John Hopkins Graduate Student Housing

Technical Report 2

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10/19/2011

Table of Contents

Executive Summary -

Technical report 2 aims at selecting and analyzing viable floor systems for the John Hopkins Graduate Student Housing project. Floor systems chosen for the pro-con analysis were:

- 2-way, post-tensioned slab (existing)
- Precast hollow core planks
- Composite steel beam and concrete slab
- 2-way flat plate

Each floor system was analyzed in the same corridor or bay to allow for an equal comparison. They were all analyzed under the load case 1.2D + 1.6LL as well. Computer and hand calculation were used to design and check preliminary member sizes for strength and deflection criteria. Each system was then compared to one another based on structural and non-structural criteria such cost, weight, architectural impact, construction impact, vibrations, and others.

Throughout the report is a detailed summary of each system including the design summary. A table summarizing the findings as well as an in-depth feasibility study can be found near the end of the report. The end conclusions showed that the precast and composite steel systems were the most viable. A 2-way flat plate in the John Hopkins Graduate Student Housing project was not economical nor provided sufficient advantages.

All images in this report are provided by Education Realty trust and Marks, Thomas Architects unless otherwise noted in the reference section.

Introduction –

Located just outside the heart of Baltimore, 2 blocks from John Hopkins campus, is the site for the new John Hopkins Graduate Student Housing. This housing project is being constructed in the science and technology park of John Hopkins. A developing "neighborhood", the science and technology park is over 277,000 sq. ft. which is planned to host at least five more buildings dedicated to research for John Hopkins University. The site is also directly across from a 3 acre green space. This location is ideal because it places graduate students within walking distance of the schools hospitals, shopping, dining and relaxing.

Figure 1 - Showing glass and brick facade along with curtain wall

John Hopkins Graduate Student Housing project is a new building constructed with brick and glass facades for a modern look. Upon completion, the building's main function is predominantly for graduate residential use, providing 929 bedrooms over 20 floors. There are efficiencies, 1, 2, and 4 bedroom apartments available. Other features include a fitness room and rooftop terrace. A secondary function of the building is three separate commercial spaces located on the first floor. Retail spaces provide a mixed use floor, creating a welcoming environment and bringing in additional revenue. At the 10^{th} floor, the typical floor size decreases, creating a low roof and a tower for the remaining ten floors. Glass curtain walls on two corners of the building also begin on the 10^{th} floor and extend to the upper roof.

The façade of John Hopkins GSH is composed mainly of red brick and tempered glass with metal cladding. Large storefront windows will be located on the first floor and approximately 6' x 6' windows in the apartments. The curtain wall is to be constructed of glass and metal cladding that can withstand wind loads without damage. There is a mechanical shading system in the windows to assist in the LEED silver certification.

Figure 2 - an overhead showing the green roof and large green area across the street

John Hopkins GSH is striving to achieve LEED silver certification. Most of the points accumulated to achieve this level come from the sustainable sites category. A total of 20/26 points were picked up in this category due to a number of achievements such as; community connectivity, public transportation access, and storm water design and quality control. Indoor air quality is the next largest category where the building picks up an additional 11 points for the use of low emitting materials throughout construction. Several miscellaneous points are picked up for using local materials and recycling efforts as well. Shading mechanisms are also implemented throughout the design as well as an accessible green roof.

There are three different types of roofs on this project. Above the concrete slab on the green roof is a hot rubberized waterproofing followed by polystyrene insulation, a composite sheet drying system, and finally the shrubbery. The sections of roof containing pavers will be constructed using the same waterproofing, a separation sheet, the insulation and finally pavers placed on a shim system. The remaining portions of the roof will be constructed using a TPO membrane system.

Structural Systems –

Foundations:

A geotechnical report was created based on 7 soil test borings drilled from 80' to 115' deep. Four soil types were found during these tests: man placed fill from previous construction 7-13 feet deep, Potomac group deposits of silty sands at 40-75 feet, and competent bedrock at 80-105 feet. Soil tests showed a maximum unconfined compressive strength of 12.37 ksi. The expected compression loads from the structure were 2400k and 1100k for the 20 and 9 floor towers respectively. The foundation system will also have to support an expected uplift and shear force of 1400k per column and 180k per column. Based on preexisting soils and heavy axial loads it was determined that a shallow foundation system was neither suitable nor economical.

In order to reach the competent bedrock, John Hopkins GSH sits on deep caissons 71-91 feet deep. Caissons range in 36-54" in diameter and are composed of 4000psi concrete. Grade beams, 4000psi, sit on top of the caissons followed by the slab on grade. Slab on grade consists of 3500 psi reinforced with W2.9XW2.9 and rests on 6" of granular fill compacted to at least 95% of maximum dry density based on standard proctor.

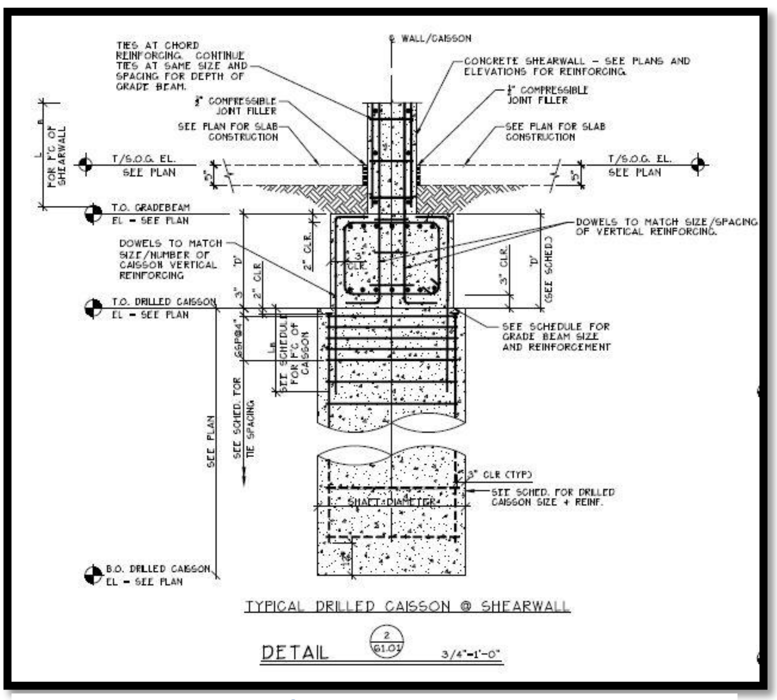


Figure 3 - a detail section of a caisson and column

According to the geotechnical report, the water table is approximately 10 feet below the first floor elevation, therefore a sub drainage system was not necessary.

Floor Framing:

Dead and live loads are supported in John Hopkins GSH through a 2 way post-tensioned slab. The slab is typically 8” thick normal weight 5000 psi concrete reinforced with #4 bars at 24” on center along the bottom in both directions. The tendons are low relaxation composed of a 7 wire strand according to ASTM A-416. Effective post tensioning forces vary throughout the floor, but the interior bands are typically 240k and 260k. This system is typical for every floor except for the 9th which supports a green roof and accessible terrace. Higher loads on this floor require a 10” thick 2 way post tensioned slab reaching a maximum effective strength of 415k. The bottom layer of reinforcing in this area is also increased to #5 bars spaced every 18”. One bay on the 9th floor (grid lines 7-8) is constructed with a 10” cast in place slab. Plans of this floor can be found in appendix E.

Mechanical penthouses exist on the 9th and 20th roof constructed with a steel moment frame. Typical sizes for the 9th floor penthouse are W10’s and W12’s with 1.5” 20 gage “B” metal deck. As for the 20th floor penthouse, the typical beam size is W16x26. Equipment will be supported on concrete pads typically 4” thick. Two air handling units and cooling towers on the roof will require 6” pads.

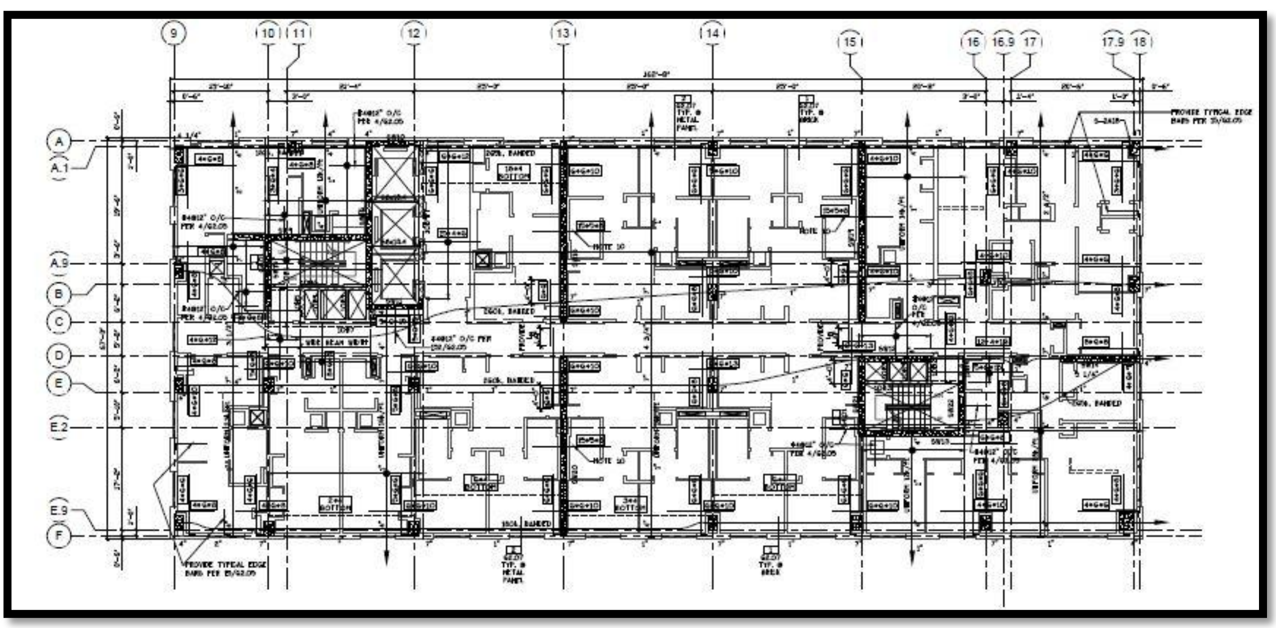


Figure 4 - Typical floor plan of upper tower

The loads will flow through the slab and reinforcement to the columns eventually making their way down to the foundation. To tie the slab and framing system into the columns, two tendons pass through the columns in each direction. To further tie the systems together, bottom bars have hooked bars at discontinuous edges. Dovetail inserts are installed every 2' on center to tie the brick façade in with the superstructure. Columns are typically 30"x20" and composed of 4ksi strength in the northern tower (9 floors), while columns in the southern tower vary from 8ksi at the bottom, and 4 ksi at the top.

