Floor System Exploration
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Executive Summary

The purpose of Technical Report 2 is to design three alternative floor systems and compare them to the analysis performed on the existing structural system of the University Sciences Building (USB). This is accomplished through both hand and computer-aided calculations performed on a typical 36'-4"x21'-0" bay spanning in the North-South direction from column lines C to D and in the East-West direction from column lines 1 to 2. The systems were compared on the basis of general conditions (weight, cost per square foot, and structural depth), architectural conditions (fire rating and other impacts), structural conditions (foundation impact and lateral system impact), serviceability conditions (maximum deflection and vibration control) and construction concerns (additional fire protection required, schedule impact, and constructability). The existing floor system is an 8" thick voided filigree slab with 18" deep voided filigree beams. The three systems designed in this report include:

- Composite Steel Framing with Composite Steel Deck
- Post-Tensioned Concrete
- Precast Hollow core Plank on Steel Girders

The design of the composite steel system results in 3 ½" concrete topping on 3" Vulcraft 3VLI19 composite deck. The framing is W10x22 infill beams spanning 21'-0" with W18x60 girders spanning 36'-4". This is nearly half the weight of the existing system, and has a comparable cost. It receives its strongest benefit from its additional constructability as well as the potential to reduce the required foundations, but it also is the only system considered in this report on which the floor system can be cored without significant structural impact. Its largest flaw is the addition of structural depth, and the requirement for fireproofing that would probably necessitate a drop ceiling. However, the flexibility of this system makes it a viable alternative to the filigree system.

A 7" thick slab with a 14"Dx3'-6"W wide-shallow beam resulted from the post-tensioning design. To achieve this, (18) ½" Ø 7-wire unbonded tendons were used in the distributed direction and (15) ½" Ø 7-wire unbonded tendons were used in the banded direction, grouped into bundles of 5. This system weighed less than the filigree system, and cost essentially the same amount per square foot. Its major advantages were the reduction in structure depth and the preservation of architectural elements. The only perceived drawbacks are the increased construction difficulty due to the post-tensioning tendons and the fact that the slab cannot be easily cored in the event of future space renovation. With so much in favor of this system, it is clear that it is a feasible system.

Nitterhouse Concrete Products was the selected manufacturer for the precast hollow core. Using their product information, a 6" thick hollow core with 2" topping was chosen to maintain the required fire rating and provide topping for floor leveling purposes and diaphragm action. These are supported by W18x175 girders. The resulting structural depth is slightly greater than the filigree slab/beam depth, but the weight is less. The largest drawbacks to this system are the cost, which exceeds the cost of the filigree system by nearly 60%, and the extreme fabrication and construction difficulties associated with the support condition chosen for the precast to help reduce structural depth. The only real advantage of the system is that it is less weight, and thus may have a positive impact on the foundations. Due to high costs, poor constructability, and potential conflicts between the lateral and gravity systems, this system was deemed to be an unacceptable choice.
Building Introduction

The University Sciences Building (USB) is a new building located on an urban university campus in the Northeast USA. The site chosen was previously a parking lot serving adjacent campus buildings (See Figure 1). However, the USB provides a much more appealing image on this busy street corner. It is a departure from typical campus architecture in both material usage and architectural style. These differences serve as a visible indication of the university’s new commitment to building sustainable, functional buildings.

While most other campus buildings have brick facades with narrow, strip-like windows, the USB is clad largely in a prefabricated natural stone panel with aluminum-honeycomb back-up, which enables the façade to be very light. Seemingly in homage to the surrounding buildings, the USB also utilizes tall, narrow windows. However, they are of varying widths and placement on the building, which adds interest to the façade (See Figure 2). An additional feature is the 5 story atrium that forms the core of the building. It provides significant focal points such as a sweeping spiral staircase and a four-story “biowall,” the first of its kind on a US university campus (See Figure 3). The biowall is used to help mitigate air quality within the building, and it is just one of many features that will help to earn the building a LEED Silver rating upon completion.

The USB is a multi-use building, incorporating four large lecture-hall style classrooms, an auditorium, several teaching and research laboratories, and faculty offices. It locates the large classrooms and administrative functions on the ground floor of the building for easy public access, but removes the laboratories and offices to the upper four stories for additional privacy. Including the mechanical penthouse, the building stands 94’-3” above grade with a partial basement. It provides the university with 138,000 square feet of new space, and has a construction cost of approximately $50 million. Construction began in August of 2009, and has an expected completion date of September 2011.
Structural Overview

The University Sciences Building rests on drilled concrete caissons ranging in diameter from 36” to 58” capped by caisson caps and then grade beams. The lower five floors utilize a voided filigree slab and beam system with cast-in place concrete columns. The mechanical penthouse, however, uses steel columns and floor framing. The lateral system consists of several shear walls spanning from ground to various heights. Masonry infill walls are used between columns on the lower floors to help dampen sound from the surrounding urban environment. These non-structural walls are used solely as back-up walls to support the cladding, and were not a part of this technical report, but their design is an important consideration.

The importance factors for all calculations were based on Occupancy Category III. This was chosen because the USB fits the description of a “college facility with more than 500 person capacity,” which requires Occupancy Category III.

Foundations
Geosystems Consultants, Inc. performed several test borings on the proposed site of the USB in October 2007. They found that the subsurface conditions consisted largely of extremely loose brick and rubble fill, followed by alluvium and finally residual soils with relatively low load-bearing capabilities. However, comparatively intact bedrock was encountered approximately 25 feet to 34 feet below the surface of the site.

In light of these conditions, traditional shallow spread footings would not be acceptable. Both driven steel H-piles and drilled caissons were considered as options for deep foundations, but H-piles were rejected due to vibration concerns within the subway station adjacent to the site, as well as noise concerns for the surrounding academic buildings. Instead, drilled caissons ranging in diameter from 36” to 58” were chosen to carry the loads from grade beams to the bedrock below. It was also recommended that the fill under the slab on grade (SOG) comprising the majority of the first floor be removed to a level of approximately 4 feet below the surface, followed by heavy compaction of subsurface materials, and then backfilled with structural fill to minimize settlement of the SOG due to the extremely poor load-bearing capacity of the brick/rubble fill.

Lastly, groundwater observation wells were installed, and groundwater was found to be present approximately 13 feet to 18 feet below the surface of the site. This is a potential concern, because some of the basement walls are 14 feet underground, and could encounter some loading due to hydrostatic pressure, particularly in seasons where the groundwater table rises due to rain. This was not evaluated in this technical report, but is a consideration for future design.

Framing System
The columns in the lower five stories of the USB are all cast-in-place concrete. The columns closest to the atrium on the ground floor are round columns 2 feet in diameter. Most are changed at the second level to 36”x16” rectangular columns. All other columns are 36”x16” columns from their base to the penthouse, rotated as required to fit into walls. At the penthouse level, the columns change to A572 steel W-shapes. These columns range in size from W8x40 to W8x67.

Lateral System
Shear walls are the main lateral force resisting system in the USB. They are scattered throughout the building to best resist the lateral forces in the building (See Figure 4). All of these walls are 12” thick cast-in-place concrete. Most span from ground level to the roof, but since roof heights vary, they are not
necessarily the same height. They are anchored at the base by grade beams that run the full length of the walls. This is a potential overturning concern due to the large forces that can occur on a shear wall. This concern was not investigated in depth in this technical report. Another issue not investigated for this technical report, but that will be of concern later, are the checks for force transfer at the thin, link-like elements to ensure that the lateral forces are able to reach the shear walls.

**Roof Systems**

There are six different roofs on the USB, due mostly to architectural reasons. Figure 5 shows these roofs and their heights above the ground reference elevation of 0'-0". The Office roof (shown in red) is at the same elevation as the fifth floor. Its structure is a 10" flat plate filigree slab system, similar to the office floors below it. The “Ledge” roof (shown in orange) is at the same level as the Penthouse floor, and is a continuation of the 10" voided filigree slab (V.F.S.)/24" voided filigree beam (V.F.B.) system used in the adjacent AHU Mechanical Room. The atrium roof, 5th Level Mechanical Room roof, and AHU Mechanical Room roof (shown in yellow, green, and purple, respectively) are all 3" P2404 Canam roof deck on steel W-shape framing. The Chiller Mechanical Room roof (shown in blue) is 3" of cast-in-place concrete topping on 3" P2432 Canam composite deck (6" total depth) supported by W-shape framing. This heavier structure is necessary because this roof supports two large cooling towers and a diesel generator. This roof is also the only one with a parapet, which serves as a screen to hide the mechanical equipment and stretches from this roof level to 94'-3".

Regardless of the underlying structure, all roofs receive the same finish. This consists of sloped rigid insulation under Thermoplastic-Polyolefin (TPO) single-ply membrane.

*Figure 4 Typical floor plan taken from Sheet S203. Shear walls are indicated in blue.*

*Figure 5 Modified keyplan image from Sheets S205, & S206 showing different roof heights in relation to 0'-0"*
Design Codes

According to Sheet S001, the original building was designed to comply with:

- 2006 International Building Code (IBC 2006) with Local Amendments
- 2006 International Mechanical Code (IMC 2006) with Local Amendments
- 2006 International Electrical Code (IEC 2006) with Local Amendments
- 2006 International Fuel Gas Code (IFGC 2006) with Local Amendments
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-05)
- Building Code Requirements for Structural Concrete (ACI 318-08)
- Masonry Construction for Buildings (ACI 530)
- AISC Manual of Steel Construction, Load Resistance Factor Design (LRFD)

These are also the codes that were used to complete the analyses contained in this technical report, with heavy emphasis on the use of ACI 318-08, AISC Manual, and ASCE 7-05.
Materials Used
Due to the variety of structural types on this project, there are also many different kinds of materials. These are listed in Table 1 below. All information was derived from Sheet S001.

<table>
<thead>
<tr>
<th>Concrete Usage</th>
<th>Weight</th>
<th>Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Caissons</td>
<td>Normal</td>
<td>3000</td>
</tr>
<tr>
<td>Caisson Caps</td>
<td>Normal</td>
<td>3500</td>
</tr>
<tr>
<td>Footings</td>
<td>Normal</td>
<td>3500</td>
</tr>
<tr>
<td>Foundation Walls</td>
<td>Normal</td>
<td>4500</td>
</tr>
<tr>
<td>Shear Walls</td>
<td>Normal</td>
<td>4500</td>
</tr>
<tr>
<td>Slab-on-Grade</td>
<td>Normal</td>
<td>3500</td>
</tr>
<tr>
<td>Columns</td>
<td>Normal</td>
<td>5000</td>
</tr>
<tr>
<td>Structural Slabs/Beams</td>
<td>Normal</td>
<td>4500</td>
</tr>
<tr>
<td>Precast</td>
<td>Normal</td>
<td>5000</td>
</tr>
<tr>
<td>Housekeeping Pads</td>
<td>Normal</td>
<td>3500</td>
</tr>
<tr>
<td>Concrete on Steel Deck</td>
<td>Normal</td>
<td>3000</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>Steel Type</th>
<th>Standard</th>
<th>Grade</th>
</tr>
</thead>
<tbody>
<tr>
<td>W-Shaped Structural Steel</td>
<td>ASTM A572</td>
<td>50</td>
</tr>
<tr>
<td>Hollow Structural Sections (HSS)</td>
<td>ASTM A500</td>
<td>C</td>
</tr>
<tr>
<td>Anchor Rods</td>
<td>ASTM F1554</td>
<td>N/A</td>
</tr>
<tr>
<td>Bolts, Washers, and Nuts</td>
<td>ASTM A325</td>
<td>N/A</td>
</tr>
<tr>
<td>3/4&quot;x4 1/2&quot; Long Welded Shear Studs</td>
<td>ASTM A496</td>
<td>N/A</td>
</tr>
<tr>
<td>Steel Deck</td>
<td>ASTM A653</td>
<td>A or B</td>
</tr>
<tr>
<td>Deformed Reinforcement Bars</td>
<td>ASTM A615</td>
<td>60</td>
</tr>
<tr>
<td>Welded Wire Fabric</td>
<td>ASTM A185</td>
<td>N/A</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Masonry Type</th>
<th>Standard</th>
<th>Strength (psi)</th>
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<tbody>
<tr>
<td>Concrete Masonry Units</td>
<td>ACI 530</td>
<td>2175</td>
</tr>
<tr>
<td>Mortar</td>
<td>ASTM C270</td>
<td>N/A</td>
</tr>
<tr>
<td>Grout</td>
<td>ASTM C475</td>
<td>3000-5000</td>
</tr>
</tbody>
</table>

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

Table 1 Summary of materials used on the USB project with design standards and strengths.
Gravity Loads

As a part of this technical report, dead, live and snow loads were all calculated and compared to loads listed on the structural drawings. Following basic load documentation, several gravity members in the structure were checked to verify their adequacy. Detailed calculations for these gravity member checks can be found in Appendix B.

