Executive Summary

The purpose of this report is to analyze, design, and evaluate four alternative floor systems in the building. The report will give preliminary sizes of members, depths, and other pertinent information about each system. Figures from Handbooks are present as well as hand calculations and tables.

The four alternative floor systems that I chose were Hollow Core Planking on Steel Supports, One-Way Concrete Joist System, Two-Way Flat Plate System, and finally Hollow Core Planking on Concrete Beams and Masonry Bearing Walls.

Through calculations and tables, I have decided that all systems, including the original system require further investigation. The only system that I am remotely unsure about is the Hollow Core on Masonry Bearing Walls. It is difficult to tell if placing bearing walls in place of the columns will result in a change in the architecture or physical spaces in the building. Therefore, I will still investigate further into the system.
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1. **Introduction**

Completed in 1997, standing eight stories above ground, and encompassing 77,000 square feet within its walls, Vickroy Hall provides Living and Learning areas for up to 280 upper class students of Duquesne University. The living quarters are suites with two double rooms and an attached private bathroom. The learning quarters are multiple meeting rooms complete with tables and lounge chairs.

The building sports multiple protrusions to give it interesting dimension when compared to the buildings around it. There are also two story columns on the exterior of the building to give it a ‘floating’ look and add to its prestigious façade. The first two floors are atypical due to the columns and the need for a lobby, large meeting rooms, and offices. However, the floors above take on a more typical structure.

This report is designed to take a closer look at the typical floor structure of Vickroy Hall and undertake the task of designing alternative systems that could have worked in the building. It will evaluate four alternative floor systems and compare them to the original system.

2. **The Current System**

2.1 **Current System in Drawings**

The main structural system consists of structural steel members including W-shapes and C-channels. The W-shapes are the framing for typical members and the C-channels provide support for the cantilevers and other protrusions. They are usually oriented perpendicular to the other framing members. The main members extending from column to column are detailed as moment connections. These moment connections are either classified as a wind moment connections or a moment resisting connections. The typical floor plan generally calls for W12 to W16’s. (See partial framing plan below or Figure 1 in the Appendix which illustrates the typical full original framing plan.)
The floor system is a non-composite metal and concrete deck. On a typical floor, the deck is 2” – 20 gage corrugation with 3-1/4” light weight concrete and 6x6 – W2.9 x W 2.9 welded wire fabric. The deck was to be welded to the supporting structural member. (See photo below)

2.2 Analysis

My analysis used the typical floor system to design the typical members of the system. I did not take into account wind or seismic forces, but designed strictly for gravity loads. The moment connections, as they exist, were taken into account as fixed-fixed beams when designing. All other connections were assumed to be simply supported. My loads were revised from Technical Assignment 1 to reflect IBC 2003 instead of BOCA 1993. With this revision, some of my members were the same as the original, but most differed. (See Framing Plan) Calculations for the current system can be found in the Appendix as Figures 6-9. The typical member sizes of my analysis versus that of the original are shown in the table below.
<table>
<thead>
<tr>
<th>Typical Beam Name (Current/Analyzed)</th>
<th>Current Size</th>
<th>Analyzed Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>Typical Beam 4/Typical Beam 1</td>
<td>W 10x15</td>
<td>W 10x15</td>
</tr>
<tr>
<td>Typical Beam 5/Typical Beam 2</td>
<td>W 18x35</td>
<td>W 10x15</td>
</tr>
<tr>
<td>Typical Beam 2/Typical Beam 3</td>
<td>W 21x44</td>
<td>W 10x15</td>
</tr>
<tr>
<td>Typical Beam 1/Typical Beam 4</td>
<td>W 14x22</td>
<td>W 12x22</td>
</tr>
<tr>
<td>Typical Beam 3/Typical Beam 5</td>
<td>W 12x19</td>
<td>W 12x19</td>
</tr>
<tr>
<td>Typical Beam 14/Typical Girder 1</td>
<td>W 21x62</td>
<td>W 18x71</td>
</tr>
<tr>
<td>Typical Beam 24/Typical Girder 2</td>
<td>W 21x62</td>
<td>W 21x73</td>
</tr>
</tbody>
</table>

2.3 Evaluation of the System

This evaluation will highlight the pros and cons of the current system.

**Current System Pros**
- Has withstood the test of time
- Steel is constructed relatively fast
- Building did not show stress cracking in masonry facade
- Relatively light system
- Plenty of plenum space between floors for MEP

**Current System Cons**
- Moment Frames are expensive
- Moment Frames take longer constructability time
- No shear walls – moment connections take all of the wind and seismic loads

3. Alternative System 1: Hollow Core Planking on Steel Supports

3.1 The System

Hollow core planking is a type of precast concrete system that can be constructed a multitude of ways. The planks are cast in long lengths and cut to size to accommodate the project. The hollow cores can be filled with grout for added strength if need be. A topping slab may also be added for either structural purposes or strictly leveling. For this system, the precast will be supported by structural steel members. The system I analyzed has a two-inch topping for both structural integrity and to make sure the floor is level. The Nitterhouse Concrete Products website provided free specifications and details for their typical planks and coinciding connections.

3.2 Analysis

From the Nitterhouse Concrete Products site, I chose the J952 planking system. The full PDF of the specifications can be found in the Appendix as Figure 2. The planks are four stranded 8” x 4’ wide members. The weight of each plank is 82.5 psf or 330 plf. The strength of the member is 3000 psi when it arrives on site, and the 28-day strength is 5000 psi. The allowable loads are located on the bottom of the PDF from Nitterhouse Concrete Products.
Following the original floor plan, the columns were kept the same and the orientation of the planks followed that of the original beams for simplicity at the cantilevered and protruding sections. There were four typical planks. The planks were assumed to be simply supported with minor tack welds to the supporting members. The sections where planks were not designed for were atypical, such as around the core of the building, which houses the elevator shafts and stairwells. Such analysis was beyond the scope of this report. The typical supporting members were also designed for. However, they were not designed as fixed-fixed members as in the original system, but simply supported.

Other Assumptions:
- Deflection of the planks is calculated into the allowable loads given in the Nitterhouse Concrete Products specifications
- Since there is topping on the planks, the planks will not serve as point loads on the supporting members, but as a distributed load along the length of the member.
- Supporting members are unbraced except for necessary tack welds along the length as defined by the precastor
- Only gravity was accounted for in the floor system

The typical floor plan of this system is shown below. The beams and girders are labeled as typical member (#). The summary for each member is shown in the table below. Deflection controlled for most members and the most economical size was not chosen. The member with the closest value to the controlling property was chosen.