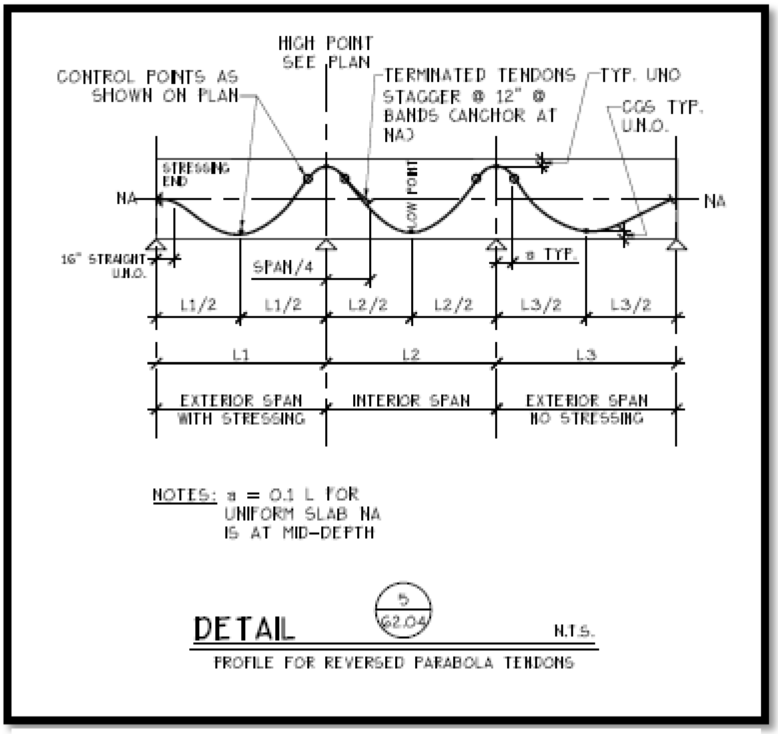


Figure 5- Typical detail for post tensioned tendon profile

Lateral System:

John Hopkins GSH is supported laterally through a cast in place reinforced concrete shear wall system. All of the shear walls are be 12" thick and are located throughout the building and around stairwells and elevator shafts. Shear walls in the 9 floor tower are poured with 4000psi strength concrete while shear walls in the 20 floor tower vary in three locations. From the foundation to 7th floor, 8ksi concrete was required, 6ksi from 7th to below 14th floor, and 4ksi for walls above the 14th floor. The shear walls are tied into the foundation system through bent vertical bars 1' deep into the grade beam as shown in figure 6. Shear walls are shown below in the figure with N-S walls highlighted in blue and E-W walls red. Walls in the center of the building will support lateral stresses directly, while those on the end support the torsion effects caused by eccentric loads. Elevations of shear walls can be found in appendix E.

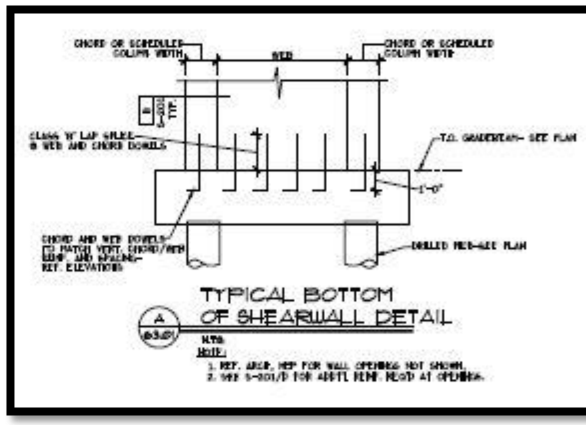


Figure 6 - detail tying shear wall into foundation

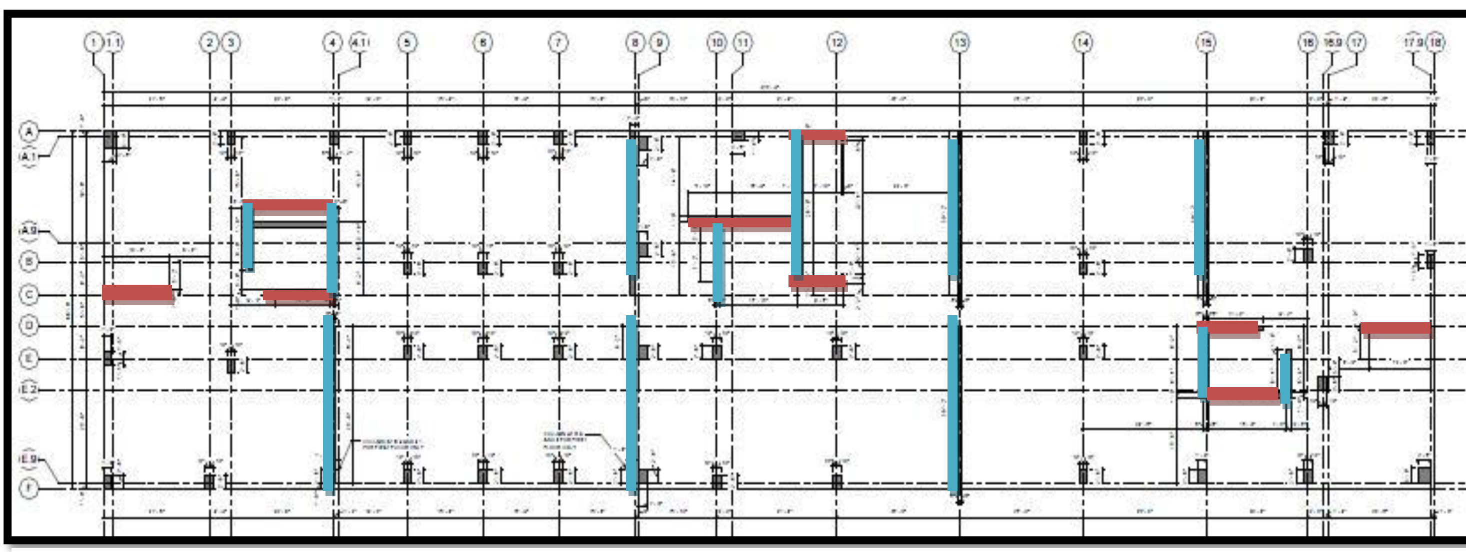


Figure 7 - Shear wall layout

Building Code Summary –

	John Hopkins GSH was designed to comply with:	My Thesis analysis/design will be based on:
General Building Code	IBC 2006	IBC 2006
Lateral Analysis	ASCE7	ASCE7-05
Concrete Specifications	ACI 301, 318, 315	ACI 318-08
Steel Specifications	AISC and AWS D1.1	AISC 2006
Masonry Specifications	ACI 530.1/ASCE 6	ACI 530.1-08/ASCE 6-08

Table 1- Building Code Comparison

Material Strength Summary –

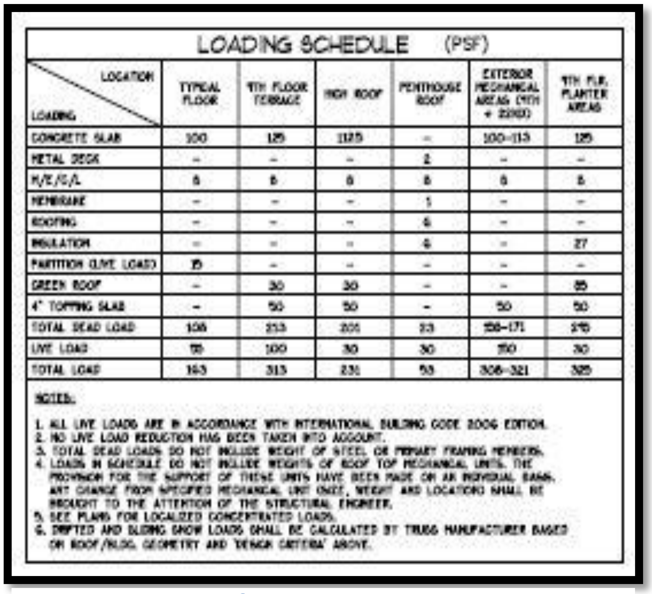
Material Strengths		
Concrete		
Material	**Weight (lbs/ft^3)**	**Strength (psi)**
Footings	145	4000
Pile Caps	145	4000
Caissons	145	4000
Grade Beams	145	4000
Slab-on-grade	145	3500
Slabs/beams	145	5000
Slab on metal deck	115	3500
Columns	145	Vary-see schedule
Shearwalls	145	Vary-see schedule
Steel		
Shape	**Grade**	**Yield Strength** (ksi)
W Shapes	A992	50
S, M and HP Shapes	A36	36
HSS	A500-GR.B	42
Channels, Tees, Angles, Bars, Plates	A36	36
Reinforcing Steel	GR. 60	60

Table 2 - Material Strength Summary

Load Calculations –

Dead Loads:

The dead loads calculated in appendix A have confirmed the dead loads that were provided in the loading schedule as seen in table 3. It appears that the designer used ASD in their analysis because the total load does not have any factors applied to it. The analysis in this tech report will be LRFD which typically results in a more aggressive design.