Dead and Live Loads

The structural drawings list superimposed dead loads, summarized in Table 2. Analyses found that these loads are accurate, although conservative in some cases. The ceiling and mechanical load applied is potentially higher than usual, but this can be explained by the large ductwork required to bring 100% outside air into the laboratory spaces. The uniform application of housekeeping pad loads to mechanical and electrical spaces is conservative because these pads are scattered over these spaces. However, these loads seem to be calculated by weight of concrete required for the depth of the pad specified. The masonry walls in the structure are 8" concrete masonry unit (CMU), weighing approximately 60 pounds per square foot (psf). Thus, the masonry wall load corresponds to a 14 foot high 8" CMU wall.

Following the verification of the superimposed dead loads, estimations were made in order to calculate the overall building weight (which was also used in seismic calculations). By looking at typical sections through filigree slabs and beams, it was decided to consider the slabs 80% solid concrete and the beams 90% solid concrete.

Also considered in the building weight calculation were the weights of the columns, shear walls, superimposed dead loads, roofs, and wall loads (both exterior and interior). The exterior walls were considered to be 60 psf, as they are 8" CMU back-up walls with a cladding that weighs approximately 1 psf. The results of this calculation are summarized per level and for a typical level in Table 3. The overall

<table>
<thead>
<tr>
<th>Description</th>
<th>Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>1st Level Ceiling/Mechanical</td>
<td>10 psf</td>
</tr>
<tr>
<td>Other Levels Ceiling/Mechanical</td>
<td>15 psf</td>
</tr>
<tr>
<td>Electrical Room 4&quot; Housekeeping Pad</td>
<td>55 psf</td>
</tr>
<tr>
<td>Mechanical Rooms 6&quot; Housekeeping Pads</td>
<td>80 psf</td>
</tr>
<tr>
<td>Roofing</td>
<td>20 psf</td>
</tr>
<tr>
<td>Topping on Office Roof</td>
<td>36 psf</td>
</tr>
<tr>
<td>Masonry Wall</td>
<td>840 plf</td>
</tr>
</tbody>
</table>

Table 2 Summary of Superimposed Dead Loads.

<table>
<thead>
<tr>
<th>Level</th>
<th>Area (ft²)</th>
<th>Weight (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground</td>
<td>25,459</td>
<td>131.62</td>
</tr>
<tr>
<td>2nd</td>
<td>21,135</td>
<td>217.83</td>
</tr>
<tr>
<td>3rd</td>
<td>21,135</td>
<td>216.39</td>
</tr>
<tr>
<td>4th</td>
<td>21,135</td>
<td>216.39</td>
</tr>
<tr>
<td>5th</td>
<td>22,215</td>
<td>234.24</td>
</tr>
<tr>
<td>Penthouse</td>
<td>22,602</td>
<td>265.50</td>
</tr>
<tr>
<td>Roof</td>
<td>12,780</td>
<td>170.28</td>
</tr>
</tbody>
</table>

Table 3 Summary of building weight per level and a typical level.
building weight was found to be approximately 30,500 k.

Live loads were also listed on the structural drawings. These were compared to the live loads in Table 4-1 in ASCE 7-05 based on the usage of the spaces, and the results are summarized in Table 4. Although many of these loads matched their ASCE 7-05 counterparts, some exceed the minimum significantly.

The large classrooms on the first floor were all designed for 100 psf, which is the design load for assembly areas with movable seating. These classrooms all have fixed seating, but it is possible that this was not yet decided at the time of the initial structural design, and therefore the more conservative load was used. There is no provision for laboratories in classroom or research facilities, so the provision for "Hospitals – Operating Rooms, Laboratories" was used for comparison. It is possible that this was exceeded because most of these labs are to be teaching facilities, where occupant loads could exceed typical values depending on class sizes. The last major discrepancy was the live load on the Office Roof. This roof was accessible during construction, and was used for materials storage during this phase of the building’s life. It is possible this load was increased to account for the loads associated with this, such as workers on the roof to access materials stored there.

It was also noted on the structural drawings that live load reduction was used wherever possible for all gravity calculations in this technical report.

<table>
<thead>
<tr>
<th>Space</th>
<th>Design Live Load (psf)</th>
<th>ASCE 7-05 Live Load (psf)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Atrium</td>
<td>100</td>
<td>100</td>
<td>N/A</td>
</tr>
<tr>
<td>Large Classrooms</td>
<td>100</td>
<td>60</td>
<td>Fixed Seating in all</td>
</tr>
<tr>
<td>Laboratories</td>
<td>80</td>
<td>60</td>
<td>Based on &quot;Hospitals - Laboratories&quot;</td>
</tr>
<tr>
<td>Offices</td>
<td>50+20</td>
<td>50+20</td>
<td>Office Load+Partition Load</td>
</tr>
<tr>
<td>Links/Stairs</td>
<td>100</td>
<td>100</td>
<td>N/A</td>
</tr>
<tr>
<td>5th Level Lab</td>
<td>80+20</td>
<td>60+20</td>
<td>Based on &quot;Hospitals - Laboratories&quot;+ Partition Load</td>
</tr>
<tr>
<td>5th Level Mech. Room</td>
<td>100</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Electrical Room</td>
<td>150</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Office Roof</td>
<td>50</td>
<td>20</td>
<td>May be due to construction loading</td>
</tr>
<tr>
<td>AHU Mechanical Room</td>
<td>100</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Chiller Mechanical Room</td>
<td>150</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Other Roofs</td>
<td>20</td>
<td>20</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Table 4 Summary of design live loads, compared to ASCE 7-05 typical live loads.

<table>
<thead>
<tr>
<th>Flat Roof Snow Load Calculations</th>
<th>Variable</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground Snow Load, p_s (psf)</td>
<td>30</td>
<td></td>
</tr>
<tr>
<td>Temperature Factor, C_t</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>Exposure Factor, C_e</td>
<td>1.0</td>
<td></td>
</tr>
<tr>
<td>Importance Factor, I_i</td>
<td>1.1</td>
<td></td>
</tr>
<tr>
<td>Flat Roof Snow Load, p_f (psf)</td>
<td>23.1</td>
<td></td>
</tr>
</tbody>
</table>

Snow Loads
The roof snow load was calculated using the procedure outlined in Chapter 7 of ASCE 7-05, and the factors required for this calculation are summarized in Table 5. The structural drawings used a C_t of 0.8, but this does not seem to be permissible by code. Therefore, the drawings used a flat roof snow load of 20 psf, whereas 23.1 psf was calculated (and used for all subsequent calculations) in this technical report.

Table 5 Summary of roof snow load calculations.
Floor Systems

Although it may not appear so upon first glance at the very irregular shape of the building, the bay sizes are relatively consistent throughout the USB. It simply rotates the bays as necessary to accommodate the different rotations of the wings of the building. Figure 6 shows a typical floor plan with the different bay sizes highlighted with different colors. The legend lists the bay sizes with the span required for the slab first, and then the span required for the girder (if one is present).

The main objective of this technical report was to analyze the existing floor system, and then design three other floor systems. For ease of comparison, all analysis and design was conducted on a 21'-0"x36'-4" bay spanning North-South between column lines C and D and East-West between column lines 1 and 2 (included as Figure A.6 in Appendix A). All four systems were then compared on the basis of general conditions (weight, cost per square foot, and structural depth), architectural conditions (fire rating and other impacts), structural conditions (foundation impact and lateral system impact), serviceability conditions (maximum deflection and vibration control) and construction concerns (additional fire protection required, schedule impact, and constructability).

Voided Filigree Slab/Beam

The elevated floors of the USB are a voided filigree system. This is a hybrid of precast, prestressed concrete and cast-in-place concrete. In essence, it consists of 2 ¼" of precast, prestressed concrete that functions as leave-in formwork. This is assembled and shored on site, followed by the placement of top and additional bottom reinforcing (if required, placed on rebar chairs on the bottom of the precast), and then further concrete is cast in place to unite the system. To help reduce the weight of the structure, polystyrene voids are incorporated where the concrete is not required for structural strength. Wire joists referred to as “filigree trusses” are used to transfer horizontal shear over the cold joint between precast and cast-in-place concrete.

Different systems were used depending on the required spans and uses. For the area under consideration in this technical report, an 8" voided filigree slab (V.F.S.) was used to span between 18" deep voided filigree beams (V.F.B.). A schematic layout of this type of system, used in the majority of the building, is shown in Figure 7. Spot checks were performed on a V.F.S. panel, the V.F.B. along column line D, and the interior column on column line D (Column D/2). These results can be found in Appendix B.

General

The filigree slab system was found to weigh 127 pounds per square foot (psf), which served as a baseline to compare to the other flooring systems. At approximately $17.50/sf, this is the least expensive system of all systems considered, which may have been the driving force behind its selection for the USB. This cost is an assemblies estimate based on data from RS Means which includes the precast production, transportation, and installation and the cast-in-place concrete materials and placement (including the columns). This estimating method carries an error of approximately 15%. Costs in this technical report do not reflect any schedule effects as a result of altered construction method.
The depth of the structure is 8" in the slab region and 18" under the beams. This is important because almost the entire remaining ceiling cavity is being consumed by the large ductwork required to bring in the required 100% outside air for the laboratories. Therefore, any additional structural depth would either require significant mechanical redesign or additional building height. The building height is not limited by zoning, but by the requirements to remain a non-high-rise building by IBC 2006. This code states in section 403.1 that high-rise provisions only apply to buildings with an occupied floor above 75 feet above the lowest level of fire department access. The highest occupied level at present is the 5th Level, located 57'-2" above grade. Therefore, the building can easily add as much as 17'-10" of height before this becomes truly problematic.

**Architectural**
The system has a minimum of the required 2 hour fire rating, and since the building was designed around this system, there are no additional architectural impacts. It should be noted that there are several locations in the building where the bottom of the structure was left exposed, which was made possible by the smooth surface of the precast concrete.

**Structural**
Drilled concrete caisson foundations and a cast-in-place concrete shear wall lateral system were designed for this system and are unchanged should this system remain.

**Serviceability**
For the purposes of this report, maximum deflection was calculated by adding maximum slab/infill beam total load deflection to the maximum total load deflection of the girder. This choice was made because the calculated value better represents the overall deflection of the system. For the filigree slab system, this value was found to be 1.07 inches, which is acceptable. It is also the lowest deflection calculated for all of the systems.