<table>
<thead>
<tr>
<th>Typical Member</th>
<th>Typical Member Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>Typical Beam 1</td>
<td>W16x89</td>
</tr>
<tr>
<td>Typical Beam 2</td>
<td>W21x83</td>
</tr>
<tr>
<td>Typical Beam 3</td>
<td>W10x22</td>
</tr>
<tr>
<td>Typical Girder 1</td>
<td>W14x38</td>
</tr>
<tr>
<td>Typical Girder 2</td>
<td>W16x77</td>
</tr>
<tr>
<td>Typical Girder 3</td>
<td>W12x106</td>
</tr>
</tbody>
</table>

Further calculations may be found in the Appendix as Figures 10-15.
3.3 Evaluation of the System

This evaluation will highlight the pros and cons of the precast hollow core plank on structural steel members system.

<table>
<thead>
<tr>
<th>Hollow Core System Pros</th>
<th>Hollow Core System Cons</th>
</tr>
</thead>
<tbody>
<tr>
<td>o Durable system</td>
<td>o Cannot change column spacing due to façade</td>
</tr>
<tr>
<td>o Inherently Fire Resistant</td>
<td>o Much cutting will be needed to size to the original column spacing as planks come in 4’ sections</td>
</tr>
<tr>
<td>o Fast Installation</td>
<td>o While spanning in shorter direction</td>
</tr>
<tr>
<td>o Noise Attenuation</td>
<td>o Easier to construct but more pieces</td>
</tr>
<tr>
<td>o Less expensive</td>
<td>o Cantilevered sections</td>
</tr>
<tr>
<td></td>
<td>o Larger supporting member sizes</td>
</tr>
</tbody>
</table>

4. Alternate System 2: One Way Concrete Joist System with Concrete Beams

4.1 The System

One way concrete joist systems can basically take a bay that can be a two way system and force it to be a one way system. The joist system is a ‘monolithic combination of regularly spaced joists (ribs) and a thin slab of concrete cast in place to form an integral unit with the supporting beams, columns, or walls.’ The system uses forms repeatedly to construct the floor system. Many sizes and depths are available. The system was ‘developed to save dead weight and reinforcement.’

4.2 Analysis

For the analysis, I chose a form that would require the least amount of atypical formwork. Vickroy Hall has many unusual spans that do not necessarily divide easily into the forms that are generally used for the concrete joist system. The form that was analyzed was thirty inches plus another six inches for the rib. The smallest depth was chosen to allow for plenum space and to keep the typical floor to ceiling height. This height consists of a ten inch deep rib with a three inch top slab for a total depth of thirteen inches. To use the Joist tables, the unit weight per length must be known. In this case, it was 138 psf, which was determined from the current system floor load.

Other Assumptions:
- o At large openings:
  - o Will need header joists and more reinforcing
- o Deflection: all load capacities have been investigated for deflection by CRSI
- o Unequal Continuous Spans:
  - o Cause differences in moments
  - o Limitations
    - ▪ Live Load <= 3 Dead Load
- Check 40< 3(60)
  - Larger span to adjacent span shall not be greater than 20% the length of the shorter span
- Check span: 14’ to 19’-10” : 42% greater, therefore, must span in the long direction (lettered column lines)

- Material Strengths
  - f’c = 4000 psi
  - Normal weight concrete
  - fy = 60000 psi
- Loadings
- CRSI Factors loads such that: 1.4D + 1.7L
  - This will be conservative to IBC 2003

CRSI Handbook Charts for Concrete Joist Band Beams
## Technical Assignment 2

### Donna Kent

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<table>
<thead>
<tr>
<th>Clear Span</th>
<th>SPAN 1</th>
<th>SPAN 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>18'-0&quot;</td>
<td>18'-0&quot;</td>
<td>18'-0&quot;</td>
</tr>
<tr>
<td>20'-0&quot;</td>
<td>20'-0&quot;</td>
<td>20'-0&quot;</td>
</tr>
<tr>
<td>22'-0&quot;</td>
<td>22'-0&quot;</td>
<td>22'-0&quot;</td>
</tr>
<tr>
<td>24'-0&quot;</td>
<td>24'-0&quot;</td>
<td>24'-0&quot;</td>
</tr>
<tr>
<td>26'-0&quot;</td>
<td>26'-0&quot;</td>
<td>26'-0&quot;</td>
</tr>
</tbody>
</table>

### Properties for Design (Concrete at 41°F, SPAN 1)

<table>
<thead>
<tr>
<th>Property</th>
<th>Value 1</th>
<th>Value 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gross Area, SF</td>
<td>60.6</td>
<td>60.6</td>
</tr>
<tr>
<td>Steel Area, SF</td>
<td>3.2</td>
<td>3.2</td>
</tr>
<tr>
<td>E0 MPA</td>
<td>190,000</td>
<td>190,000</td>
</tr>
<tr>
<td>E0 ksi</td>
<td>27,500</td>
<td>27,500</td>
</tr>
<tr>
<td>r</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>r</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Capacity at elastic deflection, k=0.30</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Properties for Design (Concrete at 41°F, SPAN 2)

<table>
<thead>
<tr>
<th>Property</th>
<th>Value 1</th>
<th>Value 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gross Area, SF</td>
<td>60.6</td>
<td>60.6</td>
</tr>
<tr>
<td>Steel Area, SF</td>
<td>3.2</td>
<td>3.2</td>
</tr>
<tr>
<td>E0 MPA</td>
<td>190,000</td>
<td>190,000</td>
</tr>
<tr>
<td>E0 ksi</td>
<td>27,500</td>
<td>27,500</td>
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<td>10</td>
<td>10</td>
</tr>
<tr>
<td>r</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td>Capacity at elastic deflection, k=0.30</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
A typical floor plan is shown below. All of the members are the same size, but hold different loads. Their size is 12.5” x 24”. Further calculations may be found in the Appendix in Figures 16-19.
4.3 Evaluation of the System

This evaluation will highlight the pros and cons of the One way concrete joist system.

**One Way Concrete Joist System Pros**
- Easy to construct
- Shallow System
- With a drop ceiling, allows for a large plenum space beneath members
- Inherently fire resistant
- Less dead load

**One Way Concrete Joist System Cons**
- May take longer to construct
- MEP would have to drill holes or go beneath the members
- Many atypical corners and widths
- Atypical spaces may increase costs

5. Alternative System 3: Two Way Flat Plate System

5.1 The System

The two-way flat plate system that I analyzed had no drop panels or beams. I chose strictly a flat slab supported by columns. This was to increase the amount of plenum space and decrease the amount of obstructions for the MEP systems. ‘The two-way flat plate is one of the most efficient structural systems for economy.’

The formwork for the system is very easy, with little oddities, even in atypical spans and protrusions, as is the case with Vickroy Hall. I used the CRSI Handbook tables for ease of design.