LOADING SCHEDULE (PSF)

LOCATION / LOADING	TYPICAL FLOOR	7TH FLOOR TERRACE	HIGH ROOF	PENTHOUSE ROOF	EXTERIOR MECHANICAL AREAS (7TH + ROOF)	7TH FLR. PLANTER AREAS
CONCRETE SLAB	100	125	115	–	100–115	125
METAL DECK	–	–	–	2	–	–
M/E/C/L	6	6	6	6	6	6
MEMBRANE	–	–	–	1	–	–
ROOFING	–	–	–	6	–	–
INSULATION	–	–	–	6	–	27
PARTITION (LIVE LOAD)	15	–	–	–	–	–
GREEN ROOF	–	30	30	–	–	85
4" TOPPING SLAB	–	50	50	–	50	50
TOTAL DEAD LOAD	106	213	201	23	156–171	295
LIVE LOAD	55	100	30	30	150	30
TOTAL LOAD	161	313	231	53	306–321	325

NOTES:

1. ALL LIVE LOADS ARE IN ACCORDANCE WITH INTERNATIONAL BUILDING CODE 2006 EDITION.
2. NO LIVE LOAD REDUCTION HAS BEEN TAKEN INTO ACCOUNT.
3. TOTAL DEAD LOADS DO NOT INCLUDE WEIGHT OF STEEL OR PRIMARY FRAMING MEMBERS.
4. LOADS IN SCHEDULE DO NOT INCLUDE WEIGHTS OF ROOF TOP MECHANICAL UNITS. THE PROVISION FOR THE SUPPORT OF THESE UNITS HAVE BEEN MADE ON AN INDIVIDUAL BASIS. ANY CHANGE FROM SPECIFIED MECHANICAL UNIT SIZE, WEIGHT AND LOCATIONS SHALL BE BROUGHT TO THE ATTENTION OF THE STRUCTURAL ENGINEER.
5. SEE PLANS FOR LOCALIZED CONCENTRATED LOADS.
6. DRIFTED AND SLIDING SNOW LOADS SHALL BE CALCULATED BY TRUSS MANUFACTURER BASED ON ROOF/BLDG. GEOMETRY AND 'DESIGN CRITERIA' ABOVE.

Figure 8 - Summary of loads used by designer

Live Loads:

It seems John Hopkins used loads very similar to the ASCE7-05 standards. Exterior mechanical loads were not specified in the standard, but I am assuming the equipment can cause significant loads while operating. The 30psf on non-assembly roof areas is most likely a judgment call to account for the maintenance that would be required for a green roof. Although not specified on the table, the 100psf required in the corridor and stairwells are most likely balanced by the large banded post tensioned tendons running parallel to the corridor and around the stairwells.

Area	Designed for – (psf)	ASCE7-05 (psf)
Typical Floor	55 (includes partitions)	40 (residential) + 15 (partitions)
Corridors	N/A	100
Stairs	N/A	100
Assembly	N/A	100
First story retail	N/A	100
Roof used for garden/assembly	100	100
Exterior Mechanical areas	150	N/A
High Roof	30	N/A
Penthouse Roof	30	N/A
Planter Areas	30	N/A

Table 3 - Live Load Comparison

Floor System Comparison –

The original floor system was compared with three other viable systems. Typical members were designed for each system to satisfy strength and serviceability requirements. Results were obtained for these designs from hand and computer calculations. Cost information was found through RS means unless otherwise noted. Details of calculations can be found in the appendix.

To keep the systems as comparable as possible, the bay sizes were kept the same as the original design. The typical floor plan is a long rectangle with typical spans of 25 feet. The alternate systems were analyzed in a critical section, the central corridor shown below in figure 9. The blue represents the bay analyzed while the red represents the strip. Although the width of this bay is five feet shorter than the edge bays, the loads here are considerably larger. The ASCE7-05 recommendation of 100psf for live load was used, but not the partition load. The partition load was assumed to only occur in the tenant living areas where it is more likely to occur. The systems analyzed were:

- 2-way, post-tensioned slab (original system)
- Hollow-core planks
- Composite steel beam and concrete slab system
- 2-way flat plate

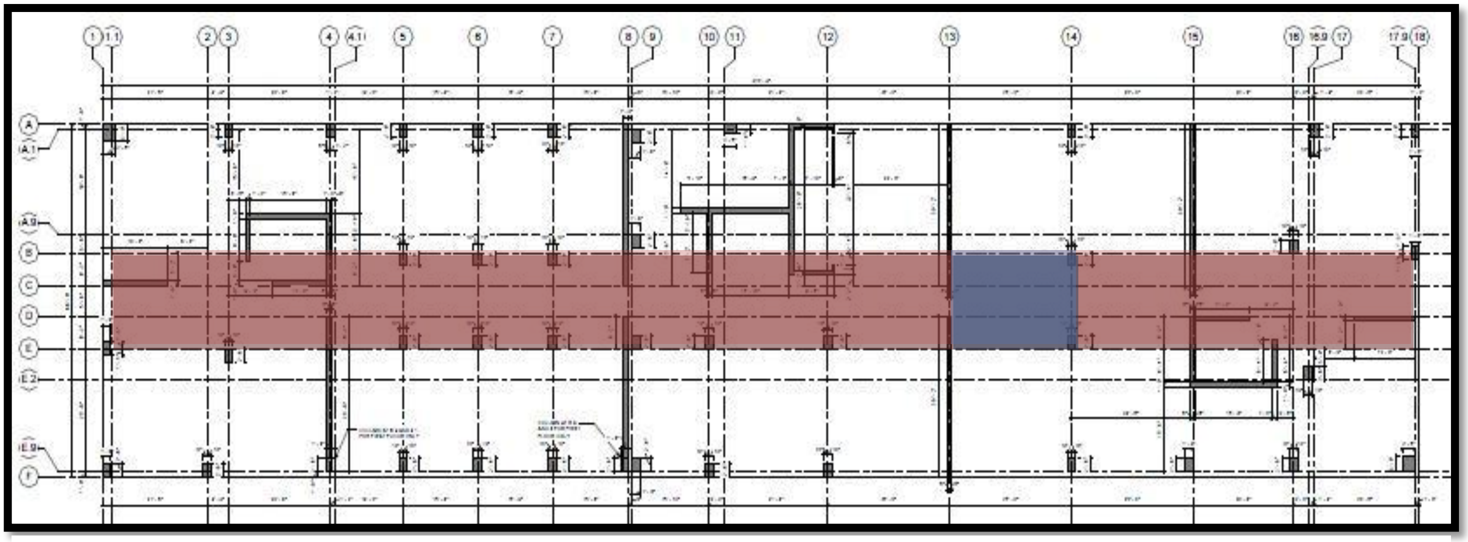


Figure 9 - Typical plan analyzed

2-way, Post-Tensioned System:

The long rectangular shape of this building is ideal for a 2-way, post-tensioned system due to its efficiency. When the slab is poured, sleeves are embedded in the concrete and a greased tendon is fed through. Once the concrete has reached adequate strength, 3750 psi, the tendon is then stressed. The stressing and tendon profile can be used to counter the vertical loads whether they are causing positive or negative moments. The PT system was modeled through RAM concepts for strength and deflection. Hand calculations were performed to analyze the allowable stresses caused by an individual tendon. Strength results were calculated using the load case 1.2D + 1.6LL. Results for these calculations can be found in appendix B.

Advantages

2-way, Post-Tensioned floor slabs are ideal for a dormitory setting because it can easily span medium to large bay sizes while maintaining a low floor to floor height. Providing a low floor to floor height allows the building to maximize floor area while minimizing overall height. This system was able to achieve a 9’4” floor to floor height, saving money on other aspects of the building such as the façade, mechanical ducts, and electrical wiring. The 8”, 5000 psi slab is redundant from floor to floor, enabling the contractor to reuse the formwork, thus saving money. Formwork is approximately 50% - 60% of the total cost of a concrete system according to Mr. David Holbert of Holbert Apple Assoc. The reuse of formwork also allows for a quicker schedule because the workers can develop a pattern and do not have to lay custom formwork at every floor.

Architecturally, the slab will not be seen by most, as the ceiling is gypsum wallboard on metal studs. The ceiling height in the unit spaces and corridor is 8’ as mandated by IBC ‘06. PT slab systems are also excellent for controlling deflections. This serviceability is evident in the results, where the maximum deflection was found to be .155”. This system also meets the minimum 2 hour fire rating prescribed in IBC table 720.1. An exact cost/sq ft for post tensioning was not found. However, according to Stephanie Slocum of Hope Furrer Associates, the cost for a PT slab would be similar to a flat plate minus the difference of the weight of the rebar. In the PT slab, the bottom reinforcement consisted of #4 bars every 2’ compared to the flat plate which has #5 bars every 6”. This significant reduction in rebar makes the PT slab much more economical than the flat plate. A flat plate costs approximately $14.75/ sq. ft. so a PT slab would cost significantly less than that.

