Technical Report II

Nicholas Leonard
Structural Option
Advisor: Kevin Parfitt

12 October 2012

*Courtesy of Ballinger

UMMC Trauma Critical Care Tower
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Executive Summary

The purpose of this report is to analyze the existing floor structure of a typical bay, and become familiar with its construction. Once completed, I was to design three alternative floor systems that could replace the existing, and compare and contrast all four.

Through my research and analysis, the three additional systems chosen are as follows: concrete flat plate, fully composite steel and deck, and prestressed hollow core planks. A full-out comparison between the four was performed, which included topics such as: cost per square foot, floor depth, floor weight, fire rating, man hours for construction, and other topics. Many have unique characteristics, but after an extensive analysis, I would prefer to use the hollow core plank system. Out of all four, it is the lightest and fastest to construct, while spanning distances with the shallowest floor depth. Cost is on the relative low side. I feel it is for superior to all the other systems.

Using all these materials/systems in practice has reinforced my knowledge and helped me become a better engineer. While I learned most of the material in classes, for the design of the hollow core system, I was forced to expand and learn on my own time. What I discovered was an engineering masterpiece. In the following pages, I will go into more detail about the layout of the four systems, and then go into an extensive comparison between all four.
# General Building Information

<table>
<thead>
<tr>
<th>Building Name</th>
<th>Trauma Critical Care Tower</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td>22 South Greene Street</td>
</tr>
<tr>
<td></td>
<td>Baltimore, MD 21201</td>
</tr>
<tr>
<td>Occupant Name</td>
<td>University of Maryland Medical Center</td>
</tr>
<tr>
<td>Function</td>
<td>Additional Operating and Patient Rooms</td>
</tr>
<tr>
<td>Total Floor Area</td>
<td>140,000 SF</td>
</tr>
<tr>
<td>Levels</td>
<td>7 Stories + Penthouse</td>
</tr>
<tr>
<td>Anticipated Completion</td>
<td>2013</td>
</tr>
<tr>
<td>Cost</td>
<td>$89,225,671.00</td>
</tr>
<tr>
<td>Delivery Method</td>
<td>Design-Bid-Build</td>
</tr>
<tr>
<td>Contract Type</td>
<td>Guaranteed Max Price</td>
</tr>
</tbody>
</table>

## Project Team

<table>
<thead>
<tr>
<th>Owner</th>
<th>University of Maryland Medical System</th>
<th><a href="http://www.umm.edu/">http://www.umm.edu/</a></th>
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</thead>
<tbody>
<tr>
<td>Civil &amp; Landscape</td>
<td>Site Resources, Inc.</td>
<td><a href="http://www.siteresourcesinc.com/">http://www.siteresourcesinc.com/</a></td>
</tr>
<tr>
<td>Architect</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Protection</td>
<td></td>
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</tr>
</tbody>
</table>

Advisor: Parfitt
Architecture

With an increased and still rising demand for medical services, it was necessary for the University of Maryland Medical System to expand their resources. As a result, a decision was made to build the 140,000 SF Trauma Critical Care Tower, emanating 116+ feet over the streets of Baltimore. When completed in 2013, the tower will service an additional five operating/trauma rooms, as well as 60 patient rooms.

Though not a stand-alone building, but rather, an expansion to the already existing Medical Center, the tower’s program blends well with the floor plans of the Medical Center, resulting with a smooth, homogeneous circulation. Proven to be the most effective layout, the tower has incorporated the use of singly loaded corridors, with nurse stations and storage occupying the center, ergo providing quick access when necessary.

To utilize the as much space as possible, air space was purchased to allow the architects to innovatively design a protruding cantilever beginning at the third story and spanning out over the sidewalk, providing both an awning, but more importantly, an aesthetic appeal. The materials that compose of the façade were selected to match the existing façade and ornament of the Medical Center. Yet, the addition of large quantities of glass, aluminum, and terra cotta stone, give the building a more contemporary feel. An example of the façade can be seen in Figure 1. Overall, the tower emanates a welcoming essence that provides a quality and comfortable atmosphere for doctors, patients, and students alike.

Figure 1: Terra Cotta Facade
*Courtesy of Ballinger

Advisor: Parfitt
Materials

The most commonly used parameters for the design of the floor systems are indicated in red in the following tables.

—Concrete

<table>
<thead>
<tr>
<th>Description</th>
<th>Compressive Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab—on—Grade</td>
<td>4000</td>
</tr>
<tr>
<td>Framed Slabs, Beams, Walls, and Mat Foundation</td>
<td>5000</td>
</tr>
<tr>
<td>Columns</td>
<td>4500, 6000, 8000</td>
</tr>
<tr>
<td>Shear Walls</td>
<td>6000</td>
</tr>
<tr>
<td>Roof Slab, Joists, and Beams</td>
<td>6000</td>
</tr>
</tbody>
</table>

Table 1

—Reinforcing Bars

<table>
<thead>
<tr>
<th>Description</th>
<th>ASTM Standard</th>
<th>Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>All Deformed</td>
<td>A—615</td>
<td>Gr 60</td>
</tr>
<tr>
<td>Welded</td>
<td>A—706</td>
<td>Gr 60</td>
</tr>
<tr>
<td>Stainless Steel</td>
<td>A—955</td>
<td>Gr 60</td>
</tr>
<tr>
<td>Threaded (Dywidag)</td>
<td>A—722</td>
<td>Gr 75</td>
</tr>
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</table>

Table 2

—Welded Wire Fabric

<table>
<thead>
<tr>
<th>Description</th>
<th>ASTM Standard</th>
<th>Grade</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete Framed Slabs</td>
<td>A—185</td>
<td>Gr 65</td>
<td>6”x6” – W5.4/W5.4</td>
</tr>
<tr>
<td>Concrete Slab—on—Grade</td>
<td>A—185</td>
<td>Gr 65</td>
<td>6”x6” – W2.9/W2.9</td>
</tr>
<tr>
<td>Mechanical Slab</td>
<td>A—185</td>
<td>Gr 65</td>
<td>6”x6” – W6.5/W6.5</td>
</tr>
</tbody>
</table>

Table 3
Typical Clear Cover Schedule

<table>
<thead>
<tr>
<th>Description</th>
<th>Minimum Cover</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cast Against Earth</td>
<td>3”</td>
</tr>
<tr>
<td>Exposed to Earth or Weather</td>
<td></td>
</tr>
<tr>
<td>No. 6 and larger bars</td>
<td>2”</td>
</tr>
<tr>
<td>No. 5 and smaller bars</td>
<td>1 – 1½”</td>
</tr>
<tr>
<td>Not Exposed to Earth and Weather</td>
<td></td>
</tr>
<tr>
<td>Slabs and Walls</td>
<td>3/4”</td>
</tr>
<tr>
<td>Beams and Columns</td>
<td>1 – 1½”</td>
</tr>
</tbody>
</table>

Table 4

Structural Steel

<table>
<thead>
<tr>
<th>Description</th>
<th>ASTM Standard</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plates, Angles, Channels, Bars, and Rolled S, M, and HP Shapes</td>
<td>A–36</td>
</tr>
<tr>
<td>Rolled Wide Flange Shapes</td>
<td>A–992’ Grade 50</td>
</tr>
<tr>
<td></td>
<td>A–572’ Grade 50</td>
</tr>
<tr>
<td></td>
<td>AH–36’ Grade 50</td>
</tr>
<tr>
<td></td>
<td>A–588’ Grade B</td>
</tr>
<tr>
<td>Tubular Shapes</td>
<td>A–500’ Grade B</td>
</tr>
<tr>
<td>Pipe Shapes</td>
<td>A–53’ Type E or S’ Grade 50</td>
</tr>
</tbody>
</table>

Table 5

Fasteners

<table>
<thead>
<tr>
<th>Description</th>
<th>Standard</th>
</tr>
</thead>
<tbody>
<tr>
<td>Anchor Bolts</td>
<td>F–1554, Grade 36</td>
</tr>
<tr>
<td>Bolts</td>
<td>A–325</td>
</tr>
<tr>
<td>Headed Stud Type Connectors</td>
<td>A–108, Grade 1015 or 1020</td>
</tr>
<tr>
<td>Welds</td>
<td>AWS D1.1–2002</td>
</tr>
</tbody>
</table>

Table 6
Masonry

<table>
<thead>
<tr>
<th>Description</th>
<th>Standard</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solid Masonry</td>
<td>C–90, Grade N1</td>
<td>May be as low as 75% solid, f’m=1900psi</td>
</tr>
<tr>
<td>Hollow Masonry</td>
<td>C–90, Grade N1</td>
<td>f’m=1900psi</td>
</tr>
<tr>
<td>Mortar</td>
<td>Type S, C–270</td>
<td>1800 psi at 28 days</td>
</tr>
</tbody>
</table>

Codes Used

- International Building Code 2006
- ACI 318–05
- ACI 530–05/ASCE 5–05
- AISC 13th Edition
- NFPA 101 2006
- ASCE 7–05
- Electric Building Code 2008
Design Loads

All design loads were calculated in accordance with ASCE 7–05.

—Live Loads

<table>
<thead>
<tr>
<th>Load Type</th>
<th>Design Load Used (psf)</th>
<th>ASCE 7–05 Minimum Loads (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Typical Floors/Stairs</td>
<td>100</td>
<td>Operating Rooms = 60 Patient Rooms = 40 First Floor Corridors = 100 Corridors Above 1st Floor = 80 Stairs = 100</td>
</tr>
<tr>
<td>Mechanical Room</td>
<td>150</td>
<td>NA</td>
</tr>
<tr>
<td>Equipment Room</td>
<td>150</td>
<td>NA</td>
</tr>
<tr>
<td>Electrical Substation</td>
<td>100</td>
<td>NA</td>
</tr>
<tr>
<td>Helistop</td>
<td>100 or 26 kip helicopter impact</td>
<td>NA</td>
</tr>
<tr>
<td>Roofs</td>
<td>30 + snow drift</td>
<td>20</td>
</tr>
</tbody>
</table>

*Table 8*

The live load chosen for design is 100 psf as indicated in Table 8 above. Even though the typical bay of interest supports patient rooms, which only requires a live load of 40 psf as written by ASCE 7–05, I felt it was more appropriate to use 100 psf. Because we can never be sure on the function of the building during its life span, it is conservative to design the entire structure to the maximum expected live load. In that case, if the rooms were to change function, say become storage, the floor is able to support the increased load. Secondly, because a hospital is a mission critical facility, designing for extreme events is always a wise decision. Finally, it becomes easier for analysis when the live load is constant across the entire floor, which leads to fewer mistakes.
Existing Floor System

-Concrete One-Way Slab and Joists

For the purpose of this report, I have chosen a typical bay on the fifth floor, which supports two patient rooms. Because the majority of the floor space in the new addition is designated for patient care, I felt it was appropriate to perform analyses on those rooms. Figure 2 shows a callout of the aforementioned bay on the fifth floor.

