USE 11''

BEAM SHEAR

@PANEL: $V_u = 0.277(25.83' \cdot 15') = 107.5^k$

$\phi V_c = 0.75 \cdot 2 \cdot \sqrt{5000} \cdot (10 \times 12) \cdot (9.75) = 124^K$

ASSUME 10' DROP PANEL

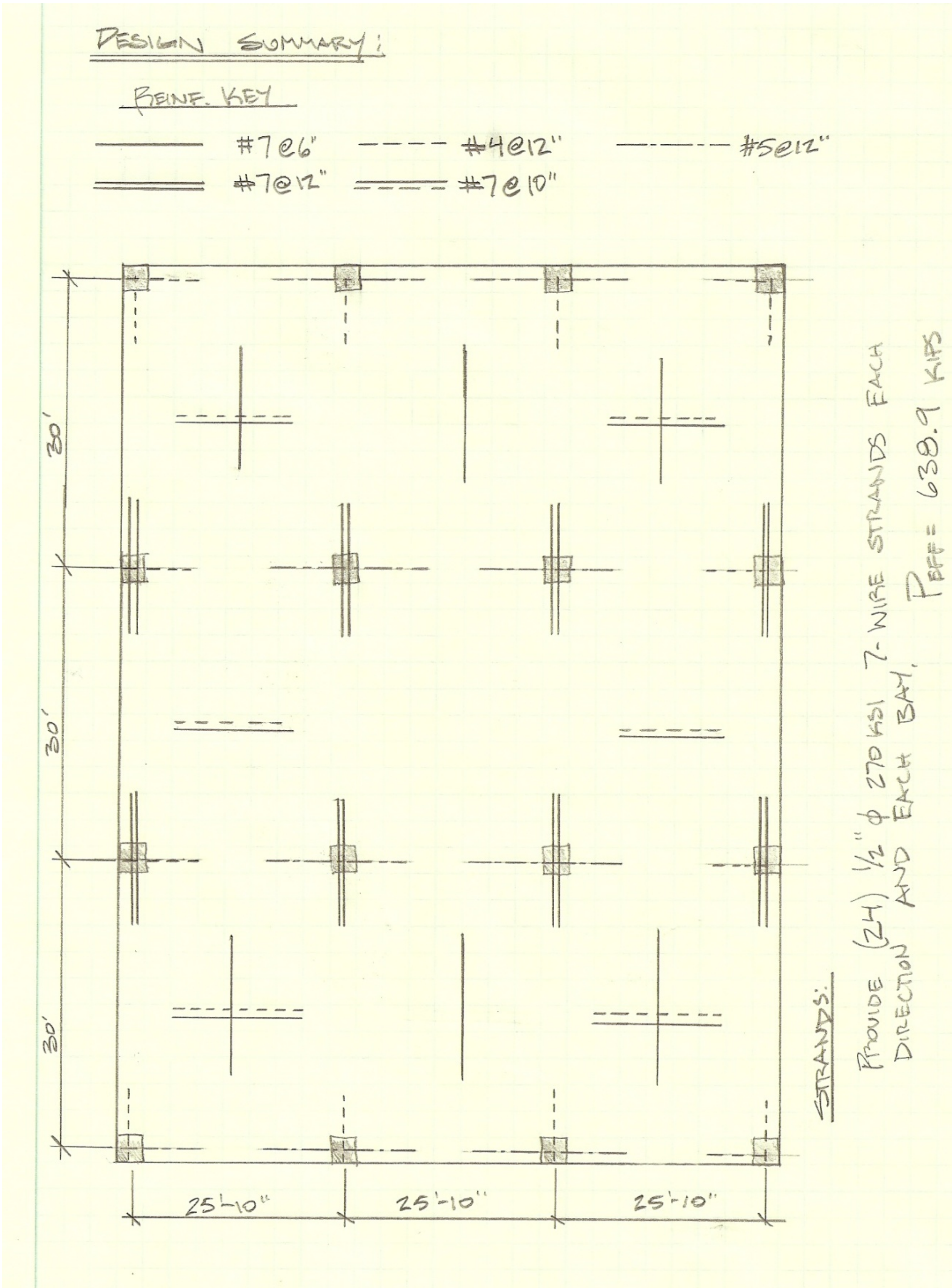
$\phi V_c > V_u$ OK ✓

@SLAB: $V_u = 83.5^K$

$\phi V_c = 0.75 \cdot 2 \cdot \sqrt{5000} \cdot (25.83 \cdot 12) \cdot 5.75 = 189^K$