<u>Disadvantages</u>

With the tendons being immovable, and the thin slab, renovations on PT systems are difficult. Any undersigned penetration in the slab requires the consulting of a structural engineer and the exact location of the tendons through an x-ray. One mistake in a renovation process, such as cutting a tendon, would be a catastrophic one. Another downfall of PT systems is the expertise and skilled labor required to install them. For this particular system, the structural engineer requires the field foreman to have at least three years of experience in this type of construction.

Hollow Core Planks:

Hollow core planks were selected as a viable alternative because of the similarity to the post tensioned system. The slab is precast with prestressed tendons spaced every 5.5" and can span lengths up to 35'. The middle of the slab has hollow cores as seen in figure 10, taking weight out of the slab. Using the Nitterhouse design tables, it was found that an 8" x 4' plank with a 2" topping would satisfy strength and fire durability requirements of 2 hours. Calculations for the strength and deflection of the planks, as well for the supporting girder can be found in appendix C. The cost for hollow core planks is typically $13.86/ sq. ft, another cheap system with respect to the PT slab.

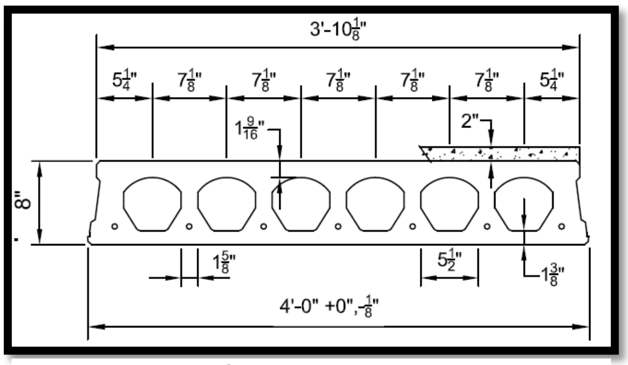


Figure 10 - Section of hollow core slab

SAFE SUPERIMPOSED SERVICE LOADS																				IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)
Strand Pattern		SPAN (FEET)																		
		17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
4 - 1/2"ø	LOAD (PSF)	280	256	226	199	190	170	151	137	119	106	93	82	72						
6 - 1/2"ø	LOAD (PSF)	366	341	318	299	271	245	223	211	196	176	159	143	129	113	98	85	74	63	53
7 - 1/2"ø	LOAD (PSF)	367	342	320	300	282	265	243	221	202	189	180	165	151	134	118	104	91	80	69

Figure 11- Design table used to size plank

Advantages

Hollow core planks can span large distances while minimizing depth of the structure. A distinct advantage of the hollow core system is that the precast bottom is able to be used as a clean exposed ceiling. If the coordination between trades is willing to work together, electrical conduit and small mechanical pipes can run through the cores. Cutting holes no larger than 560 mm in the bottom of the planks can be useful for down lighting, which is a typical fixture throughout this building.

Figure 12 - Example of down light in hollow core plank

Precast planks would also speed up the construction schedule because there would be no wait for curing of concrete or placing of formwork. Planks can also be erected during winter because no curing is involved. The hollow core planks weigh 61.25 psf, removing a approximately 59% of the weight from the floors. This weight reduction significantly reduces the seismic effect on the building. It is hard to say how this weight would affect the foundations. From the geotechnical report, suitable rock for a building this size isn't reached until 80'. There may be some reduction in the size of the caissons, but the overall depth would most likely remain the same.

Disadvantages

As with all prestressed elements, you must be cautious with the camber. As stresses are induced in the bottom of the slab, it causes an upward deflection in the middle of the slab. Camber will need to be addressed further if this design is used later in depth. The edges of the hollow core planks need to be sealed well to prevent water from entering the cores to increase its durability. With the reduction of mass and prestressed tendons, vibration would be more of an issue with this system than with the PT. An in-depth study of vibration was not done in this report, but it is reasonable to assume that in a dormitory setting of graduate students, vibration would not be the controlling factor. With pre-cast elements, there are increased shipping costs, hoisting and erection costs, as well as connection costs.

Composite steel beam and concrete slab system:

A composite steel beam and concrete slab was selected to compare steel to concrete, while minimizing the floor to floor heights. The theory behind the system is that the concrete and steel will work together to resist the load. With the systems working together, a smaller weight can be achieved instead of considering the beam to withstand the entire load. Concrete has excellent compressive properties, but is poor in tension which is where the steel comes into play.

This system was designed to minimize the depth of the structural system, not for system weight. This would ideally save money in the long run with savings on the façade, mechanical ducts, and electrical wiring. Calculations can be found in Appendix D. The results showed that the vertical loads and fire ratings can be satisfied with a 2VL deck with a 2" topping, W10x22 beams, and W12x30 girders. The average cost for a composite steel system in the city of Baltimore is $21.06/sq. ft. This is relatively high compared to the PT system, but savings could be found elsewhere due to the steel structure.

Advantages

Steel systems are generally lighter than concrete which reduces the force due to earthquake loads. This significant reduction in weight could lead to a smaller foundation. Steel frames can also often be erected quicker than cast in place concrete systems. With regards to construction, steel erection doesn't require skilled labor. Steel also provides ductile behavior, so in the event of severe loading, it will yield before failure. Steel frames also have the ability to be easily modified in the future should the owner choose to renovate the apartments.

Disadvantages

Although the concrete and decking pass the 2 hour fire rating, the beams and girders currently do not; therefore, the steel framing would need an unsightly fireproof coating. This fireproofing would require some sort of drop ceiling to cover it, thus increasing the floor to floor height. With a 12" girder plus fireproofing and ceiling, the minimum floor to floor height that can be achieved is 9'9". The steel system currently laid out also impacts the shear walls. The design was based on a column at the shear wall location, which would interrupt the continuity of the wall. Steel also has an issue with vibration due a lack of mass.

2-way Flat Plate:

A 2-way flat plate is used for large square bays and contains reinforcement at the top and bottom of the slab. Top reinforcement is required near the columns to integrate the slab with the column and resist negative moments. A flat plate is similar to a PT slab with regards to construction. The laying of formwork and pouring of concrete is the same process and costs the same as a PT slab. Where a flat plate increases cost is in the rebar. The weight of rebar in a 2-way flat plate is much greater than that of a 2-way PT system. This can be seen by the # 4 bars spaced every 2' in the PT system compared to the #5 bars spaced every 6" in the flat plate. A flat plate costs approximately $14.75/sq. ft. Laying the formwork and pouring the concrete is virtually the same procedure as for a PT slab; therefore, the price will not be less than an efficient PT slab.

<u>Advantages</u>

A 2-way flat plate can span longer distances than a one way system and also removes beams from the system. Removal of beams reduces the overall weight compared to a one way system, but compared to the PT system it is heavier. Architecturally, a flat plate will look identical to the PT system, except provide a thicker structural system. Flat plates are also more easily modified in the future compared to the PT slab. Flat plates handle vibrations extremely well due to their large mass. Along with a large mass is fire protection. A flat plate system can easily achieve a 2 hour fire rating without additional requirements.

<u>Disadvantages</u>

As previously stated, the flat plate is heavier than the PT slab and will increase earthquake loads. The foundation size might also need to be increased to account for a larger bearing pressure. The thicker slab also reduces the floor to floor height by a few inches. Deflection control is also often an issue with flat plates, short and long term. When this system was designed, it was assumed that columns were located where there currently shear walls. This impacts the lateral system and adds the additional cost of framing a portion of a column into a shear wall.

System Summary -

System Summary				
	Existing 2-way PT slab	**Hollow core plank**	**Composite steel beam/slab**	**2-way flat plate**
Cost ($/sq. ft)	<14.75	13.86	21.06	14.75
Weight (psf)	150	62.5	54	150
Foundation	Existing	Smaller	Smallest	Possibly a larger foundation
Impact lateral systems?	Existing	Need a column in shear walls to tie in planks	Yes, moment or braced frames need to be investigated	Concrete columns within the shear walls
Structural Depth	8”	10”	15”	9.5”
Fire Protection	2 hour- no extra requirements	2 hour- no extra requirements	2 hour- beams and girders need a fireproof coating	2 hour- no extra requirements
Architectural (does it need drop ceiling)	Existing uses drop ceiling	No drop ceiling needed	Drop ceiling needed	Would most likely utilize drop ceiling
Vibration	Very good	OK	Less than ok	Excellent
Construction impact	Existing	Significantly Accelerated Schedule	Slightly accelerated Schedule	About the same schedule
Constructability	Skilled labor - intensive	Medium-heavy lifts, detailed connections	Medium-heavy lifts, detailed connections	Easy – basic form work
Feasible	**Existing**	**Yes**	**Yes**	**No**

Table 4 - Systems summary chart

Feasibility –

Judging by the original system, it seems that the main goal of the John Hopkins Graduate Student Housing project is to minimize floor to floor height. This allows the owner to fit more rooms within the same height, thus make more money. A flat plate and hollow core system are the closest contenders to the PT slab with a 9.5" depth and 10" depth respectively. However, the hollow core planks also weigh less than the original system which could save cost on the foundation. With the clean bottom, further costs could be saved by eliminating the drop ceiling and running conduit through the cores.

