Technical Report 2

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**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 as 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.

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 10th 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 10th 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.
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.

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.

Figure 3 - a detail section of a caisson and column
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.
Building Code Summary –

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

Table 1- Building Code Comparison

Material Strength Summary –

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight (lbs/ft³)</th>
<th>Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Footings</td>
<td>145</td>
<td>4000</td>
</tr>
<tr>
<td>Pile Caps</td>
<td>145</td>
<td>4000</td>
</tr>
<tr>
<td>Caissons</td>
<td>145</td>
<td>4000</td>
</tr>
<tr>
<td>Grade Beams</td>
<td>145</td>
<td>4000</td>
</tr>
<tr>
<td>Slab-on-grade</td>
<td>145</td>
<td>3500</td>
</tr>
<tr>
<td>Slabs/beams</td>
<td>145</td>
<td>5000</td>
</tr>
<tr>
<td>Slab on metal deck</td>
<td>115</td>
<td>3500</td>
</tr>
<tr>
<td>Columns</td>
<td>145</td>
<td>Vary-see schedule</td>
</tr>
<tr>
<td>Shearwalls</td>
<td>145</td>
<td>Vary-see schedule</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Shape</th>
<th>Grade</th>
<th>Yield Strength (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>W Shapes</td>
<td>A992</td>
<td>50</td>
</tr>
<tr>
<td>S, M and HP Shapes</td>
<td>A36</td>
<td>36</td>
</tr>
<tr>
<td>HSS</td>
<td>A500-GR.B</td>
<td>42</td>
</tr>
<tr>
<td>Channels, Tees, Angles, Bars, Plates</td>
<td>A36</td>
<td>36</td>
</tr>
<tr>
<td>Reinforcing Steel</td>
<td>GR. 60</td>
<td>60</td>
</tr>
</tbody>
</table>

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.

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.

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

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](image-url)
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.
Disadvantages

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.
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.

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 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.

Advantages

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.

Disadvantages

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 -

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

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

Dead Loads
Typical Floor
\[ \frac{8}{12} \times 150 \text{ psf} = 100 \text{ psf} \]
Superimposed DL 8 psf (Mech, Elec, Ceiling, Lighting etc)
Through online research (www.buildernewspaper.com)
Green roof type 30-35 psf
9th Floor
\[ \frac{10}{12} \times 150 \text{ psf} = 125 \text{ psf} \]
High roof
\[ \frac{4}{12} \times 150 \text{ psf} = 112.5 \text{ psf} \]
High Elevation Mech equip \[ \frac{4}{10} \times 150 \text{ psf} = 50 \text{ psf} \]

Snow Loads - ASCE 7-05 ch 4 - Flat roof - \( R_0 = 0.76 \times C_2 \times I_0 \times p \)

From Eq. 7-1 \( R_0 = 25 \text{ psf} \)

From table 7-3 \( C_2 = 1.0 \) (All other structures)

Occupancy Category II from table 1-1
\( I = 1.0 \) from table 7-1

Site Class C from geotechnical report
Fully Exposed roof
\( C_3 = 1.0 \) from table 7-2

\[ P_1 = 0.76 \times (1.0) \times (1.0) \times 25 \]
\[ = 19.75 \text{ psf} \approx 20 \text{ psf} \]

Wind load
\( V = 13(25) + 14 \leq 300 \)
\[ = 300 \times 25 \text{ psf} \]

Leeward
\( L = 10.3 \times 3.9 \times 3.9 \)
\[ = 16 \text{ psf} \leq 0.93 \times 3.9 \]

Leeward Leeward
\( L = 10.3 \times 3.9 \times 3.9 \)
\[ = 16 \text{ psf} \leq 0.93 \times 3.9 \]

\( h_b = 16 \text{ psf} \times 3.9^2 = 132 \text{ psf} \)

\( W = 13(25) + 14 \leq 300 \)
\[ = 300 \times 25 \text{ psf} \]

Wind load
\( V = 2.41 \times 1.3^3 = 108 \)
\( h_b = 3.2 \times 1.3 = 1.8 \)