Vibrational analyses were not performed for this report, but general research was done on how the system types analyzed/designs typically behave for vibration. The filigree slab system was given a vibrational control rating of “very good.”

**Construction**
This system requires no additional fireproofing to reach the required rating. The construction schedule of over 2 years was
developed specifically for this system, and thus was not impacted.

Constructability ratings were given on the basis of difficulty of construction as well as how many different types of crews would have to be involved to successfully install the system. For the filigree slab, it was determined that a crew would be required to place the precast, and then a crew would be required to pour and cast the concrete. Handling of the precast is very difficult because of the thinness of the panels. Therefore, it was assigned a constructability rating of “medium.”

**System Pro-Con Analysis**

**Pros:**
- Low cost per square foot
- Minimal floor depth allows room for mechanical equipment
- Relatively low deflection

**Cons:**
- Relatively heavy, which requires large foundations
- Adverse effect on seismic loads
- Higher construction difficulty
- Difficult to drill through slab due to prestressing strands

Despite the fact that this system is relatively heavy, it performs well in almost every other category analyzed. Therefore, it is easy to see why this system would be chosen over the numerous other options that could have been used for the USB.
Composite Steel

The first system designed was a composite steel system, which was chosen because it seemed the most practical steel-based system to span the long bays for the relatively heavy loads. Resulting beam and girder sizes with the required camber and shear studs from the hand calculations are shown in Figure 8 (hand calculations can be found in Appendix C). The beams are topped by 3” Vulcraft 3VL19 composite deck with 3 ½” concrete topping, and composite action is achieved with 4 ½” welded studs. RAM Structural System was used to verify the hand calculations. Values obtained by RAM were almost universally different because the program simply looks for the most economical section without incorporating the need to minimize floor depth.

This layout was the result of several preliminary investigations, the calculations for which are not included in this report but are available upon request. First, beams spanning the long direction with spacing of both 7’-0” and 10’-6” were checked. This resulted in beams and girders of the same size, typically W16’s or W18’s. Such a layout would have had an overly negative impact on the mechanical plan of the building because it would make running mechanical equipment in the ceiling more difficult. Therefore, the beams were turned to span the short direction to get the recessed “cavity” between the girders in which mechanical equipment could be easily run. Beam spacing of 9’-1” and 12’-1” were checked for this layout, and the 12’-1” spacing was found to be the most economical. All layouts were evaluated for both 4 ½” topping (which automatically achieves a 2 hour fire rating) and spray-on fireproofing the entire bottom of the deck. It was found that neither provided any advantage, and therefore it was chosen to fireproof the entire deck in an attempt to minimize dead load as much as possible.

General
With a 6 ½” total thickness deck and the beams shown above, this system was found to weigh approximately 68 pounds per square foot. This is significantly less than the filigree system. However, it costs $20.40 per square foot (including framing, metal deck, pour stops, concrete for the deck,
fireproofing, columns, and erection, but does not account for impacts on the schedule or foundations). Although this is not an excessive increase, it is certainly a factor to consider. The most important impact is the increase in the overall floor thickness to 17” in the “slab” region and 25” in the girder region. The 9” increase in depth in the slab region is much easier to absorb in the mechanical layout than the 7” increase in depth under the girder.

**Architectural**

Due to the decision to fireproof the underside of the deck, the system achieves the required 2 hour rating. However, this fireproofing makes the structure impossible to leave exposed, both for aesthetic reasons as well as the concern of pieces of fireproofing flaking off on building inhabitants. A remedy to this would be the use of intumescent paint, but this is usually considered excessively expensive in comparison to simply adding a drop ceiling. The addition of a drop ceiling would provide more mechanical space in which to run equipment, and may actually absorb the effects of the increased structure depth. However, if this is not the case, an additional 7” per floor (2’-4” overall) would have to be added to the 2nd through 5th levels. This is well within the allowable height limits, and thus is not a major concern.

**Structural**

Since this system is nearly half the weight of the filigree slab/beam system, it was originally hoped that the foundations could be reduced significantly, perhaps even using shallow foundations. However, upon reviewing the geotechnical report, it was found that bearing capacities are not listed for any of the intermediate layers of soil. Even so, these soils are largely described as loose to medium density, composed of largely sand and clay, and are universally declared to have “low load-bearing capacities.” Therefore, shallow foundations seem to be impractical. However, the concrete caissons (which are limited to a minimum diameter to achieve sufficient “skin” friction) cannot be reduced in size below 36” in diameter. At this size, the caissons on this project have a service load-bearing capacity of 630 k. RAM gives the service load in Column D/2 with structural steel framing for all floors as slightly less than 490 k, and this is one of the more highly-loaded columns in the structure. Therefore, it seems as though the concrete caissons would be impractical for a structural steel frame due to grossly excessive capacity. To achieve a more reasonable capacity, alternative deep foundation systems could be explored to determine if there is a significant cost-savings in using a smaller foundation system. The most suitable alternate foundation would probably be drilled micropiles. Rammed aggregate piers (Geopiers) and driven piles were also briefly considered as options, but they were rejected due to vibrational and noise concerns for the buildings adjacent to the site. These foundation systems were not designed in this technical report, but would be important to explore further should composite steel be chosen.

The lateral system could be easily transformed into braced frames or moment frames, potentially even placing them in the same locations as the shear walls are currently positioned. This was also not considered in this analysis, but would need to be investigated if composite steel were to be used in the building.

**Serviceability**

The maximum deflection for the composite steel system was found to be 1.44 inches, approximately 35% larger than that in the filigree slab/beam. It is still well within permissible limits, but it may limit the selection of floor materials more than the filigree slab/beam. Although no vibration analyses were performed, it is known that vibrations are a much larger concern in steel. Should this system be chosen for further investigation, vibrational checks would likely be important to verify the system behaves adequately.
Construction
Structural steel requires spray-on fireproofing to reach the required fire rating, which impacts cost and construction schedule. However, the erection of steel is usually able to be completed more quickly than casting of concrete, and therefore the use of steel may reduce the construction schedule significantly. Because this system is so typical, it was given a constructability rating of “very good.”

System Pro-Con Analysis

Pros:
- Less weight
- Potential to reduce required foundations
- Positive effect on seismic loads
- May shorten construction schedule
- Relatively simple system to construct
- Ease of drilling through floor

Cons:
- Higher cost system
- Leaving structure in the ceilings exposed would be difficult and costly
- Potential height increase
- Performs worse for serviceability concerns

Although this system is less effective in comparison to the filigree slab/beam system, none of its comparative flaws render it legitimately inadequate, and therefore it merits further consideration.
Post-Tensioned Concrete

Post-tensioned design is often used to reduce the depth of a traditional concrete system, which was particularly important for the USB. The design was performed by hand calculations (which can be found in Appendix D) based on a design example published by the Portland Cement Association (PCA). Also referenced was the Post-Tensioning Institute’s (PTI) Technical Note 3, written by Dr. Bijan Aalami, and an article from the May 2003 issue of Concrete International by Dr. Bijan Aalami and Jennifer Jurgens entitled “Guidelines for the Design of Post-Tensioned Floors.”

These calculations resulted in a 7” deep slab and a 14"Dx3'-6"W wide-shallow beam. The post-tensioning required was (18) ½” Ø 7-wire unbonded tendons in the distributed direction, and (15) ½” Ø 7-wire unbonded tendons in the banded direction. Per recommendations from previous projects, as provided by Dr. Andrés Lepage, the strands in the banded direction are placed into 3 bundles of 5 strands each within the beams, as can be seen in Figure 9.

General
Despite having no voids, the reduction in depth was such that the post-tensioned system only weighs 102 pounds per square foot, which is 20% less than the filigree slab/beam system. The cost of the post-tensioning essentially offsets the cost of the precast for the filigree system, and therefore the post-tensioned costs approximately the same as the filigree at $17.95 per square foot. Since the post-tensioned slab is 7” deep and the beam is 14” deep, this system is actually an improvement on the filigree system (8" V.F.S/18" V.F.B.) in terms of floor depth.

Architectural
This system achieves the required fire rating from cover requirements on the reinforcing, all of which were incorporated into this design, and therefore the required 2 hour fire rating was maintained. The structural depth is actually less than the filigree system, and therefore may create a slightly reduced building...
height. This possibility was not explored in this technical report, because any reduction would be minimal (approximately 1’-4”). With careful construction practices, a smooth underside of the structure could be achieved, which would then allow the structure to be left exposed. However, this may be more costly than the basic costs that were evaluated in this report.

**Structural**

Although the building weight will be reduced if this system were to be used, the impact on the foundations would likely be small, consisting of smaller caissons rather than the possibility of changing foundation systems. This system would also have little or no effect on the lateral system, since concrete shear walls make the most sense for a structure that will be cast-in-place concrete.

**Serviceability**

Deflections were not directly calculated for this system, but rather were limited by acceptable span-to-depth ratios from industry practice as outlined in PTI’s Technical Note 3. It is known that post-tensioned floors also tend to perform very well under vibration loading, and thus serviceability is not likely to be a concern for this system.

**Construction**

No additional fire proofing is required to achieve the required rating. It is likely that this system will either keep the same construction schedule or potentially lengthen the schedule, depending upon how much concrete will be poured in cold-weather conditions. This system was given a constructability rating of “medium,” because although it only involves a concrete crew, this crew must be familiar with post-tensioned construction to complete the project successfully and safely.

**System Pro-Con Analysis**

**Pros:**
- Less weight
- Likely minimal impact on foundations/seismic loads
- Cost approximately comparable to filigree
- Less floor depth
- No need for finished ceilings
- Very good performance under vibration

**Cons:**
- Added construction difficulty due to post-tensioning requirements
- Difficult to drill through slab due to tendons

The benefits of this system far outweigh the negatives, and none of the negatives are such that would preclude the use of post-tensioning. Therefore, this system is a viable alternative.
Precast Hollow core/Steel Girder

The precast hollow core/steel girder system was chosen after evaluating several other possibilities, the calculations for which are not included in this report but are available upon request. The choice to investigate this system was initiated by the wish to find a steel-based system that would have less impact on the building height than composite steel. The Girder-Slab (www.girder-slab.com) was first considered, but it was found that this would not be suitable for this building because the D-beam support members could not span even the 21 foot dimension under the required loads. Next, a precast hollow core/steel girder system was considered simultaneously with an entirely precast system. It was found that the entire precast system was impractical because it added weight and structure depth unnecessarily, and thus the final design was performed on the precast hollow core/steel girder system. The system was considered with hollow cores spanning both the long and short directions, but the short direction was chosen because Nitterhouse Concrete Products’ design data indicates a 16” deep hollow core would be required to span 36'-4”, which defeated the aforementioned purpose of reducing structure depths.

The final design consists of 6"Dx4'-0"W hollow core planks from Nitterhouse Concrete Products with 2" topping spanning the 21'-0” direction and resting on W18x175 steel beams (see Figure 10) with a connection detail similar to Figure 11. The design was performed by hand calculations, which can be found in Appendix E. The design sheet for the hollow core as provided by Nitterhouse was also included in Appendix E.

**General**
This system falls in the middle of the weights calculated for the various systems in this report at 82 pounds per square foot. However, it costs by far the most at $27.45 per square foot. This cost includes the precast production, transportation, and installation, the steel framing (including the columns) and erection, the concrete topping, and fireproofing for the steel, but no schedule or foundation impacts. It has a
structural depth of 8” in the slab region, which matches the filigree slab/beam system, but is 22” deep at
the girder. This 4” increase in depth is in the region that would be very difficult to absorb without
increasing building height.

