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Executive Summary

This technical report discusses and compares three alternative floor systems to the current existing floor system of The Commonwealth Medical College. This is accomplished through hand calculations performed on a typical 26’x30’ bay. A comparison in weight of the systems, depth of the systems, cost to construct each system, and several more criterions, were made. Through analysis, these criterions were used to determine whether or not each system would be a feasible alternative. The existing floor system is a 7.5” thick composite slab with W15x55 beams and W27x84 girders. The other systems designed in this report are, non-composite on joists and joists girders, one-way slab on concrete beams, and precast plank on wide flange girders.

It was found that the existing, composite system, is the second least expensive to construct, and also the second lightest. It has a depth of 34.4”, a weight of 84 psf, and cost around $25.04 per square foot. The light weight and the ease of construction were believed to be the reasons that the composite system was chosen for the TCMC.

The non-composite with joists and joists girders system was found to be the best alternative since it has a smaller depth and weigh a lot less. However, it does cost $26.57 per square foot, $1.53 per square foot more than the composite system. It is also easy to construct since there is no shear studs involved. Overall, it was found to be an adequate alternative system.

The one-way slab on concrete beams was found to be an excellent alternative since it cost significantly less than the composite system. It does weigh around 20% more, causing a need to increase the size of the foundations. A 6” thick slab with 13.5”x22.5” beams and 15”x25.5” girders resulted from this one-way concrete design.

The precast plank on wide flange girders is an expensive alternative, at $32.9 per square foot. This is the largest setback for this system. Nitterhouse Concrete Products was the selected manufacturer for the precast plank. Using their product information sheet, an 8” thick hollow core with a 2” topping and a 2 hour fire rating was chosen. These are supported by W27x84 girders. This system has the largest structural depth, 34.7”, and this system is the second heaviest. The extreme fabrication and construction difficulties in trying to reduce the structural depth make this system hard to construct. Out of the four systems, the precast plank on wide flange girders is the worst system to use.
Building Introduction

The Commonwealth Medical College (TCMC), also known as The Medical Sciences Building (MSB), is a medical school located in the heart of Scranton, PA. Costing over $120 million, this four story building, with an additional penthouse on the roof, was completed in April, 2011. The architecture was intended to complement the existing schools and hospitals in the surrounding area. Shown in Figures 1 is the building footprint of TCMC, highlighted in yellow, and the surrounding site.

TCMC is clad in brick, stone, and glass curtain wall. The building is separated into two individual wings, west wing and east wing. The link is the lobby area that connects the two wings and it is clad largely in insulated glass units to let natural sunlight in. An additional feature is the tower which is also clad largely in glass, as shown in Figure 2. The tower, located in the east wing, is considered the main focal point of the building. The interior space of the tower is mainly corridors and small meeting rooms so the students can enjoy the view.

TCMC is a multi-use building, using all modern technology. It has a library where students go for information, Clinical Skills and Simulation Center where students learn from beyond classrooms, lecture halls that can seat up to 160 students, classrooms with Wi-Fi connections, small group meeting rooms where a team of students can work together, and a luxurious student lounge for study or relaxation. Figure 3 shows the interior lobby of TCMC. TCMC also has a garden around the link that allows the occupants to enjoy the nice green views that the city cannot offer. The building is 93 feet tall, 185,000 square feet of space, and is a composite steel framed building that utilizes moment frames for its lateral system.
Structural Overview

Design Codes

According to Sheet LS100, the building was designed to comply with:

- Mechanical   2006 International Mechanical Code
- Electrical   2005 NFPA 70/ Nation Electrical Code
- Plumbing     2006 International Plumbing Code
-            2006 International Fuel Gas Code
- Fire Protection 2006 International fire Code

All concrete work conforms to the requirements of the American Concrete Institute ACI-318-05.

Additional Code Reference from American Concrete Institute:

- ACI-211
- ACI-301
- ACI-302
- ACI-304
- ACI-305
- ACI-306
- ACI-315
- ACI-347

Regulatory Guidelines and Standards

- Accessibility  ICC/ANSI A117.1 1998
Material Properties

### Concrete

<table>
<thead>
<tr>
<th>Usage</th>
<th>Weight</th>
<th>Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MAT Slab</td>
<td>Normal</td>
<td>4000psi</td>
</tr>
<tr>
<td>Columns</td>
<td>Normal</td>
<td>4000psi</td>
</tr>
<tr>
<td>Slab on Grade</td>
<td>Normal</td>
<td>3000psi</td>
</tr>
<tr>
<td>Caisson</td>
<td>Normal</td>
<td>4000psi</td>
</tr>
<tr>
<td>Wall</td>
<td>Normal</td>
<td>4000psi</td>
</tr>
<tr>
<td>Grade Beam</td>
<td>Normal</td>
<td>4000psi</td>
</tr>
<tr>
<td>Floor Slab</td>
<td>Normal</td>
<td>4000psi</td>
</tr>
<tr>
<td>Floor Slab</td>
<td>Lightweight</td>
<td>3500psi</td>
</tr>
<tr>
<td>Floor Slab</td>
<td>Normal</td>
<td>3500psi</td>
</tr>
<tr>
<td>Lean Concrete Fill</td>
<td>Normal</td>
<td>2000psi</td>
</tr>
</tbody>
</table>

### Steel

<table>
<thead>
<tr>
<th>Type</th>
<th>Standard</th>
<th>Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforcing Bars</td>
<td>ASTM A615</td>
<td>60</td>
</tr>
<tr>
<td>Composite Floor Deck</td>
<td>ASTM A992</td>
<td>20 gauge</td>
</tr>
<tr>
<td>Roof Deck</td>
<td>ASTM A992</td>
<td>B</td>
</tr>
<tr>
<td>Galvanized Plate</td>
<td>ASTM A992</td>
<td>50</td>
</tr>
<tr>
<td>W shape Steel</td>
<td>ASTM A992</td>
<td>50</td>
</tr>
<tr>
<td>Angles</td>
<td>ASTM A992</td>
<td>50</td>
</tr>
<tr>
<td>Bolts</td>
<td>ASTM A325</td>
<td>N/A</td>
</tr>
<tr>
<td>Anchor Rods</td>
<td>ASTM F1554</td>
<td>N/A</td>
</tr>
<tr>
<td>HSS</td>
<td>ASTM A992</td>
<td>50</td>
</tr>
<tr>
<td>Welded Wire Fabric</td>
<td>ASTM A185</td>
<td>70,000psi</td>
</tr>
</tbody>
</table>

### Masonry

<table>
<thead>
<tr>
<th>Type</th>
<th>Standard</th>
<th>Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grout</td>
<td>ASTM C476</td>
<td>5000psi</td>
</tr>
<tr>
<td>Concrete Masonry Units</td>
<td>ASTM C90</td>
<td>2100psi</td>
</tr>
<tr>
<td>Mortar</td>
<td>ASTM C270</td>
<td>N/A</td>
</tr>
</tbody>
</table>

### Miscellaneous

<table>
<thead>
<tr>
<th>Type</th>
<th>Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-Shrink Grout</td>
<td>10,000psi</td>
</tr>
</tbody>
</table>

Figure 4: Tables showing materials that are used in the TCMC project
Foundations

The west wing of the TCMC is built with a mat slab foundation that is 4'-0” thick. The mat slab is designed for a soil bearing pressure of 3000psf. It is on top of a 2'-0” thick structural fill and a 4” mud slab. Figure 5 shows a typical section of the mat slab. After the mat slab, over 4’ of compacted AASHTO # 57 stone typical was placed in followed by a 5” slab on grade. Due to the confidentiality of the geotechnical report, the actual bearing capacity of the soil and the recommended type of foundations were never released.

Figure 5 A typical Section cut showing the mat slab foundation. Courtesy of Highland Associates
The east wing of the TCMC has drilled caissons ranging from 36” to 60” in diameter and is used to carry loads from grade beams to bedrock below. The typical floor slab in the east wing is 7.5” and it’s also on top of compacted AASHTO material. This can all be visualized by looking at a typical section cut from figure 6 below.

Figure 6 A section cut of a drilled caisson foundation. Courtesy of Highland Associates
Framing System

TCMC has a composite steel framed system. The sizes of the beams and columns ranged from W8x24, being the lightest, to W14x257, being the heaviest. The longest column is 44’-7” and it stopped between the third and fourth floor. An additional 48’-0” of lighter steel column is connected to this column, extending it all the way up to the penthouse.

Lateral System

The main lateral system used in TCMC consists of multiple moment frames. They are present in the west wing, east wing, and also in the link, as shown in Figure 7.1. Most frames are near the exterior wall to maximize the lateral force it can resist. The moment frames span across the entire building, from north to south and from east to west. This provides lateral resistance in each direction. The frames in the link begin on the first floor and extend to the roof, the third floor. The frames in the two wings begin on the first floor and extend to the floor of the penthouse. Figure 7.2 shows the only four frames that extend to the roof of the penthouse.

Figure 7.1 Locations of Moment Frames at TCMC. Courtesy of Highland Associates, edited by Xiao Zheng

Figure 7.2 Locations of Moment Frames at the Penthouse of TCMC. Courtesy of Highland Associates, edited by Xiao Zheng
**Roof Systems**

TCMC has over 9 different roof heights, as shown in figure 8, with the ground referenced at 0’-0”.

The link between two wings has an average roof height of 36’. The west wing goes up to 92’. The Tower, shaded in red, in the east wing goes up to 89’-4”. The rest of the east wing goes up to 81’-4” while the east wing penthouse goes up to 102’.