5.2 Analysis

The system I designed ended up being an 8.5” thick slab supported by columns in the range of 23 inches square to 34 inches square. This worked well to stay within the current floor to floor heights and even allowed an increase in the plenum space. A chart with a summary of all of the values from the CRSI Handbook, along with the typical floor plan is shown below. For the values in context, please refer to the Appendix, Figures 3-4. For further calculations, please refer to the Appendix, Figures 20-21.

<table>
<thead>
<tr>
<th>Panel</th>
<th>Column Lines</th>
<th>Span</th>
<th>Panel Type</th>
<th>Ratio (l2/l1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Typical Panel 1</td>
<td>(2-3,G-E)(4-5,G-E) (2-3,a-B)(4-5, A-B)</td>
<td>14’x24’</td>
<td>C</td>
<td>1.71</td>
</tr>
<tr>
<td>Typical Panel 2</td>
<td>(3-4,E-G)(3-4,A-B)</td>
<td>14’x24’</td>
<td>IC</td>
<td>1.71</td>
</tr>
<tr>
<td>Typical Panel 3</td>
<td>(1-2,D-E)(5-6, D-E) (1-2,B-C)(5-6, B-C)</td>
<td>26’x19’</td>
<td>C</td>
<td>1.28</td>
</tr>
<tr>
<td>Typical Panel 4</td>
<td>(D-E, 2-5)(B-C, 2-5)</td>
<td>24’x20’</td>
<td>IC</td>
<td>1.2</td>
</tr>
<tr>
<td>Typical Panel 5</td>
<td>(C-D,1-2)(C-D, 5-6)</td>
<td>26’x19’</td>
<td>C</td>
<td>1.36</td>
</tr>
<tr>
<td>Panel</td>
<td>Col Size</td>
<td>Slab t</td>
<td>Reinforcement</td>
<td></td>
</tr>
<tr>
<td>------------------</td>
<td>----------</td>
<td>--------</td>
<td>---------------</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Column Strip</td>
<td>Middle Strip</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Top Ext +</td>
<td>Bott</td>
</tr>
<tr>
<td>Typical Panel 1</td>
<td>28”x28”</td>
<td>8.5”</td>
<td>12-#5</td>
<td>11-#6</td>
</tr>
<tr>
<td>Typical Panel 2</td>
<td>23”x23”</td>
<td>8.5”</td>
<td>-</td>
<td>10-#5</td>
</tr>
<tr>
<td>Typical Panel 3</td>
<td>34”x34”</td>
<td>8.5”</td>
<td>15-#5</td>
<td>10-#7</td>
</tr>
<tr>
<td>Typical Panel 4</td>
<td>23”x23”</td>
<td>8.5”</td>
<td>-</td>
<td>10-#5</td>
</tr>
<tr>
<td>Typical Panel 5</td>
<td>34”x34”</td>
<td>8.5”</td>
<td>15-#5</td>
<td>10-#7</td>
</tr>
</tbody>
</table>

5.3 Evaluation of the System

This evaluation will highlight the pros and cons of the Two-Way flat plate system with no drop panels or beams.

Two-Way Flat Plate System Pros
- Ease of construction
- Larger Columns
- Allows for large plenum space
- Cantilevered sections
- Economical
- Must design for shear
- Inherently fire resistant
- Punching shear is typical
- Shallow system
- If current column lines are kept, there will be eccentricities in the columns

6. Alternative System 4: Hollow Core Planks on Concrete Beams and Masonry Bearing Walls

6.1 The System

Hollow core planking is a type of precast concrete system that can be constructed a multitude of ways. The planks are cast in long lengths and cut to size to accommodate the project. The hollow cores can be filled with grout for added strength if need be. A topping slab may also be added for either structural purposes or strictly leveling. For this system, the precast will be supported by concrete beams and/or masonry bearing walls. The system I analyzed has a two-inch topping for both structural integrity and to make sure the floor is level. The Nitterhouse Concrete Products website provided free specifications and details for their typical planks and coinciding connections.

6.2 Analysis

From the Nitterhouse Concrete Products site, I chose the J952 planking system. The full PDF of the specifications can be found in the Appendix as Figure 2. The planks are four stranded 8” x 4’ wide members. The weight of each plank is 82.5 psf or 330 plf. The strength of the member is 3000 psi when it arrives on site, and the 28-day strength is 5000 psi. The allowable loads are located on the bottom of the PDF from Nitterhouse Concrete Products.
Following the original floor plan, the columns were kept the same and the orientation of the planks followed that of the original beams for simplicity at the cantilevered and protruding sections. There were four typical planks. The planks were assumed to be simply supported with minor tack welds to the supporting members. The sections where planks were not designed for were atypical, such as around the core of the building, which houses the elevator shafts and stairwells. Such analysis was beyond the scope of this report. The typical supporting members were also designed for. However, they were not designed as fixed-fixed members as in the original system, but simply supported.

Other Assumptions:
- Deflection of the planks is calculated into the allowable loads given in the Nitterhouse Concrete Products specifications
- Since there is topping on the planks, the planks will not serve as point loads on the supporting members, but as a distributed load along the length of the member.
- Supporting members are unbraced except for necessary tack welds along the length as defined by the precastor
- Only gravity was accounted for in the floor system

After reviewing the typical architectural floor plan, I determined the places where masonry bearing walls could be placed without ruining the architectural beauty of the building. (see Appendix Figure 5 for Typical Architectural Floor Plan) The reason I chose these spots were because of the large amount of empty space between the columns that were not being used in the current system. Therefore, if it was not being used in the current system, it could be used in the alternative system. The plank layout is the same as the alternative system 1 due to logistics (See framing below for bearing walls and plank layout). For further calculations, please refer to Figure 22 in the Appendix.

6.3 Evaluation of the System

This evaluation will highlight the pros and cons of the precast hollow core plank on structural steel members system.