Figure 2: Call out of Bay on 5th Floor

*Courtesy of Ballinger
The as-built structural frame consists of a 4–1/2" concrete one-way slab, supported by a series of concrete joists spaced at 6’–3” apart. Fire is of the utmost importance in a hospital, so the 4–1/2” slab provides a 2 hr fire rating that requires no additional fire protection, unlike steel.

Figure 3 shows a typical section through the joists. Reinforcement typically consists of No. 7 bars for the joists; however, the slab contains welded-wire-fabric. I prefer the use of WWF for slabs whenever feasible because of the ease of placement for the reinforcement. When the construction process moves quicker, that generally means more profit for the Contractor and Engineer. I would like to also point out that in this particular section, the WWF is draped at the mid-span. This allows the steel to engage the positive moment region of the slab without placing another layer of WWF. The joists are supported by a concrete girder on either side, the same depth as the joists, but a rather large width in the range of 30”–36” compared to the 9” for the joists. After a quick comparison of this system to others outlined later in the report, I find that this system to be a more favorable one. Not only does it provide significant stiffness in lateral support in the form of a rigid diaphragm, it provides decent resistance to vibrations because of mass, which could be very helpful in a hospital setting due to a variety of equipment. On the following page Figure 4 illustrates a clear perspective and call out of the one-way slab system.
Figure 4: Existing one-way slab system
Alternate Floor Systems

— Flat Plate

As one of the more efficient and popular floor designs, I was prompted to use a flat plate for my first alternate floor system. It is popular because of its simplicity, which tends to win the Contractor’s bids. Illustrated in Figure 5 below is the typical flat plate for the bay in the Shock Trauma Tower.

Due to the nature of a flat slab, I was able to reduce the depth of the floor from 18.5” to 12”. This was primarily achieved through the two-way behavior of the slab. The moments are distributed more evenly in both directions, thus reducing the overall maximum design moment. It was soon discovered during analysis using RAM Concept (Appendix D) that strength was not the governing factor, since the capacity of the system maxed out at 71.6% in flexure. Rather, serviceability controlled. Two-way slabs are generally stiffer than one way, thus you are able to span greater distances with small members. Also, at any one column, punching shear reached a maximum capacity of 68.2%, ergo no additional reinforcement was needed.

Advisor: Parfitt
Fully Composite Deck

Since the first two designs were constructed out of concrete, I wanted to shift my attention to a new material for the framing system. I decided to attempt a design with a composite steel deck, with the calculated framing plan indicated in Figure 6.

With the spans approaching longer distances, it became more efficient to utilize full composite action of the beam and deck. Because of the room partitions of the patient rooms above, and the position of the joists with respect to the mullions, it was more practical to use a spacing of 6’-3” previously used in the existing design. To save costs from shoring, 1.5VL22 deck was chosen for its ability to meet the unshored span requirement. Including the 4-1/2” concrete topping for a 2 hr fire rating, the total slab depth came out to be 6”; however, with the addition of the wide-flange steel girder, the total floor depth was 22”, a significant increase from 18.5”. The 314 shear studs spread throughout the bay allow the steel to fully engage the concrete. Figure 7 shows a perspective of the composite bay.
Prestressed Hollow Core Planks

Figure 8: Call out of H.C Plank Layout (typ).

Figure 9: Support of H.C Planks

Figure 10: Steel Framing wrt H.C Plank

Figure 11: H.C. Plank Detail

Advisor: Parfitt
By far, the prestressed hollow core plank system was the easiest to design. Using the product manual from Oldcastle Precast Building Systems, I selected an 8” deep H.C. plank, able to stretch the entire span, girder to girder, thus opening up the floor-to-floor height. This allows the possibility for higher ceilings, or the allowance for more mechanical equipment/ductwork in the ceiling. The H.C planks rest as simply-supported members on two wide-flange girders, without the need for a concrete topping while maintaining the requirements for a 2hr fire rating. Because of the simplicity of the structure, this method allows for easy constructability, making this structure a very desirable alternative, and my favorite up-to-date.

Within the next couple of pages of this report, I will go into more depth about each system by comparing and contrasting all four (4) systems, and explaining the work behind the numbers. These past few pages were just an introduction to get you familiar with each system before diving in.


Compare and Contrast

<table>
<thead>
<tr>
<th></th>
<th>One-Way Slab System</th>
<th>Flat Plate</th>
<th>Fully Composite</th>
<th>Prestressed Hollow Core Planks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cost/S.F.</td>
<td>$16.38</td>
<td>$16.36</td>
<td>$18.43</td>
<td>$16.69</td>
</tr>
<tr>
<td>Depth</td>
<td>18.5 in</td>
<td>12 in</td>
<td>22 in</td>
<td>8 in/29 in</td>
</tr>
<tr>
<td>Largest Capacity of a Single Element</td>
<td>97.5%</td>
<td>71.6%</td>
<td>63.1%</td>
<td>97.5%</td>
</tr>
<tr>
<td>Fire Rating</td>
<td>2 HR</td>
<td>2 HR</td>
<td>2HR</td>
<td>2HR</td>
</tr>
<tr>
<td>Weight</td>
<td>132 psf</td>
<td>150 psf</td>
<td>85 psf</td>
<td>74 psf</td>
</tr>
<tr>
<td>Man Hours</td>
<td>114 hrs</td>
<td>113 hrs</td>
<td>55 hrs</td>
<td>37 hrs</td>
</tr>
<tr>
<td>Constructability</td>
<td>Difficult</td>
<td>Moderate</td>
<td>Moderate</td>
<td>Easy</td>
</tr>
<tr>
<td>Cutting Holes</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>Lateral Stiffness</td>
<td>Very Rigid</td>
<td>Rigid</td>
<td>Rigid</td>
<td>Flexible</td>
</tr>
</tbody>
</table>

Table 9

- Cost

With just a quick glance at Table 9, except for fully composite, all of the other systems have relatively similar costs per square foot and seem reasonable to actual industry costs. Though the cost of concrete is relatively cheap to the cost of steel, since concrete requires more preparation in the form of formwork, we tend to see higher costs in labor when placing concrete. If I were to look at costs alone, a choice of one-way slabs, flat plate, or hollow core would be excellent.

Flat plate and one-way shear were nearly identical. After performing a quantity takeoff for the two systems, I have flat plate requiring 27 CY of while the other only needs 17 CY. So why do these two systems have identical costs? The difference is that flat plate requires very little formwork and effort the erect the formwork into place, thus significantly offsetting the cost of the additional concrete. Because of the intricate system of the one-way slab, more effort is requires to erect formwork, thus driving up the cost.

Advisor: Parfitt
The next question is why the fully composite system is relatively expensive? That is because of the higher quantities of steel, in the form of structural steel and decking. Though it’s a fairly easy system to erect, the advantage is offset by the cost of the steel. Also, we need to account for the welding of the shear studs. There are a little over 300 shear studs in this one bay, and each one costs a significant amount to mount. As we’ll see later, the extra cost is compensated for the significant strength of the system.

- Depth

As per usual, depth of the floor is of major consideration, especially to the architect and the owner. For the owner, reducing the floor-to-floor height for every story in a 30 story high rise can significantly reduce the cost of the structure. For the Architect, they can achieve more with their allocated space to present their ideas.

Though out the systems, I encountered a wide variety of depths, ranging from 8” to 29” (Nearly a 2 foot difference!). As expected, the depth of the flat slab came in first with a low uniform 12”. As mentioned earlier, the control for that depth was in fact the serviceability requirement, or small deflections. Because the flat plate utilizes 2-way action, the slab is able to distribute those moments to reduce the maximum design moment. I would like to mention that earlier I said depths ranged from as low as 8”. The reason I still put flat plate as the smallest at 12” is because the hollow core plank does span a large ~25 ft span with just 8” (which is wonderful for the mechanical engineers and architects); however, a 21” deep wide flange is needed to support those planks. If prudent planning is achieved, those girders can be hidden and the building could utilize the 8” thick slab. The reason the hollow core planks can achieve such a short depth is because they are prestressed, typically forming a camber. Typically, the slab thickness is determined by the fire rating required by IBC. Good rule of thumb is 4–1/2” for a 2hr fire rating for unprotected concrete.
Capacity

In case there is still confusion on this category, this number represents the max utilization of capacity of a particular element in its respective system. I choose to list to category to show how efficient the systems are utilized. With the industry pushing engineers to build more with less, this category becomes a huge factor. If we want to become more efficient in our design, it’s imperative to design close to full capacity. Some could argue though that because of the mistakes that could happen in the field (or elsewhere for the matter), having reserve capacity is a good thing. It is up to the supervisor in charge to ensure things are done correctly on the field to help alleviate this issue.

Notice, similar to depth, that there are a wide variety of ranges for capacity. Notice that the composite is as max of 63.1%. Also, note that the composite system has a few elements that are as low as 20%. As an outsider looking at this, there are two possible reasons for this: (1) the engineer in charge wanted to be extremely conservative, and thus oversized all of the members. While this is good for safety reasons, unless there are special conditions, this would be overkill. These loads and equations that engineers use on a daily basis are already hit with safety factors multiple times, so it would be too conservation to design with such beefy members. (2), which is in my case here, strength is not the controlling factor. In the design for this composite system, wet concrete deflection was the controlling factor.