The composite steel beam and slab system is the second most feasible alternative. The structural depth is slightly larger at 15", and also requires a drop ceiling. Increasing depth increases the cost of the façade, but only minimally. This cost could be offset by the major reduction in size of caissons. Decreasing the weight of the structure significantly could result in wind controlling the lateral system over earthquake. A further investigation of this and the differing lateral resisting frame could be included in tech report 3.

The flat plate system has advantages compared to other concrete systems, but to a PT slab, it is clearly inferior. The only advantage the flat plate has over the PT system is the ease of installation and being able to be modified during renovations. The structural depth has been increased, increasing cost and weight of the building. This increases the loads caused by earthquakes and loads on the foundation. There are no major advantages the flat plate system possesses over the PT slab making it not feasible for the John Hopkins Graduate Student Housing project.

Conclusions –

Upon completing the calculations and analyzing the results, it was found that two of the three systems were feasible. The structural systems were analyzed and designed through hand and computer calculations. Results were compiled and the systems were compared based on structural and non structural criteria.

The hollow core planks were deemed most feasible, followed by the composite steel beam and slab. Planks were designed to 8” x 4’ plank with a 2” topping on top to meet the 2 hour fire rating. Hollow core planks were able to achieve minimal structural depth while maintaining a low cost. Another main advantage of the planks were the ability to reduce the weight of the building, possibly leading to smaller foundations. For construction, precast systems also increase the schedule significantly. Issues to look into further with the precast planks are camber issues in the field, and how the planks will affect the lateral system.

A steel composite system is the second most feasible floor analyzed. Typical beams were designed to be W10x22 while girder were designed for W12x30. This system was designed to minimize the depth of the structural system. Although the structural depth and cost were higher than the PT system, the overall building weight was reduced dramatically. A steel system could lead to smaller foundations and a more dynamic response in the event of an earthquake.

The flat plate system showed no real promise into further investigation. A flat plate slab in this scenario could only be designed to 9.5” to limit deflections. The only real advantages the flat plate had over the PT were easier labor issues since no tendons were used, and an easier time modifying the structure down the road. These were not enough reasons to validate the increase of cost, weight, and altering the shear wall system to add more columns.

Appendix A – Load verification

Brad Oliver | AE 481 | Load Calculations

Dead Loads

Typical Floor

$\frac{8"}{12'} \times 150\ pcf = 100\ psf$

Superimposed DL - 8 psf (Mech, Elec, Ceiling, lighting etc)

Through online research... (www.bae.ncsu.edu)
green roof typ 30-35 psf

9th floor

$\frac{10"}{12'} \times 150\ pcf = 125\ psf$

High roof - $\frac{9"}{12'} \times 150\ pcf = 112.5\ psf$

4" slabs for Mech equip - $\frac{4"}{12'} \times 150\ pcf = 50\ psf$

Snow Loads - ASCE 7-05 ch 7 - Flat roof - $p_f = .7 C_e C_t I p_g$

From Fig 7-1 $p_g = 25\ psf$

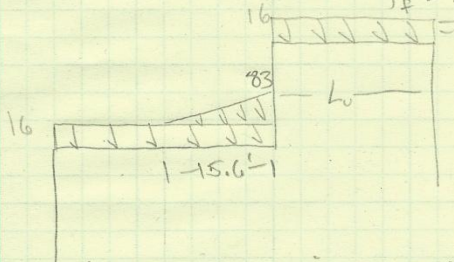
From table 7-3 $C_t = 1.0$ (All other structures)

Occupancy category II from table 1-1
$\therefore I = 1.0$ from table 7-4

Site Class C from geotechnical report

Fully Exposed roof
$\therefore C_e = .9$ from table 7-2

$p_f = .7(.9)(1.0)(1.0)(25)$
$= 15.75\ psf \cong 16\ psf$



$L_u = 163'$

$h_d = 3.9'$

$h_b = 16\ psf \times \frac{ft^3}{17.25\ lbs} = .93' < 3.9'$

$\gamma = .13(25) + 14 < 30$
$= 17.25 < 30\ pcf$

Leeward
$L_u = 163'$ $h_d = 3.9'$

$W_d = 4(3.9') = 15.6'$

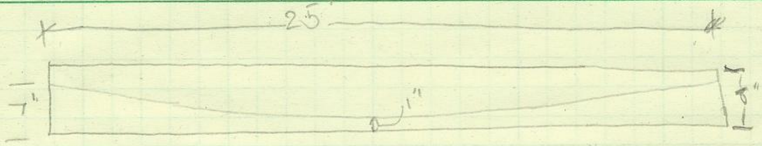
$P_d = 3.9'(17.25\ pcf) = 67\ psf$
Max snow Load $= 67 + 16$
$= 83\ psf$

Wind Ward
$L_u = 241' - 163' = 108'$
$h_d = 3.2' \times .75 = 1.8'$ Leeward controls

TOPS

Appendix B – Post Tensioned System

Brad Oliver | AE 481 | PT Check (1)



8" thick
$f'_c = 5000$ psi
$f'_{ci} = 3500$ psi
Additional #4 @ 24" both ways

Preliminary Thickness

$L/H = 45$ $\quad H = L/45 = (25 \times 12)/45 = 7"$

designed was 8"
Close enough

Dead Load - $(8"/12")(150\ pcf) = 100\ psf$
SDL - 8 psf
108 psf

LL - 55 psf

Assume targeted load balancing of 75% of DL

$.75(108) = 81\ psf$

tendon ordinate	Tendon CG Location	
int support	7"	
int midspan	3"	$a = 7 - 3 = 4"$

Weight to balance

$81\ psf\left(4' + \frac{23'}{2}\right) = 1661\ lb/ft = 1.66\ K/ft$

Force req in tendon

$P = w_d L^2/8a$

$= 1.66(25^2)/8/\left(\frac{4}{12}\right)$

$= 389K$

Tendon properties

7 Wire strand A-416
1/2" diameter tendon Area - .153 in²
Ultimate Strength - 270 ksi

Brad Oliver | AE 481 | P.T. Check (2)

Estimate losses @ 15 ksi

$$f_{se} = .7(270) - 15 = 174 \text{ ksi}$$

$P_{eff} = .153(174) = 26.6$ kips/tendon ≈ 27 k/tendon designed for.

# of tendons to balance load

$$389/26.6 = 14.6 \approx 15 \text{ tendons}$$

$$P_{actual} = 15(26.6) = 399 \text{ K}$$

399 >> 260K designed.

This could be due to smaller
self weight balance, or the fact
there is another 260 K tendon
running // 18' away to make up
for this difference.

Balanced load adjustment

$$\frac{399}{389}(1.66) = 1.7 \text{ K/ft}$$

$$\frac{P_{Act}}{A} = \frac{399 \text{ K}}{(9\times12 + \frac{23}{2}\times12)(8'')} = 203 \text{ psi} > 125 \text{ psi min} \checkmark$$

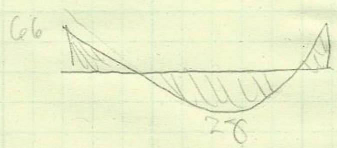
< 300 psi max ✓

Brad Oliver | AE 481 | P.T. check (3)

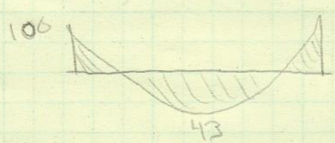
Check slab stresses

$W_{DL} = 108 \text{ psf } (9 + \frac{23}{2}) = 2.21 \text{ K/ft}$ $\quad M_o = \frac{2.21(25^2)}{8} = 173 \text{ 'K}$

$W_{LL} = 55 \text{ psf } (9 + \frac{23}{2}) = 1.13 \text{ K/ft}$ $\quad M_o = 88 \text{ 'K}$



$W_b = -1.7 \text{ K/ft}$ $\quad M_o = 133 \text{ 'K}$



Stresses immediately After jacking

$f'_{ci} = 3500 \text{ psi}$

Mid span

$f_{top} = (-M_{dL} + M_{bal})/S - P/A$

$S = bh^2/6 = (108+138)(8^2)/6 = 2624 \text{ in}^3$

$(-55 + 43)(12000)/2624 - 203$

$= -256 \text{ psi comp} < .6(3500) = 2100 \text{ psi}$ ✓

$f_{bot} \; (M_{dL} - M_{bal})/S - P/A$

$(55 - 43)(12000)/2624 - 203$

$= -148 \text{ psi comp} < 2100 \text{ psi}$ ✓

Support

$f_{top} = (-129 + 100)(12000)/2624 - 203$

$= -132 \text{ comp} < 2100 \text{ psi}$ ✓

$f_{bot} = (129 - 100)(12000)/2624 - 203$

$= -70 \text{ psi comp} < 2100 \text{ psi}$ ✓

Brad Oliver | AE 481 | P.T. deck (4)