![Figure 8 Plan showing the different roof heights; the darker, the higher.](image)

The main roof is constructed of 1.5” type B wide rib, 22 gauge, painted roof deck supported by W-shape framing. A typical roof section cut is shown on figure 9. The typical roofing system has two layers of 2” rigid roof insulation. The walls around the roof extend 4’ higher than the steel deck so that it can be used as railings.
Figure 9 Typical roof section cut showing the roof deck. Courtesy of Highland Associates
Gravity Loads

The dead, live, and snow loads were calculated under this section for TCMC using IBC 2006, ASCE 7-05, and estimation.

Dead and Live Loads

For the dead load calculations, the materials that have the most impact on the dead weight of the building were found and then calculated. The west wing primarily uses composite 3” steel deck with concrete slab that weighs 75 psf according to Vulcraft Steel Deck catalog. The east wing and the hallway use 2” steel deck, lightweight concrete, so it only weighs 42 psf. Then W-shape Steel Beams and Columns are assumed as 15 psf that covers that whole entire building. The heaviest exterior wall is chosen and is assumed throughout the building at 1000plf. Then these weights are multiplied by the area or the length that they occupied in to get the weight in pounds. A sample of this calculation is shown for the 2nd floor of the TCMC in Figure 10 below. Doing this for every level, a weight in psf and lbs are both obtained. Then the total dead weight is found to be around 22,378 kips and will be used later in seismic calculations. A breakdown of the weight per Level is shown in Figure 11.

<table>
<thead>
<tr>
<th>Material</th>
<th>Weight (psf)</th>
<th>Area or Length</th>
<th>Total Weight (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal Weight Conc Slab with Deck</td>
<td>75 (psf)</td>
<td>20408 sf</td>
<td>1,530,600</td>
</tr>
<tr>
<td>Light Weight Conc Slab with Deck</td>
<td>42 (psf)</td>
<td>24952 sf</td>
<td>1,047,984</td>
</tr>
<tr>
<td>W-Shape Steel</td>
<td>15 (psf)</td>
<td>45360 sf</td>
<td>680,400</td>
</tr>
<tr>
<td>Exterior Walls</td>
<td>1000 (plf)</td>
<td>1418 lf</td>
<td>1,418,000</td>
</tr>
<tr>
<td><strong>Total Weight</strong></td>
<td></td>
<td></td>
<td><strong>4,676,984</strong></td>
</tr>
<tr>
<td>Total Weight per sf (close to design average dead load of 93 psf)</td>
<td></td>
<td></td>
<td><strong>103.11</strong></td>
</tr>
</tbody>
</table>

Figure 10 Total Weight per square foot of TCMC

<table>
<thead>
<tr>
<th>Level</th>
<th>Area (ft²)</th>
<th>Weight (psf)</th>
<th>Weight (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st</td>
<td>51,348.00</td>
<td>99.3</td>
<td>5099</td>
</tr>
<tr>
<td>2nd</td>
<td>45,360.00</td>
<td>103.1</td>
<td>4677</td>
</tr>
<tr>
<td>3rd</td>
<td>40,425.00</td>
<td>106.0</td>
<td>4286</td>
</tr>
<tr>
<td>4th</td>
<td>40,422.00</td>
<td>106.0</td>
<td>4286</td>
</tr>
<tr>
<td>Penthouse</td>
<td>10,337.00</td>
<td>209.2</td>
<td>2163</td>
</tr>
<tr>
<td>Roof (all level)</td>
<td>40,455.00</td>
<td>46.0</td>
<td>1867</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>228,347.00</strong></td>
<td></td>
<td><strong>22378</strong></td>
</tr>
</tbody>
</table>

Figure 11 Total Weights per Level of TCMC

The design live load for the TCMC can be found in the drawings on sheet S201A and S201B. A comparison of it to the minimum live load requirement from ASCE 7-05 can be seen on Figure 12. Notice that most design load are the same as the minimum required live load. However, some are design live loads for several locations are higher because more live loads are expected.
### Design Live Loads for West Wing

<table>
<thead>
<tr>
<th>Location</th>
<th>Design Live Load (psf)</th>
<th>ASCE 7-05 Live Load (psf)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Offices</td>
<td>50</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>Lobbies/ Corridors</td>
<td>100</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>Corridors above 1st</td>
<td>80</td>
<td>80</td>
<td></td>
</tr>
<tr>
<td>Stairs</td>
<td>100</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>Classrooms</td>
<td>40</td>
<td>40</td>
<td></td>
</tr>
<tr>
<td>Laboratories</td>
<td>100</td>
<td>60</td>
<td>Larger equipment needed in TCMC Labs</td>
</tr>
<tr>
<td>Storage Rooms</td>
<td>125</td>
<td>125</td>
<td>Light warehouse</td>
</tr>
<tr>
<td>Restrooms</td>
<td>60</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>Mechanical Room</td>
<td>150</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>Mechanical Roof</td>
<td>30</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>Roof</td>
<td>20</td>
<td>20</td>
<td>ordinary flat</td>
</tr>
<tr>
<td>Partitions</td>
<td>15</td>
<td>15</td>
<td></td>
</tr>
</tbody>
</table>

### Design Live Loads for Rest of Building

<table>
<thead>
<tr>
<th>Location</th>
<th>Design Live Load (psf)</th>
<th>ASCE 7-05 Live Load (psf)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Offices above 1st</td>
<td>65</td>
<td>50</td>
<td>Partitions and some heavier office equipment</td>
</tr>
<tr>
<td>Lobbies/ Corridors</td>
<td>100</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>Corridors above 1st</td>
<td>80</td>
<td>80</td>
<td></td>
</tr>
<tr>
<td>Stairs</td>
<td>100</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>Classrooms</td>
<td>50</td>
<td>40</td>
<td></td>
</tr>
<tr>
<td>Storage above 1st</td>
<td>125</td>
<td>125</td>
<td></td>
</tr>
<tr>
<td>Restrooms above 1st</td>
<td>75</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>Auditorium</td>
<td>100</td>
<td>100</td>
<td>if seats are fixed, then only 60psf</td>
</tr>
<tr>
<td>Bookstore</td>
<td>150</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>Lecture Halls</td>
<td>60</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>Mechanical Room</td>
<td>150</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>Library</td>
<td>75</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>1st floor offices</td>
<td>65</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>1st floor restrooms</td>
<td>75</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>Roof</td>
<td>30</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>Mechanical Roof</td>
<td>30</td>
<td>N/A</td>
<td></td>
</tr>
<tr>
<td>1st floor storage</td>
<td>125</td>
<td>100</td>
<td></td>
</tr>
</tbody>
</table>

*Figure 12 Design live load is compared to ASCE 7-05, required live load*
Snow Loads

The variables needed for snow load calculations are found on sheet S201B of the drawings. Figure 13 shows all the loads and variables that are from Sheet S201B of the structural drawing. Also, because of the many different roof heights, snow drifts can happen in over 10 different areas of the building. One of these areas is calculated and shown under Appendix A, snow load calculations. The result of that area is that the snow accumulated in the corner reached over 73 psf, more than double the amount compared to the regular flat roof amount of 30 psf. Snow drift is an important factor when designing TCMC.

<table>
<thead>
<tr>
<th>Flat Roof Snow Load Calculations</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Variable</td>
<td>Value</td>
</tr>
<tr>
<td>Ground Snow Load (P_G)</td>
<td>35 psf</td>
</tr>
<tr>
<td>Flat Roof Snow Load (P_F)</td>
<td>30 psf</td>
</tr>
<tr>
<td>Snow Exposure Factor (C_E)</td>
<td>1.0</td>
</tr>
<tr>
<td>Importance Factor (I_S)</td>
<td>1.1</td>
</tr>
<tr>
<td>Thermal Factor (C_T)</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Figure 13 Variable for snow load obtained from S201B


**Floor Systems**

The existing floor system of the TCMC is held up by W-shaped steel columns and composite steel beams. Figure 14 shows the floor plan with different bay sizes in different colors. Bay sizes are shown along with the figure, with the span required for the slab first and the span required for the girder next, match with their colors. Small bays sizes are not shown in Figure 14.

The floor is composite steel deck with concrete topping. The typical floor plan in the west wing is shown in Figure 15 along with two section cuts, Figures 16 and 17. It is a 4.5” normal weight concrete topping on a 3” lok-floor 20 gauge galvanized composite floor deck, giving it a total slab construction of 7.5”. The east wing, and the link, has different slab thickness than the west wing. They are 3.25” lightweight concrete topping on U.S.D. 2” lok-floor 20 gauge galvanized composite floor deck, making the total thickness of 5.25”.

The main focus in this technical report was to analyze the existing floor system, and then design three other alternative floor systems. All analysis and design was conducted on a 26’x30’ typical bay in the west wing. All four systems were then compared to see which systems is the best for TCMC.
Figure 16 Section cut 11 from Figure 15

Figure 17 Section cut 9 from Figure 15
Composite Slab System

The existing floor system of TCMC consists of composite slab and decking with composite steel beam and girders. Through a series of spot checks on the typical bay, the slab, beam, and girders were found to be adequate to carry the loads. Figure 18 shows the existing floor system on the typical bay. The design was spot-checked by hand calculations, which can be found in Appendix B.

Advantages

A composite system is relatively light compared to a concrete system or even a non-composite system. This makes the building lighter in design, which reduces the need of large foundations. The concrete slab resists compression and the steel beam resists tension, maximizing the efficiency of the system. A composite system also helps minimize deflections when the beam is chambered; 1.12 in total system deflection in this case. Additionally, it is easy to construct, which is preferred when a schedule is tight.