On the opposite end of the spectrum, the one-way slab and hollow core planks are very efficient. Because, these two systems are similar in cost, it will come down to the amount of material used, or other factors. For example, if a building is shooting for LEED credits, using less material for the same result would be desired. Just based on strength, I would prefer to use the one-way slab system because of its higher stiffness to help contribute lateral resistance.
- Fire Rating

Because of the code provisions in NFPA 101 2006, all the floors must have a 2 hr fire rating. All systems are designed for the 2 hr fire rating; however, one might prefer one system over another. For example, I personally would like concrete systems because of their inherent ability to resist fire. If the fire proofing were to at any point be compromised, any steel would be vulnerable to heat and potentially fail. Though concrete can still falter in heat, it is known to be tougher than steel, and thus my preferred choice of material. As a side note, because steel requires additional fire proofing, that tends to drive up the cost for steel, which could persuade engineers to choose concrete.

- Weight

Weight is a very important characteristic, especially for upper floors in a multistory building. If the loads are reduced due to weight, that leaves the potential to size down any lower elements, thus reducing the cost of the building.

In my comparison in Table 9, the flat plate weight significantly more than the one-way slab, this can also be deceiving, because the flat plate is shallower. Twelve inches of concrete are spread evenly throughout the floor, while the one-way slab has the depth of just the beams and joists of 18.5". Since I had just performed an analysis on the fifth floor, I would have to consider the effect of the columns below if I were to increase the weight of all my floors to a flat plate. Some repercussions would be a repeated look at second order (P-Delta) effects, as well as, the foundation design.

Another consideration would be the weight in relation to an earthquake response. When using ASCE 7-05, since the base shear is a function of building weight, the heavier the building, the greater the earthquake forces. For example, the difference in weight between
a flat plate and hollow core planks is nearly two. That means that for the flat plate, the earthquake forces will be twice as large. If possible, I would like to reduce the amount of lateral influence because lateral forces have a greater effect on structures compared to gravity loads.

Oppositely, it should be cautioned to not use too much of a lightweight system. This pertains mainly towards vibrations and stability issues. When a structure is lighter, it tends to have a lower frequency. Many pieces of machinery and other sources frequently use a lower frequency; ergo, there is a greater chance for the structure to reach resonance. In my designs, I feel that all of these structures are plenty heavy. I am more concerned about saving weight, so if I were to base a selection on just weight, I would most definitely go for the hollow core plank.

— Man Hours

Often running closely with constructability, and also taken from RS Means while performing a quantity take-off, man hours indicates how much time is required to install the respective system. One of a Contractor’s main concerns is the ability to stay on schedule. When a project over shoots a deadline, for each day the project is still over-due, the owner 9 times out of ten loses a gratuitous amount of money. With systems that require shorter installing durations, a project is often completed on time and saves the owner a significant amount of money.

In my opinion, there is direct correlation of man hours on a project and the probability for something to go wrong. More often than not, the longer a project takes, the more complicated the system, and as a result, something goes wrong. In the case of the difference between a one-way slab system and hollow core planks, the one-way system contains many elements of varying sizes and shapes, and as a result, takes three times more time to get the floor
erected (and that is excluding the time to allow the green concrete to cure). Not only that, the one-way slab has reinforcement at different levels that require special attention to sequencing, which is only another ingredient to the recipe for disaster (or in this case, the rebar misplacement, or the concrete with the wrong water/cement ratio).

Composite decking is very popular because of its speed of erection, along with all the other advantages, such as strength as mentioned earlier.

— Miscellaneous

When designing floor systems, it is imperative to keep in mind the function of the building. In the case of the Shock Trauma Center in Baltimore, as most obviously deduced, this building is a hospital. Hospitals tend to have a lot of equipment that specifically require punching through the slab to feed wire and tubes. Because some floor systems are vulnerable to this, choosing the right system for the allowance for versatility of drilling holes in the slab is desired.
In the case of the flat slab, the delicacy of the two-way slab prevents the tampering of the system because of redistribution of moment forces. For the hollow core planks, you run the risk of cutting through one of the prestressed strands, and thus reduction of strength of the member. If clearly stated to the owner of the building that punching holes in the slab is forbidden, than either of these systems are a viable choice. The composite and one-way slab systems have enough redundancy that allow for such drilling.

Lastly, I would like to briefly just comment of the effect of lateral stiffness from the influence of each of these structures. Because of the nature of the one-way and composite systems, those would provide a more rigid diaphragm, aiding in the resistance in lateral deformations.
Conclusion

The scope of this report was to compare the existing floor system for a typical bay with three other alternatives. The four systems include:

- One-way Concrete Slab with Joists
- Flat Plate
- Fully Composite Deck
- Prestressed Hollow Core Planks

Each one has their own advantages and disadvantages. The one-way concrete slab provides a redundant, stiff system that is fairly efficient and is a relatively low cost. The flat plate allows for shallower floor depths, while still providing rigidity and low cost; however, the weight of the concrete could be negative. Fully composite steel and decks provide a significant increase in strength, relatively fast to construct, lightweight, and rigid, however cost a significant amount more. Lastly, and what I would recommend for a floor system, would be the prestressed hollow core planks. They are the lightest, and fastest to construct out of all the systems. A fairly strong and efficient system, and provides shallow depth in between supports. Most importantly, it is relatively cheap. The only downside is that it is not versatile when it comes to future modifications and does not provide much rigidity for lateral support. Though negative, this can be offset with additional bracing, which in natural is an effective lateral resisting method.

Through my analysis of the four different floor systems, I was able to better understand the relationship between these floor structures, through advantages and disadvantages of each. I would like to give a special thanks to Mr. Parfitt and Mr. Taylor for their help on this assignment.
Appendices
Appendix A: Floor Framing Plans

Foundation
+2\textsuperscript{nd} Floor
+3rd Floor
+4th Floor
+5th Floor
+6th Floor
+Penthouse
+Equipment Platform
+Roof
+Helipad
Appendix B: Floor System Calculations

**Existing Structure**

**Typical Joist Section:**

- 1½" CUL
- (4) #9 Top
- (2) 9/16" CUL
- (3) STUDS @ 8" OSCT
- 4½" H25

**Slab Analysis:**

- **L.L.** = 100 psf [UNREDUCIBLE]
- **D.L.** = \((150 \text{ pcf})(4.5''/12''/4'') = 56.25 \text{ psf}\)

*Note: Per ASCE 04-05, only 40 psf is required to account for self-weight of building and allowance for fixtures and extra equipment. Consideration to say L.L. = 100 psf.**

**Conclusion:**

\[1.2D + 1.6L = 1.2(50.25) + 1.6(100) = 229.5 \text{ psf}\]

Original System 1
DEFLECTION CHECK:

ACCORDING TO ACI 318-11, TABLE 9.5(a)

For one way slabs, both ends continuous

\[ h_{min} = \frac{L}{2 \phi} = \frac{95}{28} = 3.4 \text{ in.} < 4.5 \text{ in.} \ \text{[Good]} \]

ANALYZING A 1' STRIP OF SLAB

BY USING ACI MOMENT COEFFICIENTS

\[ \text{CONTROLLING} \ \frac{W_n l^2}{11} \ [\text{NEGATIVE MOMENT @ ALL OTHER INTERIOR SUPPORTS}] \]

\[ \frac{W_n l^2}{16} \ [\text{POSITIVE MOMENT @ INTERIOR SPANS}] \]

\[ W_n = (227.5 \text{ psf})(14) = 227.5 \text{ lb/ft} \]

\[ M_n = \frac{(227.5 \text{ psf})(5.5 \text{ ft})^2}{11} = 626 \text{ lb-ft} \ \text{[NEGATIVE MOMENT]} \]

\[ h_n = \frac{d}{12} = \frac{4.75 - 1/4}{12} = 0.08 \]

\[ \lambda = \frac{(0.100 \text{ in}^2)(65 \text{ ksf})}{0.85(65 \text{ ksf})(12 \text{ in})} = 0.128 \]

\[ \rho = \frac{0.1080}{(12'')(2.95')} = 0.0024 \]

\[ \rho_{min} = 0.0012 \ \text{FOR SPAN, RACE AND TEMP} \]

\[ M_n = (0.100 \text{ in}^2)(65 \text{ ksf}) \left[ 3.75'' - \frac{0.128}{2} \right] \]

\[ M_n = 25.84 \text{ k-in} = 2.15 \text{ k-ft} \ \text{[Good]} \]

\[ E = \frac{6000}{0.193''} \left[ 3.75'' - 0.135'' \right] = 0.062 > 0.005 \]

\[ \phi = 0.9 \]

\[ \delta \frac{M_n}{0.4(2.15 \text{ k-ft})} = 19.35 \text{ lb-ft} \ % \ \text{CAPACITY} \]

\[ \frac{227.5 \times 100}{19.35} = 32.4\% \]
Because the WF is laid out in the same position as the negative moment area, the positive moments will also pass.