Max snow load = 67 + 16
\[ = 83 \text{ psf} \]
Appendix B – Post Tensioned System

Preliminary Thresses
\[ V_A = 115 \quad \text{and} \quad L = \frac{V}{A} = \frac{115}{12 \times 12} = 0.95 \text{ in.} \]

Dead Load - (3/12)(1000 lb)\( \text{ft} \)\( \times \) 1000\( \text{lb} \)\( \text{ft} \)
\[ 5\text{ in.} / 12 \times 1000 \text{lb} = 125\text{ lb ft} \]

Assume target load balancing of 75\% of DL
\[ 0.75 \times 125 = 93.75 \text{ lb per in.} \]

Int. support

Int. mid-span

Weight to balance
\[ 81 \text{ lb} (9 + \frac{3}{2}) = 1661 \text{ lb} / \text{in.} = 166 \text{ kN} / \text{m} \]

Force req in tendon
\[ P = \frac{W L}{A} \]
\[ = 166 \times (25) \times \frac{V}{A} \]
\[ = 383 \text{ kN} \]

Tendon properties
- Wire strand, A-416
- \( \frac{1}{2} \) in. diameter tendon
- Area - 1.53 in.²
- Ultimate strength - 270 ksi
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Estimate loads @ 15 ksi:

\[ f_{se} = 0.7(270) - 15 = 194 \text{ ksi} \]

\[ P_{ext} = 0.15(194) = 29.1 \text{ ksi/tendon} \leq 27 \text{ ksi/tendon designed} \]

\[ \text{Number of tendons to balance load} = \frac{29.1}{26.6} = 11.1 \approx 15 \text{ tendons} \]

\[ P_{loss} = 0.15(26.6) = 3.99 \text{ ksi} \]

3.99 ksi > 2.0 ksi designed

This could be due to smaller self weight, balance, or the fact there is another 26.6 ksi tendon running // 15' away to make up for this difference.

Balanced load adjustment:

\[ \frac{3.99}{1.66} = 2.4 \text{ ksi/tendon} \]

\[ \frac{P_{loss}}{A} = \frac{3.99 \text{ ksi}}{(0.75 + 0.25)(15')} > 125 \text{ psi min in V} \]

\[ \leq 300 \text{ psi max} \]
Check slab stresses

\[ W_{DL} = 108 \text{ psi} \left( 9.1 + \frac{23}{2} \right) = 2.21 \times 10^4 \text{ kips} \]

\[ M_0 = 2.21 \left( \frac{23}{8} \right) = 7.3'k \]

\[ W_{LL} = 35.2 \text{ psi} \left( 9.1 + \frac{23}{2} \right) = 1.12 \times 10^4 \text{ kips} \]

\[ M_0 = 88'k \]

\[ W_1 = -1.78'k \]

\[ M_0 = 133'k \]

Stresses in Actually: After packing

\[ f_{\text{emp}} = \left( M_0 + M_{\text{w}} \right) / s = 17.4 \text{ ksi} \]

\[ s = 0.13 \text{ ksi} \left( 108 + 133 \right) / 81 / 4 \]

\[ = 862.4 \text{ ksi} \]

\[ f_{\text{emp}} = (M_0 + M_{\text{w}}) / s = 17.4 \text{ ksi} \]

\[ s = 0.13 \text{ psi} \left( 108 + 133 \right) / 81 / 4 \]

\[ = 862.4 \text{ ksi} \]

\[ f_{\text{emp}} = (M_0 + M_{\text{w}}) / s = 17.4 \text{ ksi} \]

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\[ f_{\text{emp}} = (M_0 + M_{\text{w}}) / s = 17.4 \text{ ksi} \]

\[ s = 0.13 \text{ psi} \left( 108 + 133 \right) / 81 / 4 \]

\[ = 862.4 \text{ ksi} \]
Stresses & Service Load

\[ f_{\text{top}} = \frac{-M_{\text{top}} - M_{\text{left}} + M_{\text{right}}}{5} \sigma - \frac{F}{A} \]
\[ = \frac{-55 - 28 + 130}{5} \sigma - 60 \]
\[ = 385 \text{ psi} < \frac{145}{500} \times 2250 \text{ psi} \checkmark \]
\[ f_{\text{bot}} = \frac{M_{\text{top}} + M_{\text{left}} - M_{\text{right}}}{5} \sigma - \frac{F}{A} \]
\[ = \frac{55 + 28 - 130}{5} \sigma - 20 \]
\[ = 20 \text{ psi} < 2250 \text{ psi} \checkmark \]