$\phi V_c > V_u$ OK ✓

Technical Assignment #2



Technical Assignment #2

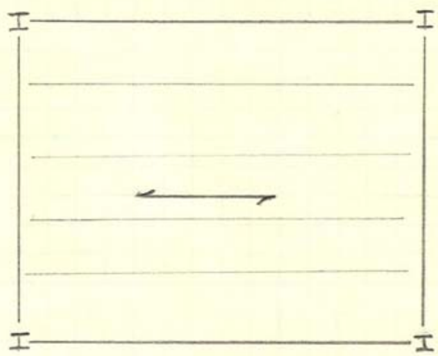
SYSTEM WEIGHT

$$S.W. = \frac{\frac{7}{12} \cdot 150 \cdot 30' \cdot 25.83' + \frac{4}{12} \cdot 150 \cdot 10' \cdot 10'}{30' \cdot 25.83'}$$

$$S.W. = 94 \text{ PSF}$$

Appendix D – Pre-Cast Hollow Core Planks on Steel Beam

PRE-CAST HOLLOW CORE PLANK ON STEEL BEAMS



TYPICAL INTERIOR BAY:

30'×25'-10"

f'_c = 6000 PSI
f_y = 50 KSI

LOADING:

DL = 37 PSF + S.W.
LL = 80 PSF

DESIGN PLANK

$W_u = 1.2(37) + 1.6(80) = 172.4$ PSF

TRY NITTERHOUSE 10"×4'-0" HOLLOW CORE TOPPED PLANK

- @ 30' SPAN ⟶ PROVIDES 233 PSF (CONTROLLED BY SHEAR)
- 2 HOUR FIRE RATING
- S.W. = 68 PSF + $\frac{2}{12} \cdot 115$ = 87.17 PSF
  ↳ L.W. TOPPING
- $W_u = 1.2(37 + \frac{2}{12} \cdot 115) + 1.6(80) = 195.4$ PSF < 233 PSF

STILL OK ✓

USE NITTERHOUSE 10"×4'-0" W/ (7) 0.6" ϕ STRANDS W/ 2" OF LIGHT WEIGHT CONC. 30' LENGTH.

ASSUME Δ IS W/IN CONC. DEFLECTION LIMITATIONS

STEEL BEAM DESIGN

UNBRACED LENGTH = 25.83'

$W_u = 1.2(37 + 19.17 + 68) + 1.6(80) = 277$ PSF

$W_{u_{DIST}} = 0.277$ KSF $\times 30' = 8.31$ KLF

$M_u = \frac{8.31(25.83)^2}{8} = 693$ FT-K $\qquad V_u = 108^K$

TRY W24×104 → MOST ECONOMICAL SHAPE

$\phi M_n = 747$ FT-K OK✓

$\phi V_n = 361^K$ OK✓

$\Delta_{LL} \leq \frac{L}{500} = 25.83 \cdot 12 / 500 = 0.62''$

$\Delta_{LL} = \frac{5}{384} \frac{(2.4) 25.83^4 \cdot 1728}{29000 \cdot 3100} = 0.27'' < 0.62''$ OK✓

$\Delta_{TL} \leq \frac{L}{240} = \frac{25.83 \cdot 12}{240} = 1.29''$

$\Delta_{TL} = \frac{5}{384} \cdot \frac{(2.4 + 3.725) 25.83^4 \cdot 1728}{29000 \cdot 3100} = 0.68'' < 1.29''$ OK✓

USE W24×104

$WT = \frac{104 \cdot 25.83 \cdot 2 + (68 + 19.17) 30 \cdot 25.83}{30 \cdot 25.83} = 94$ PSF

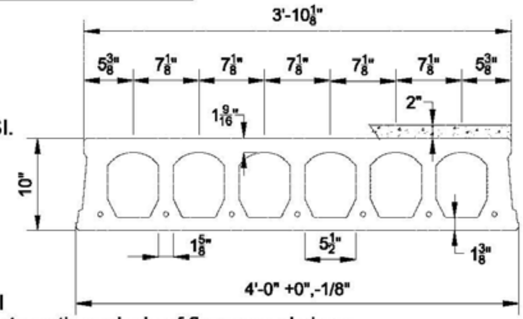
75 PSF
w/ NO TOPPING.

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 S_{bc} = 824 in.^{3}
I_c = 5102 in.^{4} — Topping S_{tc} = 1242 in.^{3}
Y_{bc} = 6.19 in. — Precast S_{tc} = 1340 in.^{3}
Y_{tc} = 3.81 in. — Wt.= 272 PLF
Wt.= 68.00 PSF



DESIGN DATA

1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI or 4000 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)...
   7-1/2"Ø, 270K = 192.2 k-ft
   7-0.6"Ø, 270K = 256.4 k-ft
7. Maximum bottom tensile stress is $7.5\sqrt{f'c}$ = 580 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. 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 2003 & ACI 318-02 (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
7 - 1/2"ø	LOAD (PSF)	234	210	189	170	153	137	123	110	98	87	77	68	60	52					
7 - 0.6"ø	LOAD (PSF)			256	24[illegible]	233	222	202	185	168	154	140	128	116	106	96	87	78	70	63

NITTERHOUSE
CONCRETE PRODUCTS

2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17201-0813
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.