Maximum spacing of WF strands:

\[ S_{min} = 18'' \times 3 = 3(4.5) = 13.5'' \]

\[ 6'' \leq 13.5'' \quad \text{Good} \]

One-way shear:

Load at face of support:

\[ \frac{V_n}{2} = \frac{W_n \cdot l_n}{2} = \frac{(229.5 \text{ lb})(5.5)}{2} = 626 \text{ lb} \]

* Code allows to take shear off away from support, but to fully conservatively, will take shear from support face.

\[ V_n = V_c + V_s \rightarrow 0 \quad \text{[no shear reinforcement required]} \]

\[ V_c = 2\sqrt{5200 \cdot (3.75')(12')} = 6364 \text{ lb} \]

\[ /2 \beta V_c = /2(0.45')(6364 \text{ lb}) = 2386 \text{ lb} \]

% capacity = \[ \frac{626}{2386} \times 100\% = 26.3\% \]

Comments:

Though moment and shear capacity utilization are low, there must be some other reason why slab is 4\(1/2\)'' thick. Detection only requires 2\(3/4\)'' of thickness. Though, after a search for fire ratings, it is required to have 4\(1/2\)'' for a 2hr fire rating.
**Analysis:**

The area per foot = 9.5” x 28.2” = 150 ft²

- Live Load (LL) = 100 psf
- Dead Load (DL) = 56.28 psf + 15 psf + (150 psf)(9.5/18.5)(6.25) = 93 psf

Total Load = 272 psf

**Deflection Check:**

\[
W_0 = \frac{(292 \text{ psf})(6.25 \text{ ft})}{18.5} = 17.7 \text{ k/ft}
\]

**Ultimate Moment Using ACI 318-11 Moment Coefficient Method**

Negative Moment @ Exterior:

\[
W_m \cdot \frac{L^2}{24}
\]

Positive Moment @ End Span:

\[
W_m \cdot \frac{L^2}{14}
\]

Negative Moment @ Interior:

\[
W_m \cdot \frac{L^2}{10}
\]

- \(L = 28.5\) in - 15” = 13” = 24.9” = 20.13’

**Nominal Strength of Negative Section (ND T-Bend Behavior)**

- \(d = 10.2” - 0.593” = 9.607” = 10.12”
- \(A_s = 4(0.6) = 2.4 \text{ in}^2
\)
- \(\rho = \frac{2.4 \text{ in}^2}{(2.4 \text{ in})(0.6)} = 0.5165
\)
- \(a = \frac{(2.4)(0.6)}{0.85(10.12)} = 3.76”
\)
- \(L = 5.96 / 0.8 = 7.47”
\)

- \(m_n = (1.41)[16.2 - 1.7/2] = 181.8 \text{ k-ft}
\)

- \(E_n = \frac{0.003(181.8)}{4.7} (16.2 - 5.7) = 0.0093
\)

- \(0.9 \cdot 181.8 = 155 \text{ k-ft}
\)

- % Capacity = \(\frac{92.25}{1.55} = 46.7%
\)
Nominal Strength of Positive Moment at Mid-Span

T-Bent Section

\[ \text{Effective Width} \quad b_{\text{eff}} = \frac{h}{2} \]

\[ b_{\text{eff}} = \frac{39.5}{2} = 19.75 \text{ in} \]

\[ \text{Effective Depth} \quad d_{\text{eff}} = \frac{h}{2} \]

\[ d_{\text{eff}} = \frac{39.5}{2} = 19.75 \text{ in} \]

\[ \text{Effective Moment of Inertia} \quad I_{\text{eff}} = I_{\text{cross}} \]

\[ I_{\text{eff}} = 142,000 \text{ in}^4 \]

\[ \text{Effective Plastic Modulus} \quad W_{\text{pl}} = \frac{I_{\text{eff}}}{d_{\text{eff}}} \]

\[ W_{\text{pl}} = 7,000 \text{ in} \]

\[ \text{Check for T-Bent Behavior} \]

\[ \beta = \frac{0.85(d)}{0.95(h)} \]

\[ \beta = \frac{0.85(4.5)}{0.95(39.5)} = 0.226 < 4.5^\circ \]

Rectangular Behavior

\[ \frac{\rho_{\text{min}}}{\rho_{\text{cr}}} = \frac{0.226}{0.8} = 0.283 \]

\[ \text{Min. Moment Capacity} \quad M_{\text{pl}} = (1.2)(0.95) \left[ (16.2 - 0.25) \right] = 19.5 \text{ k-ft} \]

\[ \varepsilon_{x} = \frac{0.064}{0.223} \left[ (16.2 - 0.25) \right] = 0.166 \quad \therefore \phi = 0.9 \]

\[ \phi M_{\text{pl}} = 0.9 \left[ 19.5 \right] = 17.6 \text{ k-ft} \]

\[ \% \text{ Capacity} = \frac{51.6}{86.9} = 59.5\% \]

Shear Capacity

\[ V_{\text{u, max}} = 1.15 \left( \frac{V_{u}}{2} \right) = 1.15 \left( \frac{1.75(1.74 \times 180)}{2} \right) = 20.2 \text{ k} \]

\[ V_{u} = V_{t} + V_{s} \]

\[ V_{t} = 2 \left( \frac{1.75(180)}{2} \right) = 20.6 \text{ k} \]

\[ V_{s} = 0.75 \left( \frac{20.6 + 26.7}{2} \right) = 35.5 \text{ k} \]

\[ V_{s} = 2 \left( \frac{1}{8} \right) \left( 60 \times 39.5 \right) = 26.7 \text{ k} \]

\[ \% \text{ Capacity} = \frac{26.7}{35.5} = 59.4\% \]

Original Section 5
Girder Analysis:

- 5EH [36" x 18.5"]

- Column DA [12" x 24"]

- TECB

- Joint 5.54

- 4 1/2"

- 36"

- 18 1/2"

- 18 1/4"

- Beam 28 - 9"

- L.L. = 100 psf

- D.L. = 93 psf + Water Density

- \[ W_e = \left(150 \text{ psf}\right) \left(\frac{36\times18.5}{144}\right) \]

- \[ = 693.8 \text{ klf/sf} \]

- \[ \text{Total Water} = 28\frac{5}{12} + 12\frac{1}{2} = 12.941\]

- \[ W_u = 1.2 \left[93(12.94) + 693.8\right] + 1.6 \left[100(12.94)\right] \]

- \[ W_u = 4.35 \text{ k/lf} \]

- Using ACI 515-11 Moment Coefficient Method:

- Since the top and bottom rebar is the same, only need to analyze negative moment area

- \[ M_n = \frac{W_u L_n^2}{11} = \frac{(4.35 \text{ k/f}) (27.25)^2}{11} = 293.7 \text{ k*ft} \]

- \[ V_n = \frac{W_u L_n}{2} = \frac{(4.35)(27.25)}{2} = 59.3 \text{ k} \]
L → NOMINAL FLEXURAL STRENGTH → NEGATIVE MOMENT

L = 18" L = 1½" GLK

\[ \text{Area} = 4(1.29 \text{ft}^2) = 5.16 \text{ in}^2 \]

\[ \rho = \frac{5.08 \text{ in}^2}{(36)(15.9)} = 0.0089 \]

\[ a = \frac{(5.08)(60)}{0.85 (5)(36)} = 2" \]

\[ \rho_m = \frac{2(5000)}{60000} = 0.0035 \quad \text{Good} \]

\[ \phi_c = \frac{0.0032}{2.5} \left[ 15.9 - 2.5 \right] = 0.0161 \]

\[ \phi = 0.9 \]

\[ m = 0.9(397.5) = 358 \text{ lb ft} \]

\[ N = \frac{293.7}{340.5} = 86.3\% \]

L → NOMINAL SHEAR CAPACITY

\[ V_c = \frac{2(5000)}{(36)(15.9)} = 90.9 \text{ k} \]

\[ V_{\text{min}} = \frac{0.75(5000)}{(36)(8)} \]

\[ V_s = \frac{2(0.24^2)(60 \text{ k}) (15.9)}{8} = 47.4 \text{ k} \]

\[ V_{\text{min}} = \frac{0.255 \text{ in}^2}{60000} \quad \text{Good} \]

\[ \phi V_n = 0.75(88.9 + 47.4) = 96.5 \text{ k} \]

\[ \% \text{ CAPACITY} = \frac{96.5}{96.5} = 61.5\% \]

L → TORSION CONSIDERATION

\[ T_n = 2 \times W_n \phi_n / 24 = \phi(1.9)(20.63) / 24 = 60.3 \text{ k ft} \]

Note: Ultimate torsion comes from the moments induced by joint connections, each support will require 2 joint connections.

Threshold Torsion:

\[ A_{ef} = 3\pi \times 18.5 = 0.66 \text{ in}^2 \]

\[ T = 0.75(5000) (666^2 / 109) = 18 \text{ k ft} \]

\[ \rho_{cp} = 2(36 \times 15.5) \times 10^4 \text{ in} \]

ORIGINAL SECTION 7

Advisor: Parfitt
* The threshold torsion is less than $T_u$, torsional reinforcement must be considered.

$$T < T_u$$

* $H$ studrup

$$17.5" \text{ cur (ft)}$$

$$A_{pl} = \left[ \frac{96-3}{51.5} \right] \cdot \frac{19.5-3}{51.5} = 8.15 \text{ in}^2$$

$$A_p = 0.85 \left( \frac{51.5}{51.5} \right) = 43.5 \text{ in}^2$$

$$T_n = \frac{2 \left( 43.5 \text{ in}^2 \right) \left( 0.2 \text{ in}^2 \right) \left( 60 \text{ kips} \right)}{8"} = 97 \text{ kips}$$

$$\phi T_n = 0.75 \left( 97 \right) = 81.6 \text{ kips}$$

\% Capacity = \frac{61.2}{81.6} = 75.1\%$$

**Shear-torsion interaction:**

$$\frac{53200}{103.6} \left( \frac{\left( \frac{923600 \text{ kips} \cdot \text{in}^2}{51.5^2} \right) \left( 99.9 \right)}{29403} \right)^2 = 0.75 \left( 10 \sqrt{53200} \right)$$

$$158.1 \text{ psi} \leq 530,3 \text{ psi} \quad \text{GOOD}$$

$$A_{shear} = \frac{\left( 0.2 \text{ in}^2 \right) \left( 99.9 \right)}{8"} = 2.43 \text{ in}^2$$

**Total longitudinal steel at support face:**

$$(8) \times 10 = 81.29 = 10.16 \text{ in}^2$$

$A_1$ for flexure = 5.08 in²

$A_1$ for torsion = 2.43 in²

$A_{pl} = 7.51 \text{ in}^2 < 10.16 \text{ in}^2 \quad \text{GOOD}$

* Original section 8
$S88 \ (30'' \times 18.5'')$

- Column D2 \[20'' \times 84''\]
  - $f_c = 5000 \, \text{psi}$
  - $f_y = 60000 \, \text{psi}$
  - $q_u = 292 \, \text{psf}$ \text{ [From Previous Analysis]}