Architectural
The precast portion of the structure achieves the required 2 hour rating for fire protection simply through
its design. However, the steel girders would require fire proofing wherever they are left exposed to
reach the appropriate rating. Therefore, it would be difficult to leave these beams architecturally
exposed. It is possible that it might be easier to build soffits for the beams for this system, but it is likely
that the most economical solution to the required fireproofing is to provide a drop ceiling. The addition of
a drop ceiling would provide extra mechanical space, and therefore potentially alleviate the issues
caused by the increased structural depth. However, if this is not enough, the overall building height would
be increased by approximately 1’-4”, which is well below the maximum allowable height increase.

Structural
This system is significantly lighter than the filigree slab/beam system, and may merit a different
foundation system, similar to the composite steel system. Since the vertical columns are steel, it is likely the
lateral system would have to be either steel moment or braced frames. However, both systems would be
complicated significantly by the connection of the precast to the girders. Since fabrication costs for this
system are already high (due to the stiffener and shelf angles required to support the precast), unique or
difficult moment/braced frame connections would only serve to exacerbate the problem.

Serviceability
The deflection for this system was the worst of all the systems calculated in this report at 1.74 inches
(approximately 63% greater than the filigree slab/beam). Although this is within permissible limits, it
may limit selection of floor finishes. Most of this deflection comes from the girder, and the only economical
way to reduce this deflection is to provide a deeper girder. Since this is undesirable, the deflection
presents itself as an unsolvable problem. The behavior of this system under vibration is unknown, and
would have to be investigated carefully if this system were to be chosen for further consideration.

Construction
Spray-on fireproofing would be required to ensure the rating of the steel girders. Due to the extremely
complicated construction process, which will likely include tack-welding of precast planks during erection,
it is possible that the construction schedule for this system would be longer than the one for the filigree
slab/beam. Due to the number of trades required to complete this system, as well as how uncommonly
used it is (and therefore how little familiarity most contractors will have with the system), it was given a
constructability rating of “difficult.”

System Pro-Con Analysis

Pros:
- Less weight
  - Potential to reduce required foundations
  - Positive effect on seismic loads

Cons:
- Very high cost
- Leaving structure in the ceilings exposed would be difficult and costly
- Potential height increase
- Unknown performance in vibration
- Potentially increased construction schedule
- Construction very difficult
- Difficult to drill through slab due to prestressing

The drawbacks to this system are insurmountable by its benefits, which renders this system not a feasible
option for the USB.
Summary of Systems

Figure 12 summarizes the results discussed in the preceding sections in a tabular format.

<table>
<thead>
<tr>
<th>Consideration</th>
<th>Filigree Slab</th>
<th>Composite Steel</th>
<th>Post-Tensioned Concrete</th>
<th>Precast Hollow core/Steel Girder</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight (psf)</td>
<td>127</td>
<td>68</td>
<td>102</td>
<td>82</td>
</tr>
<tr>
<td>Cost ($/SF) *</td>
<td>17.50</td>
<td>20.40</td>
<td>17.95</td>
<td>27.45</td>
</tr>
<tr>
<td>Floor Depth (inches)</td>
<td>8 slab/18 beam</td>
<td>17 slab/25 girder</td>
<td>7 slab/14 beam</td>
<td>8 slab/22 beam</td>
</tr>
<tr>
<td>Fire Rating</td>
<td>2 hr</td>
<td>2 hr</td>
<td>2 hr</td>
<td>2 hr</td>
</tr>
<tr>
<td>Architectural</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Other Impacts</td>
<td>Bottom of slab currently left exposed in some locations</td>
<td>Adds 6&quot;-9&quot; of height per floor, cannot be left exposed</td>
<td>Can only be left exposed if underside is well finished</td>
<td>Adds 4&quot; of height per floor, beams cannot be left exposed</td>
</tr>
<tr>
<td>Foundation Impact</td>
<td>Existing cast-in-place caissons &amp; grade beams</td>
<td>May reduce required foundations</td>
<td>May slightly reduce required foundations</td>
<td>May reduce required foundations</td>
</tr>
<tr>
<td>Lateral System Impact</td>
<td>Existing cast-in-place shear walls</td>
<td>Steel braced/moment frames would be considered</td>
<td>None - shear walls would remain</td>
<td>Steel braced/moment frames would be considered</td>
</tr>
<tr>
<td>Maximum Deflection (inches)</td>
<td>1.07</td>
<td>1.44</td>
<td>N/A</td>
<td>1.74</td>
</tr>
<tr>
<td>Vibration Control</td>
<td>Very Good</td>
<td>Average</td>
<td>Very Good</td>
<td>Unknown</td>
</tr>
<tr>
<td>Additional Fire Protection Required</td>
<td>None</td>
<td>Spray-on for beams/deck</td>
<td>None</td>
<td>Spray-on for beams</td>
</tr>
<tr>
<td>Construction Schedule Impact</td>
<td>N/A</td>
<td>May reduce construction schedule</td>
<td>Will likely have no effect/potentially increase duration</td>
<td>Will likely have no effect/potentially increase duration</td>
</tr>
</tbody>
</table>

Feasible? | N/A | Easy | Medium | Difficult |

* - All costs are calculated using RS Means Assemblies Costs (which carries an approximate error of ±15%) for a typical interior bay with dimensions of 36'-4"x21'-0". They include materials (ie steel/concrete/fireproofing), installation, and labor, but do not include impacts on the foundations, construction schedule or architectural elements (ie facade or drop ceilings)

**Figure 12** Summary chart of this report’s findings.
Conclusion

Technical Report 2 analyzed the existing floor system and compared it to three additional floor systems, all of which were also designed as a part of the technical report. The analysis/design of all systems was performed at a typical bay. Major factors in the comparison of the systems were cost, weight, structural depth, and architectural impact, although several other considerations were also included. It was desirable to reduce the weight of the building without adversely affecting the cost or structural depth.

The existing 8” voided filigree slab and 18” voided filigree beam system remains the least costly system, but by a narrow margin. It is the heaviest system that was considered in this report. It was found in this technical report that the structural depth achieved in this system is very difficult to reduce, or even match. It was verified to be a very reasonable choice as the structural system of the USB.

Composite steel was found to be slightly more expensive but significantly lighter than the voided filigree system. However, it has several negative impacts on the building architecture, such as the potential of increased height (due to higher structural depth) and the inability to leave the structure exposed. Despite these concerns, the system has a great deal of inherent flexibility, and it is possible that with further refinement, these concerns could be resolved. It also can utilize either a braced frame or moment frame lateral system, which provides additional opportunities to adjust the design to suit the building. For these reasons, it was deemed to be a viable alternative.

Of the alternative systems considered, the 7” post-tensioned concrete slab supported by 14” deep wide-shallow beams was by far the most successful. It reduces the weight of the structure by approximately 20%, costs nearly the same, provides a lesser structural depth, and has no architectural impacts whatsoever. The only major drawback of this system is the additional construction difficulty associated with the post-tensioning process. However, this was not deemed to overwhelm the obvious benefits of the system, and it is therefore a feasible option.

The only system of those investigated that was found to be inadequate was the 6” precast hollow core with 2” topping on W18x175 steel girders. Although it reduced the building weight by 35% and the structural depth increase was minimal, the system cost was excessive (approximately 60% more than the current filigree slab system) and the architectural impacts would be difficult to accommodate. It also lacks the flexibility of the composite steel system due to the precast-to-steel connection that would make both braced frames and moment frames difficult to design. It was therefore rejected, and will no longer be considered as an alternative.
Appendices

Appendix A: Typical Plans

Figure A.1 Typical Floor plan, taken from S202. See following figures for sections indicated on the plan.
Figure A.2 Section 1 through portion of building at 0° rotation (see Figure 1), taken from 3/A401.
Figure A.3 Section 2 through portion of building at -15° rotation (see Figure 1), taken from 2/A402.
Figure A.4 Section 3 through portion of building at -45° rotation (see Figure 1), taken from 4/A402.
Figure A.5 Section 4 through portion of building at -20° rotation (see Figure 1), taken from 3/A403.
Figure A.6 Enlarged floor plan for the area in which the gravity checks were performed, taken from S202 (levels 2 through 4 are identical, and reinforcing is only displayed on level 2). Slab design moments are boxed (k-ft/ft), beam design moments are enclosed in an oval (k-ft), and the location of the first void in the beams with relation to the face of columns is enclosed in a prism-like shape.
Appendix B: Voided Filigree Slab/Beam Calculations

Notes:
- Structural drawings use 500 sf
- Loads listed on section are service loads from the column schedule, Sheet 5201

Assumptions:
- Voided Filigree Slabs (V.F.S) 80%
- Solid Filigree Slabs (S.F.S) 20%
- Voided Filigree Beams (V.F.B) 90%
- Solid Filigree Beams (S.F.B) 10%

self weights of slabs/Beams

V.F.S, W/ 8x12" V.F.B ->
[(1/2) (0.9)] [(1/2) (0.8)] 150 psf = 105 psf

V.F.B, W/ 8x18" V.F.B ->
[(1/2) (0.9)] [(1/2) (0.6)] 150 psf = 130 psf

Column Loads

Roof
\[
P_3 = 560(15 + 20 + 20) = 47690 \text{ k} \rightarrow 50 \text{ k}
\]

\[
P_3 = 560(20) = 12960 \text{ k} \rightarrow 15 \text{ k}
\]

Penthouse

\[
P_3 = 560(90 + 165) + 450(22.5) + 10210 = 1540 \text{ k}
\]

\[
P_3 = 560(135) = 560 \text{ k} \rightarrow 65 \text{ k}
\]

5th

\[
P_3 = 560(15 + 130) + (1/2) 900(14.25) = 900 \text{ k}
\]

\[
P_3 = 560(0.6)(80) = 26.9 \text{ k} \rightarrow 30 \text{ k}
\]

4th

\[
P_3 = 560(15 + 130) + (1/2) 900(14) = 900 \text{ k}
\]

\[
P_3 = 560(0.5)(80) = 22.4 \text{ k} \rightarrow 20 \text{ k}
\]

3rd

\[
P_3 = 90 \text{ k} \quad (\text{see 4th})
\]

\[
P_3 = 0.25 + (\text{4th}) = 0.47 \rightarrow 0.5 \text{ k}
\]

\[
P_3 = 560(0.5)(80) = 22.4 \text{ k} \rightarrow 20 \text{ k}
\]

2nd

\[
P_3 = 90 \text{ k} \quad (\text{see 3rd})
\]

\[
P_3 = 20 \text{ k} \quad (\text{see 3rd})
\]

Totals

\[
P_3 = 564 \text{ k} \rightarrow P_3 = 1.2(564) + 1.4(108) = 794 \text{ k}
\]

\[
P_{\text{overall}} = 12(400) + 14(180) = 1010 \text{ k} \quad (\text{7% def})
\]
COLUMN IS CHECKED FOR PURE COMPRESSION BECAUSE IT IS AN INTERIOR COLUMN NOT IN A MOMENT FRAME OR NEAR A SHEAR WALL. THEREFORE, COLUMN ECCENTRICITY (e) IS NEGLIGIBLE.

\[
\phi P_0 = \phi [0.85 f'_c A_c + A_{sv}] = 0.65 \left[ 0.85 (5 \text{ ksi}) (16 (\text{in}^2)) - 9.48 \text{in}^2 \right] + 9.48 \text{in}^2 (60 \text{ ksi})
\]