<table>
<thead>
<tr>
<th>Hollow Core System Pros</th>
<th>Hollow Core System Cons</th>
</tr>
</thead>
<tbody>
<tr>
<td>o Durable system¹</td>
<td>o Cannot change column spacing due to facade</td>
</tr>
<tr>
<td>o Inherently Fire Resistant¹</td>
<td>o Much cutting will be needed to size to the original column spacing as planks come in 4’ sections</td>
</tr>
<tr>
<td>o Fast Installation¹</td>
<td>o While spanning in shorter direction</td>
</tr>
<tr>
<td>o Noise Attenuation¹</td>
<td>o Easier to construct but more pieces</td>
</tr>
<tr>
<td>o Less expensive</td>
<td>o Cantilevered sections</td>
</tr>
<tr>
<td>o Less empty space between columns</td>
<td>o Planks must bear on beams which in turn bear on bearing walls</td>
</tr>
<tr>
<td>o Bearing walls take the place of columns</td>
<td></td>
</tr>
</tbody>
</table>

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### 7. Overall Evaluation

<table>
<thead>
<tr>
<th>System</th>
<th>Current</th>
<th>Hollow Core on Steel</th>
<th>One-Way Joist</th>
<th>2-Way Flat Plate</th>
<th>Hollow Core on Masonry</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Features</strong></td>
<td>o Moderate member sizes</td>
<td>o Light system</td>
<td>o Light system</td>
<td>o Easiest to construct</td>
<td>o Less empty spaces</td>
</tr>
<tr>
<td></td>
<td>o Easy Constructability</td>
<td>o Easy to construct</td>
<td>o Reusable formwork</td>
<td>o Largest floor to floor height</td>
<td>o Easy to construct</td>
</tr>
<tr>
<td></td>
<td>o Withstood test of time</td>
<td>o Fire Resistant</td>
<td>o saves money</td>
<td>o Fire Resistant</td>
<td>o Fire Resistant</td>
</tr>
<tr>
<td><strong>Cost</strong></td>
<td>o Moment frames are expensive</td>
<td>o Atypical spaces</td>
<td>o Atypical spaces</td>
<td>o Atypical spaces</td>
<td>o Atypical spaces may prove to be pricey</td>
</tr>
<tr>
<td></td>
<td></td>
<td>may prove to be pricey</td>
<td>may prove to be pricey</td>
<td>may prove to be pricey</td>
<td></td>
</tr>
<tr>
<td><strong>Least Depth</strong></td>
<td>Moderate Depth</td>
<td>Largest of 5</td>
<td>Moderate Depth</td>
<td>Least Depth</td>
<td>2nd Largest</td>
</tr>
<tr>
<td><strong>Further Evaluation</strong></td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Maybe – Placement of bearing walls may become and issue architecturally</td>
</tr>
</tbody>
</table>
Appendix

Figure 1: Typical Framing Plan
Figure 2: Nitterhouse PDF for J952 Hollow Core Planking

Prestressed Concrete
8"x4' SpanDeck—U.L.—J952
(2" C.I.P. Topping)

Physical Properties
Composite
$A = 295$ in$^2$
$S_h = 488$ in$^2$
$E = 2524$ in$^4$
$S_y = 1096$ in$^2$ (At Top of SpanDeck)
$Y_h = 5.61$ in.
$S_y = 597$ in$^2$ (At Top of Topping)
$Y_h = 2.38$ in. (To Top of SpanDeck) $W_T = 330$ PLF
$Y_T = 4.38$ in. (To Top of Topping) $W_T = 82.5$ PSF

Design Data
1. Prestress Strength @ 28 Days = 5000 PSI.
2. Prestress Strength @ release = 3000 PSI.
3. Prestress Tensile = 150 PEF (Top and Web).
4. Strand = 1/2", 260 K Lo-Relaxation.
5. Composite Strength = 3000 PSI.
6. Composite Density = 150 PEF.
7. Strand Yield = 270 K.
8. Ultimate moment capacity (undeveloped): 4 - 1/2", 270K = 85.5 K,
   6 - 1/2", 270K = 124.7 K.
9. Maximum bottom flange stress is $6\sqrt{C} = 424$ PSI.
10. All superimposed load is treated as live load in the strength analysis of flexure and shear.
11. Pinned strength capacity is based on stress/strand relationships.
12. Shear values are the maximum allowable shear reinforcement is required.
13. Deflection limits were not considered when determining allowable loads in this table.
14. Load values to the left of the solid line are controlled by ultimate strength. Load values to the right are controlled by service stress.
15. All loads shown refer to allowable loads applied after the topping has hardened.

<table>
<thead>
<tr>
<th>Strand Pattern</th>
<th>Allowable Superimposed Load (PSF)</th>
<th>Span (Ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Prestress 6  1/2&quot;</td>
<td>250, 260, 270, 280, 290, 300, 310</td>
<td>10, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27, 28, 29, 30, 31, 32</td>
</tr>
<tr>
<td>Prestress 4  1/2&quot;</td>
<td>300, 310, 320, 330, 340, 350, 360</td>
<td>10, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27, 28, 29, 30, 31, 32</td>
</tr>
<tr>
<td>Prestress 6  3/4&quot;</td>
<td>350, 360, 370, 380, 390, 400, 410</td>
<td>10, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27, 28, 29, 30, 31, 32</td>
</tr>
<tr>
<td>Prestress 4  3/4&quot;</td>
<td>400, 410, 420, 430, 440, 450, 460</td>
<td>10, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22, 23, 24, 25, 26, 27, 28, 29, 30, 31, 32</td>
</tr>
</tbody>
</table>

NITTERHOUSE
CONCRETE PRODUCTS

2635 HOLLY PITCHER HWY, SOUTH, BOX N
CHAFFEEBURG, PA 17016-0013
717-267-1935 • FAX 717-267-4529

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, spans or beam openings and reverse spans.
Figure 3: Flat Plate System from CRSI
Figure 4: Flat Plate System from CRSI
Figure 5: Typical Floor showing Architectural Elements
Floor loading
  Dead load
    deck  3 psf
    reinforcing  2 psf
  lightweight concrete 38 psf = 140 psf (3.25"/12"")
  floor covering  2 psf
  ceiling  2 psf
  MEP  10 psf
  collateral  5 psf
  601 psf

* partition load not needed

Live load
  dwelling units  40 psf

* Assume fixed-fixed for girders due to moment frame detail
* Assume simply supported for most beams, fixed for 2 Typical
* Assume gravity loads only
  Revision of Technical Report 1
  - floor loading
  - end conditions for girders

\[ W_u = \frac{1}{2}(w_0 + 1.6(w_0)) \]
\[ W_u = 138 \text{ psf} \]

Typical Beam 1
  span = 19'-6"
  simply supported
  tributary width = 6'

\[ W_u = 138 \text{ psf} (c) = 8.12 \text{ plf} \]
\[ M_{max} = \frac{w_0 c^2}{8} = 21'k \]
\[ V_{max} = \frac{w_0 c}{2} = 6'k \]
\[ \Delta_{max} = \frac{1}{384} = 0.00026 \]
  \[ I_{req} = \frac{3b_c(b_c)w_0^2}{8} = \frac{1800(828)(49)}{8} = 584 \text{ in}^4 \]
  \[ s = \frac{584}{(79000000)} = 76\text{ in} \]
  \[ I_{req} = 73 \text{ in}^4 \]

use W10x15
  \[ I = 1.89 > 73 \text{ in}^4 \ldots \text{ ok} \]
  \[ M_{pl} = 140 > 21'k \ldots \text{ ok} \]
  \[ N_{pl} = 62 > 6'k \ldots \text{ ok} \]
Figure 7: Current System Calculations Page 2