- Trip width: $25' - 10\frac{3}{4}'' + 25'\frac{3}{4}''$

- $W_n = \frac{292 \, \text{psf} \times 24.5}{12} + \frac{1.2 \times 150 \, \text{psf} \times (30' - 18.5'') \times 0.44}{16}$

- $W_n = 7.08 \, \text{k/lf}$

- Deflection check:
  - By ACI 218-11, Table 9.5 (a)
  - Beam L/Both ends Continuous

- $W_n = \frac{L}{2} = \frac{345''}{2} = 172.5'' \leq 18.5'' \, \text{[Good]}

- Ultimate moment and shear using ACI 318-11 Moment Coefficient Method

- Neg. mom: $W_n \frac{L}{11} = \frac{(7.08 \, \text{k/lf})(26.25)}{11} = 444 \, \text{k.ft}$

- Pos. mom: $W_n \frac{L}{16} = \frac{(7.08 \, \text{k/lf})(26.25)^2}{16} = 305 \, \text{k.ft}$

- Shear: $W_n \frac{L}{2} = \frac{(7.08)(26.25)}{2} = 92.9 \, \text{k}$
NEGATIVE MOMENT CAPACITY

1.5" CEE (TR8)

\( f_{c} = 9 \text{ ksi} \)

\( A_s = \frac{1}{2} (1.0) = 9 \text{ in}^2 \)

\( f_y = 60,000 \) psi, 12% NC

\( d = 18.5 - 1.5" - 0.5" - 1.125/2 = 15.94" \)

\( \rho = \frac{A_s}{(15.94)(30)} = 0.0188 \Rightarrow \rho_{mu} = 0.0035 \text{ Good} \)

\( a = \frac{(9.0)(60 \text{ ksi})}{0.85 (0.65)} = 4.24 \)

\( M_n = (9.0)(60) \left[ 15.94 - 4.24^2/2 \right] = 621.9 \text{ k.l.f.} \)

\( \% \text{ capacity} = \frac{621.9}{559.7} = 99.3\% \)

POSITIVE MOMENT CAPACITY

\( a = \frac{4 (1.0)}{0.85 (\xi)(36.2)} = 0.0084 \)

\( d_{eff} = \max \left\{ \frac{4}{36.2}, \frac{30}{16 (4.5)} \right\} = 86.84" \Rightarrow \text{ controlling} \)

\( 30" + 16 (4.5) = 102" \)

\( 24.5" = 24.9" \)

\( a = \frac{(4)(60)}{0.85 (0.65)(36.2)} = 0.65" \Rightarrow 4.5" \text{ T-Berm behavior} \)

ORIGINAL SECTION 10
* Using doubly reinforced T-beam moments and Excel calculation, the values resulted are:

- $a = 1.41''$, $M_a = 347.5$ k-ft, $\epsilon_a = 0.0241$
- $c = 1.76''$, $M_c = 312.8$ k-ft, $\epsilon_c = 0.09$

% Capacity = \[ \frac{303}{312.8} = 97.5\% \]

Shear nominal strength

\[ V_a = 2 \sqrt{5000 (15.94^2)} = 67.6 \text{k} \]

\[ V_c = \frac{(2.14^2) \times 2 (60.871 \times 15.94)}{6.01} = 62.7 \text{k} \]

\[ \phi V_a = 0.95 \left( 69.6 + 63.9 \right) = 98.5 \text{k} \]

% Capacity = \[ \frac{98.5}{98.2} = 99.3\% \]

* No torsion is required because beam is loaded on both sides approximately evenly.
**COST ESTIMATE:**

**Line Number:**
- 033.0534.0259

- **ONE-WAY SLAB:** 80% PANS, 125 psf Snow Load, 25' span
- **INCLUDES:** Forms (4 sides), Reinforcing Steel, Concrete, Placement and Finishes

**Usage Costs**

<table>
<thead>
<tr>
<th>Mat.</th>
<th>Labor</th>
<th>Equipment</th>
<th>Total Unit</th>
<th>Daily Output</th>
</tr>
</thead>
<tbody>
<tr>
<td>$410.00</td>
<td>$252.00</td>
<td>$24.00</td>
<td>$676.00</td>
<td>31.15</td>
</tr>
</tbody>
</table>

**Volume of Concrete:**

- **Slab:**
  \[(28.95' \times 22.875')(\frac{\pi}{12}) = 257 \text{ ft}^3\]

- **Joints:**
  \[(20.63')(9.14')/(144) = 12.1 \text{ ft}^3\]

- **Beams:**
  \[(24.25')/(36.14') = 95.4 \text{ ft}^3\]
  \[(26.25')/(30.11') = 96.5 \text{ ft}^3\]

**Cost:**

\[\text{Cost} = \frac{2670}{\text{cy}} \times 17 \text{ cy} = 11730.00\]

**SF of Pany:**

\[(28.75' \times 22.875') = 686 \text{ ft}^2\]

\[\frac{11730.00}{114 \text{ ft}^2} = 169.8 \text{ \$/SF}}

**Original Section 12**
Alternate System 1 - Plate Plate:

Establish slab thickness through deflection control:

- Using ACI 318-11, Table 9.5(c)

\[ h_{min} = \frac{L}{f_p} \]

- Without drop panels
- Bearing panels
- Without edge beams

Typical bay layout:

\( \ell_1 = 286.5'' - 18'' - 12'' = 256.5'' \)

\( \ell_{12} = 345'' - 18'' = 327'' \) => Controlling

\[ h_{min} = \ell_{12} / 30 = 327'' / 30 = 10.9'' \] => Use 12''
PUNCHING SHEAR CHECK:

** Earliest Column will be on Foundation: Column CA**

**L.L.** = 100 psf

**D.L.** = (150 psf) * (144) + 15 psf = 165 psf

* Dead Load

1.2D + 1.6L = 1.2(165) + 1.6(100) = 358 psf

\[358 \text{ psf} \left( \frac{28.75}{2}(12.94) - \left( \frac{28.75 \times 12.94}{100} \right) \right)\]

\[28.75 \quad V_a = 132.1 \text{ k}\]

Assume \(d = 12" - 1.5" = 10.5"\)

- Nominal shear strength:
  \(\frac{d}{2} = \frac{10.5}{2} = 5.25"\)

\[V_c = \min \left( \frac{2}{k/3} \right) = \left( \frac{2}{1.33} \right) = 5.00\]

\[\left( \frac{2 \times d}{b_0} + 2 \right) \times \sqrt{5} b_0 d\]

\[4 \Rightarrow \left( \frac{30 \times 10.5}{8.7} + 2 \right) = 5.62\]

\[\beta = \frac{24}{18} = 1.33\]

\[\alpha = 3.0 \left[ \text{Edge Column} \right]\]

\[b_0 = 2(29.25) + 28.5 = 87"\]

\[\frac{\theta}{\Phi} \text{はもちろん} = \frac{132.1}{193.8} = 69.2\%\]

**Flat Plate 2**


d = 10.5''

\( \beta = \frac{3w}{l_0} = 1.8 \)

\( \alpha_s = 40 \)

\[ b_0 = 2 \left[ \frac{46.5 + 33.5}{2} \right] = 154'' \]

\[ V_u = 358 \text{ psf} \left[ 24.44 \times 25.63 = \frac{20.94}{100} \right] \]

\[ V_u = 222.5 \text{ k} \]

\[ V_c = \min \left( \frac{2}{\sqrt{1.03}}, \frac{40 \times 10.5}{154} + 2 \right) = 4.93 \times \sqrt{5.6 \ b_0 \ d} \]

\( Y \Rightarrow \text{concrete} \)

\[ V_c = 4 \sqrt{5000} (154) (10.5) = 457.4 \text{ k} \]

\[ \phi V_c = 0.75 (457.4) = 343 \text{ k} \]

\% capacity = \( \frac{222.5}{343} = 64.9\% \)

**Moment Plan:** From Ram Concept

**Flat Plate 3**
MOMENTS W/tr 1' SLAB SECTION

* N-S DIRECTION

COLUMN C1: 
\[ -68.1 \text{k-ft} \quad \frac{1}{2} A = -34.0 \text{k-ft} / ft \]

COLUMN C2: 
\[ -90.1 \text{k-ft} \quad \frac{1}{2} A = -45.0 \text{k-ft} / ft \]

COLUMN D1: 
\[ -51.8 \text{k-ft} \quad \frac{1}{2} A = -26.0 \text{k-ft} / ft \]

COLUMN D2: 
\[ -150 \text{k-ft} \quad \frac{1}{3} A = -50 \text{k-ft} / ft \quad \text{CONTROLLING} \]

* E-W DIRECTION

COLUMN C1: 
\[ -346.6 \text{k-ft} \quad \frac{1}{1.5} A = -231 \text{k-ft} / ft \]

COLUMN C2: 
\[ -90.2 \text{k-ft} \quad \frac{1}{3.83} A = -27.5 \text{k-ft} / ft \quad \text{CONTROLLING} \]

COLUMN D1: 
\[ -34.1 \text{k-ft} \quad \frac{1}{1.5} A = -22.7 \text{k-ft} / ft \]

COLUMN D2: 
\[ -53.4 \text{k-ft} \quad \frac{1}{1.67} A = -22.0 \text{k-ft} / ft \]

NOTE: Because positive moments are relatively small compared to negative moments, for simplicity, top and bottom reinforcement will be the same.