Stresses @ service load

Midspan

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

$$= (-55 - 28 + 43)(12000)/S - P/A$$

$$= -386 \text{ psi} < .45(5000) = 2250 \text{ psi} \checkmark$$

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

$$= (55 + 28 - 43)(12000)/2624 - 203$$

$$= -20 \text{ psi} < 2250 \text{ psi} \checkmark$$

Support

$$f_{top} = (-129 - 66 + 100)(12000)/2624 - 203$$

$$= -637 \text{ psi} < 2250 \text{ psi} \checkmark$$

$$f_{bot} = (129 + 66 - 100)(12000)/2624 - 203$$

$$= 231 \text{ psi tension} < 6\sqrt{5000} = 424 \text{ psi} \checkmark$$

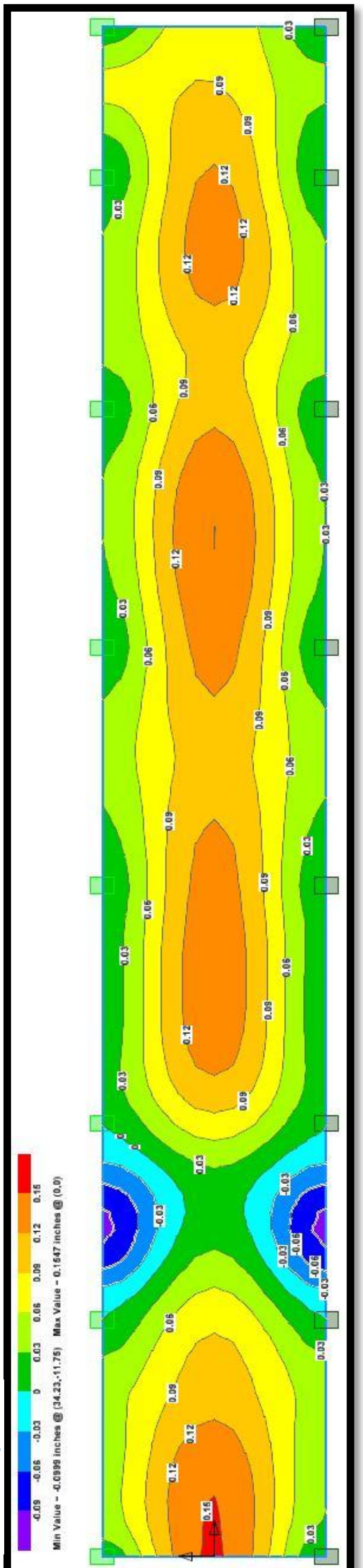


Figure 14- Deflection reaction for PT system. Maximum displacement is shown in red at .155 inches

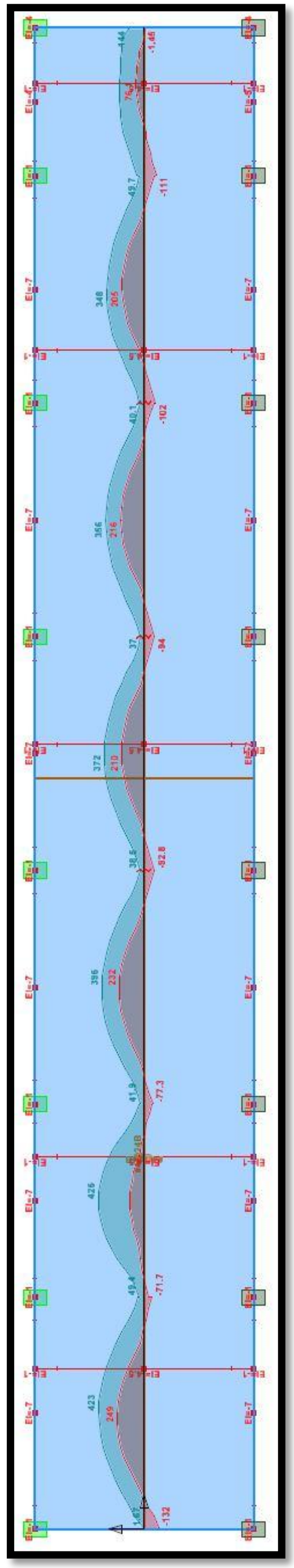


Figure 13- strength results. Blue line represents moment capacity while red represents required

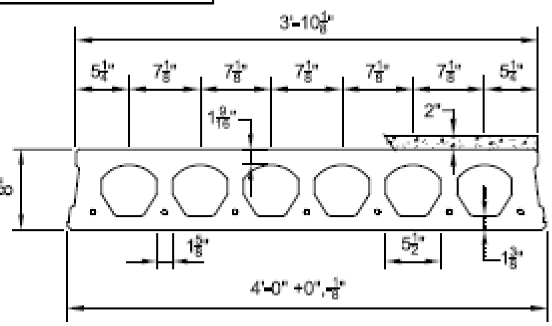
Appendix C – Hollow Core Planks

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

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES Composite Section	
A_c = 301 in.^{2}	Precast b_w = 13.13 in.
I_c = 3134 in.^{4}	Precast S_{bcp} = 616 in.^{3}
Y_{bcp} = 5.09 in.	Topping S_{tct} = 902 in.^{3}
Y_{tcp} = 2.91 in.	Precast S_{tcp} = 1076 in.^{3}
Y_{tct} = 4.91 in.	Precast Wt. = 245 PLF
	Precast Wt. = 61.25 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 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)..
   4-1/2"Ø, 270K = 92.3 k-ft at 60% jacking force
   6-1/2"Ø, 270K = 130.6 k-ft at 60% jacking force
   7-1/2"Ø, 270K = 147.8 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 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 allowable service stresses.
15. Load values will 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)																		
		17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
4 - 1/2"ø	LOAD (PSF)	280	256	226	199	190	170	151	137	119	106	93	82	72						
6 - 1/2"ø	LOAD (PSF)	366	341	318	299	271	245	223	211	196	176	159	143	129	113	98	85	74	63	53
7 - 1/2"ø	LOAD (PSF)	367	342	320	300	282	265	243	221	202	189	180	165	151	134	118	104	91	80	69

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 1 Hour & 0 Minute fire resistance rating.

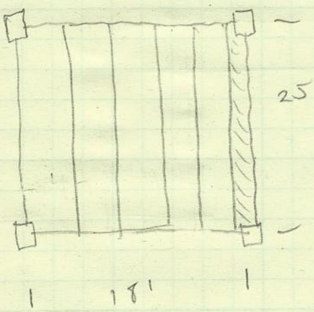
11/03/08

8SF1.0T

Brad Oliver | AE 481 | Hollow core 1

$W_u = 1.2(8) + (1.6)(100) = 170$ psf < 190 psf allowable by table.

*Dead load of plank included in table.



= 2' section. Nitterhouse is able to cut 4' planks in half.

Weight = 61.25 psf

Girder to support planks

$W_u = 1.2(61.25 + 8) + 1.6(100)$
$= 243$ psf

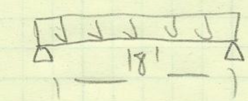
$W_L = 243(18') = 4.37\ ^{K}/_{ft}$

$M_u = 4.37(25^2)/8 = 341'K$

Unbraced length = 18'

Using table 3-10 Try W12X72 - chosen to limit depth
$I = 597\ in^4$

LL defl – $\Delta_{LL} = \dfrac{5(.1\times25)(18^4)(1728)}{384(29000)(597)} = .341''$



$\Delta_{LL\ All} = \ell/360 = 18\times12/360 = .6 > .341''$ ✓

TL defl $\Delta_{TL} = \dfrac{5(.1+.008+.061)25'(18^4)(1728)}{384(29000)(597)} = .58''$

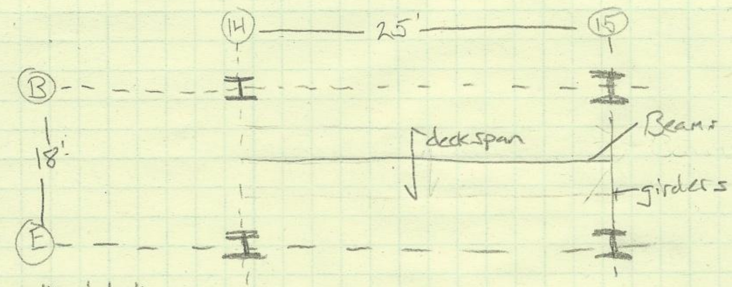
$\Delta_{TL\ Allow} = \ell/240 = 18\times12/240 = .9'' > .58''$ ✓

use W12X72 girder in short direction to support planks.

Appendix D – Composite Steel Beam with slab

Brad Oliver | AE 481 | Steel-composite floor 1

going to analyze 14-15 & B-E Bay b/c it is one of the largest with the largest loads. Most critical case.



Assumptions & deck

Assume beams spaced 9', steel columns, NW 5000psi conc

Loads- corridor has no partition load, only 100psf LL & self weight
will be using LC 1.2D+1.6L

2 VLI composite deck w/2" topping. gage 19

SDI Max unshored const = 9'4 > 9'0" ∴ No shoring necessary

@ 9' LL_{Allow} = 157psf > 100psf ✓

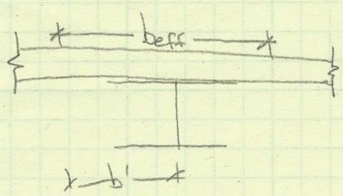
Weight = 33 psf conc + 2.49 psf deck

This deck sufficiently gives 2 hour fire rating UL design - D743 #

Beam

Dead- 8 (SDL) + 33 + 2.49 + self = 43.5 psf + self
Live- 100 psf (No partitions b/c corridor)

W_u = 1.2(43.5) + 1.6(100)
= 212 psf
× 9 ft
1910 lb/ft



b' < span/8 = 25×12/8 = 37.5" ☆ controls

b' < ½ dist to Adj beam = ½(9'×12) = 54"

b_{eff} = 75"

Brad Oliver | AE 481 | Slab composite 2

$M_u = \frac{w_u L^2}{8} = \frac{1910(25^2)}{8(1000)} = 149'K$

Assume fully composite $a = 1"$ ∴ $Y_2 = 3.5 - \frac{1}{2}" = 3"$

Try W10x19 $\Sigma Q_n = 201\ K$ Additional Mom to self weight - $\frac{1.2(19)(25^2)}{8(1000)} = 1.8'K$