Typical Beam 2
Span 14'-0"
fixed-fixed
Tributary Width 6'

\[ W_u = 828 \text{ plf} \]

\[ M_{\text{max}} = \frac{W_u L^2}{12} = 14\, k \]

\[ V_{\text{max}} = \frac{W_u L}{2} = 6\, k \]

\[ \Delta_{\text{max}} = \frac{4W_u L^2}{3600} = 0.004\, \text{in} \]

\[ I_{\text{req}} = \frac{12 t^4}{12t^4} = 360(828)(14)^3 (10) \]

\[ I_{\text{req}} = 11,100\, \text{in}^4 \]

Use: W10x15

\[ I = 48.9\, \text{in}^4 > 11\, \text{in}^4 \quad \therefore \text{ok} \]

\[ \Delta_{\text{req}} = 60.0\, \text{in} > 14\, \text{in} \quad \therefore \text{ok} \]

\[ V_n = 52\, \text{k} > 6\, \text{k} \quad \therefore \text{ok} \]

Typical Beam 3
Span 19'-10"
fixed-fixed
Tributary Width 6'

\[ W_u = 828 \text{ plf} \]

\[ M_{\text{max}} = 27\, k \]

\[ V_{\text{max}} = 8\, k \]

\[ \Delta_{\text{max}} = \frac{31}{10} \text{in}^4 \]

Use: W10x15

\[ I = 48.9\, \text{in}^4 > 31\, \text{in}^4 \quad \therefore \text{ok} \]

\[ \Delta_{\text{req}} = 60.0\, \text{in} > 27\, \text{in} \quad \therefore \text{ok} \]

\[ V_n = 62\, \text{k} > 8\, \text{k} \quad \therefore \text{ok} \]

Typical Beam 4
Span 19'-10"
simply supported
Tributary Width 6'

\[ W_u = 828 \text{ plf} \]

\[ M_{\text{max}} = 4.1\, k \]

\[ V_{\text{max}} = 8\, k \]

\[ \Delta_{\text{max}} = \frac{150,000}{3894} = 150\, \text{in}^4 \]

Use: W12x22

\[ I = 156\, \text{in}^4 > 150\, \text{in}^4 \quad \therefore \text{ok} \]

\[ \Delta_{\text{req}} = 110\, \text{in} > 4.1\, \text{in} \quad \therefore \text{ok} \]

\[ V_n = 86.3\, \text{k} > 8\, \text{k} \quad \therefore \text{ok} \]
Typical Beam
Span: 10' 8",
Simply Supported
Wu = 8'0" plf

M\text{max} = 30'k
V\text{max} = 8'k
\Delta\text{max}:
I\text{req} = 126\text{in}^4

Use W12x19
I = 130\text{in}^4 > 126\text{in}^4 :: \text{ok}
\Omega_{mp} = 92'6'k > 36'k :: \text{ok}
\Omega_{vn} = 77'4'k > 8'k :: \text{ok}

Typical Girder
Span: 24'-0"
Fixed-Fixed

\text{by superposition: (maximums at center)}
M\text{max} = \frac{P/2}{2} + \frac{8}{3} \cdot \frac{14'k}{12'}
M\text{max} = 14'k · 2 \cdot \left( \frac{21'12' - 14'18'x}{24'} \right)
M\text{max} = 314'k

\Delta\text{max} = \frac{PL^3}{192EI} + 2\left( \frac{Pb^2x^2}{6EI} \right)^{\frac{1}{2}}

\text{W18}x71
I = 1170\text{in}^4 > 150\text{in}^4 :: \text{ok}
\Omega_{mp} = 59'3'k > 314'k :: \text{ok}
\Omega_{vn} = 24'k > 2'k :: \text{ok}
Typical Girder 2
Span 25'-4"
Fixed - Fixed

$V_{\text{max}} = 24k$

By Superposition (maximums at middle point load assumption)

$M_{\text{max}} = \frac{2Pa^2b^2}{L^3} + \left( \frac{R_1x - P_{\text{max}}b^2}{L^3} \right) + \left( \frac{R_1x - P_{\text{max}}b^2}{L^3} \right)$

$= \frac{2600(13.33)^2(12)^2}{25.33^2} + \frac{16000(13.33)(12)^2}{25.33^2} - \frac{16000(13.33)(12)^2}{25.33^2}

\Rightarrow M_{\text{max}} = 6171k$

$A_{\text{max}} = \frac{1}{3} \frac{V_{\text{max}}}{EIn} = \frac{Pa^2b^2}{EIl} + \frac{Pb^3}{EIl} \left( \frac{3a_x^2 - 3a_x x - bx^2}{EIl} \right)$

$\Rightarrow A_{\text{max}} = \frac{25.33}{600} \left( \frac{16000(13.33)^2(12)^2}{3(13.33)(12)^2} + \frac{16000(13.33)(12)^2}{3(13.33)(12)^2} - \frac{16000(13.33)(12)^2}{3(13.33)(12)^2} \right)$

$0.84 = \frac{0.046}{3} + \frac{33.33}{3}$

$0.84 I = 86.95$

$I = 104 \text{ in}^4$

Therefore, $W21 \times 73$

$I = 1600 > 104 \text{ in}^4$, \( \therefore \text{OK} \)

$\phi M_p = 0.95 \times 25.33 = 78.09$, \( \therefore \text{OK} \)

$\phi V_n = 740 > 24 k$, \( \therefore \text{OK} \)

Summary

<table>
<thead>
<tr>
<th>Typical Beam</th>
<th>Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>W10x15</td>
</tr>
<tr>
<td>2</td>
<td>W10x15</td>
</tr>
<tr>
<td>3</td>
<td>W10x15</td>
</tr>
<tr>
<td>4</td>
<td>W12x22</td>
</tr>
<tr>
<td>5</td>
<td>W12x19</td>
</tr>
<tr>
<td>Girder 1</td>
<td>W10x71</td>
</tr>
<tr>
<td>2</td>
<td>W11x73</td>
</tr>
</tbody>
</table>
Figure 10: Hollow Core On Steel Support Calculations Page 1

HOLLOW-CORE PLANKING ON STEEL

Weight: 3300 lbs or 82.5 psf
4 typical plank sizes
- Not including around elevator shafts
- 3 simply supported: spans = 14'-0", 19'-10", 18'-8"
- 1 with edge cantilever: span = 14'-0" with 1 cantilever
4' widths, 8" depth, 1' CIP concrete topping

* See Nitterhouse PDF in Appendix
* Assuming deflection is within limits on specifications