DIAGRAM: *

NOMINAL FLEXURAL STRENGTH OF SLAB

* N-S DIRECTION:

\[ M_u = 50 \text{k-ft} \]

\[ d = 12" - 0.75" - 0.5" = 10.75" \]

\[ f_c = 5000 \text{ psi}; \]

\[ f_y = 60 \text{ ksi}; \]

\[ A_t = 2(0.199) = 1.58 \text{ in}^2 \]

\[ \rho_{\text{min}} = \frac{1.58 \text{ in}^2}{(10.75)(12")} = 0.0122 \Rightarrow \rho_{\text{min}} = 0.0065 \text{ (Good)} \]

（FLAT PLATE）
\[ a = \frac{(1.58 \, \text{m}^2)(60)}{0.85 \, (5)(12)} = 1.86 \text{ in} \quad \epsilon = \frac{0.005}{2.32} \quad \text{and} \quad 2.32' \]

\[ M_u = (0.58)(60) \left[ 10.95 - 1.86 \right] = 77.6 \, \text{k} \quad \epsilon_u = \frac{0.005}{2.32} \left[ 10.95 - 2.32 \right] \]

\[ \phi M_u = 0.9 \left( 77.6 \right) = 69.8 \, \text{k} \cdot \text{ft} \]

\[ \% \text{ CAPACITY} = \frac{50}{69.8} = 71.6\% \]

- **E-W DIRECTION**

\[ M_u = 42 \, \text{k} \cdot \text{ft} \]

\[ d = 10.95 - 1.0 = 9.95' \]

\[ (2) \times 8 \, \text{top} \quad a = \frac{(1.58 \times 60)}{0.85 \times (5)(12)} = 1.86' \quad \epsilon = 2.32' \]

\[ M_u = (1.58 \times 60) \left[ 9.95 - 1.86 \right] = 69.7 \, \text{k} \cdot \text{ft} \]

\[ \epsilon_u = \frac{0.005}{2.32} \left[ 9.95 - 2.32 \right] = 0.0096 \quad \phi M_u = 0.9 \left( 69.7 \right) = 62.7 \, \text{k} \cdot \text{ft} \]

\[ \% \text{ CAPACITY} = \frac{42}{62.7} = 67.1\% \]

\[ S_{\text{bars}} = 15 \left( \frac{42}{40} \right) - 2.5 \left( 6.95 \right) = 13.11 \, \text{lb} \]

**SEQUENCING:**

1. **E-W BARS**
2. **N-S BARS**

**REINFORCEMENT LAYOUT**

**FEAT: FUTURE 5**
COST ESTIMATE:

LINE ITEM: 0730 5340 2150
ELEVATED SLAB, FLAT PLATE, 126 psi subimposed load, 25' SPAN
L2 includes forms (4' x 3'), reinforcing steel, concrete, placement;
and finishes

BARE COSTS

<table>
<thead>
<tr>
<th>MATERIAL</th>
<th>LABOR</th>
<th>EQUIPMENT</th>
<th>TIME</th>
<th>UNIT</th>
<th>DAILY OUTPUT</th>
</tr>
</thead>
<tbody>
<tr>
<td>9 234</td>
<td>$185</td>
<td>$44.95</td>
<td>$493.95</td>
<td>CY</td>
<td>49.20</td>
</tr>
</tbody>
</table>

VOLUME OF CONCRETE:

\[
\text{Volume} = \left(28.95\text{ ft}^3\right) \times \left(24.975\text{ ft}^3\right) \times \left(1\text{ in}\right) = 915\text{ ft}^3 = 24.99\text{ CY} = 27\text{ CY}
\]

TOTAL COST = \[
\left(\frac{\$493.95}{\text{CY}}\right) \times \left(27\text{ CY}\right) = \$11,717.00
\]

4.14MH
\[
\left(\frac{\text{MH}}{\text{CY}}\right) \times 27 = 113\text{ MH}
\]
Alternate System 2 - Fully Composite Deck:

Deck Sizing:

2 hr Fire Rating → Unprotected Deck: 4½" NWC

Spacing of Joists: 6'-3" → Same Layout as Existing System

Superimposed L.L = 100 psf + 15 psf = 115 psf → Superimposed Mechanical

1.5 V/L 22 = Total Slab Depth 6" (6' = 4.5")

- 3 Span Unshored = 6' - 4' = 6' - 3' → Good

- Strength @ 6'-6" = 3.85 psf >> 115 psf → Good

Joist Sizing:

- Load: D.L. = 68 psf + 3% beam wt + 15 psf sup. = 83.3 psf
  L.L. = 115 psf
  1.2D + 1.6L = 1.2(83.3) + 1.6(110) = 256.4 psf

Net Width = 6'-3"

\[ W_n = \left(\frac{256.4}{6.25}\right) = 1.6 \text{ k/ft} \]

\[ M_n = \frac{W_n^2}{8} = \left(\frac{1.6}{9}\right)(23.875)^2 = 114 \text{ k-ft} \]

- Construction Load/Wet Concrete

\[ q_u = 1.2\left[68 + 1.0\right] + 1.6\left[20\right] = 110 \text{ psf} \]

Deck wt. + 6% self wt. + construction live load

\[ \text{Composite Deck} \]

Advisor: Parfitt
\[ a = \frac{(6.48 \text{ in}^2)(50 \text{ kips})}{0.85 (\delta) (91.6 \text{ in})} = 1.06 \text{ in} \]
\[ d = 6'' + \frac{12.3}{2} = 12.15'' \]
\[ M_w = (6.48 \text{ in}^2)(50 \text{ kips}) \left[ 12.15 - \frac{1.36}{2} \right] = 313.7 \text{ kips-ft} \]
\[ M_0 = 0.9(23.9) = 215.1 \text{ kips-ft} \]
\[ \% \text{ Capacity} = \frac{114}{282} = 40.9 \% \]

* Check construction load:
\[ M_{cc} = 49 \text{ kips} \text{ ft} \]

\[ M_{cc} \text{ or } M_0 = 125 \text{ kips-ft} \text{ Good} \]

* Check for deflections - construction:
\[ w = (63)(6.86) + 22 \times (2.0)(6.25) = 541 \text{ lb/ft} \]
\[ L = 23.875 \text{ ft} \]
\[ \Delta = \frac{5wL^2}{384EI} = \frac{5(541 \text{ lb/ft})(23.9 \text{ ft})^4(12)}{384(10000)(195\%)} = 0.98'' \text{ Good} \]
\[ \frac{L}{240} = \frac{(23.9 \times 12)}{240} = 1.20'' \]

Composite deck 2
* Check for deflections - live load

\[ W_L = (100 \text{ psf})(6.25') = 625 \text{ lb/ft} \]
\[ l = 23.9 \text{ ft} \]
\[ Y_1 = 0 \quad Y_2 = 6'' - 1.1/2 = 5.47'' \pm 5.5 \]
\[ I_{lb} = 596 \text{ in}^4 \]
\[ A_L = \frac{5(625 \text{ lb/ft})(23.9 \text{ ft})}{25 \text{ in}^4(596 \text{ in}^4)} \approx 0.27'' \text{ Good} \]

\[ \frac{l}{360} = \frac{23.9 \text{ ft}}{360} = 0.179'' \]
\[ \phi = 324 \text{ k} - 3/4 \phi \]
\[ \frac{324 \text{ k}}{17.2 \text{ k}} = 18.8 \rightarrow 19 \times 2 = 38 \text{ studs} \]
\[ 23.9''/28 = 0.95'' \]
\[ S_{\text{min}} = 4 \times 0.75'' = 3'' \checkmark \]

* Sizing in deck checks good
  To use 1 stud/8'8"

* Check shear

\[ V_k = \frac{(1.6 \times 23.9)}{2} = 19.12 \text{ k} \]
\[ \phi V_n = 94.5 \text{ k} \quad \% \text{ capacity} = \frac{19.12}{94.5} = 20.2\% \]
Girder Sizing:

$M_A = 0$:

$R_B(28.75) = 19.12 \left(6.25 + 12.5 + 18.75 + 28\right)$

$R_B = 41.6 \text{k}$

$M_{A} = 0$:

$R_A = \frac{41.6}{4} = 34.88 \text{k}$

Construction Load:

$D.L = 63 \text{ psf} + \frac{7}{16} = 67.4$

$L.L = 20 \text{ psf}$

$1.2[67.4] + 1.6[20] = 113 \text{ psf}$

$w_u = \left[113 \text{ psf}\right][12.94] = 1.46 \text{ k/l}f$

$M = \frac{\left[1.46 \text{ k/l}f\right][28.75]^2}{8} = 151 \text{ k-ft}$

Composite Deck 4
TRY W 16 x 36

Steel Properties:

\[ A_s = 10.6 \text{ in}^2 \]
\[ I_x = 448 \text{ in}^4 \]
\[ d = 15.9'' \]
\[ b_f = 9.00'' \]

\[ b_{eff} = b_1 + b_2 \]

\[ b_1 = 6.52'' \]
\[ b_2 = 3.5'' \]
\[ b_{eff} = 3.5'' + 43.1'' = 46.6'' \]

\[ \beta = \frac{1/4}{29.75} = 43.1'' \]
\[ 296.5/2 = 143.25'' \]

\[ a = \frac{(10.6)(50)}{0.85(5)(146.4)} = 2.68\]
\[ d = 6'' + 15.9/2 = 15.95'' \]

\[ M_u = (10.6)(50)[15.95 - 2.68/2] = 554 k\cdot ft \]

\[ \phi M_u = 0.9(554) = 501.2 k\cdot ft \]
\[ \%\text{ safety} = \frac{516.5}{501.2} = 63.1\% \]

- Check construction limits
  \[ M_{cs} = 151 k\cdot ft \leq \phi M_u = 240 k\cdot ft \text{ Good} \]

- Check construction deflections
  \[ w = \frac{67.4 + 20}[12.44''] = 1.13 k/ft \]
  \[ \Delta = \frac{5(1.13 k/ft)(28.75)''(1928)}{284 (29,000)(448)} = 1.33'' \text{ Good} \]
  \[ \frac{l}{240} = \frac{28.95}{240} = 1.44'' \]

Composite Deck 5
* CHECK LIVE LOAD DEFLECTIONS

ωₜₐₜ = (100 psf)(12.94) = 1.29 k/ft

\[ Y_1 = 0 \quad Y_2 = 6 - \frac{2.68}{2} = 4.66 \approx 4.5 \]

\[ I_{lb} = 1270 \text{ in}^4 \]

\[ \Delta_{ul} = \frac{5(1.29)(20.75)^4 (1929)}{394 (24000) (1270)} = 0.54'' \text{ [Good]} \]

\[ l/360 = \frac{345''}{360} = 0.96'' \]

* SHEAR STUDS

\[ \Sigma Q_n = 530 \text{ k} \quad \# \text{STUDS} = \frac{530}{17.2} = 31 \approx 30 \text{ STUDS} \]

\[ 345''/62 = \frac{5.58''}{\text{STUD}} \quad S_{min} = 4 \times \frac{3/4''}{4} = 3'' \checkmark \]
**Cost Estimate**

- **Steel Decking** - Form, Galvanized
  - 22 Gauge, 1-5/16" deep
  - $2.19/S.F.