= 19.35 k

PURE COMPRESSION LIMITED BY \( \alpha \) (ACCOUNTS FOR MINIMUM ECCENTRICITY)

\[
\phi P_0 = \alpha \phi P_0 = 0.8 (19.35 k) = 1548 k \gg P_0 \quad \text{OK}
\]

\[\rho = \frac{9.48}{36 (\text{in})} = 0.016 > 1\% \quad \text{(MINIMUM ALLOWABLE \( \rho \) PER ACI 318-08 SECTION 10.9.1)}
\]
CHECK AN INTERIOR VOIDED FILIGREE SLAB (V.F.S.) PANEL SPANNING BETWEEN COLUMN LINES C & D ON THE 4TH LEVEL

ASSUMPTIONS:
- VOIDED FILIGREE SLAB (V.F.S.) IS 80% SOLID
- VOIDED FILIGREE BEAMS (V.F.B.) ARE 90% SOLID

DESIGN MOMENT GIVEN ON STRUCTURAL DRAWINGS $= 51x \text{ft}^2$

FROM COMPARISON WITH MOMENTS ON SLAB AT ADJACENT BAYS, IT SEEMS AS THOUGH ACI MOMENT COEFFICIENTS WERE USED

SPAN A-B $M_{u} = 8.0 \times \text{ft}^2 = \frac{wL^2}{8}$
SPAN B-C $M_{u} = 5.5 \times \text{ft}^2 = \frac{wL^2}{8}$
SPAN CD $M_{u} = 5.1 \times \text{ft}^2 = \frac{wL^2}{8}$

$M_{u} = \frac{wL^2}{8} \Rightarrow x = 17.25 \text{ ft}$

LOADS FROM STRUCTURAL DRAWINGS:
- SDL = 15 psf
- LL = 80 psf

SLAB SELF-WEIGHT $= \left[ \left( \frac{w_{n}}{2} \right) \left( \frac{w_{n}}{6} \right) \left( 0.9 \right) + \left( \frac{w_{n}}{12} \right) \left( \frac{w_{n}}{2} \right) \left( 0.8 \right) \right] 150 \text{ plf} = 130 \text{ psf}$

CLEAR LENGTH $= L = 21 - 2 \left( \frac{w_{n}}{2} \right) \left( \frac{w_{n}}{6} \right) = 18 \text{ ft}$
$W_{n} = 130 + 15 = 145 \text{ psf} \Rightarrow W_{u} = 1.2 \times \text{psf} + 1.6 \times \text{plf} = 302 \text{ psf} \Rightarrow 0.802 \text{ kips/ft}$
$W_{n} = 0.802 \left( \frac{W_{n}}{2} \right) = 0.302 \left( \frac{W_{n}}{2} \right) = 5.76 \text{ kips/ft} \Rightarrow M_{u, \text{serv.}} = \frac{5.76 \times \text{ft}^2}{1.5} = 3.84 \text{ kips/ft}$
$M_{u} = 0.302 \left( \frac{W_{n}}{2} \right) = -8.9 \text{ kips/ft}$
$V_{n} = 0.302 \left( \frac{W_{n}}{2} \right) = 2.77 \text{ kips/ft}$
**Positive Moment Check**

\[ \text{Bottom Reinforcing } \rightarrow (6) \frac{3}{8} \text{ in P.S. Strands} \]

\[ P_e = 0.085 \times 0.875 \times (6 \text{ Strands}) (0.6) (270 \text{ KSI}) = 110.2 \text{ kN} \]

\[ M = (4+4+8+8) (1.75) (0.875) + 2.25^2 (96) (2.25) = 2.55 \text{ in} \]

\[ I = \frac{(4+4+8+8) (1.75)^3 (2.25-0.875)^3}{12} + \frac{96 (2.25) (2.25)(2.25-2.25)^2}{12} = 242.5 \text{ in}^4 \]

\[ S = \frac{I}{M} = \frac{242.5 \text{ in}^3}{2.55 \text{ in}} = 60.025 \text{ in}^3 = 5b \]

\[ A = \left[ (4+4+8+8) (1.75) + (96) (2.25) \right] = 510 \text{ in}^2 \]

\[ M_{\text{req}} = 3.64 \times \frac{f_{y}}{f_{c}} (8 \text{ ft}) (12 \text{ in}) = 369 \text{ k-in} \]

\[ \frac{M}{S} = \frac{369 \text{ k-in}}{60.025 \text{ in}^3} = 6.09 \text{ kSI} \]

\[ \frac{P_{e}}{A} = \frac{110.2 \text{ kN}}{514 \text{ in}^2} = 0.214 \text{ kSI} \]

\[ \frac{P_{e}}{S} = \frac{110.2 \text{ kN}}{60.025 \text{ in}^3} = 1.84 \text{ kSI} \]

\[ f_{y} = 6.09 - 0.214 - 5.23 = 0.64 \text{ kSI} \]

\[ f_{c} = 0.45 f_{c} = 0.45 (5) = 2.25 \text{ kSI} \]

\[ f_{y} = 1.5 \times (0.64) = 0.969 \text{ kSI} \]

\[ f_{c} = 1.5 \times (0.45) = 0.975 \text{ kSI} \]

\[ \frac{f_{y}}{f_{c}} = 0.975 < \frac{f_{y}}{f_{c}} \]

**Negative Moment Check**

**Reinforcing Provided** \#5 @ 12” with WWF (for shrinkage & temp)

\[ A_{0} = 0.31 \text{ in}^2 \]

\[ A = 8 - 0.75 - \frac{1}{2} (9.84) = 4.94 \text{ in} \]

\[ \alpha = \frac{A_{0} f_{y}}{A_{0} f_{c}} = 0.31 \times 60 \text{ kSI} = 0.365 \text{ in} \]

\[ \phi M = 0.8 A_{0} f_{y} (d - \frac{b}{6}) = 0.9 (0.31 \times 60 \text{ kSI}) (4.94 - \frac{9.84}{2}) = 9.3 \times 10^3 > M_{u} = 8.9 \times 10^3 \]
VERTICAL SHEAR CHECK

\[ \phi V_c = 0.2 \sqrt{\frac{1}{2}} \times 10.5\text{in} = 0.75(2) \sqrt{5000} \left(12\frac{\text{in}}{\text{ft}}\right) \left(0.94\text{ in} \right) \left(\frac{1}{1500} \right) = 8.83 \text{ k/ft} \]

\[ \phi V_c > V_c = 2.72 \text{ k/ft} \quad \text{OK} \]

HORIZONTAL SHEAR CHECK

\[ \frac{0.969}{x} = \frac{1.611}{2.3} \]

\[ 0.969 \times 2.3 = 1.611 \times x \]

\[ x = 3.00 \text{ in} \]

\[ V_{0n} = \frac{1}{2} (0.242 \text{ ksi})(12\frac{\text{in}}{\text{ft}})(0.75 \text{ in}) = 1.09 \text{ k/ft} \]

FROM ACI 318-08 SECT. 17.5.3.1,

\[ \phi V_{0n,k} = \phi (80 \text{ ksi}) \left(12\frac{\text{in}}{\text{ft}}\right) \left(0.94\text{ in} \right) \left(\frac{1}{1500} \right) = 5.0 \text{ k/ft} > V_{0n} \quad \text{OK} \]

DEFLECTION

PER ACI 318-08 SECT. 9.5.5.1, COMPOSITE MEMBERS CAN BE CONSIDERED EQUIVALENT TO A MONOLITHICALLY CAST MEMBER DUE TO USE OF SHORING DURING CASTING OF ADDITIONAL CONCRETE.

TRY CALCULATING DEFLECTIONS AS IF SIMPLY SUPPORTED

L1 THIS IS CONSERVATIVE, AND IF IT PASSES DEFLECTION REQUIREMENTS, NO MORE SPECIFIC/COMPLICATED CALCULATIONS WILL BE REQUIRED

\[ \Delta_{LL} = \frac{5}{384} \left(0.001\text{ ksi}\right) \left(18\frac{\text{in}}{\text{ft}}\right) \left(1728\text{ in}^2\right) \left(10^6 \text{ in} \right) = 0.155 \text{ in} \quad \text{OK} \]

\[ \Delta_{TL} = \frac{5}{384} \left(0.302\text{ ksi}\right) \left(18\frac{\text{in}}{\text{ft}}\right) \left(1728\text{ in}^2\right) \left(10^6 \text{ in} \right) = 0.365 \text{ in} \quad \text{OK} \]
CHECK 36'-4" BEAM SPANNING BETWEEN COLUMN LINES 1 & 2 ALONG COLUMN LINE D ON THE 4th LEVEL.

ASSUMPTIONS:
- VOIDED FILIGREE SLAB (V.F.S.) IS 30% SOLID
- VOIDED FILIGREE BEAM (V.F.B) IS 90% SOLID

IT SEEMS LIKELY THAT ACI MOMENT COEFFICIENTS WOULD BE USED FOR THE BEAMS, SINCE THEY WERE USED FOR THE SLAB. THEREFORE, THIS IS HOW I CHECKED THE BEAM.

LOADS FROM STRUCTURAL DRAWINGS:
- SDL = 15 psf
- LL = 80 psf

SLAB SELF-WEIGHT: \[ \left( \frac{1}{2} \right) \left( \frac{15}{12} \right) (0.9) + \left( \frac{2}{3} \right) \left( \frac{15}{12} \right) (0.3) \] 150 plf = 130 psf

CLEAR LENGTH: \[ L = 36.33 - 2 \left( \frac{3}{4} \right) \left( \frac{15}{12} \right) = 35 ft \]

LL = 0.25 + \[ \frac{15}{(25.3)(0.63)} + 0.63 > 0.5 \text{ OK} \]

\[ \omega_3 = 21 (130 + 15) = 3,045 \text{ plf} = 3,045 \text{ klf} \]
\[ \omega_2 = 21 (0.63)(80) = 1056 \text{ plf} = 1056 \text{ klf} \]
\[ M_{u} = \frac{wL^3}{12} = \frac{5,346(35)^3}{12} = 595 \text{ k-ft} \rightarrow M_{u,\text{serv}} = \frac{595 \text{ k-ft}}{1.5} = 397 \text{ k-ft} \]
\[ M_{u,1} = \frac{wL^2}{16} = \frac{5,346(35)^2}{16} = 409 \text{ k-ft} \]
\[ M_{u,2} = \frac{wL^2}{k} = \frac{5,346(35)^2}{k} = 728 \text{ k-ft} \]
\[ V_u = 5.346(35)(3) = 93.6 \text{ k} \]
Bottom Pre-Stressing → (26) 3/8" 270 kpsi strands

P_e = 0.085 \frac{12^2}{2} \text{Strands} \cdot (26) (0.370 kips) = 358 kips

Bottom Reinforcing → (4) #6

A_s = 0.44 \frac{\text{in.}^2}{\text{in.}^2} \cdot 1.76 = 0.78 in.

\alpha = \frac{A_s F_y}{0.85 f_{y} b} = \frac{1.76 (40)}{0.85 (5)(0.37)} = 0.259 in.