Support
\[ f_{\text{top}} = \frac{-129 + 66 + 100}{2624 - 203} \]
\[ = 637 \text{ psi} > 2250 \text{ psi} \checkmark \]
\[ f_{\text{bot}} = \frac{129 + 66 - 100}{2624 - 203} \]
\[ = 221 \text{ psi} < \sqrt{2000 \times 424} \text{ psi} \checkmark \]
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

<table>
<thead>
<tr>
<th>PHYSICAL PROPERTIES</th>
</tr>
</thead>
<tbody>
<tr>
<td>Composite Section</td>
</tr>
<tr>
<td>$A_c = 301 \text{ in}^2$</td>
</tr>
<tr>
<td>$t_c = 3134 \text{ in}^4$</td>
</tr>
<tr>
<td>$V_{wc} = 5.09 \text{ in}$</td>
</tr>
<tr>
<td>$V_{wc} = 2.91 \text{ in}$</td>
</tr>
<tr>
<td>$V_{wc} = 4.91 \text{ in}$</td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>

DESIGN DATA

1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2" @ 270 K 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 $10\sqrt{f_c} = 775 \text{ PSI}$
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 2006 & 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.

<table>
<thead>
<tr>
<th>SAFE SUPERIMPOSED SERVICE LOADS</th>
<th>IBC 2006 &amp; ACI 318-05 (1.2 D + 1.6 L)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strand Pattern</td>
<td>SPAN (FEET)</td>
</tr>
<tr>
<td>4 - 1/2&quot;Ø</td>
<td>LOAD (PSF)</td>
</tr>
<tr>
<td>LOAD (PSF)</td>
<td>280</td>
</tr>
<tr>
<td>6 - 1/2&quot;Ø</td>
<td>LOAD (PSF)</td>
</tr>
</tbody>
</table>

MITTERHOUSE CONCRETE PRODUCTS

This table is for simple spans and uniform loads. Design data for any of these special conditions is available or request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, conditions, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 1 Hour & 0 Minute fire resistance rating.

8SF1.0T

NITTERHOUSE CONCRETE PRODUCTS
2855 Molly Pitcher Hwy, South, Box N
Chambersburg, PA 17202-4203
717-267-4505 Fax 717-267-4518

11/03/08
Brad Oliver

Hollow core 1

\[ w_o = 1.2(5) + (1.6)(100) = 170 \text{ psf} < 190 \text{ psf allowable by table}. \]

*Dead load of plant included in table.

- 2' section, Hollowcore is able to cut 4' planks in half.

- Weight > 61.25 psf

Girder to support planks

\[ W_o = 1.2(6)(25+8) + 1.6(100) = 243 \text{ psf} \]

\[ W_t = 2.43(18') = 43.7 \text{ kips} \]

\[ M_o = W_o(25)/8 = 5311 \text{ kip-ft} \]

Unbraced length = 18'

Using table 3-10

Try \( W_{12} \times 72 \) - chosen to limit depth

\[ I = 597 \text{ in}^4 \]

LL defl -

\[ \Delta u = \frac{5(11925)(18')(1224)}{284(24000)(597)} = .341" \]

\[ \Delta u_{int} = \frac{56}{2840} = \frac{19}{1224} = .16 > .341" \]

TL defl -

\[ \Delta u = \frac{5(.1008 + .00184)(25)(18')(1728)}{284(24000)(597)} = .58" \]

\[ \Delta u_{int} = \frac{56}{2840} = \frac{19}{1224} = .16 > .58" \]

Use \( W_{12} \times 72 \) girder in short direction to support planks.
Appendix D – Composite Steel Beam with slab

Going to analyze 14-15 at B-E Bay. It is one of the largest with the largest loads. Most critical case.