Plank 1:
- Span 14'-0" using J952
- Weight = 4620 lbs

Max deflection: \( \frac{5wL^4}{384EI} \)
- I from Nitterhouse = 2624 in^4
- Strength = 5000 psi
- \( E_c = 33 \times 10^6 \) psi where \( W_e = 193 \) psi
- \( E_c = 4.07 \times 10^6 \) psi
- \( E_c = 4.07 \times 3 \) ksi

\[ \Delta = \frac{5(148)(435000)^4}{384(4.07)(2624)} \]
= 0.012"

\[ \Delta^2/L^4 = \frac{198}{560} \]
= 0.37 > 0.012" - OK

Allowable superimposed load from Nitterhouse (Assuming 4 Strand Pattern)
- Tension = 462 psf
- Shear = 382 psf

Load (from Technical Report 2)
- TL = 1.2D + 1.6L = 1.2(70 psf) + 1.6(10 psf)
- TL = 148 psf

* Both allowable loads are greater than imposed, therefore OK
**Hollow-Core Planking**  

**Technical Report 2**  

**Donna Kent**

---

**Plank 2:**

Span: 19'-10"  
Using 1952  
\( w_p = 320 \text{ lb} \)  
8" x 4'  
4 strand

Weight = 6545 lbs

\( E = 9.07 \times 10^6 \text{ psi} \)

\( I = 2428 \times 10^{-6} \text{ in}^4 \)

Max Deflection:

\( \Delta_{\max} = 0.048 \text{ in} \)

\( \Delta_{\text{allowable}} = 0.01 \text{ in} \)

\( \Delta_{\max} < \Delta_{\text{allowable}} \) therefore ok.

Allowable loads at 20° with 4 strand pattern

- Flexure: 191 psf > 198 psf therefore ok.
- Shear: 219 psf

---

**Plank 3:**

Span: 18'-8"  
Using 1952  
8" x 4'  
4 strand

Weight = 6160 lbs

\( E = 9.07 \times 10^6 \text{ psi} \)

\( I = 2428 \times 10^{-6} \text{ in}^4 \)

Max Deflection:

\( \Delta_{\max} = 0.037 " \)

\( \Delta_{\text{allowable}} = (18+8/12)/360 " = 0.052 " \)

\( \Delta_{\max} < \Delta_{\text{allowable}} \) therefore ok.

Allowable Superimposed loads at 19° assuming 4 Strand pattern

- Flexure: 218 psf > 198 psf therefore ok.
- Shear: 255 psf
Figure 12: Hollow Core Planks on Steel Supports Page 3

Hollow Core Planking

Plank 4

Span: 12'-0" wy 4' cantilever
8"x4' 250K 4xbrd

E = 40.50 kpsi
I = 26x24 in^4

Max Deflection
Noncantilever: \[ \Delta = \frac{wL^4}{24EI} \]
\[ = \frac{100(7^2)(7(1+4.6^2) + 4x^2 - 2x^4 + 2x^6)}{24(4.6)26(24)(7^2)} \]
\[ \Delta = 0.0094" \]

Cantilever: \[ \Delta = \frac{wxL}{24E} \]
\[ = \frac{100(7^2)(4.6)(7^2) - 1x2^2 - 4x^3 - 1x4^3 - 1x6^3}{24(4.6)26(24)} \]
\[ \Delta = 0.0094" \]

Allowable:

Allowable loads ok \( \Rightarrow \) refer to plank 1

Typical Beam 1:

Beam supporting abutment of Plank 1 and 2.
Since topping will be placed on planks, the planks will not be point loads, but
instead, distributed evenly along beams.

Plank 1: self weight 4670 lbs \( \Rightarrow \) shear = 23.10 K \( \Rightarrow \) w = 578 plf
imposed load = 148 psf \( \Rightarrow \) \( w = 1034 \) plf

Plank 2: self = 6543 lbs \( \Rightarrow \) shear = 3273 lbs \( \Rightarrow \) w = 819 plf
imposed load = 148 (19+10/12)/2 \( \Rightarrow \) 1468 plf

Total load on beam: 578 + 1034 + 3273 + 819 + 1468 plf
TL = 3901 plf

Span: 24'-0" (unbraced length)
Assuming simply supported

\[ m_{max} = \frac{wL^2}{8} \]
\[ = \frac{280(24)^2}{8} \]
\[ = 2811.599 \]

\[ V_{max} = \frac{wL}{2} \]
Figure 13: Hollow Core Planks on Steel Supports Page 4

Typical Beam 2:

Plant 2 to Plant 3 abutment support

Plank 2: 228 ft pl6

Plank 3: self weight: 1600 lb ⇒ shear = 3080 lbs ⇒ w = 740 pl6
imposed: 148. (18 + 8/2) ⇒ w = 138 pl6

Total load = 4439 pl6

Span (unbraced length): 25' - 9" ⇒ 25.33' = 26'

\[ M_{\text{max}} = 375'k \]

\[ V_{\text{max}} = 58'k \]

\[ I_{\text{required}} = \frac{1800 \text{ in}^4}{384E} = \frac{1800(384)(24)^3}{384(290000000)} \]

\[ I = 18116 \text{ in}^4 \]

Try W21x85 I = 1880 in^4 > I_{\text{req}}

\[ \left(\frac{M_p}{1355} \right) = \frac{735'k}{375'k} = \frac{2}{3} \quad \text{ok} \]

\[ \left(\frac{N_n}{298'k} \right) = \frac{94.6}{298'k} = 0.31 \quad \text{ok} \]

Typical Beam 3:

Plant 4 canthlever support

will be holding shear from Plant 1 and 4' canthlever

4' weight: 4'(62.5p/lb) = 250 pl6
imposed: 148.14' = 592 pl6
Plant 1: 1619 pl6

Total load = 1619 pl6

Unbraced length = 14'
Hollow-Core Planking

Technical Report

Donna Kent

Page 5

Figure 14: Hollow Core Planks on Steel Supports

Inscribed = \( \frac{3420}{384} = \frac{3420(5)(16)}{384(29,000,000)} \)

\[ I = 10.5 \text{ in}^4 \]

Try W10x22:

\[ I = 118,320 \text{ in}^4 \]

\[ f_{pm} = 7750 \text{ psi} > f_{cm} = 30 \text{ psi} \]

\[ \phi_{wl} = 6.41 > 0.7 \]

Deepest System:

<table>
<thead>
<tr>
<th></th>
<th>db</th>
<th>drawn</th>
<th>dropping</th>
<th>total</th>
</tr>
</thead>
<tbody>
<tr>
<td>W16x89</td>
<td>16.8</td>
<td>8''</td>
<td>8''</td>
<td>26.8''</td>
</tr>
<tr>
<td>W21x83</td>
<td>21.4</td>
<td>8''</td>
<td>8''</td>
<td>31.4''</td>
</tr>
</tbody>
</table>