\Phi_{M_n} = \Phi_{A_s F_y} \left( \frac{d}{2} \right) = 0.9 \left( 1.76 \right) \left( 15.5 - \frac{0.37}{2} \right) = 122 k-ft

M_0 = 39.7 - 122 = 275 k-ft \times 12 \frac{ft}{in} = 3300 k-in

M_+ = (14^{"}+14^{\circ}+70^{\circ})(4.75^{\circ}) (3.75^{\circ}) + 96 (2.25^{\circ})(7.875^{\circ}) = 5.175 in

I_+ = \frac{(14^{"}+14^{\circ}+70^{\circ})(4.75^{\circ}) + (14^{"}+14^{\circ}+70^{\circ})(4.75^{\circ})}{12} = 394.6 in^4

S_+ = \frac{I_+}{y} = \frac{394.6 in^4}{4.75} = 438 in^2

A = \left( (96)(2.25^{\circ}) + (14^{"}+14^{\circ}+70^{\circ})(6.75^{\circ}) \right) \frac{2}{1080} in^2

\sigma_+ = \frac{M_+}{S} = \frac{5.175}{438 in} = 0.012 ksi

f_{\sigma} = 0.45 \frac{\sigma}{f_{y}} = 0.45 \frac{0.012}{(14)} = 0.008 ksi < 0.45 f_{\sigma} = 0.008 ksi < 0.45 f_{\sigma} = 0.008 ksi

f_{\sigma} = 0.042 ksi < 0.45 f_{\sigma} = 0.042 ksi

f_{\sigma} = 0.042 ksi < 0.45 f_{\sigma} = 0.042 ksi

f_{\sigma} = 0.042 ksi < 0.45 f_{\sigma} = 0.042 ksi

f_{\sigma} = 0.042 ksi < 0.45 f_{\sigma} = 0.042 ksi
Technical Report 2

Kathryn Gromowski | Structural Option

October 27th, 2010

University Sciences Building | Northeast USA

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GRAV. CHECK → V.F.B.  TECH 1

NEGATIVE MOMENT CHECK @ 1

REBAR PROVIDED → (8) #9

Aₚ = 8 (1.0) = 8.0 in²

d = 18 - 0.75 - ½(¾") = 16.49 in

a = Aₚ F_y / 0.85 f'f_b = 8(60) / 0.95(590) = 1.18 in

Mₙ = φ Aₚ F_y (d - a) / 12 = 0.9 (8 in²)(60 ksi)(16.49 in - 1.18 in) = 579 k·ft > N₀ = 409 k·ft

OK ✔

NEGATIVE MOMENT CHECK @ 2

REBAR PROVIDED → (12) #9

Aₚ = 12 (1.0) = 12 in²

d = 16.49 in

a = Aₚ F_y / 0.85 f'f_b = 12(60) / 0.95(590) = 1.76 in

Mₙ = φ Aₚ F_y (d - a) / 12 = 0.9 (12 in²)(60 ksi)(16.49 in - 1.76 in) = 854 k·ft > N₀ = 728 k·ft

OK ✔

VERTICAL SHEAR

Vₙ = φ 2 1/2 b d = 0.75 (2) 15000 (9.6") (16.49") = 170 k > V₀ = 93.6 k

OK ✔

HORIZONTAL SHEAR

1.14 = 2.13 / 16 - x

1.14(16) - 1.14x = 2.13x

x = 6.23 in

Vₓₙ = φ (0.732 ksi)(9.6") (4.03") = 142 k

OK ✔

Vₓₙ = φ 80 b w d = 0.75 (80)(9.6") (16.49") (1000) = 96.1 k < Vₓₙ : MUST CONSIDER STEEL

FROM ESR 96-14, A TYPICAL FILIGREE TRUSS IS MADE OF 6 mm DIAMETER WIRE AND PENETRATES THE HORIZONTAL SHEAR PLANE EVERY 5°. THERE ARE THREE TRUSES.
DEFLECTION

PER ACI 318-05 Sect. 9.5.5.1, composite members can be considered equivalent to a monolithically cast member due to use of shoring during casting of additional concrete.

Try calculating deflections as simply supported.

This is conservative, and if it passes deflection requirements, no more specific/complicated calculations will be required.

\[ \Delta_L = \frac{5}{384} \left( \frac{(1058 \text{ kips})(100^\text{in})}{384 (57000)^{1.5}} \right) \left[ \frac{15.4^4 - (12.9)^4 - (23.9)^4 + 12.9 (12.9) (15.4)}{1.6028} + 35^3 \right] \]

\[ \Delta_{LL, \text{max}} = \frac{5}{384} \left( \frac{(1058 \text{ kips})(100^\text{in})}{384 (57000)^{1.5}} \right) \left[ \frac{15.4^4 - (12.9)^4 - (23.9)^4 + 12.9 (12.9) (15.4)}{1.6028} + 35^3 \right] \]

Assume more accurate calculation required for total load deflection also.
\[
\Delta_{\text{max}} \text{ occurs at } x = \frac{1}{2} + \frac{M_1 - M_2}{w} = \frac{35}{2} + \frac{409 - 728}{5.3468(35)} = 15.80 \text{ ft}
\]

\[
\Delta_{TL} = \frac{5.3468(15.8)(728)}{24(575000)[2(33.46)]} \left[ \frac{15.8^3 - (2)(35) + \frac{4(409) - 4(728)}{5.3468(35)}}{5.3468} \right] 15.8^2 + \frac{12(409)(15.8)}{5.3468}
\]

\[
\Delta_{TL} = 0.701 \text{ in} < \frac{\Delta_{TLY, \text{max}}}{{2}40 = 1.75 \text{ in}}
\]

\[
\text{OK}
\]
Appendix C: Composite Steel Calculations

COMPOSITE STEEL

Choose Deck → Use Vulcraft Manual (Canam Not Readily Accessible)

→ Use 15’ or 3’ Deck (Sizes being Used on Project Already)

→ 4.5’ Topping or Spray Fireproofing on Bottom of Deck

→ 2 HR. Fire Rating

→ Spans Are Very Long, Choose to Fireproof to Reduce Weight

→ Superimposed Load

→ SD = 15 psf
→ LL = 80 psf

→ Design for 2-Span Condition for Safety
→ Use 3.5’ Topping

→ 1.5 VLR (Pd. 50 in Catalog, 56 psf for 5” Total Thickness)

→ Span Not Acceptable

→ 3 VLI (Pd. 54 in Catalog, 63 psf for 6.5” Total Thickness)

→ Span OK for > 3VLI 19
→ Load OK for > 3VLI 20

Choose 3VLI 19 w/ Total Thickness = “6” . . . s.w. = 63 psf

Design Interior Beam
→ Assume Simply Supported
→ Add 5 psf for Beam Self-Weight
→ Consider Camber to Reduce Structure Depth
→ L_min = “6”
→ L_max = “1” (Practical Limits)

W_v = (15 psf + 63 psf + 5 psf) (12.083 ft) = 1003 plf = 1.003 klf

LL_v = 0.25 + \frac{12(0.92)(0.21)}{3(4)} = 0.92

W_h = 80 psf (0.92) (12.083 ft) = 889 plf = 0.889 klf

W_s = 1.2 (1.003) + 1.1 (0.089) = 2.63 klf

M_v = W_h \frac{L^2}{8} = 2.63 \times (21.4)^2 = 145 k-ft

V_v = 2.63 klf (21.4) (0.4) = 27.6 k

From AISC Table 3-21, Find Q_n
→ Deck
→ Weak Position
→ 1 Stud Per Rib
→ \frac{1}{4}” Ø
→ f’ = 3 ksi
→ w_x = 145 psf

Q_n = 17.2 k
\[ \Delta_{LL, \text{max}} = \frac{4}{300} + 1 = 21 \times \frac{12}{300} + 1 = 1.7 \text{ in} \]

\[ \Delta_{LL} = \frac{5 \cdot w_0 \ell^4}{384 \cdot E \cdot I_{LL}} \]

\[ I_{LL, \text{min}} = \frac{5 \left[ 1.6 \left( 0.889 \text{ ksi} \right) \left( 21.4 \right)^4 \right]}{384 \left( 29,000 \text{ ksi} \right) \left( 1.7 \text{ in} \right)} = 126 \text{ in}^4 \]

Use AISC Tables 3-19 & 3-20 to find options

\( \text{Assume } \frac{1}{2} = 5.5 \text{ in} \)

1. \( w_0 \times 17 \) w/ \( \Sigma q = 150 \text{ k} \) \( \phi_{MM} = 156 \text{ k-ft} \) & \( I_{LL} = 291 \text{ in}^4 \)

\( \text{% Composite} = 60\% \)

\( \text{# of studs} = 159 \div 7.3 = 8.7 = 9 \text{ studs} \)

\( \text{Economy} = 19(21) + 18(10) = 537 \)

\( \text{Total} = 18 \text{ studs} < 21 \text{ ok} \)

2. \( w_0 \times 19 \) w/ \( \Sigma q = 122 \text{ k} \) \( \phi_{MM} = 156 \text{ k-ft} \) & \( I_{LL} = 286 \text{ in}^4 \)

\( \text{% Composite} = 43.4\% \)

\( \text{# of studs} = 197 \div 7.2 = 27.1 = 8 \text{ studs} \)

\( \text{Economy} = 19(21) + 16(10) = 559 \)

\( \text{Total} = 16 \text{ studs} < 21 \text{ ok} \)

3. \( w_0 \times 22 \) w/ \( \Sigma q = 81.1 \text{ k} \) \( \phi_{MM} = 152 \text{ k-ft} \) & \( I_{LL} = 263 \text{ in}^4 \)

\( \text{% Composite} = 25\% \)

\( \text{# of studs} = 204 \div 7.2 = 28.7 = 5 \text{ studs} \)

\( \text{Economy} = 22(21) + 10(10) = 562 \)

\( \text{Total} = 10 \text{ studs} < 21 \text{ ok} \)

Do not try \( w_0 \times 12 \) because \( w_0 \times 19 \) 's work and shallower is better in this case; try \( w_0 \times 22 \) (4 of bare beam usually controls)

\[ \Delta_{LL} = 5 \left[ 1.6 \left( 0.889 \text{ ksi} \right) \left( 21.4 \right)^4 \right] \]

\[ \Delta_{TL} = 5 \left[ 2.43 \text{ kft} \left( 21.4 \right)^4 \right] \]

\[ M_{u, \text{limit}} = 1.2 \left( 0.06 \right) + 1.6 \left( 0.03 \right) \left( 12.083 \right) \left( 21.4 \right)^2 = 75.7 \text{ k-ft} \]

\[ Q_{u, \text{limit}} = 5 \left[ 1.2 \left( 0.06 \right) + 1.6 \left( 0.03 \right) \left( 12.083 \right) \left( 21.4 \right)^2 \right] \]

\[ \text{Check SW assumption:} \quad \frac{22 \text{ psi}}{12.083 \text{ psi}} < 5 \text{ psi} \quad \text{OK} \]

\[ \Delta_{u, \text{limit}} = \frac{1.76 \text{ in} - 2 \times 1.51 \text{ in}}{2 \times 0.5} = 0.71 \text{ in} \]

\[ \phi_{\Delta} = 73.2 \text{ k} > \phi_{u} \text{ OK} \]

Use \( w_0 \times 22 \) w/ 10 studs and \( \frac{1}{2} = 3.5 \text{ in} \)
**Composite Steel**

**Design Girder**
- Assume simply supported
- Add 1 k to each point load for girder self-weight
- Consider camber to reduce structure depth

**Practical Limits**

\[ P_0 = 1.003 \text{ klf} (21 \text{ ft}) = 21.1 \text{ k} \]

**Load Calculations**

\[ P_L = 0.25 \left( \frac{15}{12} \right) (3.43) \]

\[ P_L = 0.08 \text{ klf} (12.83 \text{ ft})(21 \text{ ft})(0.63) = 12.9 \text{ k} \]

\[ P_L = 1.2 (21.1) + 1.6 (12.9) + 1 = 46.0 \text{ k} \]

\[ M_U = 55.4 \text{ k-ft}, \quad V_U = 44.0 \text{ k} \]