Assumptions:
- Assume beams span 91', steel column, NW 5000 pl. care
- Corridor has no partition load, only 100 pl. LL 4.5kW weight
- Will be using LC 1.2D + 1.6L

2.5' composite deck w/2' topping, gage 19

SDT Max unboxed con = 9'4" + 9'6" + 10" = 29'6" - No shoring necessary.

@ 9'4"
- LL allow = 157 psf > 100 psf
- Weight = 33 psf con + 2.49 psf deck

This deck sufficiently gives 2 hour fire rating. UL design: D7113±

Beam:
- SDS: 8' + 3'3" + 2'49" + self = 43.6 psf (total)
- Live = 100 psf (No partition 1/2 pl. care)

b' < 5sin1/8 = 25x32/8 = 37.5' = control

b' < 1/8 dat to Adj beam = 5/8 (9x2) = 54'

best = 75'
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\[ M_a = \frac{w_{dir} L^2}{8} = \frac{1940 \times (25)^2}{8 \times (100)} = 149 \text{ kN} \]

Assume fully composite: \( a = 1'' \implies y_a = \frac{3.25 - 0.25}{2} = 1.5'' \)

Try W10x19

\[ \frac{60}{2} \text{ kN/m} \]

Try W10x22

\[ \frac{60}{2} \text{ kN/m} \]

Additional Man. Spt. Weight: \[ \frac{2.5 \times 12}{8} \text{ kN/m} \]

Additional Man. Spt. W: \[ \frac{2.5 \times 2}{8} \text{ kN/m} \]

\[ \frac{150.8}{157} \text{ kN/m} \]

\[ Q_a = \frac{5 \times (40)}{8} = 30.9 \text{ kN/m} \]

\[ = 1.6 \times 144 = 230 \text{ kN/m} \]

\[ = 14.6 \text{ kN/m} \]

\[ = 12.1 \text{ kN/m} \]

For W10x19, \[ \frac{60}{17.2} = 3.46 \text{ Stud/bm} \leq 25 \text{ Stud/bm} \]

For W10x22, \[ \frac{60}{17.2} = 3.46 \text{ Stud/bm} \leq 25 \text{ Stud/bm} \]

Total weight for W10x19: \[ 19.15 \text{ m} \times 17.2 \text{ m} = 329 \text{ kN} \]

W10x22: \[ 19.15 \text{ m} \times 22 \text{ m} = 422 \text{ kN} \]

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

Check Assumption

\[ a = \frac{50}{5 \times 17.2} = 0.03 \text{ in} \]

\[ y = 2'' \text{ is OK} \]
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Check Beam for Unshored strength:

\[ W_{10\text{cm}} = 0.694 \times \left( \frac{1}{8} \right) = 81.1k \]

Check Deck Width:

\[ b = 60 + 2.5 = 62.5 \]

Considering only DL: \[ 1.0 \times (53 + 2.54) = 1.0 \times (55.54) \]

Construction Load:

\[ 1.2 \times (53 + 2.54) = 1.2 \times (55.54) \]

Check Beam:

\[ M_u = \frac{W_u \times L^2}{8} = \frac{69.4 \times (25)^2}{8} = 54.1k < 81.1k \]

Check Wet Concrete Deflection:

Service Loads:

\[ W_{f} = 33.6 \times 25.75^{0.9} = 1.76k \]

\[ I_x = \frac{I_{beam}}{10} = \frac{960k}{10} = 96k \]

\[ \Delta_{wc} = \frac{5.11}{384E} = \frac{5.11}{384(20000)(9063)} = 0.06" \]

\[ \Delta_{min} = 0.02 = 25.125/200 = 0.125" > 0.06" \]

Check LL: \[ \Delta_{ll} = 0.02 = 25.125/200 = 0.125" > 0.06" \]