Disadvantages

Although a composite system is easy to construct, it does require a large amount of labor. The welding of shear studs to the beams required a lot of work. Also, fireproofing is required, compared to a concrete system which usually doesn’t. This also increases the cost for the system.
Analysis

The composite system used in the TCMC had a weight of 84 psf, and a depth of 34.4”. This fit right in the middle of the other three systems. Spray fireproofing was added to the beams and girders to achieve the 2-hour fire rating required. Using a steel frame system allows the building to use moment frames as a lateral system, which does not add additional weight to the building. This system cost about $25.04 per square feet. All cost figures are found in 2013, RSMeans Assemblies.

Model

For the steel model, it will just be a quick check to see the moment that was created by the loads on the typical bays. Figure 18.1 shows the typical bays used in the model. The maximum moment caused on the beam is 243.5 kip-feet, shown in Figure 18.2, which is compared to 256.5 kip-feet in the hand calculations. The model was more accurate because hand calculations tend to be more conservative. The concrete model will be more complex than this, showing more results.

Figure 18.1 ETABS model of the Composite System
Figure 18.2 ETABS model of the Composite System showing the maximum moment on the beam.
Non-Composite Slab with Joist and Joist Girders

The first alternative floor system that was investigated was a non-composite slab with joist and joist girders. Keeping the original 26’x30’ bay size, it was found that a 3C18 non-composite deck with 4.5” concrete topping is required to carry the load. The joists required for this system were 26K9 at 2’-10.66” on center, and the joist girders required were 28G9N19.4F at 30’ in length. Figure 19, on the next page, shows the typical bay used for this system for TCMC. The design was performed by hand calculations, which can be found in Appendix C.

Advantages

A non-composite deck with joist and joist girders is a very economical choice for several reasons. It is the lightest of all four systems, by more than half the weight per square foot. Joists are very light and can span greater distances than a concrete beam. This system is easy to construct and quick to erect. This is the best system that allows a large, open floor plan, which is preferred for offices and classrooms. The depth is 3.3” smaller also, so the ceiling can be higher or the building can be shorter, which will a little extra cost.
Disadvantages

This system has a total deflection of 1.45” if used in TCMC, which is more than 30 percent than the existing system. Although, it is still within the deflection limit, it may not be what the owners want. Because many joists are used, this system cost almost $2 per square feet more than the existing structure. That is close to half a million more on the project. There is also a longer lead time for this steel system, which will add stress to the construction schedule. Lastly, vibrations would be expected to be the greatest in this system compared to the other three. This can be one primary reason why this system was not chosen for TCMC.

Analysis

The weight of the non-composite, joist and joist girder system, was determined to be 34.8 psf, which makes it the lightest system among the four being compared. Because of the light weight, the size of the foundation system can be greatly reduced. Because more joists are used to support the slab, it will not span as far, therefore, it will not be as thick. Through analysis using Vulcraft Steel Deck catalog, a 3” total slab thickness is adequate to carry the load.

The non-composite slab with joists and joist girders was found to cost around $26.57 per square foot using 2013, RSMeans Assemblies. This includes the price of additional fire proofing for the slab and steel joists.
One-Way Slab on Concrete Beams

The one-way slab on concrete beams was chosen as the second alternative to the existing system. The same typical bay size of 26’x30’ was chosen for this analysis. The beams span in the 30’ direction, the girders spanning the 26’ direction, and the slab spanning over 13’. Through analysis of this system, a 6” thickness would be required for the slab, a 13.5”x22.5” beam would be required to span over 30’, and a 15”x25.5” girder would be required to span over 26’. Figure 20 shows the typical bay used for this one-way slab design. The design was performed by hand calculations, which can be found in Appendix D.

![Diagram of one-way slab on concrete beams](image)

**Figure 20 One-way slab**

**Advantages**

There are many reasons why a one-way slab is economical. It has a high compressive strength, and the concrete floor system is fire-rated without any extra fire protection. Its large mass provides an excellent vibration control. Concrete is widely available, cheap, and easy to construct. In the city of Scranton, concrete more preferred in construction than steel. That is because it is cheaper, and buildings are not as high.
Disadvantages

A one-way slab floor system has a larger system depth and weight a lot more than a steel deck and beam system. Concrete is very poor in tension so steel reinforcement must be added to help carry the flexural loads. Although concrete is cheap, formwork can be costly. Additionally, shrinkage and creep are also problems that a concrete system must face, later in the life of the structure.

Analysis

The one-way slab system has a weight of 103.7 psf, 20 psf more than the current system. Because it’s heavier, the foundations need to be increased.

The estimated cost of this system is around $19.09 per square foot. That is around $6 per square foot less than the current system. Compared to the other three systems, this system cost the least. This will be a huge saving in cost, which is a very good thing for the owner.

The one-way slab floor system has a total system depth of 25.5”, making this system the shortest depth among the other three. It is 9” shorter in depth compared to the current system, but this does not mean the building height can be decreased. The building height might still need to be increased for a mechanical system. Because there are no height restrictions in Scranton, this height increase will not be a big problem. However, it is preferably not to increase the building height because that would increase the weight as well as the surface area of the building and hence, would increase both seismic and wind forces.

Through this investigation, a one-way slab would be a viable system. Although it is the heaviest compared to the other three systems, it is the cheapest to construct, and the most capable of handling vibrations, which makes it appealing to the owner. Foundations do need to be increased and shear walls need to be added for lateral resistance. And because it is a popular material in Scranton, it makes it an attractive alternative.

Model

The model of the one-way slab is designed on spSlap. The output of deflection, moment, and shear was used to compare with the hand calculations. According to the model, the maximum deflection is 0.284” while the hand calculation resulted in 0.524”. This could also be that hand calculations are more conservative. The moment and shear outputs came out to be close to the hand calculations. The model has 170.52 kip-feet for the moment and 55.2 kip-feet for the shear. In hand calculations, the moment is 259 kip-feet 73.8 kips.
Figure 20.1 spSlap Model: Loads on the system

Figure 20.2 spSlap Model: Deflections on the system
Figure 20.3 spSlap Model: Moment

Figure 20.4 spSlap Model: Shear
Precast Plank with Wide Flange Girders

The third alternative floor system that was investigated was a precast plank with wide flange girders. The same typical bay size of 26’x30’ was chosen for this analysis. It was found that an 8”x4’-0” hollow core plank, from Nitterhouse Concrete Products, with a 2” normal weight concrete topping is required to carry the load. The hollow cores were chosen to span on the shorter direction, 26’, because it requires a much larger hollow core plank to span on the longer direction, 30’. The plank rest on W27x84 steel girders, which span 30’. The design was performed by hand calculations, which can be found in Appendix E. The design sheet from Nitterhouse Concrete Products, for the hollow core plank, was also in Appendix E. Figure 21 shows the typical bay used for this system for TCMC.

Advantages

Not many advantages can be found from this system. Its weight is similar to the two steel systems so the foundations can be kept the same. However, it does have a short lead time, reducing the stress for the construction schedule.

Disadvantages

This system has a very high cost. The construction of this system is very difficult. The performance of this system in vibration is unknown. Because the hollow core planks are pre-stressed, it is very difficult to drill through the slab when needed, and TCMC may need to drill through the slab in the near future.
Analysis

The weight of this system, at 89.5 pounds per square foot, falls in the middle for the four systems in this report. However, it costs the most, at $32.9 per square foot. This cost includes the precast production, transportation, installation, steel girders, erection, concrete topping, and fireproofing for the steel. The precast portion of the slab achieves the required 2 hour rating for fire protection by its design.

This system has the largest depth, at 34.7”. This does not create major changes to the original design because the difference is relatively small. It also does not handle well in deflection compared to the one-way slab system. One possible reason is that because the span was over 26’ while the one-way slab system span, 13ft. The lateral system does not need to be changed since steel girders and columns are still in use. Overall this system is not preferred because it is the most expensive with very little to no benefits compared to the other three systems.
## Comparison of Systems

<table>
<thead>
<tr>
<th>Criterion</th>
<th>Composite System</th>
<th>Non-Composite</th>
<th>One-Way Slab</th>
<th>Precast Plank</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight of System</td>
<td>84 psf</td>
<td>34.8 psf</td>
<td>103.7 psf</td>
<td>89.5 psf</td>
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<tr>
<td>Depth of Slab</td>
<td>7.5&quot;</td>
<td>3.0&quot;</td>
<td>6&quot;</td>
<td>10&quot;</td>
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<tr>
<td>Depth of System</td>
<td>34.4&quot;</td>
<td>31&quot;</td>
<td>25.5&quot;</td>
<td>34.7&quot;</td>
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<tr>
<td>Cost ($/SF)*</td>
<td>25.04</td>
<td>26.57</td>
<td>19.09</td>
<td>32.9</td>
</tr>
<tr>
<td>Deflection</td>
<td>1.12&quot;</td>
<td>1.45&quot;</td>
<td>.524&quot;</td>
<td>1.32&quot;</td>
</tr>
<tr>
<td>Architectural Impact</td>
<td>No change in bay size</td>
<td>No change in bay size</td>
<td>No change in bay size</td>
<td>No change in bay size</td>
</tr>
<tr>
<td>Fire Rating</td>
<td>2 hr</td>
<td>2 hr</td>
<td>2 hr</td>
<td>2 hr</td>
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<tr>
<td>Fire Protection</td>
<td>Unprotected Deck and spray on for beams</td>
<td>Spray-on for deck and joists</td>
<td>None</td>
<td>Spray-on for beams</td>
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<tr>
<td>Foundation Impact</td>
<td>N/A</td>
<td>May reduce required foundations</td>
<td>Needs to be increased</td>
<td>Needs to be increased slightly</td>
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<tr>
<td>Vibration</td>
<td>Moderate</td>
<td>Moderate High</td>
<td>Minimal</td>
<td>Unknown</td>
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<tr>
<td>Lateral System</td>
<td>No Change</td>
<td>No Change</td>
<td>Shear Walls</td>
<td>No Change</td>
</tr>
<tr>
<td>Constructability</td>
<td>Easy</td>
<td>Easy</td>
<td>Moderate</td>
<td>Hard</td>
</tr>
<tr>
<td>Lead Time</td>
<td>Long</td>
<td>Long</td>
<td>Short</td>
<td>Short</td>
</tr>
<tr>
<td>Viable System</td>
<td>N/A</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
</tbody>
</table>

* All costs are calculated using 2013, RS Means Assemblies Costs, which carries an approximate error of ±15%. Included in the cost are materials, installation, fire proofing, transportation, and labor.
Conclusions

Technical Report Two was prepared with the intention of providing three other alternative floor systems that could be used in the construction of The Commonwealth Medical College. The composite system was compared with a non-composite deck on joist and joist girders, a one-way slab on concrete beams, and a precast plank on wide flange girders. The comparison made included, system weight, system cost per square foot, system depth, deflections, impact on foundations, impact on lateral system, impact on architecture, susceptibility to vibration, and fire protection.