From AISC Tab 3-21, find \( Q_a \)

\[ \frac{w}{x} \text{ ksf} = 1.55 > 1.5 \]

\[ Q_a = 21.0 \text{ k} \]

**Deflection Calculations**

\[ \Delta L, \text{max} = \frac{1}{350} + 1 + 1.5 = 2.71 \text{ in} \]

**Economy Analysis**

\[ I_{lb, \text{min}} = 0.036 [1.6 (12.9 \text{ k})] (36.33 \text{ ft})^3 (1728) = 783 \text{ in}^4 \]

**Use AISC Tab 3-19 & 3-20 to find options**

1. W18 x 60, \( w/ \leq Q_a = 220 \text{ k}, \quad P_{lf} = 630 \text{ k-ft} \) & \( I_{lb} = 1690 \text{ in}^4 \)

   - 9% Composite = 25%
   - A of studs = 25/20k = 10.9 = 11 studs

   **Economy** = 60 (36.33) + 22 (10) = 2400

Next size listed is W21, to keep floor depth to a minimum, try W18 x 60.

\[ \Delta L = \frac{0.036 [16 (12.9 \text{ k})] (36.33 \text{ ft})^3 (1728)}{2900 \text{ k-ft} (1145 \text{ in}^3)} = 1.26 \text{ in} \]

\[ \Delta x \]
\[
\Delta_{u} = 0.036 \frac{(46.0 \text{ k}) (36.3 \text{ ft})^3 (1728)}{29000 \text{ ksi (1690)}} = 2.8 \text{ in} - 1.82 \text{ in} = 0.98 \text{ in} \quad ; \quad c = 1" \\
M_{u,\text{const}} = 0.33 \left[ 12 (0.005) + 1.6 (0.02) \right] (21 \text{ ft}) (12.083 \text{ ft}) (36.33 \text{ ft}) = 3.46 \text{ k-ft} \\
\phi M_{u,\text{bare beam}} = 4.61 \text{ k-ft} > M_{u,\text{const}} \quad \text{OK} \\
\Delta_{\text{const}} = 0.036 \left[ 1.3 (0.008) + 1.6 (0.02) \right] (21 \text{ ft}) (12.083 \text{ ft}) (36.33 \text{ ft})^3 (1728) = 3.01 \text{ in} \\
= 3.01 - 1.82 = 1.19 \text{ in} \quad ; \quad c = 1\frac{1}{4}" \\
\phi V_n = 0.77 \text{ k} > V_u \quad \text{OK} \\
\text{CHECK S.W. ASSUMPTION} \rightarrow 60 \text{ plf} \times 12 \text{.083 ft} = 725 \text{ lb} = 0.73 \text{ k} < 1 \text{ k} \quad \text{OK} \\
\text{USE W18 x 60 w/ 22 STUDS AND } c = 1\frac{1}{4}"
\]

\text{DESIGN INTERIOR COLUMN}

\[P_u = 1.5 (46.0) + 2 (27.6) = 124 \text{ k at 14 ft UNBRACED LENGTH} \]

\text{USE } k = 2.0 \text{ (VERY CONSERVATIVE) } \text{K}L = 28 \text{ ft}

\text{FROM AISC TBL 4-1 ON PG 4-1B,}

\text{USE W12 x 53, W/ } \phi P_h = 192 \text{ k}

\text{RAM RESULTS DIFFER BECAUSE RAM DEFAULTS TO THE MOST ECONOMICAL SECTION, WHEREAS I WAS SEEKING TO MINIMIZE DEPTHS.}

\text{THE COLUMN DIFFERS BECAUSE RAM SIZED IT TO HAVE THE SAME COLUMN ALL THE WAY UP THE BUILDING, DESPITE BEING GIVEN SPICE LEVELS}

\text{CAMBER IN RAM WAS ALSO LIMITED UNIFORMLY TO 1", AS IT DOES NOT APPEAR POSSIBLE TO APPLY DIFFERENT CAMBER ALLOWANCES TO BEAMS VS. GIRDER}
Appendix D: Post-Tensioned Concrete Calculations

**Materials**

- **Concrete:** NWC (150pcf)
  - $f'_c = 5000$ psi
  - $f'_t = 3000$ psi
- **Rebar:** $f_y = 60$ KSI

**PT → Unbonded Tendons**

- $\frac{1}{2} \varnothing$ 7-wire strands
  - $A = 0.153$ in$^2$
  - $f_{pt} = 270$ KSI

**Estimated Prestress Loss:**

- $f_{ps} = 0.7(270 $ KSI) = 15$ KSI = 174 KSI
- $P = 174$ KSI (0.153 in$^2$) = 26.6 ksf/tendon

**Preliminary Slab/Beam Sizes**

**Per PTI’s Technical Note 3, Initial Slab Thickness Should Be Based on the Shorter Span for W-S Beams/Slabs**

- \[ h = \frac{12}{40} \times 6.3 \text{ in} \Rightarrow 7” \text{ Slab} \]

- Reasonable to limit deflection
  - $h = 14”$
  - $b = 42” = 3’-6”$
LOADS

\[ DL = \left( \frac{25}{2} \right) \frac{15}{2} + \left( \frac{15}{2} \right) \left( \frac{7}{2} \right) \right] \quad 150 \text{pcf} = 102 \text{psf} \]

\[ SDL = 15 \text{ psf} \]

\[ LL = 80 \text{ psf} \rightarrow \text{REDUCE AS ALLOWED} \]

- 21' x 36'-4" BAY \[ \text{K}_{A} = \frac{7}{2}(21')(36.33') = 763 > 400 \text{ psi} \]
  \[ LL = 0.25 + \frac{15}{17} = 0.79 \]

- 21' x 17'-0" BAY \[ \text{K}_{A} = \frac{7}{2}(21')(17') = 357 < 400 \text{ psi} \text{ NO REDUCTION} \]

- 21' x 7'-3" BAY \[ \text{NO REDUCTION} \]

DESIGN N-S DIRECTION, 36'-4" BAY (DISTRIBUTED PT)

USE EQUIVALENT FRAME METHOD

BAY WIDTH = 36'-4"

IGNORE COLUMN STIFFNESS FOR SIMPLICITY.

\[ \frac{LL}{DL} = \frac{0.79(80)}{102} = 0.62 < 0.75 \text{ NO PATTERN LOADING REQUIRED} \]

(ACI 318-05 § 13.7.6.2)

SECTION PROPERTIES

\[ A = (36.33' \times 12\frac{1}{2}')(7\frac{1}{2}') = 3,050 \text{ in}^2 \]

\[ S = \frac{bh^2}{6} = \frac{(36.33' \times 12\frac{1}{2}')(7\frac{1}{2})^3}{6} = 3,360 \text{ in}^3 \]

CLASS U DESIGN PERMITS USE OF GROSS SECTION

SET DESIGN PARAMETERS

ALLOWABLE STRESSES:

- SACKING \[ f_{ci} = 3000 \text{ psi} \]
  \[ \text{COMPRESSION} \rightarrow 0.6 f_{ci} = 1800 \text{ psi} \]
  \[ \text{TENSION} \rightarrow 3 \frac{f_{ci}}{6} = 164 \text{ psi} \]

- SERVICE \[ f_{ci} = 5000 \text{ psi} \]
  \[ \text{COMPRESSION} \rightarrow 0.45 f_{ci} = 2,250 \text{ psi} \]
  \[ \text{TENSION} \rightarrow 5f_{ci} = 424 \text{ psi} \]

PRECOMPRESSION LIMITS:

\[ \text{MIN} = 125 \text{ psi} \]
\[ \text{MAX} = 350 \text{ psi} \]

TARGET LOAD BALANCE: \[ 60-80\% DL \rightarrow USE 70\% \]

LOWER REQUIREMENTS:

\[ \text{TOP} = \frac{3}{4}" \]
\[ \text{BOTTOM} = \frac{1}{2}" \text{ UNRESTRAINED} \]
\[ \frac{3}{4}" \text{ RESTRAINED} \]
**TENDON PROFILE**

- **DESCRIPTION**
  - **HEIGHT**: 3.5''
  - **ANCHOR**: 3.5''
  - **END - BOTTOM**: 1.75''
  - **INT. - TOP**: 6.0''
  - **INT. - BOTTOM**: 1.0''
  - *= MEASURED FROM BOTTOM OF SLAB

**REQUIRED PRESTRESS**

\[
\begin{align*}
W_b &= 0.7 \times (102 \text{ psi}) \times (3.33 \text{ ft}) = 2.594 \text{ kif} = 2.594 \text{ kif} \\
P_{\text{II}} &= \frac{W_b \times l^2}{8 \times a_{\text{II}}} = 5.72 \text{ k} \\
\# \text{ OF TENDONS} &= \frac{572 \text{ k}}{26.0 \text{ k/TENDON}} = 21.5 \text{ TENDONS} \rightarrow \text{USE} \ 20 \text{ TENDONS} \\
P_{\text{ACT}} &= 20 \times 26/2 = 532 \text{ k}, \\
D &= \frac{532}{3000} \times 1000 = 174 \text{ psi} > 125 \text{ psi} \ \text{OK} < 300 \text{ psi} \ \text{OK} \\
\rho_{\text{INT}} &= \frac{W_b \times l^2}{8 \times a_{\text{INT}}} = 343 \text{ k} \ \text{(LESS PRESTRESS REQUIRED)} \\
W_{D,\text{INT}} &= \frac{532 (b) (5/12)}{21^2} = 4.02 \text{ kif} \\
W_b/W_b &= 108\% \ \text{NOT GOOD} \rightarrow \text{USE} \ 18 \text{ TENDONS} \ (P_{\text{ACT}} = 479 \text{ k}) \\
D &= \frac{479}{3000} \times 1000 = 157 \text{ psi} \ \text{OK} \\
W_{b,\text{END}} &= \frac{479 (b) (5/12)}{21^2} = 2.17 \text{ kif} \ (59\% \ W_b) \ \text{OK} \\
W_{b,\text{INT}} &= \frac{479 (b) (5/12)}{21^2} = 3.62 \text{ kif} \ (98\% \ W_b) \ \text{OK} \\
P_{\text{III}} &= 479 \text{ k}
\end{align*}
\]
**POST-TENSIONING | TECH 2 | pg 4 OF 12**

**CHECK SLAB STRESSES**

\[ W_0 = (0.102 \text{ ksf}) (36.33 \text{ ft}) = 4.25 \text{ klf} \]

\[ W_L = (0.70) (0.08 \text{ ksf}) (36.33 \text{ ft}) = 2.30 \text{ klf} \]

\[ W_0 = 3.21 \text{ klf (AVERAGE ALL 7 BAYS)} \]

**STRESSES AFTER JACKING**

Midspan Stresses:

\[ f_{top} = \frac{M_{pl} + M_{bal}}{S} \]

\[ f_{bot} = \frac{M_{pl} - M_{bal}}{S} \]

Interior Span (Span A-D)

\[ f_{top} = \frac{-192.5 + 42.3}{3500 \text{ in}^2} \]
\[ f_{bot} = \frac{-192.5 - 42.3}{3500 \text{ in}^2} \]

End Span (Span A-B)

\[ f_{top} = \frac{-145 + 110}{3500 \text{ in}^2} \]
\[ f_{bot} = \frac{145 - 110}{3500 \text{ in}^2} \]
### Technical Report

**Kathryn Gromowski | Structural Option**

**October 27th, 2010 | University Sciences Building | Northeast USA**

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**Post-Tensioning**

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</table>

**Support Stresses:**

\[
\begin{align*}
s_{top} &= \frac{M_{DL} - M_{EL}}{S} - \frac{P}{A} \\

s_{bot} &= \frac{-M_{DL} + M_{EL}}{S} - \frac{P}{A}
\end{align*}
\]