Current:

<table>
<thead>
<tr>
<th></th>
<th>db</th>
<th>drawn</th>
<th>dropping</th>
<th>total</th>
</tr>
</thead>
<tbody>
<tr>
<td>W14x22</td>
<td>13.9</td>
<td>2''</td>
<td>3.25''</td>
<td>18.95''</td>
</tr>
<tr>
<td>W14x24</td>
<td>13.9</td>
<td>2''</td>
<td>3.25''</td>
<td>19.15''</td>
</tr>
<tr>
<td>W14x40</td>
<td>14.6</td>
<td>2''</td>
<td>3.25''</td>
<td>21.25''</td>
</tr>
<tr>
<td>W14x51</td>
<td>15.9</td>
<td>2''</td>
<td>3.25''</td>
<td>21.15''</td>
</tr>
</tbody>
</table>

Floor to Floor: 11'-9''

Would lose at most 10'' floor to ceiling.

Gain 10'' for plenum

Girders

Typical Girder: I:

Plank 1 Area / Plank 4 Area

Tributary Area:

12' for Plank 1
12' for Plank 4 (contour goes straight to column)

Loading:

Planks: 12" x 4" x 148 psf = 280.8 psf

Span 14'-0''

Assume Simply Supported

\[ M_{max} = 147 \text{ k}\]

\[ V_{max} = 42 \text{ k} \]
Max Deflection (I required)

\[
I = \frac{3wL^2}{48E} = \frac{3(60)(12)^2(4.42)(12)^2}{58,440,000,000}\]

\[I_{req} = 384.4 \text{ in}^4\]

Try W 14 x 38

\[I = 385 \text{ in}^4\]

\[\Phi_{Mp} = 229 \text{ k} > 147 \text{ k \ \ \ : \ \ ok}\]

\[\Phi_{Vn} = 188 \text{ k} > 42 \text{ k \ \ \ : \ \ ok}\]

Typical Girder 2

Plank 2 Area

- Same loading: 6012 psf

Span: 19'-10"

\[M_{max} = 291 \text{ k}, \ \ \ \ \ \ \ \ \ V_{max} = 160 \text{ k}\]

\[\Delta_{max} (I_{req})\]

\[I = 1092 \text{ in}^4\]

Use W 116 x 77

\[I = 1120 \text{ in}^4\]

\[\Phi_{Mp} = 506 \text{ k} > 294 \text{ k \ \ \ : \ \ ok}\]

\[\Phi_{Vn} = 203 \text{ k} > 160 \text{ k \ \ \ : \ \ ok}\]

Typical Girder 3

Plank 3 Area

- Same loading: 6012 psf

- Span: 18'-8"

\[M_{max} = 262 \text{ k}, \ \ \ \ \ \ \ \ \ V_{max} = 54 \text{ k}\]

\[\Delta_{max} : I_{req} = 910 \text{ in}^4\]

Use W 12 x 48

\[I = 953 \text{ in}^4 > 910 \text{ in}^4 \ \ \ : \ \ ok\]

\[\Phi_{Mp} = 615 \text{ k} > 262 \text{ k}\]

\[\Phi_{Vn} = 212 \text{ k} > 54 \text{ k \ \ \ : \ \ ok}\]
**Unequal Continuous Spans**
- Cause differences in moments
- Limitations:
  - Live Load ≤ 3 Dead Load
  - Check 90 ≤ 3(L)
  - Larger span to adjacent span shall not be greater than 20′
- Check span in short direction
  - 19′ to 19′10″
  - by 92″ larger, therefore must span in long direction (lettered columns)

**Material Strengths**
- f’c = 4000 psi
- f’y = 60000 psi

**Loadings**
- CESI factors loads such that: 1.4D + 1.7L
- this will be conservative to 1.8L 2003
  - Analyzing wu = 138 psi

**Span 1: Between Column lines G4 to E**

![Diagram of One-Way Concrete Joist System]

- Shaded areas disregarded in this report (for simplified check)

From Page 8-21
- Use 10″ rib, 10″ deep rib + 3.0″ top slab = 13″ total depth

**End Span**
- Use #5 @ 12″ on top
- 165 #10@6″ per rib on bottom
- Weight of bars: 105 psf
- Capacity: 165 psf
- Top bars at exterior edge
  - A5 = 3(0.51 + 0.49)/6′ = 0.083 in²/ft
  - Use #4 at 12″ with 90° end hooks
Figure 17: One-Way Concrete Joist System Page 2

ONE-WAY CONCRETE JOIST

Interior Spans
Use #4 at 8.5" on top
1#2 at 1"-5" in rib
Weight = 0.95 psf
Capacity = 1.10 psf
Deflection
\[
\frac{4L}{115.60 \sqrt{L}} = \frac{40}{115.60 \sqrt{180}} < \frac{29(1.10)}{1800} = \frac{0.29"}{0.384"} = 0.76
\]

SECTION THRU RIB

Span 2: Between D&E Column Lines

From Page 8-21
Use 36" @ 6" in rib 10" deep rib + 3.0" top = 6'-0" total depth
ext. #5 @ 12" on top
1#4 in rib
Weight of bars = 1.25 psf
Capacity through interpolation = 1.03 psf
Top bar at exterior edge, \( A_0 = \frac{0.3}{10} (2.54/180) = 0.10 \text{ in}^2/ft \)
Use #4 at 12" with 90° end hooks

Interior Spans
(same as span 1)

Deflection:
Interior (Same as span 1)
Exterior (33) \( \left( \frac{1.75(1.10)}{980} \right) \leq \frac{2933(6)}{750} \)
\[
0.29" < 0.94" = 0.30
\]
Figure 18: One-Way Concrete Joist System Page 3

**Figure 18: One-Way Concrete Joist System**

**Section Through Rib**

**Span 3**: Between Column Lines C+D

Single Span due to building core = 25.4'

From page 8-42

use 30" + 4" rib 10" deep rib + 3.0" top slab = 13" total depth (6" uniformity)

- #2 in rib
- Weight of bars = 1.36
- Capacity through interpolation: 167.4

Top bar at exterior edge:

- $A_s = \frac{42(0.60)}{12} = 0.07$
- use 3 @ 12" with 90° hooks

Deflection

$$\delta = \frac{5}{384}(25.33(40)) \leq \frac{25.33(40)}{480}$$

$$0.29" < 0.41" \Rightarrow \text{ok}$$

**Beam Design: Concrete**

From Table 8-1

Concrete weight = 0.1 psf

- CRS I Beams
  - $f_c = 4,000$ psi
  - $f_y = 60,000$ psi
  - deflections taken into account
  - assume all beams simply supported due to costing