It is found that the precast plank system would be the least economical and least efficient alternative floor system. The one-way slab would be the most economical system to use, found in this analysis.

The one-way slab system cost the least to build, comparing just the price per square foot of the floor systems, but would result in significant increase in the foundations, therefore an increase in cost there. Additionally, the lateral system will be changed to shear walls.

The existing system cost came in between the other systems. It was most likely chosen because if performs fairly well in deflection, average in cost, average in weight, easy to construct, and moderate sense of vibration control. The one-way slab would have been a more economical choice in this analysis but maybe the weight of the structure is what drove the owner or designer away. Although, the non-composite system has many advantages, it does cost more and performs poorly in deflection and vibration. Handling vibration is one of the most important factors for TCMC because of the medical usage of the building.
Appendix A

Framing Plan of the 2nd Floor, Courtesy of Highland Associates
2nd Story frame, east wing (south side), Courtesy of Highland Associates
2nd Story frame, east wing (north side), Courtesy of Highland Associates
## Appendix B

<table>
<thead>
<tr>
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</table>

### Typical Brg

**Current System - Composite System**

- **3ULI20, 4 1/2” NWC topping**
- **B1 = W19 x 55” span 30’**
- **G1 = W27 x 94” span 26’**

Current system has 2hr fire rating.

- **3/4” Ø x 6” long shear studs**
- **equally spaced**

### Plan View

#### Slab Design (From 2008 Vulcraft Decking Catalog)

**Loads**
- **OL = selfweight**
- **LL = 100psf**
- **SDL = 18psf = 10 partitions**
  - 5 M.E.P.
  - 3 Finishes

**Superimposed loads = 100 + 18 = 118psf**

- **3ULI20 3 span, clear span = 11’-0” > 8’-8” ✓**

Use 9’-0” (round 8’-8” up to be conservative)

**Carry superimposed load of 231 psf > 118 psf ✓**

Fire resistance, unprotected Deck + 4” NWC minimum controls.

**3ULI20 has 2hr fire rating,**

**3ULI20 meets all requirements**
Composite System

Composite B1

\[ A_t = 9.66'' (30') = 260 \text{ ft}^2 \]

\[ k_L = 2 \]

\[ L_L = L_0 \left( 0.25 + \frac{5}{K_L A_t} \right) = 100 \left( 0.25 + \frac{5}{520} \right) = 90.18 \text{ psi} \]

\[ W_u = 1.2 \left( 75 + 18 \right) + 1.6 \left( 90,18 \right) = 255.9 \text{ psi} \]

\[ W_{u_1} = 75 \text{ lb} + (75 + 18) \left( 90,18 \right) = 1592 \text{ psi} \]

\[ M_u = \frac{(2225 \times 30)^2}{8} = 251 \text{ k-ft} \]

\[ R_u = \frac{2225 \times 30}{2} = 33.4 \text{ k} \]

Table 3-21

Deck Perpendicular, \( R_p = 0.60 \), 1 stud per row

\[ N = \frac{3}{4} \text{ stud}, f_1 = 4 \text{ksi} \]

\[ Q_n = 17.2 \text{ k} \]

\[ b_{eff} = b'_i = \min \left[ \frac{5\text{ psi}}{8}, \frac{5\text{ psi}}{8} \right] = \frac{5\text{ psi}}{8} \]

\[ b'_i = b_{eff} = 45'' \]

\[ b_{eff} = 2 \left( 45'' \right) = 90'' \]

\[ L = a_i = 1.0 \]

\[ y_2 = 7.5 - 2.5 = 7.0'' \]

\[ Q_n = \min \left( \frac{P_{g} r_i \gamma (55.7)}{17.2} \right) = 17.2 \text{ k} \]

\[ A_{s} F_n = \gamma \left( \frac{3}{4} \text{ ksi} \right)^2 (65) = 20.7 \text{ k} \]

Table 3-19

\[ I_{18} = 1340 \text{ in.}^4, T_{18} = 18 \times 40 \]

\[ M_u = 446 \text{ k-ft} \]

\[ \leq Q_n = 14.8 \text{ k} \]

\[ 14.8 > 17.2 \Rightarrow 10 \text{ studs (2)} = 20 \text{ studs} \]

\[ I_{18} = 1340 \text{ in.}^4, T_{18} = 2.1 \times 44 \]

\[ M_u = 542 \text{ k-ft} \]

\[ \leq Q_n = 16.3 \text{ k} \]

\[ 16.3 > 17.2 \Rightarrow 10 \text{ studs (2)} = 20 \text{ studs} \]

\[ I_{18} = 1730 \text{ in.}^4, T_{18} = 18 \times 55 \]

\[ M_u = 623 \text{ k-ft} \]

\[ \leq Q_n = 20.3 \text{ k} \]

\[ 20.3 > 17.2 \Rightarrow 12 \text{ studs (2)} = 24 \text{ studs} \]
Check A: For W18x40, since it is the most economical:

\[
\frac{E \alpha n}{0.85 \ell_{e} b_{w} t_{c}} = \frac{148}{0.85(4.0)(30)} = 0.484 \leq 1'' \text{ Ok.}
\]

\[I_{w} = 1210 \text{ in}^4 \quad I_{x} = 612.2 \text{ in}^4\]

Check for unshored strength:

\[Q_{w} = 75 \text{ kips} \quad I_{w} = 114(75+18)(8.66) + 114(40) = 1146.6 \text{ kips} \]

\[Q_{w} = 1.2(75+18)(8.66') + 1.2(40) + 1.6(98.8)(8.66) = 22.8 \text{ kips}\]

\[M_{w} \geq \frac{(2.28)(30)^{2}}{8} = 256.5 < 294 \text{ kips} \text{ Ok}\]

Check for deflections:

\[w_{w} = 98.5 \text{ psi}(8.66) = 787 \text{ in}^{2}\]

\[I_{w} = 1210 \text{ in}^4 \quad \gamma = 0.2 \quad Q_{n} = 148 \text{ kips}\]

\[\Delta_{w} = \frac{5(1.787)(30)^{4}(1728)}{384(29,000)(1210)} = 0.41''\]

\[\Delta_{w} = \frac{5}{360} = 1'' > 0.41'' \text{ Ok}\]

\[w_{L} = 787 + (93)(8.66) = 165 \text{ kips}\]

\[\Delta_{L} = \frac{5(1.6)(30)^{4}(1728)}{384(29,000)(1210)} = 0.85''\]

\[\Delta_{L} = \frac{5}{240} = 1.5'' > 0.85'' \text{ Ok}\]

Check A: For 18 x 55, used in TCMC:

\[Q = \frac{203}{0.85(4.0)(30)} = 664 \leq 1'' \text{ Ok}\]

Check for deflections:

\[\Delta_{w} = \frac{5(1.787)(30)^{4}(1728)}{384(29,000)(1730)} = 0.286'' < 1'' \text{ Ok}\]

\[\Delta_{L} = \frac{5(1.6)(30)^{4}(1722)}{384(29,000)(1730)} = 0.582'' < 1.5'' \text{ Ok}\]

W18x40 met all criteria and is the most economical to use, however, TCMC uses W18x55 which may be for vibrational purposes, so use W18x55.
Composite System | Tech 2 | Rise 4 of 5

Composite Girder (interior)

\[ P_n \]

\[ E_c = \frac{w_i}{10^5} \sqrt{f'c} \\
= 150\sqrt{f'c} \\
= 3674.23 \]

\[ W = 27 \times 94 \]

\[ Q_n = 0.5 A_{Ec} \sqrt{f'c} E_c \]

\[ = 0.5 \times 10 \times \left( \frac{8}{12} \right)^2 \times 4 \times 3674 \]

\[ = 74.4 \text{ k} \]

\[ \frac{\Sigma Q_n}{\Sigma Q_n} = \# \text{ studs} \]

\[ \frac{\Sigma Q_n}{26.77} = 22 \text{ studs} \]

\[ \Sigma Q_n = 589.9 \text{ k} \Rightarrow \Sigma Q_n = 490 \text{ to be on the conservative side} \]

\[ b_1 = \min \left[ \frac{SPY}{0.35} \right] = \frac{26(2)}{8} = 39'' \Rightarrow \text{controls} \]

\[ b_2 = 39'' \text{ for both interior/secondary span} \]

\[ a = \frac{\Sigma Q_n}{0.95(4)(78)} = \frac{490}{0.95(4)(78)} = 1.85'' \]

\[ y_2 = 7.5 - 1.85/2 = 6.6'' \]

\[ \delta M_n = 1590 \text{ kft} \Rightarrow y_2 = 6.5'' \]

\[ = 1610 \text{ kft} \Rightarrow y_2 = 7.0'' \]

\[ \text{Integrate } \frac{1600 - x}{7.0 - 6.6} \Rightarrow x = 1594 \text{ kft} \]

\[ \delta M_n = 1594 \text{ kft} \]

\[ W_n = 1.2 \left[ 175 + 18 \left( 8.66 \right) + 1.6 \left( 90.8 \right) \left( 8.66 \right) \right] \]

\[ = 1.91 \times \left( 30' \right)^2 \times 2 \]

\[ = 57.3 \text{ k} < W_n \]

\[ M_n = P_n \cdot a = 57.3 \times 8.66 = 500 \text{ k-ft} \leq 1594 \text{ k-ft} \]
Appendix C

Noncomposite Deck with Joist Grinders

| 2.0" | 0.6 C28 | 2' - 11" | 4' - 7" |
| 3.0" | 0.6 C22 | 4' - 7"  |
| 3.0" | 1.0 C26  |
| 3.5" | 1.0 C22  |

| 19 spaces @ 2' - 10 3/4" on center |
| 26"

30'

N.W. Concrete (3-span)

The following floor decks have a 2-hr fire resistance rating:

- Choose deck that meets both load and span criteria.