$\phi M_n = 153'K$

$\phi M_n > 150.8\ 'K$ ✓

Try W 10x22 $\Sigma Q_n = 169K$

$\phi M_n = 160'K$ Additional Mom self W - $\frac{1.2(22)(25^2)}{8(1000)} = 2'K$

$\phi M_n > 151'K$ ✓

$Q_n = .5 A_{sc} \sqrt{f'_c E_c} \leq R_g R_p A_{sc} F_u$

$E_c = w_c^{1.5} \sqrt{f'_c} = 145^{1.5} \sqrt{5}$
$= 3904$

$A_{sc} = \pi \left(\frac{.75}{2}\right)^2 = .442$

$Q_n = .5(.442)\sqrt{5(3904)} = 30.9K$

$$\left.\begin{aligned} &= 1.0(.6)(.442)(65) = 17.2K\ \text{1 stud/rib weak pos} \\ &= .85(.6)(.442)(65) = 14.6K\ \text{2 stud/rib " "} \\ &= .7(.6)(.442)(65) = 12.1K\ \text{3 stud/rib " "} \end{aligned}\right\}\ \text{govern over 30.9K}$$

For W10x19 $\frac{201}{17.2} = 11.6 \Rightarrow 24$ Studs/beam < 25 ✓

For W10x22 $\frac{169}{17.2} = 9.8 \Rightarrow 20$ Studs/beam < 25 ✓

total Weight for W10x19 - $19\ \text{lbs/ft}(25') + 10\ \frac{\text{lbs}}{\text{Stud}}(24) = 715$ lbs

" " " W10x22 - $22(25) + 10(20) = 750$ lbs

W10x19 most economical while striving to maintain low floor to floor heights

check Assumption

$a = \frac{\Sigma Q_n}{.85 f'_c b_{eff}} = \frac{201}{.85(5)(75)} = .63"$ ∴ $Y_2 = 3"$ is ok

TOPS.

Brad Oliver | AE 481 | slab composite 3

Check Beam for unshored strength

W10X19 ⇒ $\phi_b M_p = 81'K$ — table 3-2 AISC

slab, deck, width, self

considering only DL – $1.4(33+2.49)\,9' + 1.4(19) = .474\ K/ft$

const LL

construction loads – $1.2(33+2.49)\,9' + 1.2(19) + 1.6(20)(9) = .694\ K/ft$ ☆

$M_u = \frac{w_u L^2}{8} = \frac{.694(25^2)}{8} = 54'K < 81'K$ ✓

Check wet concrete deflection

Service Loads $W_{wc} = 33\,psf\,(9') + 19\ lb/ft = .316\ K/ft$

$I_x = I_{beam}$ b/c no strength in concrete yet
$= 96.3\ in^4$ table 3-2 AISC

conversion

$\Delta_{wc} = \frac{5wL^4}{384EI} = \frac{5(.316)(25^4)(1728)}{384(29000)(96.3)} = .99''$

$\Delta_{Allow} = L/240 = 25\cdot12/240 = 1.25'' > .99''$ ✓

Check LL Δ

use $I_{Lower\ Bound}$ now b/c full composite Action
$I_{LB} = 251\ in^4$ by table 3-20 AISC

$W_{LL} = (100)\,9' = .9\ K/ft$

$\Delta_{LL} = \frac{5 W_{LL} L^4}{384EI} = \frac{5(.9)(25^4)(1728)}{384(29000)(251)} = 1.09''$

$\Delta_{LL\ All} = L/360 = 25\times12/360 = .83$ NG try W 10X22

Check Assumption $a = \frac{169}{.85(5)(75)} = .53'' < 1''$ ∴ $y_2 = 3''$ is ok

Unshored strength

$\phi M_p = 97.5'K$

DL – $w = 1.4(33+2.49)\,9' + 1.4(22) = .528\ K/ft$

const Load $w = 1.2(33+2.49)\,9' + 1.2(22) + 1.6(20)(9') = .698\ K/ft$ ☆

$M_u = \frac{.698(25^2)}{8} = 54.5'K < 97.5'K$ ✓

Wet conc defl

Service $W_{wc} = (33+2.49)\,9' + 22 = .341\ K/ft$ $I_x = 118\ in^4$

$\Delta_{WC} = \frac{5(.341)(25^4)(1728)}{384(29000)(118)} = .88'' < .99''$ Allowable ✓

Check LL defl $I_{LB} = \frac{5(.9)\,25^4\,(1728)}{384(29000)(263)} = 1'' > .83''$ Allow NG

Change Assumption so PNA is in top of flange ⇒ $\Sigma Q_n = 324\ K$
$a = 324/.85(5)(75) = 1''$ $a/2 = .5''$ $y_2 = 3.5 - .5 = 3''$
OK!

$\boxed{\text{W10X22 W/10 studs/beam}}$

TOPS

Brad Oliver	AE 481	Slab composite 4

With validity of new assumption, moment capacity increases to 197'K
& $I_{LB} = 330$ in^4. More Aggressive design.

$$\Delta_{LL} = \frac{5(.9)(25^4)1728}{384(29000)(330)} = .827'' < .833'' \checkmark$$

Check Δ_{TL}

$$w_{TL} = (\underset{\text{conc}}{33} + \underset{\text{deck}}{2.49} + \underset{\text{SDL}}{8})9' + \underset{\text{self}}{22} + \underset{\text{LL}}{100}(9) = 1.41\ ^K/_{ft}$$

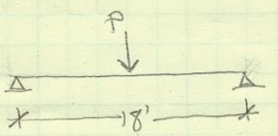
$$\Delta_{TL} = \frac{5(1.41)(25^4)1728}{384(29000)(330)} = 1.29''$$

$$\Delta_{TL\ Allow} = L/_{240} = 25\times 12/_{240} = 1.25'' \lessapprox 1.29''$$

Close. You could camber beam to solve this issue. I_{LB} is also a conservative calculation

Use W 10X22 w/ 10 studs for beams in long direction

Girder



$$P_u = 1.2(33+2.49+8)9' + 1.2(22) + 1.6(100)(9')$$
$$= 1936\ ^{lb}/_{ft} \cdot 25'$$
$$= 48.4K$$ Don't divide by 2 b/c beam from either side

$$W_u = 1.2(\text{Self Weight}) - M = W_u L^2/8$$

$$M \text{ from point load} = \frac{PL}{4} = \frac{48.4(18)}{4} = 218'K$$

Go into table with same Assumptions $a = 1''$ $y_2 = 3''$
Designing to limit depth

$$b' < \text{span}/8 = 18\times 12/8 = 27''$$
$$< 1/2 \text{ dist Adj beam} = (25\times\tfrac{1}{2})\times 12 = 150''$$

$$b_{eff} = 27''\times 2 = 54''$$

Try W12X22 $\phi M_n = 223'K$
$\Sigma Q_n = 324K$ $\quad a = \frac{324}{.85(5)(54)} = 1.4''$ $\therefore y_2 = 2.8$

also try W12X26 $\phi M_n = 233'K$
$\Sigma Q_n = 259K$ $\quad a = \frac{259}{.85(5)(54)} = 1.13''$ NG

W 12X30 $\quad \phi M_n = 227'K$
$\Sigma Q_n = 131$ $\quad a = \frac{131}{.85(5)(54)} = .57$ $\therefore$ assump Okay

Adjusted M_u
$30(18^2)$ .018 = 219'K < 227'K ✓

Brad Oliver | AE 481 | Slab composite 5

Shear stud strength for ribs // girder

$W_r/h_r = 1.75/1.5 = 1.12 < 1.5 \therefore R_g = .85$, $R_p = .75$

$Q_n = .5(.442)\sqrt{5(3904)} = 30.9K$

$= .85(.75)(.442)(65) = 18.3K$ 1 stud/rib

For 12X30 $\frac{131}{18.3} = 7.16 \Rightarrow$ 16 studs/beam

check girder for unshored strength

W12X30 $\phi_b M_p = 71.8'K$

Considering only DL: $.474\ ^k/_{ft}(25') + 30\ ^{lb}/_{ft} = 11.85K + .03\ ^k/_{ft}$

$M_u = \frac{11.85(18)}{4} + \frac{.03(18^2)}{8} = 54.5'K$

Const loads: $.694\ ^k/_{ft}(25') = 15.3K$

$M_u = \frac{15.3(18)}{4} = 69'K < 71.8'K$ ✓

check wet conc defl

$W_{wc} = .316\ ^k/_{ft}(25') = 7.9K$

$\Delta = \frac{PL^3}{48EI} = \frac{7.9(18')^3(1728)}{48(29000)(238)} = .24''$

$\Delta_{All} = L/240 = 18 \cdot 12/240 = .9'' > .24''$ ✓

check LL Δ

$P_{LL} = .9\ ^k/_{ft}(25') = 22.5K$ $\quad I_{LB} = 408\ in^4$

$\Delta_{LL} = \frac{22.5(18^3)(1728)}{48(29000)(408)} = .39''$

$\Delta_{LL\ Allow} = 18 \times 12/360 = .6''$

check total Load Δ

$P_{tot} = 1.41\ ^k/_{ft}(25') = 35.25K$ $\quad$ self $w = .03\ ^k/_{ft}$

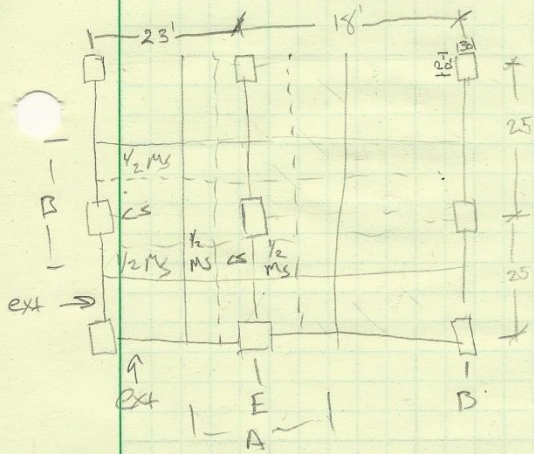
$\Delta_{tl} = \frac{35.25(18^3)(1728)}{48(29000)(408)} + \frac{5(.03)(18^4)(1728)}{384(29000)(408)} = .63'' < .9''$ Allow ✓

$\boxed{\text{Use W 12X30 w/16 studs for girders}}$

TOPS

Appendix E – Flat Plate System

Brad Oliver | AE 481 | 2 Way Flat Plate 1



Preliminary slab thickness

using table 9.5c to limit deflection

$l_n/33$

$l_n = 25' - \frac{10''}{12'} - \frac{10''}{12'} = 23.3'$

$23.3 \times 12/33 \sim 9.3''$. Try 9.5" slab

Assume 5 ksi concrete

#5 bars.