- **Concrete** 5,000 psi
  - $0.31 $0.25 - $0.40
  - $1.09/C.Y.

- **Placing Concrete**
  - Elevated slabs, less than 6" thick, pumped
  - $22.75/C.Y.

- **Structural Steel Projects**
  - $0.12 $0.09 - $0.06
  - High tensile, 7 to 15 degrees
  - $3.66/Ton

- **Sprayed Concrete Paving**
  - 1" thick, beam, including tamping
  - $1.11/S.F.

**Quantities:**

- \((24.9') \times (28.75') = 716 \text{ ft}^2\) of decking
- \(716 \text{ ft}^2 \times \left( \frac{\text{in}}{\text{ft}} \right) \times \frac{1}{12} = 18.25 \approx 18 \text{ C.Y.}\)
- \((22 \frac{11}{16} / \text{ft}) \times (33.9') \times (5) + 38 \times (\text{lbs}) (18) + (36 \frac{3}{8} / \text{ft}) (28.75') (2) + 62(10)\)
  - 5699.16 \approx 2.85 \text{ tons}\n
*Composite Deck 7*
**Surface Area of Steel**

\[ SA = 2(4.0) + 2(12.3) + 2(4.03 - 0.26) \]

\[ 12.3^\prime = 3.35 \text{ ft}^2/\text{ft} \]

**Total Length**

\[ 5(28.9') = 119.5' \]

\[ (3.36)(119.5') = 410 \text{ ft}^2 \]

\[ S.A. = 2\left[ 7 + 15.01 \right] + 2\left[ 7 - 0.295 \right] \]

\[ 15.9'' = 4.97 \text{ ft}^2/\text{ft} \]

**Total Length**

\[ 2(25.75') = 57.5' \]

\[ (4.43)(57.5') = 283 \text{ ft}^2 \]

**Area of Steel:** 682 S.F

**Cost Breakdown:**

\[ 2.12/\text{SF} \times 716 \text{ SF} = 1568.06 \text{ SF Steel Decking} \]

\[ 109/\text{CY} \times 14 \text{ CY} = 1526.00 \text{ Concrete} \]

\[ 22.95/\text{CY} \times 14 \text{ CY} = 319.00 \text{ Concrete Placement} \]

\[ 3166/\text{TON} \times 2.85 \text{ TON} = 9023.00 \text{ Steel} \]

\[ 1.1/\text{SF} \times 682 \text{ SF} = 758.00 \text{ Fireproofing} \]

\[ 1.14/\text{SF} \times 682 \text{ SF} = 1319.00 \text{ Composite Deck} \]

**Advisor:** Parfitt
**Alternate System 2 - Oldcastle Precast Hollowcore Planks**

*Note: Manufacturer: Oldcastle Precast Building Systems*

**Supposed Load:**
- 100 psf + 15 psf = 115 psf [Unfactored]

**Span:**
- 24 ft

**8" x 4' Section - 20-08706**
- No Topping - 2 HR Fire Rating

- $f_c = 5000$ psi
- $f_{cu} = 3000$ psi
- $S_{pu} = 240$ ksu
- $54$ psf Dead Weight

- $d_{allow} = 123$ psf > 115 psf [Good]

**Check $\Delta_L$:**

- $\Delta_L = \frac{5(400 \cdot 0.6)(23.9)^4}{384(4030 \cdot 500)(1072)} = 0.24"$

*Shear Controlled Section*

- Hollow Core 1

**Advisor:** Parfitt
EXPENSE CURDER CD DESIGN

LOADS

\[ DL = 54 \text{ psf} + 15 \text{ psf} + 7\% \text{ wind} = 73.8 \text{ psf} \]

\[ LL = 100 \text{ psf} \]

\[ 1.2D + 1.6L = 1.2(73.8) + 1.6(100) = 249 \text{ psf} \]

\[ W_u = (249 \text{ psf})(1.25) = 3.22 \text{ k/ft} \]

\[ M_u = \frac{(3.22 \text{ k/ft})(20.75 \text{ ft})^2}{8} = 332.7 \text{ k-ft} \]

USE A \[ W(21 \times 44) \rightarrow \gamma M_u = 358 \text{ k-ft} \]

\[ \% \text{ CAPACITY} = \frac{332.7}{358} = 93.0 \% \]

DEFLECTION CHECK

\[ w_{L} = (100 \text{ psf})(12.95) = 1.3 \text{ in} \]

\[ \Delta_{L} = \frac{5(1.3)(29.75)^3(1928)}{384(29800)(843)} = 0.82 \text{ in} \]

\[ \frac{L}{360} = \frac{345}{360} = 0.96 \text{ in} \]

FOR SERVICE LOAD

\[ W_{sd} = \left[ 73.8 + 100 \right](12.95) = 2.25 \text{ k/ft} \]

\[ \Delta_{sd} = \frac{5(2.25)(28.95)^4(1928)}{384(29000)(843)} = 1.42 \text{ in} \]

\[ \frac{L}{240} = \frac{346}{240} = 1.44 \text{ in} \]
INTERIOR CHAPEL CD DESIGN

- $q_v = 249 \text{ psf}$
- $w = (249 \text{ psf})(24.441) = 6075 \text{ k/ft}$
- $M_v = \frac{(6075)(28.93)^2}{8} = 629 \text{ k-ft}$

Use $A = \sqrt{L^2 + h^2}$ to obtain $M_v = 645 \text{ k-ft}$

- % Capacity: $\frac{629}{345} = 97.5\%$

DEFORMATION CHECK

- $w = (100 \text{ psf})(24.441) = 2444 \text{ k/ft}$
- $\Delta_v = \frac{5(2.444)(28.93)^4}{2094(29,000)(16.06)} = 0.81'' \text{ Good}$

- Service Load

- $w_{s,v} = (93.8 \times 100)(24.441) = 4125 \text{ k/ft}$
- $\Delta = \frac{5(4125)(28.93)^4}{384(29,000)(16.06)} = 1.41'' \text{ Good}$

- $\frac{L}{240} = \frac{545}{240} = 1.44''$
COST ESTIMATE

- Structural Steel, Hospitals, 9 to 5 Stories $85,123.57 - 09 AM
  $510/ton
  4.074 tons
  0.0646 ton / $510 = $3,246.00

- Structural Columns, Fireproofing, 1" thick, Beam, Includes
  Taping $0.981 $16.10 - 04 AM
  $0.016 $/lin ft x 739 lin ft = 5.42 $/lin ft

- Precast Slab Planks, Tree Stepped, Grooved, Hollow, 8" thick
  $2.11 $/lin ft
  0.025 0.025 x 716 lin ft = 6.95 $/lin ft

QUANTITIES

- (44 ft) (28.95) + (73 lin ft) (28.95) = 1,140 tons - Steel
- (29.14) (28.95) = 716 sf - Hollow Core Planks

\[
SA = 2 \left( \frac{20.9 + 6.5}{2} \right) + 2 \left( \frac{6.5 - 0.85}{2} \right)
\]
\[
= 5.56 \text{ ft}^2 / \\text{lin ft}
\]

TOTAL SURFACE AREA: \[
5.56 + 6.22 = \frac{29.58}{2} = 339.42
\]

COSTS

- Steel: $510/ton x 1.67 ton = $831.90
- Foam: $8.74 $/lin ft x 399.82 = $3,576.00
- MC Blocks: $8.97 $/lin ft x 716 lin ft = $6,258.00

\[
SA = 2 \left( \frac{21.2 + 8.3}{2} \right) + 2 \left( \frac{8.5 - 0.455}{2} \right)
\]
\[
= 6.22 \text{ ft}^2 / \\text{lin ft}
\]

Hollow Core 4
Introduction

The purpose of this technical guide is to provide assistance in selecting and detailing precast concrete hollowcore plank manufactured by Oldcastle Precast, Inc. Additional information and standard specifications are available at oldcastlesystems.com.

The load tables presented herein are intended as a guide only. Final design is determined by our engineering department based on information presented in the final plans and specifications. To ensure the optimum selection for your application, please contact us for assistance.

Although core has been taken to provide the most accurate data possible, Oldcastle Precast, Inc. does not assume responsibility for errors and omissions.

The Manufacturing Process

Elematic® is a machine extruded, precast, prestressed hollowcore plank. The planks are manufactured on 600-foot-long beds in standard widths of 48 inches and thicknesses of 8, 8, 10, 12 and 16 inches. High strength prestressing strands are cast into the planks at the spacing and location required for the given span, loading and fire cover conditions. The planks are cut to length for each project using a diamond-blade saw. After the planks are cut, they are removed from the casting beds and placed into storage.

All Elematic® materials equal or exceed the requirements of applicable ASTM specifications. The concrete mix is designed to have release strength of 3,000 psi or 3,500 psi, and a 28-day compressive strength of 5,000 psi. The prestressing strands are uncoated, seven wire, low relaxation with a minimum ultimate strength of 270 ksi.

Load Table Design Criteria

The tables herein list allowable live loads in pounds per square foot for uniformly distributed loading. Non-uniform loading conditions resulting from point loads, line loads, openings and cantilevers require special design consideration.

The allowable load is usually governed by the ultimate capacity of the section. As a design aid, the ultimate moment capacities in governing criteria for short spans may be the horizontal shear stress between the plank and the topping.