- DL = self weight
- SDL = 18 psf
- LL = 100 psf
- Given S201A

- 10 partitions
- 5 M.E.P.
- 3 Finishing
- 18 SDL Assumed

- Superimposed uniform load
- 100 + 18 = 118 psf

- Span over 8' - 8"

- Use span 9' - 0" to be conservative in the Mulcraft catalog.

- Starting with 2.0", 0.6 C28 spanning 3' - 0"
- (conservative cal)
- For total load: F_b = 36,000 ksi
- 28 gage carries 119 psf at 3' - 0" span

Total Load:
- LL = 100
- SDL = 18
- DL = 23 Deck + CC
- 141 psf > 119 psf so deck needs to increase.
- Use stronger deck.

Find deck with strength over 141 psf with additional dead weight.

- 3.0" 1.0 C26 spanning 3' - 0"
- For total F_b = 36,000
- 26 gage carries 232 psf at 3' span > 141 psf

Total Load:
- LL = 100
- SDL = 18
- DL = 31 Deck + CC
- 141 psf < 232 psf so works.
For live load \( \frac{1}{240} \), use 10c26.

Summary: Use 10c26 with 2.0" NW concrete topping (3.0" total) because it satisfies all the above.

Required \( \leq \) Allowed by slab

- 2'-10" \( \leq 13' - 4" \)  ✔
- 119 psf \( \leq 232 \) psf  ✔
- 100 psi \( \leq 188 \) psi  ✔

Joint

Unfactored Loads

- SIO = 18 psf
- LL = 100 psf
- DL = 31 psf slab only

The joints in this bay span 30 feet long.

\[
W_{ld} = \left[ 1.2 \left( 31 + 18 \right) + 1.6 \left( 100 \right) \right] \left( 2.89 \right) = 633 \text{ plf}
\]

\[
W_{ld} = (31 + 18 + 100)(2.89) = 431 \text{ plf}
\]

Both needs to include joint dead weight letter.

\[
\Delta_{\text{max}} = \frac{1}{240} \text{ for floor}
\]

From K-series economy table: SJI ps C-4

\[
26K9 \leq W_{nt} = 625 \text{ plf} > 633 + 1.2(12.2) = 648 \text{ plf} ✔
\]

Check deflection for 26K9

\[
W \text{ for } \frac{1}{320} = 459 \text{ given on table}
\]

\[
W \text{ for } \frac{1}{240} = 459(1.5) = 688.5 \text{ plf} > 431 \text{ plf} ✔
\]

So use joint 26K9
Jost. Girdler

\[ U_{\text{ult}} = [1.2(31+15) + 1.6(100)](2.891) + 1.2(12.2) = 647.0 \text{ p.l.f.} \]

\[ P_n = \frac{647.0}{300} = 19.9 \text{ k} \]

Let's use 26" because it economical from Jost. Girdler table.

28G9N19,4F girdler will be used.

This girdler weights about 82 p.l.f.

- **Deflection Check for Joints**

26K9 \[ \Rightarrow I_j = 26.76 \text{ in.}^2(10^{-6}) \]

\[ w_{ul} = 459.1 \text{ p.s.f.} \]

\[ L = (30' - 0.33') \]

\[ I_j = 26.76 \text{ in.}^4(30' - 0.33')^3(10^{-6}) \]

\[ = 321 \text{ in.}^4 \]

\[ \Delta_{ul} = \frac{5w_{ul}L^4}{384EI_x} = \frac{5(289.1)(30')^4(12')^3}{384(25,000)(321)} = 5.66 \text{ in.} \]

\[ \frac{h}{360} = \frac{30'(12')}{360} = 1 \text{ in.} > 5.66 \text{ in.} \checkmark \]

\[ \Delta_{TL} = \frac{5(43)(30')^4(12')^3}{384(25,000)(321)} = 8.43 \]

\[ \frac{h}{240} = \frac{30'(12')}{240} = 1.5 \text{ in.} > 8.43 \text{ in.} \checkmark \text{satisfies deflection} \]

- **Deflection Check for Jost. Girders**

28G9N24.0F \[ \Rightarrow I_j = 0.018 \text{ N.p.l.d.} \]

\[ \Delta_{ul} = \frac{5(2.6')(26')^4(12')^3}{384(25,000)(225')^4} = 0.018 \text{ in.} \]

\[ \frac{w_{ul} = 26(100)}{2600 \text{ p.l.f.}} \]

\[ w_{ul} = 26(149) = 3.74 \text{ p.l.f.} \]

\[ = 0.403 \text{ in.} < \frac{h}{360} = 1 \text{ in.} \checkmark \]

\[ \Delta_{TL} = \frac{5(3.88)(26')^4(12')^3}{384(25,000)(235')^4} = 1.601 \text{ in.} < \frac{h}{240} = 1.5 \text{ in.} \checkmark \text{satisfies deflection} \]
Appendix D

One-way Slab

Tech 2

Page 1 of 11

2hr fire Rating, min 5 in slab thickness.

SDL = 18 psf

LL = 100 psf, Gaus 5201A

DL = self weight

10 psf partitions

5 psf MEP.

3 psf Finishig

18 psf SDL Assumed

Use ACI 318-11

Use all N.W. concrete

Grade 60 reinforcement

$f_c = 4000$ psi

Try $\# 5$ rebar for slab $\# 9$ rebar for beam

$d_6 = .625$ in

$A_6 = .311$ in$^2$

$W = 1.043 \frac{1}{12}$ ft

$\frac{1}{2} f_y = 1.278$ in

Try $\# 6$ rebar for slab $\# 10$ rebar for beam

$d_6 = .75$ in

$A_6 = .444$ in$^2$

$W = 1.502 \frac{1}{12}$ ft

$\frac{1}{2} f_y = 1.270$ in

*$Use 24 \times 24$ columns,

$f_c = 4000$ psi

$f_y = 60,000$ psi

Plan view

- Slab Design

$h_{min} = \frac{d}{2} for interior, Table 9.5(a)$

$= \frac{2h(x)}{2b} = 11.74'' \geq 11.50'' > 5'' min for 2hr fire rating.$

SW of slab = $(150 \text{ psf})(11.50 \frac{1}{12}) = 143.75 \text{ psf}$

Minimum Concrete Cover = $3/4''$

$d = h - cc - d_f \geq 11.5'' - 3/4'' - .625'' = 10.445''$

$A_{min} = .002bh$, use 1" wide section.

$= .002(12'')(11.5'') = .276 \text{ in}^2/ft^2 \text{ of slab required}$

$S_{min} = \frac{15(40,000)}{40,000} - 2.5(3%) \leq 12''$

$f_s = \frac{3}{2} f_y = \frac{3}{2}(60,000) = 40,000 \text{ psi assumed}$

$= 13.1'' \leq 12'' \Rightarrow max 12'' spacing.$
One-way Slab

Try #5 rebars spaced at 10” o.c.

\[ A_s = \frac{51(12)}{10} = 372 \text{in}^2/\text{ft} > 276 \text{in}^2/\text{ft} \text{ req'd} \checkmark \]

\[ \rho = \frac{A_s}{bd} = \frac{372}{(12)(10.44)} = 0.0297 \]

\[ a = \frac{A_{sf}}{0.8\epsilon_{cl,b}} = \frac{(372)(60,000)}{785(4000)(12)} = 5.47 \]

\[ \Phi_{Mn} = \Phi A_{sf}(d - 0.2) \]

\[ = 0.91(372)(60)(10.44 - 5.47) \]

\[ = 204.2 \text{ kips} \]

\[ w_m = 1.2(143.75) + 1.6(100) = 354.1 \text{ psf} \]

\[ = 354.1 \text{ psf} \text{ per 1 ft section} \]

\[ M_n = \frac{w_m l^2}{8} = \frac{3541}{2} = 29.93 \text{ k-ft} = 359.1 \text{ k-in} \]

\[ 359.1 \text{ k-in} > 204.2 \text{ k-in} \text{ so N.I.C. redesign.} \]

Try #6 rebars spaced at 7” o.c.