**Typical Beam 1**: Exterior Beam between Columns G+D, A+B

Span: 14'-0"

Tributary Width = 12'

$W_u = 32'psf$

$M_{max} = 18'k$

$V_{max} = 5'k$

From page 12-80 $h = 12.5''$ $b = 24''$ will hold 3.2 k/ft \(\Rightarrow\) ok
Typical Beam 2: Interior Beam Between Column lines G4-E3, A+8
Span = 14' 0"
Tributary width = 24'
Wu = 1464 plf
Mmax = 36'k
Vmax = 10'k

From Page 12-94 h = 12.5' b = 24" will hold 4.2 k/ft: ok

Typical Beam 3: Exterior Beam between Column lines D+ E, B+C
Span = 19'-10"
Tributary width = 12.17'
Wu = 745 plf
Mmax = 37'k
Vmax = 8'k

From page 12-81 w/ h = 20', h = 12.5' b = 24" will hold 1.6 k/ft: ok

Typical Beam 4: Interior Beam between Column lines D+E, B+C
Span = 19'-10"
Tributary width = 24.17'
Wu = 1474 plf
Mmax = 73'k
Vmax = 15'k

From Page 12-95 h = 185" b = 24" will hold 2.1 k/ft: ok
- from 20' span

Typical Beam 5: Exterior Beam between Column lines C+D
Span = 18'-8"
Tributary width = 12.17'
Wu = 743 plf
Mmax = 38'k
Vmax = 7'k

From Page 12-81 w/ h = 20', h = 12.5' b = 24" will hold 1.6 k/ft: ok
Figure 20: Two-Way Flat Plate System Page 1

2-WAY FLAT PLATE

CRSI
- Material Strengths
  - $f_{c} = 4000$ psi (Normal weight concrete)
  - $f_{y} = 60,000$ psi

- Design Factors
  - $1.4D + 1.7L$ - conservative for IBC 2003

- Requirements Satisfied
  - minimum thickness
  - deflection
  - crack control

- Limitations
  - 3 spans continuous in each direction ✓
  - Ratio lengths to width not exceeding 2.0 ✓
  - Successive spans not to differ more than 1/8 the span of longer ✓
  - Individual columns not to be offset more than 10% of span ✓
  - Live load ≤ 2Dead load

- Assumptions for non-square
  - When $b_{1}/b_{2}$ is close to 1.0, use values for larger span
  - No shear heads required

Design
$w_{d} = 13.8$ psf as determined from current system analysis

Typical Panel 1: Column lines (2-3, G-E), (4-5, G-E), (23, A-B), (4-5, A-B)
- Span $4'0" \times 24'0"
- $L_{e}/L_{i} = 1.71$
- Will use larger values due to large ratio
- Square Edge Panel C
- Minimum square column to hold $28\times28", 8.5\text{" slab}$

Typical Panel 2: Column lines (3-4, E-G), (3-4, A-B)
- Span $14'6" \times 24'0"
- Interior panel, 1C
- $L_{e}/L_{i} = 1.71$
- Use larger values due to large ratio
- Minimum square column to hold $23\times23", 8.5\text{" slab}$

Typical Panel 3: Column lines (1-2, D-E), (5-6, D-E), (1-2, BC), (5-6, BC)
- Span $25'4" \times 19'-10"
- Edge Panel, C
- $L_{e}/L_{i} = 1.28$
- Compare reinforcing due to closeness to 1.0
- Use $2\times20", 8.5\text{" slab}$
Figure 21: Two-Way Flat Plate System Page 2

<table>
<thead>
<tr>
<th>2-WAY FLAT PLATE</th>
<th>TECHNICAL REPORT 2</th>
<th>DONNA KENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Col Strip</td>
<td>Middle</td>
<td>Steel</td>
</tr>
<tr>
<td>Top Ex + Bolt Top Int</td>
<td>Bott. Top Int</td>
<td>C</td>
</tr>
<tr>
<td>20' 11-#4 3 9-#5 11-#6</td>
<td>9-#4 10-#4</td>
<td>2.33</td>
</tr>
<tr>
<td>20' 13-#5 10-#7 13-#8</td>
<td>12-#5 10-#5</td>
<td>3.70</td>
</tr>
</tbody>
</table>

*cannot really cut back on steel, so use 20' reinforcing
*minimum square to hold: 34"x34"

Typical Panel 4: Between column lines (BE, 2-5) + (BC, 2-5)
Span: 24' x 19' 10" = 20'
I_{2/6} = 1.2
Interior Panel, IC
compare reinforcing

<table>
<thead>
<tr>
<th>Col Strip</th>
<th>Middle</th>
<th>Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top Ex</td>
<td>Bott Ex</td>
<td>Top Ex</td>
</tr>
<tr>
<td>20' 11-#4 9-#4 10-#9 1-#4</td>
<td>2.41</td>
<td></td>
</tr>
<tr>
<td>24' 13-#7 10-#5 13-#7 12-#4</td>
<td>3.11</td>
<td></td>
</tr>
</tbody>
</table>

use 24' reinforcing
*minimum square column to hold: 23" x 23", slab = 8.5"

Typical Panel 5: Between column lines (CD, 1-2) and (CD, 5-6)
Span: 25-4" x 18-8" = 24 x 19
I_{2/6} = 1.36 + use larger reinforcement,
Exterior Panel, IC
*minimum square column to hold: 34"x34", slab = 8.5"
Figure 22: Hollow Core Planking On Masonry Bearing Walls and Concrete Beams

<table>
<thead>
<tr>
<th>Hollow Core Planking On Masonry + Beams</th>
<th>TECHNICAL REPORTZ</th>
<th>DONNA KENT</th>
<th>PI</th>
</tr>
</thead>
</table>

**Beam Design**

- Using designs from other system for loading

**Typical Beam 1**

- Span: 24'
- Total Load = 5036 plf
- M_max = 2.47 k
- V_max = 8.6 k

From Page 12-25 use L_n = 24', h = 16', b = 16' will hold 3.1 k/ft

**Typical Beam 2**

- Span: 26'
- Total Load = 3174 plf
- M_max = 2.68 k
- V_max = 9.1 k

From page 12-25 use L_n = 26', h = 16', b = 16' will hold 3.1 k/ft

**Typical Beam 3**

- Span 11' + 4' cantilever
- Total Load = 1614 plf
- M_max = 40 k
- V_max = 15 k

From page 12-50, use L_n = 18', h = 12', b = 10'; will hold 1.7 k/ft = ok

Concrete Beams will bear on masonry bearing walls - did not design because it is the bearing system of the floor system.