$W_d = (9.5''/12)(150\ pcf) = 119\ psf$ self w.

8 psf SDL

$W_L = 100\ psf$ Worst case (corridor)

$W_u = 1.2(119+8) + 1.6(100) = 312\ psf$

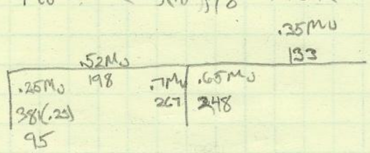
$M_u = \frac{w_u l_2 l_n^2}{8}$

Frame A – $M_u = 312(18')(23.3^2)/8 = 381'K$

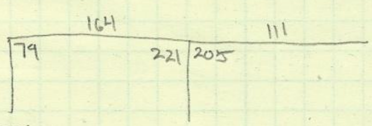
$l_n = \left(\frac{23+18}{2}\right) - \frac{30}{12} = 18'$

Frame B – $M_u = 312(25')(18^2)/8 = 316'K$

A



B



% to col strip

– M @ ext support 100

+ M 60

– M @ int support 75

Brad Oliver | AE 481 | 2 Way Flat Plate 2

Frame A	total width - 20	Col strip - 10'	MS - 10'		
total Moment	-95	+148	-267	-248	+133
Mom in Col strip	-95	+119	-200	-186	+80
Mom in MS	0	+79	-67	-62	+53
		ext span		int span	

CVR

$d = 9.5 - 3/4 - \frac{1}{2}(.625)$
$= 8.43"$

Frame B	total width 25'	col strip - 12'6"	MS - 12'6"		
Total M	-79	+164	-221	-205	+111
Mom in CS	-79	+98	-166	-154	+67
Mom in MS	0	+66	-55	-51	+44

$d = 9.5 - 3/4 - .625 - \frac{.625}{2}$
$= 7.81"$

Design of slab reinforcement in Frame A Col strip

Description	Ext Span M^-_{ext}	Ext Span M^+	Ext Span M^-_{int}	Int span M^-	Int span M^+
1. Mom M_n 'k	-95	119	-200	-186	+80
2. Wth CS, (in)	120	120	120	120	120
3. effective d (in)	8.43	8.43	8.43	8.43	8.43
4. M_u (in kip/in)	-9.5	11.9	-20	-18.6	8.0
5. $M_n = M_u/.9$	-106	132	-222	-207	89
6. $R = M_n/bd^2$ (psi)	-149	186	-312	-291	125
7. ρ from table A.5a	.0025	.0032	.0054	.005	.0021
8. $A_s = \rho bd$	2.53	3.24	5.46	5.06	2.12
9. $A_{s min} = .0018bt$	2.05	2.05	2.05	2.05	2.05
10. $N =$ value of 8 $/.31$ (#5)	8.2 - 9	10.4 - 11	17.6 - 18	16.3 - 17	6.8 - 7
11. $N_{min} = W_{strip}/2t$	7	7	7	7	7

Design Reinforcement Frame A MS.

1. Mom M_n 'K	0	79	-67	-62	53
2. Width MS	120"	120"	120	120	120
3. effective d	8.43	8.43	8.43	8.43	8.43
4. $M_n = M_u/.9$		88	-74	-69	59
5. $R = M_n/bd^2$		124	-104	-97	83
6. $\rho =$		.0021	.0016	.0017	.0014
7. $A_s = \rho bd$		2.12	1.62	1.72	1.42
8. $A_{s min} =$		2.05	2.05	2.05	2.05
9. $N =$		6.8 - 7	6.6 - 7	7	7
10. N_{min}		7	7	7	7

TOPS.

Brad Oliver | AE 481 | 2 WAY Flat Plate 3

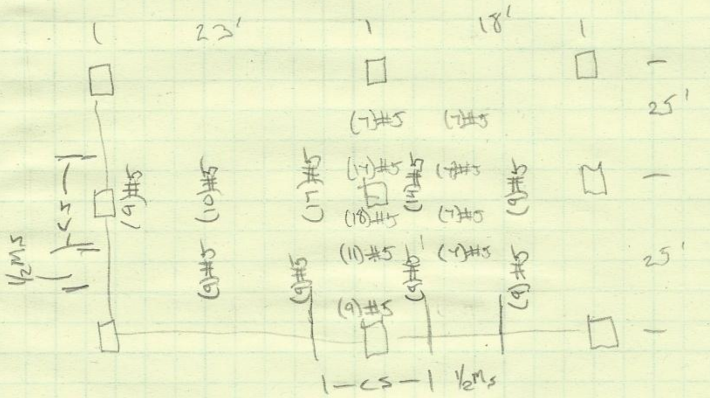
Design of Slab reinforcement Frame B C.S.

Description	Ext Span M_{ext}	M^+	M^-_{int}	Int Span M^-	M^+
1. Mom M_u 'K	-79	98	-146	-154	67
2. Width CS	150	150	150	150	150
3 effective d	7.81	7.81	7.81	7.81	7.81
4 $M_n = M_u/.9$	-88	109	-184	-171	74
5 $R = M_n/bd^2$	-115	143	-241	-224	97
6 $\rho =$	.002	.0024	.0043	.0037	.0017
7 $A_s = \rho bd$	2.34	2.81	5.04	4.33	1.49
8 $A_{min} = .0018bt$	2.57	2.57	2.57	2.57	2.57
9 N-	8.3-(9)	9.1-(10)	16.3-(17)	13.9-(14)	(9)
10 N_{min}	7.8-8	8	8	8	8

Design Reinforcement Frame B M.S.

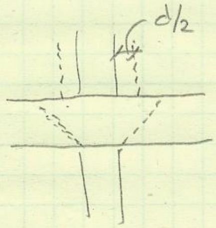
1. Mom 'K	0	66	-55	-51	44
2 Width	150	150	150	150	150
3 effective d	7.81	7.81	7.81	7.81	7.81
4 $M_n = M_u/.9$		73	-61	-57	49
5 R		96	-80	-75	64
6 ρ		.0017	.0013	.0012	.0011
7 A_s		1.99	1.52	1.41	1.29
$A_{s\,min}$		2.57	2.57	2.57	2.57
N-		(9)	(9)	(9)	(9)
N_{min}		8	8	8	8

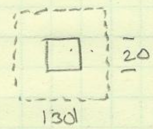
Calculation of deflection not necessary b/c code ch 9



Brad Oliver | AE 481 | Flat Plate Punching Shear | 4

Punching Shear Check



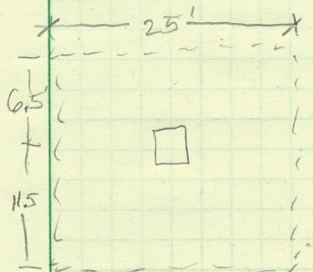


$d_{Avg} = \frac{8.43 + 7.81}{2} = 8.12''$

$d/2 = 4.06''$

Length $= 30'' + 8.12 = 38.12$
Width $= 20' + 8.12 = 28.12$

Perim $= b_o = 132.5''$



$b_l / b_s = 30/20 = 1.5 < 2 \therefore$

$V_c = 4\sqrt{f'_c}\, b_o d$

$V_c = 4\sqrt{5000}(132.5)(8.12'')$

$= 304\text{K}$

$V_u = W_u A$

$W_u = 1.2\left(\underbrace{\frac{9.5}{12}\times 150}_{\text{self}} + \underbrace{8}_{\text{SDL}}\right) + 1.6(100)$

$= .312\text{ ksf}$

$V_u = .312\left(25\times(11.5+6.5)\right)$

$= 140\text{ K}$

$\phi V_n = .75(304) > V_u$

$= 228\text{K} > 140\text{K}$ ✓ punching shear OK

Appendix F – References

Cost analysis was performed through online RS Means Costworks.

http://www.meanscostworks.com/

Precast Design tables were obtained from Nitterhouse Inc.

http://www.nitterhouse.com/

Information about hollow core slabs and provided figure 12

http://www.cma.org.za/UploadedMedia/CMA%20Prestress%20%28Multi%20Purpose%292%281%29.pdf

Further Information on hollow core slabs

http://web.eng.fiu.edu/prieto/HeavyConstruction/HC-Lecture19-PrecastConcrete.pdf

2-way Post tension slab design aid

http://www.cement.org/buildings/Timesaving-2WayPost-IA.pdf