Use Unshored Beam now be Full Comp. Action

\[ I_{beam} = 25.125 \text{ by table 3.20 AISC} \]

\[ \Delta_{u} = \frac{5W_{u}L}{384E} = \frac{5.11}{384(20000)(9063)} = 0.06" \]

\[ \Delta_{min} = 0.02 = 25.125/200 = 0.125" > 0.06" \]

Check Assumption:

\[ c = \frac{169}{3.20(1.7)} = 0.53" > 0.1" \text{ in ok} \]

Unshored Strength:

\[ M_{up} = 97.5k \]

\[ DL = 1.4 \times (55.54 + 2.54) = 1.4 \times (58.08) \]

\[ I_{beam} = 1.2 \times (55.54 + 2.54) = 1.2 \times (58.08) \]

\[ M_{u} = 69.4 \times (25) = 54.1k < 97.5k \]

Wet Core defl:

Service:

\[ W_{f} = 33.6 \times 25.75^{0.9} = 1.76k \]

\[ I_x = 118 \text{ in}^4 \]

\[ \Delta_{wc} = \frac{5.11 \times (25)^{0.9}}{384(20000)(9063)} = 0.08" < 0.1" \text{ Allowable} \]

Check LL defl:

\[ I_x = 118 \text{ in}^4 \]

\[ 5.11 \times (25)^{0.9} = 118 \text{ in}^4 \]

Calc. Assumption 20% PNA 9 in top of floor = 20 \times 32.4k

\[ a = \frac{20 \times 32.4}{3.20(1.5)} = 1" \]

\[ 0.125" > 3.5" \text{ and 3.0"} \text{ ok} \]

W10 22 4/10 stud/beam
With validity of new assumption, moment capacity increases to 197 k
# I_m = 236 in^4. More aggressive design.
\[
\Delta_m = \frac{5(9.1) (25)}{30000} = 0.022" < 0.033" \checkmark
\]

Clock \Delta_m:
\[
\Delta_m = \frac{5(1.41V (25))}{300} = 1.79"
\]

\[
\Delta_m, \min = \frac{1.25}{2} = 0.625" < 1.25" x 0.5" = 1.25" x 0.5"
\]

Close, you could camber beam to make this issue. Input is also a conservative calculation.

Use W10 x 22 9/10 studs for beams in leg direction.

\[P = 1.2(33+2.4+8) + 1.2(22) + 1.6(100)(9) = 48.4K, \text{ divide by } 2 \frac{1}{2} \text{ beam from each side.}\]

\[U_2 = 1.2(\text{Self Weight}) - M = \frac{\pi^2 f}{8} = 2.18 K\]

Go into table with same assumption \( a = 1/2, \ v_2 = 3/4 \).

Designing to limit depth:
\[b_1 \times \pi^2 f_1 = 18\times \frac{3}{2} = 27" \checkmark\]
\[\leq \frac{1}{2} \text{ in Ab beam } = (25+\frac{1}{4})\times 2 = 130"
\]
\[\text{max } = 27" \times 2 = 54"\]

Try W12 x 22, \( \phi N_2 = 225K \)
\[\phi N_2 = 225K \]
\[\alpha = \frac{225}{235} = 0.96\checkmark\]

Try W10 x 26, \( \phi N_2 = 225K \)
\[\phi N_2 = 225K \]
\[\alpha = \frac{225}{235} = 0.96\checkmark\]

W12 x 30, \( \phi N_2 = 225K \)
\[\phi N_2 = 225K \]
\[\alpha = \frac{225}{235} = 0.96\checkmark\]

Adjusted: \( \phi N = 225K \)
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\[ \text{Check for shear strength:} \]
\[ \frac{W}{k} = \frac{1.25}{1.12} = 1.12 < 1.5 \]
\[ k = 0.85 \]
\[ P_p = 735 \]

\[ C_{\text{h}} = 0.25 \left( 0.44 \frac{1}{2} \left( 1.0 (1.0) \right) \right) = 0.99 \]
\[ = 0.25 (1.15) (1.44) (0.25) = 18.3 \]