\[ A_s = \frac{144(12)}{7} = 254.3 \text{ in}^2/\text{ft} > 276 \text{ in}^2/\text{ft} \text{ req'd} \checkmark \]

\[ d = 11.5” - 3/4” - 1.5” = 10.38” \]

\[ \rho = \frac{A_s}{bd} = \frac{254.3}{12(10.38)} = 0.0605 \Rightarrow \rho = 0.49 \]

\[ a = \frac{254.3(60,000)}{185(4000)(12)} = 2.91 \]

\[ \Phi_{Mn} = 0.91(254.3)(60)(10.38 - 1109/2) \]

\[ = 400.0 \text{ k-in} > 359.1 \text{ k-in} \checkmark \text{ good.} \]

Use a 11.5” slab w/ #6 rebars spaced at 7” o.c.
One-way Slab  |  Tech Z  | page 3 of 11

Beam Design

\[ DL = 18 \text{ psf} + 143.75 \text{ psf} + \text{Beam SW} \]
\[ LL = 100 \text{ psf} \]

\[ W_u = 1.2 (18 + 143.75) + 1.6 (100) \]
\[ = 354.1 \text{ psf} \]
\[ W_u = 354.1 \text{ psf (26')} = 9207 \text{ plf} \]
\[ M_u = 9207 \text{ klf} \]

\[ M_u = \frac{9207 (30' - 24')^2}{8} \times (1',1') = 993 \text{ klf} \]
*Use 24x24 Columns.*

Estimate beam size

\[ b d^2 = 20 M_u \]
\[ b = \frac{3}{2} d \]
\[ d = 31'' \quad b = 21'' \]
\[ h = d + 2.5'' \Rightarrow 33.5'' \]
\[ b d^2 = 21 (31)^2 = 20181 \text{ in}^3 \]

Self Weight

\[ w_{u_1} = \frac{(31')^2 (150 \text{ psf})}{144} = 733 \text{ plf} \]
\[ W_u = 9207 + 733 = 9941 \text{ klf} \]
\[ M_u = 9941 (28')^2 = 975 \text{ klf} < 993 \text{ klf} \text{ assumed O.K.} \]

Reduced Steel

\[ A_s = \frac{M_u}{4d} = \frac{969}{4(31)} = 7.82 \text{ in}^2 \]

Try (2) # A rebars \( = 8(1') = 8 \text{ in}^2 > 7.82 \text{ in}^2 \)
- Place 6 on the bottom and 2 on top of them

\[ d = 31'' - 1.128'' = 29.9'' \]
One-way Slab

Nominal Moment

\[ M = \frac{Asf_y}{0.85f'y} = \frac{8 \times (60)}{0.85 \times (1)(21)} = 6.73 \]

\[ C = \frac{d}{h} = \frac{6.73}{0.85} = 7.92 \]

\[ \varepsilon_s = \varepsilon_n \left( \frac{d-C}{C} \right) = 0.003 \left( \frac{29.9 - 7.92}{7.92} \right) = 0.083 > 0.05 \text{ yield} \]

\[ \phi M_n = \phi Asf_y (d - \frac{h}{2}) \]

\[ = 0.9(8)(60)(29.9 - 6.73/2) \]

\[ = 955 \text{ k-ft} \]

\[ M_n = 955 \text{ k-ft} < M_n = 975 \text{ k-ft} \quad \text{N.G.} \]

Try \( g = 8 \# 10 \) rebars \( g(1.27) = 10.16 \text{ in}^2 > 7.82 \text{ in}^2 \)

- Place 6 on the bottom and 2 on top of them

\[ d = 3(1) - 1.27 = 29.8'' \]

\[ a = \frac{10.16(60)}{0.85(4)(21)} = 8.54 \]

\[ C = \frac{8.54}{0.85} = 10.04 \]

\[ \varepsilon_s = \frac{0.003}{10.04} \left( \frac{29.8 - 10.04}{10.04} \right) = 0.0599 > 0.05 \text{ yield} \]

\[ \phi M_n = 0.9(10.16)(60)(29.8 - 8.54/2) \]

\[ = 1167.2 \text{ k-ft} \]

\[ \phi M_n = 1167 \text{ k-ft} > M_n = 975 \text{ k-ft} \quad \text{good} \]

Steel Area

\[ A_s, min = \frac{200}{f_y} bd = \frac{200}{60,000}(2)(3) = 2.17 \text{ in}^2 < 10.16 \text{ in}^2 \quad \text{V} \]
Max reinforcement

\[ P_{\text{max}} = 0.85 \beta_1 \frac{f'c}{f'} \left( \frac{E_u}{E_n + 0.05} \right) \]

\[ = 0.85 \left( \frac{60}{60} \right) \left( \frac{2000}{1000} \right) \]

\[ = 1.018 \]

\[ P = \frac{A_s}{bd^2} = \frac{10 \times 0.6}{21 \times 31.5^2} = 0.0162 \leq 0.0181 \checkmark \]

Use 21" x 31" beam with (8) #10 rebars

\[ d_c = 29.8 \text{ in.}, \quad d_t = 31 \text{ in.} \]
Layout Two

24” x 24” columns
Concrete beams

Some materials as layout one
unless listed below.

Try #4 rebar for slab
\( d_b = 1.5'' \)
\( A_{sb} = 2'' \)
\( w = -66.6^{\text{lb}}/\text{ft} \)

Slab Design

\[ h_{\text{min}} = \frac{3}{2} \text{ ft for interior, Table 9.5(a)} \]
\[ = \frac{3}{2} \times 5.57'' \approx 6'' \text{ slab } > 5'' \text{ min for fire rating} \]

510 of slab = (150pcf)(6/12) = 75psf

Minimum concrete cover = \( 3/4'' \)

\[ d = h - CC - d_{c2} = 6'' - 3/4'' - \frac{15}{2} = 5.0'' \]

\( A_{\text{min}} = .002(12'')(6) = \frac{114}{1,000} \text{in}^2/\text{ft} \text{ of slab required} \]

\( S_{\text{max}} = 15\left(\frac{40,000}{40,000}\right) - 2.5\left(\frac{3}{4}\right) = 12 \]

\( = 13.1'' \leq 12'' \Rightarrow 12'' \text{ max spacing} \)

Try #4 rebar spaced at 12'' O.C.

\( A_b = \frac{.20(12)}{12} = .20\text{in}^2 > .144\text{ in}^2/\text{ft}\text{ required} \)

\[ \phi = \left(\frac{20}{2\times5.0}\right) = .00337 \]

\[ a = \frac{(20)(60,000)}{.85(40,000)(12)} = 12.54 \]
One Way Slab

Tech 2

Page 7 of 11

\[ \Delta M_n = 0.9 \left( \frac{1}{2} \right) (20)(60) \left( \frac{5.0 - \frac{12}{2}}{2} \right) = 51.76 \text{ k}\cdot\text{in} \]

\[ W_n = \frac{1}{2} \left( \frac{12 + 75}{2} \right) + 1.6(100) = 271.6 \text{ psf} \]

\[ 2.716 \left( \frac{1}{1} \right) = 271.6 \text{ psf per ft section} \]

\[ M_n = \frac{Wd^2}{8} = \frac{2.716 \left( \frac{13}{2} \right)^2}{8} = 5.74 \text{ k}\cdot\text{ft} = 68.85 \text{ k}\cdot\text{in} \]

\[ 68.85 \text{ k}\cdot\text{in} > 51.76 \text{ k}\cdot\text{in} \text{ so N.G. } \Rightarrow \text{ redesign} \]

Try #6 rebars spaced at 11" o.c.

\[ A_s = \frac{1.44(12)}{11} = 1.18 \text{ in}^2 / \text{ft} > 1.276 \text{ in}^2 / \text{ft} \]

\[ d = 6" - \frac{3}{4}" - \frac{15}{2} = 4.875" \]

\[ p = \frac{480}{(12)(4.875)} = .0865 \]

\[ a = \frac{480(60,000)}{35(4000)(12)} = .705 \]

\[ \Delta M_n = 0.9 \left( \frac{1.18}{12} \right)(60) \left( 4.875 - \frac{.705}{2} \right) = 117.2 \text{ k}\cdot\text{in} \]

\[ 117.2 \text{ k}\cdot\text{in} > M_n = 68.85 \text{ k}\cdot\text{in} \text{ } \checkmark \text{ good} \]

Use a 6" slab with #6 rebars @ 11" o.c.
Beam Design

\[ DL = 18 + 75 + \text{Beam SW} \]
\[ LL = 100 \, \text{psf} \]
\[ W_u = 1.2(18 + 75) + 1/6(100) = 178.6 \, \text{psf} \]
\[ \Rightarrow 178.6 \,(131) = 2321.8 \, \text{plf} \]
\[ = 2.322 \, \text{kfl} \]
\[ Mu = \frac{2.322 \,(30' - 20')^2}{8} \left( \frac{1}{1.2} \right) = 275 \, \text{k-ft} \]

Estimate beam size

\[ bd^2 = 20Mu : \text{Try} \, b = 2/3d \]
\[ d^3 = \frac{3}{2} \left( \frac{20}{(275)} \right) \]
\[ d = 19.6' \approx 20'' \Rightarrow b = 13.5'' \]
\[ h = d + 2.5'' = 22.5'' \]
\[ bd^2 = (13.5)(20)^2 = 5400 \, \text{in}^2 \]

Self weight

\[ W_{sw} = \frac{22.5 \,(13.5)(\frac{150}{144})}{19.6} = 31.6 \, \text{psi} \]
\[ W_{u} = 2.322 + 3164 = 2.638 \]
\[ Mu = \frac{2.638 \,(20')^2}{8} = 259 \, \text{k-ft} \]
\[ < 275 \, \text{k-ft} \text{ assumed, } \checkmark \]

Required steel

\[ As = \frac{Mu}{4d} = \frac{275}{4 \,(20)} = 3.144 \, \text{in}^2 \]

\[ \text{Try} \, (5) \# \, 9 \, \text{rebars} : (4)(1) = 4 \, \text{in}^2 \]
\[ d = 20'' \]