05/14/07

10F2.0T

Appendix E – Existing Composite Floor System

Metal Decking:

It was determined from the structural design criteria, general notes for each floor, and in the specifications that the metal decking chose by the structural engineer is 3" deep, with a 40 ksi minimum yield strength, and a minimum thickness of 20 gage. The following table was taken from United Steel Deck for a 3" deep deck with a yield strength of 40 ksi:

slab depth	wc psf	Sc in^3	φVt lbs.	Ac in^2	Iav in^4	Max Unshored Spans, ft. 1 span	2 spans	3 spans	WWF
5.50	38	1.50	5249	37.6	8.1	10.25	12.79	13.22	0.023
6.00	43	1.73	5866	42.0	10.4	9.78	12.28	12.68	0.027
6.25	46	1.84	6183	44.3	11.6	9.56	12.04	12.44	0.029
6.50	48	1.96	6506	46.6	13.0	9.36	11.82	12.21	0.032
7.00	53	2.21	7125	51.3	16.1	9.00	11.41	11.78	0.036
7.25	55	2.33	7295	53.8	17.7	8.84	11.21	11.59	0.038
7.50	58	2.46	7468	56.3	19.6	8.68	11.03	11.40	0.041
8.00	62	2.71	7823	61.3	23.5	8.44	10.69	11.05	0.045

The following table was also taken from United Steel deck and displays the maximum service live load per square foot of metal decking:

Stud Spacing	Slab Depth	φMn in.k	Superimposed Live Load, psf Spans, ft. 9.0	9.5	10.0	10.5	11.0	11.5	12.0	12.5	13.0	13.5	14.0	14.5	15.0
	5.50	74.69	355	315	280	250	225	205	185	170	155	140	130	115	105
	6.00	85.06	400	360	320	290	260	235	210	195	175	160	145	135	125
	6.25	90.25	400	380	340	305	275	250	225	205	185	170	155	145	130
ONE	6.50	95.43	400	400	360	325	290	265	240	215	200	180	165	150	140
FOOT	7.00	105.80	400	400	400	360	325	290	265	240	220	200	185	170	155
	7.25	110.99	400	400	400	375	340	305	280	255	230	210	195	175	165
	7.50	116.17	400	400	400	395	355	320	290	265	240	220	200	185	170
	8.00	126.54	400	400	400	400	385	350	320	290	265	240	220	205	185

CHECK METAL DECKING

FROM STRUCTURAL DESIGN CRITERIA:

- 3 1/2" L.W. CONC. SLAB
- 3" DECKING
- 40 KSI YIELD STRENGTH
- MINIMUM 20 GAGE

FROM UNITED STEEL DECK WEBSITE:

3" LOK, 20 GAGE, 115 PCF CONCRETE

MAXIMUM UNSHORED SPAN:

$\left.\begin{array}{l}6\tfrac{1}{2}\text{" SLAB}\\ 3\text{ SPANS}\end{array}\right\}$ 12.21'

SPAN BETWEEN BEAMS: (USE W16x26'S ON C.D.'S)

$12.61' - 5.5"/12" = 12.15'$ OK ✓

CHECK SUPERIMPOSED LL:

$\left.\begin{array}{l}\text{STUD SPACING} = 1'\\ \text{SPAN} = 12.5'\\ 6\tfrac{1}{2}\text{" SLAB}\end{array}\right\}$ 215 PSF

TYPICAL FLOOR LOADING:

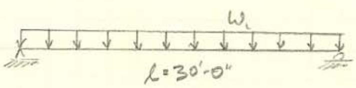
LL = 100 PSF
CDL = 51 PSF
SDL = 37 PSF

TOTAL SERVICE LOAD = 188 PSF OK ✓

USE 3" LOK-FLOOR, 20 GAGE MINIMUM
w/ FY = 40 KSI

BEAM CHECK

CLASSROOM LOADING.