For 12\times30 \[ \frac{10}{18} = 5.5 \Rightarrow 16 \text{ stud/beam} \]

\[ \text{Check for bending strength:} \]
\[ W_{c} = 12 \times 30 \]
\[ M_P = 71.8 \]

\[ \text{Consider only DL:} \]
\[ P_{\text{cl}} = 0.441 \rho (25) \]
\[ = 0.441 (25) = 11.3 \]
\[ M_c = 0.353 (25) = 5.35 \]

\[ \text{Correct loads:} \]
\[ P_{\text{cor}} = 0.441 \rho (25) \]
\[ = 0.441 (25) = 11.3 \]
\[ M_{\text{cor}} = 0.353 (25) = 5.35 \]

\[ \text{Check for deflection:} \]
\[ W_{\text{def}} = \frac{W}{k} = 12 \times 30 \]
\[ \Delta = \frac{1}{k} = \frac{0.25 (1.15) (1.44) (0.25)}{18.3 (18.3)} = 0.124 \]

\[ \Delta_{\text{all}} = \frac{h}{240} \]
\[ = 0.124 > 0.025 \quad \checkmark \]

\[ \text{Check LL:} \]
\[ P_{\text{ll}} = 0.25 \rho (25) = 5.35 \]
\[ I_{\text{ll}} = 10.58 \]

\[ \Delta_{\text{all}} = \frac{h}{240} \]
\[ = 0.124 > 0.025 \quad \checkmark \]

\[ \text{Check beam load A:} \]
\[ P_{\text{beam}} = 1.41 \rho (25) = 35.25 \]
\[ \Delta_{\text{all}} = \frac{h}{240} \]

\[ \Delta_{\text{all}} = 0.124 > 0.025 \quad \checkmark \]

\[ \text{Use} \quad W_{12} \times 30 \quad 9/16 \text{ stud for girders} \]
Appendix E – Flat Plate System

Preliminary slab thickness

Using table 9.5c to
limit deflection

\[ \frac{L_o}{f_{tk}} \]

\[ L_o = 25' - \frac{16'}{10} = \frac{25'}{10} = 2.5' \]

23.3\(\frac{kip}{ft}\) = 9.3". Try 9.5" slab
Assume 5 ksi concrete at 5 bars.

\[ W_o = (9.5"/2) (150 psi) = 119\text{ kips} \]

Total

\[ W_i = 100\text{ kips} \]

Worst case (corridor)

\[ W_o = 1.2 (119.48) + 10 (100) = 312\text{ kips} \]

\[ M_o = W_o \frac{L_o^2}{4} \]

Frame A - \[ M_o = 312 (14')(23.5')/8 = 381'k \]

Frame B - \[ M_o = 312 (20')(18')/8 = 316'k \]

% to Col Strip
- Max + moment 100
- 75% of max + moment 67.5

\[ \frac{75\text{ kips}}{22\text{ kips}} \]
### Frame A

<table>
<thead>
<tr>
<th>Description</th>
<th>Ext. Span</th>
<th>Int. Span</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Min. M.</td>
<td>0.79</td>
<td>0.66</td>
</tr>
<tr>
<td>2. Width M</td>
<td>130</td>
<td>130</td>
</tr>
<tr>
<td>3. Eff. d</td>
<td>7.43</td>
<td>7.43</td>
</tr>
<tr>
<td>4. M. / N.</td>
<td>8.8</td>
<td>8.8</td>
</tr>
<tr>
<td>5. R = M. / N</td>
<td>12.4</td>
<td>12.4</td>
</tr>
<tr>
<td>7. ( a_p )</td>
<td>1.54</td>
<td>1.54</td>
</tr>
<tr>
<td>8. A.</td>
<td>2.05</td>
<td>2.05</td>
</tr>
<tr>
<td>9. N.</td>
<td>2.05</td>
<td>2.05</td>
</tr>
<tr>
<td>10. Min.</td>
<td>7</td>
<td>7</td>
</tr>
</tbody>
</table>