Nominal Moment

\[ a = \frac{Asf_y}{0.85f_y b} = \frac{(60 \times 4)}{0.85 \times 5 \times (13.5)} = 5.23 \]
\[ \leq 9/\beta_1 = 5.23/1.15 = 6.15 \]
\[ \varepsilon = 0.003 \left( \frac{20 - 6.15}{6.15} \right) = 0.0676 > 0.005 \]
\[ \beta = 0.9 \]
\[ \Delta \varepsilon_n = 1.9(4)(60)(20 - 5.23/2) = 319.0 \, \text{k-ft} \]
\[ > M_n = 259 \, \text{k-ft} \text{ good} \]

Min Steel Area

\[ As_{min} = \frac{200}{f_y} \left( \frac{60 \, \text{ksi}}{13.5}(20) \right) = 19 \, \text{in}^2 \leq 4 \, \text{in}^2 \]

\[ \checkmark \]
One-Way Slab

Max. reinforcement

\[ P_{\text{max}} = 0.85 \left( \frac{4}{40} \right) \left( \frac{1}{0.03} \cdot \frac{1}{0.08} \right) = 0.181 \]

\[ P = \frac{4}{(13.5 \times 20)} = 0.0148 < 0.0161 \text{ in}^2 \checkmark \]

Use 13.5” x 22.5” beam with (4)# of rebar

<table>
<thead>
<tr>
<th>Girders Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Width (W) = 20 in</td>
</tr>
<tr>
<td>Depth (D) = 23 in</td>
</tr>
<tr>
<td>Height (h) = 23 in</td>
</tr>
</tbody>
</table>

Estimate girders size:

\[ b_d^2 = 20 \times W = 200 \quad b = \frac{b_d}{d} \]
\[ d^2 = \frac{3}{2} \times \left( \frac{20}{23} \right) \]
\[ d = 22.6 < 23^2 \quad b = 15^1 \]
\[ h = 23^2 + 2.5^2 = 25.5^1 \]

Self-weight

\[ W_{self} = 25.5 \times (15 - (10)) = 399 \text{ psf} \]

\[ M_u = \frac{(24)(115.5)}{8} + \frac{(395)(24)^2}{8} = 375.3 \text{ k-ft} \]

Required steel

\[ A_s = \frac{M_u}{4d} = \frac{375.3}{4(25)} = 4.08 \text{ in}^2 \]

Try (4)# of rebars "(1.27) = 5.08 in^2 > 4.08 in^2"
\[ d = 23'' \]

\[ q = \frac{(5.08 \times 60)}{0.85 (4)(15''/15')} = 5.98 \]

\[ C = \frac{5.98}{13} = 7.104 \]

\[ \varepsilon_5 = 0.003 \left( \frac{23 - 7.04}{7.04} \right) = 0.0068 \]

\[ \Delta M_n = 1.9 (5.08)(60)(23 - \frac{5.98}{2}) \]

\[ = 457.5 \text{k-ft} > 375.8 \text{k-ft} \checkmark \]

**Steel area**

\[ \text{As, min} = \frac{200}{60,000} (15')(23) = 1.15 \text{in}^2 \leq 5.08 \text{in}^2 \checkmark \]

**Max relief**

\[ P_{\text{max}} = 0.185 \left( \frac{18'6}{60} \right) \left( \frac{0.035}{0.062} \right) = 0.181 \]

\[ P = \frac{5.08}{(15' \times 23')}, 0.0147 < 0.181 \text{in}^2 \checkmark \]

So use 15'' x 25.5'' concrete girders with (4) 7# 10 rebars.

\[ h = 25.5'' \]

\[ d = 23'1'' \]

\[ b = 15'' \]

**Layout 2** is a bit more efficient so will use layout 2 instead of layout 1 in the report.

Check concrete (24'' x 24'') only pure axial.

Total \( P_n \)

\[ (15')(15') \left[ 103.7(12) + 1.2(18) + 1.6(100) \right] \left( 4 \right) + 13\left' (15') \left[ 103.7(12) + 1.2(90) + 1.6(10) \right] \]

\[ 239K + 93K \]

\[ P_n = 332k \]
\[
\varepsilon_y = \frac{f_y}{E_s} = \frac{60}{29,000} = 0.00207 \\
(4) \text{ #10 rebars} = (4)(1.27) = 5.081'' \\
v_y = 60 \text{ ksi} \\
P_0 = 0.85(4)(24.24 - 5.08) + (5.05)(60) \\
P_0 = 1941 + 304 + 304 \\
P_0 = 2245 \text{ k} > 332 \text{ k} \\
\text{Column} \ 24'' \times 24'' \text{ is more than enough for pure axial compression.}
Appendix E

Precast Plank/Steel Girder Tech 2

Used on a typical Bay
Precast Plant Slab Design

30'

24'

Plan View

Section A-A

Dead Load = Self weight
Live Load = 100 psf Found on S201A
Superimposed Dead Load = 18 psf
Use Nutterhouse hollow core planks, Partitions 10 psf
2hr fire rating, required,

Try 8'' x 4'-0'' Hollow Core Plank
2hr fire resistance rating with 2'' topping

Self weight noted in catalog (attached in report)
W = 61.25 psf + 150 psf (1/2) = 86.25

Service load
W = 86.25 + 18 + 100 = 204.25 psf

Superimposed load
W = 18 + 100 = 118 psf

Check M_u
Planks are 4' wide

M_u = \left( \frac{204.25 \times 4}{263} \right)^2 = 69.04 k\cdot ft
\Rightarrow 828.5 k\cdot in

Check for Stresses

f = 0.06 (270 ksi) (0.15 3 in^2) = 24.8 ksi

0.15 ksi = A舁wire, Vpu = 270 ksi

Cause by Loads

Precast  S_{lep} = 616 in^2

Topping Stirr. = 1076 in^2

A_c = 301 in^2

f_c = 6000 psi

f_{lep} = 5.09 psi

\epsilon = 0.03 - 1.25 = 3.34

I_c = 3134 in^4
Check for deflections

\[ \Delta_{LL} = \frac{5(1/6)(0.11 \text{ ksf})(4.3)(24.5^4)(1728)}{384(25,000 \text{ksi})(3.134 \text{ in}^4)} = 0.073 \text{ in} \leq \Delta_{LL, \text{max}} = 0.07 \text{ in} \checkmark \]

(6 since it's post-tensioned, it won't even deflect this much.

\[ \Delta_{TL} = \frac{5[1.2(0.018 + 0.08625) + 1.6(0.1)](4)(24)^2(1728)}{384(25,000)(3.134)} = 0.129 \text{ in} \leq \Delta_{TL, \text{max}} = 1.05 \text{ in} \checkmark \]

Factored Mu

\[ Mu = \frac{[1.2(0.018 + 0.08625) + 1.6(1)]4(26)^2}{8} = 96.4 \text{ k-ft} = 1157 \text{ k-in} \]

Choosing a Strand pattern of 6 - 1/2" @

\[ M = 130.6 \text{ k-ft} \checkmark 60\% \text{ jacking force} \]

\[ M = 130.6 \text{ k-ft} > 96.4 \text{ k-ft} \checkmark \text{ this is good.} \]

So use 8" x 4'-0" 2hr rating 1" toppings, 6-1/2" @ strands is good

Choosing the Girder required

\[ W_d = (18 + 86.25)(26) = 2970, 5 \text{ plf} \]

\[ L_{d} = 0.25 + \frac{15}{\sqrt{2(26)(30)}} = 0.63 \]

\[ W_l = 0.630(100)(26) = 1638 \text{ plf} \]

\[ W_u = 1.2(2.97) + 1.6(1.64) = 6.19 \text{ klf} \]

\[ I_{\text{total}} = \frac{5(1/6)(1.64)(30)^4(1728)}{384(25,000 \text{ ksf})(30)^2(360)} = 1650 \text{ in}^4 \]

\[ I_{\text{total}} = \frac{5(6.19)(30)^4(1728)}{384(25,000)(30)^2(360)} = 2593 \text{ in}^4 \]
<table>
<thead>
<tr>
<th>Precast Plain/Steel Girder</th>
<th>Tech 2</th>
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</tr>
</thead>
</table>

Check for Moment and Shear required:

\[
M_n = \frac{6.19(30)^2}{2} = 697 \text{k-ft}.
\]

\[
V_n = \frac{6.19(30)}{2} = 92.9 \text{k}.
\]

Going to Table 3-2 in steel manual to look up some beams:

- **W 24 x 68**: \( M_n = 664 \) \( V_n = 295 \) \( I = 1230 \times \)
- **W 24 x 84**: \( M_n = 840 \) \( V_n = 340 \) \( I = 2370 \times \)
- **W 27 x 84**: \( M_n = 915 \) \( V_n = 368 \) \( I = 2850 \times \)

\[ 915 > 697 \checkmark \quad 368 > 92.9 \checkmark \quad 2850 > 2593 \checkmark \]

* If ceiling height is more important, use \( W 18 \) to match existing wires.

- **W 18 x 130**: \( M_n = 1090 \) \( V_n = 388 \) \( I = 2460 \times \)
- **W 18 x 143**: \( M_n = 1210 \) \( V_n = 427 \) \( I = 2750 \checkmark \)

\[ 1210 > 610 \checkmark \quad 427 > 91.3 \checkmark \quad 2750 > 2271 \checkmark \]

Using \( W 27 x 84 \) is the most economical.