Allowable live loads for long-span, heavily reinforced sections are limited to loads that result in a bottom-tension stress equal to the cracking stress. Loads beyond this limit may result in deflections that exceed the allowable value set forth in the ACI code.

The load tables are based on a plank concrete strength of 5,000 psi. Tables for topped sections are based on a topping strength of 3,500 psi and minimum thickness of 2 inches.

Maximum spans and loads shown are not absolutes. Longer spans or heavier loads may be achieved under certain conditions or different criteria than assumed in the tables. Contact us if you need assistance.

Plank Design Considerations

The following items will affect the selection of appropriate planks sizes and should be carefully reviewed by the Architect/Engineer while developing the plans and specifications for a project.

Fire Rating

- The fire rating requirement should be clearly specified in the contract documents.

Loading Conditions

- Specify all uniform loading requirements on structural plans.
- Identify line and point loads resulting from bearing walls, masonry walls, face brick, columns, mechanical equipment, etc.
- Identify diaphragm forces and lateral loads resulting from wind or earth pressures.
- Review roof plans for vertical protrusions such as parapets, penthouses and adjacent buildings that could require designing for snow drift loads.
- Plank supporting slabs require special loading considerations.
- Large openings or closely spaced groups of smaller openings will reduce the plank load carrying capacity.

Topping

- Specify whether or not concrete topping is to be composite. Composite action requires the topping to be bonded to the top surface of the plank. Topping separated by a vapor barrier or insulation is non-composite and must be considered a superimposed load.
- Large cantilevers resulting from long spans and/or heavy loads will affect the quantity of topping, assuming a level floor is required. Two inches of composite topping at mid span is minimal, and additional thickness at the ends of the plank may be required to maintain level floor elevations.

Camber

- Camber is inherent in all prestressed products. It is the result of the eccentric prestress force required to resist design loads, and cannot be designed in, cut, or to an exact number. The amount of camber will depend upon the span, design loads and thickness of plank. Planks stored in the yard for more than 6 weeks, usually due to construction schedule changes, will experience more camber growth.
- Adjacent planks of dissimilar length, end pattern or with openings will have inherent camber differences.
Fire Rating

Fire rating specifications are as important as all other design parameters. Plank rating requirements are determined by the Architect or Engineer of Record, who is also responsible for establishing the fire rating criteria for the total project.

Three methods generally used in the Northeast for determining hollowcore plank fire-resistive ratings are:

1. 2006 International Building Code
2. Rational analysis as defined by PCI MNL 124, "Design for Fire Resistance of Precast Concrete"
3. Underwriters Laboratories Fire Resistant Ratings
4. MEA product approval (New York City only)

International Building Code “IBC” Fire Rating

The IBC code prescribes fire ratings to any hollowcore plank section. Since 2000, the IBC code has replaced the DOCA, SBC and UBC model codes in many states. The two criteria that are measured to determine the fire rating are:

1. Equivalent concrete thickness - 4.6" inches is required for 2 hrs
2. Bottom strand cover - ¾” cover is required for 2 hrs (restrained condition)

Underwriters Laboratories Fire Resistant Ratings

Prior to codes including prescriptive fire-endurance rating methods, fire tests provided the primary source of ratings classifications. While some plank sections were fire tested, others can be evaluated by UL to qualify for existing UL numbers.

The table below lists the UL ratings available with Elematic® plank. Note that these ratings are dependent upon whether or not the ends of the planks are restrained. Determination of the restraint must be made by the Architect or the Engineer of Record, as it is primarily a function of the support structure.

<table>
<thead>
<tr>
<th>UL Number</th>
<th>Rating (Hour)</th>
<th>Plank Thickness (inches)</th>
<th>Topping Thickness (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>J594</td>
<td>1½</td>
<td>8, 10, 12</td>
<td>0</td>
</tr>
<tr>
<td>J694</td>
<td>2</td>
<td>8, 10, 12</td>
<td>½” Gypsum</td>
</tr>
<tr>
<td>J694</td>
<td>3</td>
<td>8, 10, 12</td>
<td>2½” Topping</td>
</tr>
<tr>
<td>J594</td>
<td>4</td>
<td>8, 10, 12</td>
<td>3½” Topping</td>
</tr>
</tbody>
</table>

Fire Ratings by Rational Analysis

PCI MNL 124 defines the "rational analysis" method for determining the fire rating of precast, prestressed members. It is useful when a fire rating cannot be obtained by either of the two previous methods. Actual practice has shown that this method is very conservative and that the span of the hollowcore plank will have to be reduced (approx. 10% to 20%) to achieve the same fire rating from both IBC and UL.

In using this method, the reduced strength of the prestressed strands at elevated temperatures is determined and the resulting moment capacities are compared to that required for service loads. Strand temperatures are based on the amount of concrete cover and the standard fire exposure as defined by the time-temperature relationship specified in ASTM E119. Fire ratings will also be improved if the plank assembly is restrained against thermal expansion. It should be noted that the only universally accepted definition of full restraint is an interior bay of a multi-bay building.

Sound Ratings

The following tables contain values for the Sound Transmission Class (STC) and the Impact Insulation Class (IIC) of various floor systems utilizing Elematic® hollowcore plank.

Sound Transmission Class (STC)

The values for the Sound Transmission Class were determined by tests which were in accordance with ASTM E90. The STC is a measure (in decibels) of the ease at which airborne sound is transmitted through a floor system. The larger the value of the STC for a given system, the greater the sound insulation.

Impact Insulation Class (IIC)

The values for the Impact Insulation Class (IIC) were determined by tests which were in accordance with ASTM E492. The Impact Insulation Class is the resistance to impact noise transmission and is highly dependent on the floor surface and structural connection details. As with the STC, the higher IIC values are more desirable.

www.oldcastlesystems.com
Production and Erection Tolerances: (Reprinted from PCI Manual for the Design of Hollowcore Slabs)

**Product Tolerances: Hollowcore Slabs**

- **a** = Width
- **b** = Height
- **c** = Top flange thickness
- **d_1** = Bottom flange thickness

The total cumulative web thickness defined by the actual measured value of **c** shall not be less than 85% of the nominal web thickness, calculated by

- **e** = Web thickness

The total cumulative web thickness defined by the actual measured value of **c** shall not be less than 85% of the nominal cumulative web thickness calculated by **e**.

- **f** = Blockout location
- **g** = Flange angle
- **h** = Variation from specified and squareness of edges
- **i** = Slope (variations from straight line parallel to centerline of member)
- **j** = Control of gravity or static group

The group of the strand group relative to the top of the plane shall be within 3/4 in. of the nominal strand group. The position of any individual strand shall be within 1/4 in. of nominal vertical position and 3/4 in. of nominal horizontal position and shall have a maximum cover of 1/2 in.

- **k** = Position of plates
- **l** = Tipping and flushness of plates
- **m** = Local smoothness

(These tolerances do not apply to the top or bottom surface of concrete or to visually discerned surfaces)

- **n** = Plank weight. Excess concrete material in the plank internal feature is within tolerance as long as the measured weight of the member does not exceed 110% of the nominal published unit weight used in the load capacity calculation.

**Erection Tolerances: Hollowcore Floor & Roof Members**

- **a** = Plan location from building grid datum
- **b** = Top elevation from nominal elevation at member ends
- **c** = Maximum (in) alignment of matching edges
- **d** = Joint width
- **e** = Differential top elevation as noted
- **f** = Differential bottom elevation of exposed hollowcore slabs

For precast concrete erected in a steel frame building, this tolerance takes precedence over tolerance on dimensions of the members. It may be necessary to raise the edges to 1/2 in. to properly apply some roof membranes. This is a setting tolerance and should not be confused with structural performance requirements set by the architect/engineer. Unfinished installation requires a larger tolerance here.
ELEMATIC® Hollowcore Plank

UNIFORMLY DISTRIBUTED SUPERIMPOSED* LOAD IN LBS. PER SQ. FT.

<table>
<thead>
<tr>
<th>Standard Designation</th>
<th>7-Win 270 Lokax PS Strand Combination</th>
<th>PS Strand Area Sq. In.</th>
<th>Ultimate Bending Moment, ( \phi ) Min Kip- Ft. per Unit</th>
<th>SIMPLE SPAN IN FEET</th>
<th>p Value in Kips per Ft.</th>
</tr>
</thead>
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<tr>
<td>30.00804</td>
<td>4-7/16&quot;</td>
<td>0.460</td>
<td>63.88 165 268 348 315 256 220 204 182 162 149 126 112 95 77 66</td>
<td>10 11 12 13 14 15 16 17 18 20 21 22 23 24 25 26 27 29 30 31 32 33</td>
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<td>5-7/16&quot;</td>
<td>0.575</td>
<td>72.52 175 280 355 325 268 235 202 182 164 148 135 123 112 99 85 71 67 64 60 56 52</td>
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<td>0.805</td>
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<td>109.88 273 423 471 371 325 302 280 261 242 227 211 195 180 165 150 135 120 105 90 80 70</td>
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<td>10 11 12 13 14 15 16 17 18 20 21 22 23 24 25 26 27 29 30 31 32 33</td>
<td>12.30</td>
</tr>
</tbody>
</table>

*Includes the live load plus any dead load that is additional to the weight of the bare grouted planks in place.

NOTES:
1. Design Standard: ACI 318-2005
2. For complete and detailed calculations, consult Oldcastle Precast.
3. For larger spans, heavier loads, or special conditions, consult Oldcastle Precast.
4. The table indicates maximum safe loads. Camber and deflection must always be investigated by the architect and/or engineer for the contemplated loading and span so that these factors are compatible with the contiguous materials in the proposed structure.
5. Values to the left and below the heavy stepped line are controlled by shear.
6. Shaded region indicates expected camber greater than 1".

Grouted weight of plank is 54 lbs. per sq. ft.

\[ f_c = 5,000 \text{ psi} \quad f_c l = 3,000 \text{ psi} \quad \text{Area} = 207 \text{ in.}^2 \]

\[ f_{p u} = 270, 900 \text{ psi} \quad I = 1,580 \text{ in.}^4 \quad b w = 10.0 \text{ in.} \]