### Frame B

<table>
<thead>
<tr>
<th>Description</th>
<th>Ext. Span</th>
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</tr>
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<tbody>
<tr>
<td>1. Min. M.</td>
<td>0.79</td>
<td>0.66</td>
</tr>
<tr>
<td>2. Width M</td>
<td>130</td>
<td>130</td>
</tr>
<tr>
<td>3. Eff. d</td>
<td>7.43</td>
<td>7.43</td>
</tr>
<tr>
<td>4. M. / N.</td>
<td>8.8</td>
<td>8.8</td>
</tr>
<tr>
<td>5. R = M. / N</td>
<td>12.4</td>
<td>12.4</td>
</tr>
<tr>
<td>7. ( a_p )</td>
<td>1.54</td>
<td>1.54</td>
</tr>
<tr>
<td>8. A.</td>
<td>2.05</td>
<td>2.05</td>
</tr>
<tr>
<td>9. N.</td>
<td>2.05</td>
<td>2.05</td>
</tr>
<tr>
<td>10. Min.</td>
<td>7</td>
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</tbody>
</table>

Design Reinforcement Frame A: M.5.
### Design of Slab Reinforcement Frame B C.S.

<table>
<thead>
<tr>
<th>Description</th>
<th>Ext. Span</th>
<th>Int. Span</th>
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</thead>
<tbody>
<tr>
<td>1. Min. M</td>
<td>M_min</td>
<td>M_min</td>
</tr>
<tr>
<td>2. U_M 6 5</td>
<td>150</td>
<td>150</td>
</tr>
<tr>
<td>3. A</td>
<td>7.51</td>
<td>7.51</td>
</tr>
<tr>
<td>4. R = M_i 6 5</td>
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<td>1.15</td>
</tr>
<tr>
<td>5. g_x = M_i 6 5</td>
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<td>0.005</td>
</tr>
<tr>
<td>6. g_y = M_i 6 5</td>
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<td>0.002</td>
</tr>
<tr>
<td>7. A_A = M_i 6 5</td>
<td>2.34</td>
<td>2.34</td>
</tr>
<tr>
<td>8. A_N = M_i 6 5</td>
<td>9.40</td>
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</tr>
<tr>
<td>9. N_1</td>
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<tr>
<td>10. N_2</td>
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</table>

**Design Reinforcement Frame B C.S.**

<table>
<thead>
<tr>
<th>Description</th>
<th>M_S</th>
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<tbody>
<tr>
<td>1. Min. M</td>
<td>o</td>
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<tr>
<td>2. U_M 6 5</td>
<td>150</td>
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<tr>
<td>3. A</td>
<td>7.51</td>
</tr>
<tr>
<td>4. R = M_i 6 5</td>
<td>1.15</td>
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<tr>
<td>5. g_x = M_i 6 5</td>
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<td>6. g_y = M_i 6 5</td>
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<tr>
<td>7. A_A = M_i 6 5</td>
<td>2.34</td>
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<tr>
<td>8. A_N = M_i 6 5</td>
<td>9.40</td>
</tr>
<tr>
<td>9. N_1</td>
<td>1.34</td>
</tr>
<tr>
<td>10. N_2</td>
<td>7.87</td>
</tr>
</tbody>
</table>

**Calculation of deflection not necessary.**
Punching Shear Check

\[ \frac{d_h}{10} = \frac{8.45 + 7.81}{2} = 8.12'' \]
\[ d_h = 8.12'' \]
\[ d_h = 4.06'' \]

Length = 30'' + 6.12 = 36.12''
Width = 20'' + 8.12 = 28.12''

Perimeter = 3(20) = 60.5''

\[ \beta' = \frac{300}{20} = 1.5 < 2 \]
\[ V_c = \frac{4}{1.5} (125)(700) \]
\[ V_c = 234 \text{ kips} \]

\[ V_o = W_o A \]
\[ W_o = 12(125)(0.4) + 16(100) \]
\[ W_o = 212 \text{ kips} \]

\[ V_o = 312(25.3(115.75)) \]
\[ V_o = 140 \text{ kip} \]

\[ \phi V_o = 0.75(312) > V_o \]
\[ \phi V_o = 234 > 140 \text{ kip} \]
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


Further Information on hollow core slabs

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

2-way Post tension slab design aid