Check for deflection:

\[
\Delta n = \frac{100(2w)}{2.6 \times 12} = 2.16 \text{k-ft}
\]

\[
\Delta n = \frac{2600 + (18 + 86.25)(26) + 84}{5.4} \approx 5.1 \text{k-ft}
\]

\[
\Delta n = \frac{5(1.4)(30)^4(12)^3}{384(25000)(2530)} = 1.19'' < 1.5'' \checkmark
\]

\[
\Delta n = \frac{5(2.6)(30)^4(12)^3}{384(25000)(2530)} = 0.5'' < 1.0'' \checkmark
\]
Prestressed Concrete
8"x4'-0" Hollow Core Plank
2 Hour Fire Resistance Rating With 2" Topping

**PHYSICAL PROPERTIES**
Composite Section
- $A_w = 301$ in.$^2$
- Precast $b_w = 13.13$ in.$^2$
- $l = 3134$ in.$^4$
- Precast $S_{box} = 816$ in.$^3$
- $V_{tot} = 0.19$ in.$^2$
- Topping $V_{tot} = 902$ in.$^3$
- $V_w = 2.91$ in.$^2$
- Precast $V_{top} = 1076$ in.$^3$
- $d_{w} = 4.91$ in.
- Precast Wt. = 245 PLF
- Precast Wt. = 81.25 PSF

**DESIGN DATA**

1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed),
   - 4-1/2"Ø, 270k = 92.3 k-ft at 60% jacking force
   - 6-1/2"Ø, 270K = 130.6 k-ft at 60% jacking force
   - 7-1/2"Ø, 270K = 147.8 k-ft at 60% jacking force
7. Maximum bottom tensile stress is 10 $\sqrt{fc}$ = 775 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strength strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI  Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

**SAFE SUPERIMPOSED SERVICE LOADS**

<table>
<thead>
<tr>
<th>Strand Pattern</th>
<th>17</th>
<th>18</th>
<th>19</th>
<th>20</th>
<th>21</th>
<th>22</th>
<th>23</th>
<th>24</th>
<th>25</th>
<th>26</th>
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<th>32</th>
<th>33</th>
<th>34</th>
<th>35</th>
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<tbody>
<tr>
<td>4 - 1/2&quot;Ø LOAD (PSF)</td>
<td>285</td>
<td>248</td>
<td>214</td>
<td>185</td>
<td>159</td>
<td>136</td>
<td>118</td>
<td>102</td>
<td>87</td>
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<td>62</td>
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<td>42</td>
<td>42</td>
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<td>42</td>
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<tr>
<td>6 - 1/2&quot;Ø LOAD (PSF)</td>
<td>365</td>
<td>341</td>
<td>318</td>
<td>299</td>
<td>271</td>
<td>239</td>
<td>211</td>
<td>187</td>
<td>165</td>
<td>146</td>
<td>129</td>
<td>114</td>
<td>114</td>
<td>101</td>
<td>88</td>
<td>77</td>
<td>67</td>
<td>58</td>
<td>50</td>
</tr>
<tr>
<td>7 - 1/2&quot;Ø LOAD (PSF)</td>
<td>367</td>
<td>342</td>
<td>321</td>
<td>300</td>
<td>282</td>
<td>265</td>
<td>243</td>
<td>221</td>
<td>202</td>
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</table>

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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, large or slender openings in walls, and narrow widths. The allowable loads shown in this table reflect a 2-hour & 0-minute fire resistance rating.

8SF2.0T

The Commonwealth Medical College | Scranton, PA
Appendix F

Weight of Systems

<table>
<thead>
<tr>
<th>Composite System</th>
<th></th>
<th>Tech 2</th>
<th></th>
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</thead>
<tbody>
<tr>
<td></td>
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</tr>
<tr>
<td>Slab/deck = 75 psf</td>
<td>Beam = 55 psf</td>
<td>Girders = 94 psf</td>
<td>W_{sw} = 75 psf + 55 \times 0.66 + 94 \times 0.65 = 184 psf</td>
<td></td>
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</tr>
</tbody>
</table>

Non composite on Joists and Joist Girders:

| | | | | |
| Slab/deck = 31 psf | Joists = 12.2 psf | Girders = 82 psf | W_{sw} = 31 psf + 12.2 psf \times 0.66 + 82 psf \times 0.65 = 34.8 psf | |

One-way Slab on Concrete Beams:

Layout 1

| Slab/deck = 143.8 psf | Beam = 150 (33 5/12)(2 1/2) = 733 psf | Rebars = 1.502 \times 1.2 \times 2.5 = 3.8 psf | W_{sw} = 143.8 psf + 733 \times 0.66 + 3.8 psf = 175.7 psf | |

Precast Plank on Wide Flange Girders:

| Slab = 86.25 psf | Girders = 84 psf | W_{sw} = 86.25 psf + 84 \times 0.66 = 89.5 psf | |

Note: For one-way slab, there is calculation for both layout 1 and 2. Only layout 2 will be used to compared with the other 3 floor system because its more efficient.
### System Depths

<table>
<thead>
<tr>
<th></th>
<th>Tech 2</th>
<th>Page 2 of 5</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Composite System</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>slab = 7.5&quot;</td>
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<tr>
<td>beam = 18.1&quot;</td>
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<td></td>
</tr>
<tr>
<td>girder = 26.9&quot;</td>
<td></td>
<td>⇒ controls</td>
</tr>
<tr>
<td>d = 7.5&quot; + 26.9&quot; = 34.4&quot;</td>
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<td></td>
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<tr>
<td><strong>Noncomposite on Joists and Joist Girders</strong></td>
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</tr>
<tr>
<td>slab = 3.0&quot;</td>
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<td></td>
</tr>
<tr>
<td>joint = 26&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>girder joint = 28&quot; ⇒ controls</td>
<td></td>
<td></td>
</tr>
<tr>
<td>d = 3.0&quot; + 28&quot; = 31&quot;</td>
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<tr>
<td><strong>One-way slab on concrete beams</strong></td>
<td></td>
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<tr>
<td>Layout 1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>slab = 11.5&quot;</td>
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<tr>
<td>beam = 33.5&quot;</td>
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</tr>
<tr>
<td>d = 33.5&quot;</td>
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<tr>
<td>Layout 2</td>
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</tr>
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<td>slab = 6&quot;</td>
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<tr>
<td>beam = 22.5&quot;</td>
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<tr>
<td>girder = 25.5&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>d = 25.5&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Precast plank on wide Flange Girders</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>slab = 10&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>girder = 24.7&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>d = 10&quot; + 24.7&quot; = 34.7&quot;</td>
<td></td>
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</tbody>
</table>
Deflections

Composite System:

 Beam: 0.59 in = Δ_TL / 0.29 in = Δ_LL
 Girder: 0.53 in = Δ_TL / 0.22 in = Δ_LL

\[
\frac{0.59 + 0.53}{2} = 1.12\text{ in}
\]

\[
\frac{1.12}{2} = 0.56\text{ in}
\]

Noncomposite on Joints and Joint Girder:

 Joints: 0.843 in = Δ_TL / 10.566 in = Δ_LL
 Joint Girder: 0.601 in = Δ_TL / 0.403 in = Δ_LL

One-Way Slab on Concrete Beams:

 Beam: Δ_TL = \frac{5(6)(30)(1725)}{384(3605)(12,914)} = \frac{144}{14} = 10.3 in
 I = \frac{1}{12}bh^3 = \frac{1}{12}(12)(1.5)^3 = 6.9 kft
 E = 52,000 (4000) = 3605 ksi

 Beam: W_{LL} = \left(\frac{144 + 10.566}{2}\right) + 100 = 56.25 + 100 = 156.25

 Slab: W_{LL} = 100 (1) = 100

\[
I = \frac{1}{6}bh^3 = \frac{1}{6}(12)(1.5)^2 = 6.9 kft
\]

 Beam: Δ_LL = 0.168 in

 Beam: Δ_TL = \frac{5(6)(30)^2(1725)}{384(3605)(12,914)} = 0.106

 Beam: W_{LL} = 100 (1) = 100

 I = \frac{1}{12}(1512)(22.5)^3 = 19,614.5

 Girder: D_LL = P x (2x - 4x) = \frac{15.5(12)^2(2x - 4x) - 4(12)}{480} = 0.241

 Δ_{LL} = 0.6 in
<table>
<thead>
<tr>
<th>Slab</th>
<th>Deflections (continue)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Delta_{TL} = \frac{5(1.75)(13)^4(1725)}{384(36.25)(2.16)} = 0.144'' &lt; 0.5''$</td>
<td>$\Delta_{IL} = \frac{5(1.1)(15)^4(1725)}{384(36.0)(2.16)} = 0.0826'' &lt; 1''$</td>
</tr>
</tbody>
</table>

Precast Plank on whole Flange Girders:

- Slab: $0.129'' = \Delta_{TL}$, $0.073'' = \Delta_{IL}$
- Joint Girders: $1.19'' = \Delta_{TL}$, $0.53'' = \Delta_{IL}$

$0.129 + 1.19 = 1.32''$
Fire Ratings

2 hr fire rating for all systems required.

Composite System:
Using Unprotected Deck, 4½" NWC Topping, 3ULI = 2 hr fire rating
Beams and girders are 2 hr spray fire proofing

Non-composite or Joists and Jast Girders:
2 hr spray fire proofing for beam, joists and joint girders.

One-way Slab on Concrete Beams:
Using a 11.5" concrete slab is thick enough to reach a fire rating of 2 hr. No additional protection is required.
6.0" concrete slab > 5" for 2 hr fire rating. No additional protection is required.

Precast Plank on wide Flange Girders:
Hollow Core Planks are 2 hr fire ratings with 2" Concrete Topping,
Girders are 2 hr spray fire proofing.