TRIB WIDTH = 13'-0"

w_1

CDL = 51 PSF × 13'-0" = 0.663 KLF
SDL = 37 PSF × 13'-0" = 0.481 KLF
LL = 40 PSF × 13'-0" = 0.520 KLF

FACTORED LOADS

$W_u = 1.2(0.663 + 0.481) + 1.6(0.520) = 2.20$ KLF

DESIGN

$M_u = \frac{2.20(30^2)}{8} = 248$ FT-K

$V_u = 2.20 \times \frac{30}{2} = 33^K$

ASSUME $a = 1''$

$Y_2 = 6\frac{1}{2}'' - \frac{1}{2} = 6''$

TRY W16×26 $\phi M_n = 252'^K$ PNA #7

$b_{eff} = \left| \begin{array}{l} 13' \\ 30'/4 = 7.5' \end{array} \right| = 7.5' = 90''$

$a = \frac{96}{0.85(4)90} = 0.313 < 1''$ ASSUMPTION OK ✓

$\Sigma Q_n = 96^K$

SHEAR STUDS $= \frac{96^K}{17.2^K/\text{STUD}} = 6$ STUDS ×2 = 12 STUDS

CHECK DEFLECTIONS:

Δ CONSTRUCTION $I = 301\ \text{IN}^4$

$\Delta_{CDL} = \frac{L}{240}$

$\Delta = \frac{5}{384} \frac{(0.663)\,30^4 \times 1728}{29000 \times 301} = 1.38'' = \frac{L}{260} < \frac{L}{240}$ OK✓

Δ_{LL} $I = 595$ IN^4

$\Delta = \frac{5}{384} \frac{(0.520)\,30^4 \times 1728}{29000 \times 595} = 0.55'' = \frac{L}{655} < \frac{L}{500}$ OK✓

USE W16×26 [12]

$\phi M_n = 252'^k > M_u = 242'^k$ OK✓

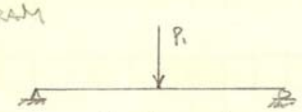
$\phi V_n = 106^k > V_u = 33^k$ OK✓

Δ_{CDL} OK✓

Δ_{LL} OK✓

GIRDER CHECK

LOADING DIAGRAM



$l = 25.83'$

P_1 (FROM BEAM CHECK)

$CDL = 0.633\text{ KLF} \times 15' + 0.035\text{ KLF} \times 15' = 10.5^k$

$SDL = 0.481 \quad \times 15' = 7.22^k$

$LL = 0.520 \quad \times 15' = 7.80^k$

FACTORED LOAD

$$P_{1u} = 1.2(10.5 + 7.22) + 1.6(7.8) = 33.75^k$$

DESIGN

$$M_u = \frac{33.75(25.83)}{4} = 218\text{ FT-K} \cdot 2 = 436\text{ FT-K}$$

$$V_u = 33.75^k$$

Δ_{CDL}

$$\Delta_c = \frac{L}{240}$$

$$\frac{25.83 \cdot 12}{240} = \frac{(2 \cdot 10.5)(25.83^3)\,1728}{48 \cdot 29000 \cdot I_{REQ'D}}$$

$$I_{REQ'D} = 348\text{ IN}^4$$

TRY W18x35 $\quad I = 510\text{ IN}^4$ OK Δ_{CDL} ✓

ASSUME $a = 1"$ $\quad Y_2 = 6\tfrac{1}{2}" - 1"/2 = 6"$ $\quad$ PNA #4 $\quad \phi M_n = 485'^k$

$$b_{eff} = \left| \begin{matrix} 30' \\ 25.83'/4 \end{matrix} \right| = 77.5'$$

$\Sigma Q_n = 323^k$

$$a = \frac{323}{0.85 \cdot 4 \cdot 77.5} = 1.22" \quad \text{ASSUMPTION NOT VALID!}$$

ASSUME $a = 2"$ $\quad Y_2 = 6\tfrac{1}{2}" - 2"/2 = 5.5"$ $\quad$ PNA #4 $\quad \phi M_n = 473'^k$

$\Sigma Q_n = 323 \quad a = 1.22"$ ASSUMPTION OK✓

$$\# STUDS = (323''/21.2) \cdot 2 = 32 \text{ STUDS}$$

CHECK Δ_{LL}

$$\Delta_{LL} = \frac{(2 \cdot 7.8)\, 25.83^3 \cdot 1728}{48 \cdot 29000 \cdot 1330} = 0.25'' = \frac{L}{1235} < \frac{L}{500} \text{ OK}\checkmark$$

USE W18x35 [32]

EXISTING SYSTEM SELF WEIGHT

3½" LIGHTWEIGHT CONC. SLAB ON 3" METAL DECKING.

$$SW = \left(\frac{3\tfrac{1}{2}'' + 3''/2}{12}\right) \cdot 115 \text{ PCF} + 3 \text{ PSF} = 51 \text{ PSF}$$

BEAMS:

$$SW = \frac{26 \text{ PLF} \times 30' \times 9 \text{ BEAMS} + 35 \text{ PLF} \times 25.83' \times 4 \text{ GIRDERS}}{(25.83 \cdot 2) \cdot (30 \cdot 2)}$$

$$SW = 3.43 \text{ PSF}$$

TOTAL: 55 PSF