**Interior Span (at Support D):**

\[
\begin{align*}
s_{top} &= \frac{(158 - 120) \times 12000}{3560 \text{ in}^3} = 157 \text{ psi} < 0.6 f_c'
\end{align*}
\]

\[
\begin{align*}
s_{bot} &= \frac{-(158 + 120) \times 12000}{3560 \text{ in}^3} = 157 \text{ psi} < 0.6 f_c'
\end{align*}
\]

**End Span (at Support B):**

\[
\begin{align*}
s_{top} &= \frac{(148 - 150) \times 12000}{3560 \text{ in}^3} = 157 \text{ psi} < 3(\frac{f_c}{3})
\end{align*}
\]

\[
\begin{align*}
s_{bot} &= \frac{-(148 + 150) \times 12000}{3560 \text{ in}^3} = 157 \text{ psi} < 0.6 f_c'
\end{align*}
\]

**Stress in Service:**

**Midspan Stresses:**

\[
\begin{align*}
s_{top} &= \frac{-M_{DL} - M_{LL} + M_{EL}}{S} - \frac{P}{A} \\

s_{bot} &= \frac{M_{DL} + M_{LL} - M_{EL}}{S} - \frac{P}{A}
\end{align*}
\]

**Interior Span (span C-D):**

\[
\begin{align*}
s_{top} &= \frac{(-82.5 + 44.7 + 62.3) \times 12000}{3560 \text{ in}^3} = 157 \text{ psi} < 0.45 f_c'
\end{align*}
\]

\[
\begin{align*}
s_{bot} &= \frac{82.5 + 44.7 - 62.3} \times 12000}{3560 \text{ in}^3} = 157 \text{ psi} < 0.45 f_c'
\end{align*}
\]

**End Span (span A-B):**

\[
\begin{align*}
s_{top} &= \frac{(-145 - 78.9 + 110) \times 12000}{3560 \text{ in}^3} = 157 \text{ psi} < 0.45 f_c'
\end{align*}
\]

\[
\begin{align*}
s_{bot} &= \frac{145 + 78.9 - 110} \times 12000}{3560 \text{ in}^3} = 157 \text{ psi} < 0.45 f_c'
\end{align*}
\]

**Support Stresses:**

\[
\begin{align*}
s_{top} &= \frac{M_{DL} + M_{LL} - M_{EL}}{S} - \frac{P}{A} \\

s_{bot} &= \frac{-M_{DL} + M_{LL} + M_{EL}}{S} - \frac{P}{A}
\end{align*}
\]

**Interior Span (Support @ D):**

\[
\begin{align*}
s_{top} &= \frac{(158 + 85.7 - 120) \times 12000}{3560 \text{ in}^3} = 157 \text{ psi} < 0.45 f_c'
\end{align*}
\]

\[
\begin{align*}
s_{bot} &= \frac{-(158 - 85.7 + 120) \times 12000}{3560 \text{ in}^3} = 157 \text{ psi} < 0.45 f_c'
\end{align*}
\]

**End Span (Support @ B):**

\[
\begin{align*}
s_{top} &= \frac{(148 + 107 - 150) \times 12000}{3560 \text{ in}^3} = 157 \text{ psi} < 0.45 f_c'
\end{align*}
\]

\[
\begin{align*}
s_{bot} &= \frac{-(148 - 107 + 150) \times 12000}{3560 \text{ in}^3} = 157 \text{ psi} < 0.45 f_c'
\end{align*}
\]

**All Stresses Acceptable**
**POST-TENSIONING**

**ULTIMATE STRENGTH**

\[ M_i = P_e \]

\[ \Rightarrow e = 0 \text{ @ EXT. SUPPORT} \]

\[ \Rightarrow e = 2.5 \text{ in} \text{ @ INT. SUPPORT} \]

\[ M_1 = 479 \text{ k}(\frac{23}{12}) = 99.8 \text{ k-ft} \]

\[ M_{sec} = M_{real} - M_1 \text{ (VARIES LINEARLY BETWEEN SUPPORTS)} \]

\[ M_0 = 1.2 M_{dl} + 1.6 M_{ll} + 1.0 M_{sec} \]

**BONDED REINFORCEMENT**

**POSITIVE MOMENT REGION:**

- **INTERIOR SPAN (SPAN C-D):**
  \[ f_{tens} = 51.8 \text{ psi} \]
  \[ 2.25 \text{ psi} = 141 \text{ psi} \]
  \[ \text{NO REINFORCEMENT REQD} \text{ (ACI 318-08 § 18.9.3.1)} \]

- **END SPAN (SPAN A-B):**
  \[ f_{tens} = 227 \text{ psi} \]
  \[ 2.25 \text{ psi} \]
  \[ \text{REIN. REQ'D} \]

\[ M_0 = 1.2(145) + 1.6(78.9) + 1.0(50.2)(50.2 - 0) = 325 \text{ k-ft} \]

\[ \text{MIN REINFORCEMENT (ACI 318-08 § 18.9.3.2)} \]
\[ y = \frac{f_{tens} \cdot h}{f_{tens} + f_{tens} \cdot 541} = 227 \text{ psi} \cdot (7 \text{ in}) = 2.07 \text{ in} \]

\[ N_c = \left( \frac{M_0 + M_1}{S} \right) 0.5 \cdot t = \left( \frac{145 + 78.9}{35(60 \text{ in}^3)} \right) 0.5 (2.07 \text{ in})(31.33 \text{ ksi})(141 \text{ in}^3) \]

\[ = 341 \text{ k} \]

\[ A_{s, min} = \frac{N_c}{f_{y}} = \frac{341 \text{ k}}{0.5(60 \text{ ksi})} = 11.4 \text{ in}^2 = 36.33 \text{ in} \cdot 0.314 \text{ in}^2/\text{ft} \]

\[ \text{TRQ } #6 \text{ @ 12'' O.C. } = 0.44 \text{ in} \cdot \text{ft} \Rightarrow \text{VERIFY MIN. AREA IS OK} \]

\[ A_{ps} = 0.153 \text{ in}^2/\text{en} \text{ (8 TENDONS)} = 2.75 \text{ in}^2 \]

\[ f_{ps} = f_{se} + 10000 + \frac{f_{ps} \cdot d}{300} = 174000 + 10000 + \frac{5000(30 \text{ ksi})(2)}{300} (2.75 \text{ in}^2) \]

\[ = (184 + 2.642) \text{ ksi} \]

\[ d = 7'' - 1.5'' - \lambda = 5.25'' \]

\[ f_{ps} = 200.5 \text{ ksi} \]

\[ q = \frac{A_{sh} + A_{f} f_{pl} \cdot 15.64 \text{ in}^2 (60 \text{ ksi}) + 275 \text{ in}^2 (200 \text{ ksi})}{0.85 (5 \text{ ksi})(36.33 \text{ ft}^2)(12^2)} = 0.81 \text{ in} \]
NEGATIVE MOMENT REGION:
INTERIOR SPAN (SUPPORT @ D)

\[ M_u = 1.2 (158) + 1.6 (78.6) + 1.0 (20.2) = -295 \text{ k-ft} \]

MIN. REINFORCEMENT (ACI 318-05 § 18.9.3.3)

\[ A_{re} = \max \left( \frac{2f_u}{360f_{cd}} \right) \geq \frac{36.33 \text{ft} \cdot \text{in}}{(7 \text{in})(12^2/4)} = 3050 \text{ in}^2 \]

\[ A_{min} = 0.00075 A_{re} = 0.00075 (3050) = 2.29 \text{ in}^2 \]

TRY (8) # 5 \( \omega/Ar = 2.48 \text{ in}^2 \)

\[ d = 7'' - 3/4'' - 3/8'' = 6'' \]

\[ f_{pu} = 184 + 2.642 (6) = 199.9 \text{ ksi} \]

\[ q = \frac{2.48 \text{ in}^2 (60 \text{ksi}) + 2.75 \text{ in}^2 (199.9 \text{ ksi})}{0.85 (5 \text{ ksi}) (36.33 \text{ ft} \times 12^2/4)} \]

\[ Q_m = \frac{Q (f_{pu} + f_{cd}) (d - \frac{d}{2})}{304 \text{ k-ft} > M_u \text{ OK}} \]

USE (8) # 5

END SPAN, INTERIOR SUPPORT (SUPPORT @ B)

\[ M_u = 1.2 (-198) + 1.6 (40.7) + 1.0 (50.2) = -339 \text{ k} \]

\[ A_{min} = 2.29 \text{ in}^2 \text{ (WON'T BE ENOUGH)} \]

TRY (8) # 4 \( \omega/Ar = 3.52 \text{ in}^2 \)

\[ q = \frac{3.52 \text{ in}^2 (60 \text{ ksi}) + 2.75 (199.9 \text{ ksi})}{0.85 (5 \text{ ksi}) (36.33 \text{ ft} \times 12^2/4)} = 0.41 \text{ in} \]

\[ Q_m = 0.9 (3.52 (60) + 2.75 (199.9)) (6 - \frac{d}{2}) = 331 \text{ k-ft} > M_u \text{ NOT OK} \]

TRY (12) # 6 \( \omega/Ar = 5.28 \text{ in}^2 \)

\[ q = \frac{5.28 \text{ in}^2 (60 \text{ ksi}) + 2.75 (199.9 \text{ ksi})}{0.85 (5 \text{ ksi}) (36.33 \text{ ft} \times 12^2/4)} = 0.47 \text{ in} \]
POST-TENSIONING

TECH 2

Pg 8 of 12

\[ \phi M_a = 0.9 \left( 5.28(60) + 2.75(148.9) \right) \left( \frac{2.5}{6} \right) = 375 \text{ k-ft > M}_u \]

**USE (12) #6**

**END SPAN, EXTERIOR SUPPORT (SUPPORT @ A)**

**MIN REINFORCING WILL BE USED**

**USE (8) #5**

**SUMMARY**

\[ \begin{align*}
(8) &\text{ HS} \\
(12) &\text{ HS} \\
(3) &\text{ U.N.O.} \\
(12) &\text{ HS} \\
(8) &\text{ HS}
\end{align*} \]

18 POST-TENSIONING TENDONS (SEE PG 3 FOR PROFILE)

**DESIGN E-W DIRECTION (BANDED PT)**

USE EQUIVALENT FRAME METHOD

IGNORE COLUMN STIFFNESS FOR SIMPLICITY

\[ \frac{H}{L} = 0.79 \left( \frac{60}{102} \right) = 0.62 < 0.75 \quad \therefore \text{NO PATTERN LOADING REQUIRED} \]

**SECTION PROPERTIES**

\[ A = 7 \times (2'' \times 12'' \text{wyf}) + 7 \times (3.5'' \times 12'' \text{wyf}) = 2060 \text{ in}^2 \]

\[ \frac{H}{L} = \frac{7 \times (2'' \times 12'' \text{wyf}) + 7 \times (3.5'' \times 12'' \text{wyf})}{7 \times (2'' \times 12'' \text{wyf}) + 7 \times (3.5'' \times 12'' \text{wyf})} = 9.5 \text{ in} \]

\[ I = \frac{2(12)'(7) + (3' \times 12')(9.5 - 10.5)^2 + (3.5' \times 12')(7)^2 + (3.5' \times 12')(9.5 - 3.5)^2}{12} = 20,752 \text{ in}^4 \]

\[ S_{bop} = \frac{I}{4} \quad 4.5 \text{ in} \]

\[ S_{bop} = \frac{I}{4} \quad 4.5 \text{ in} \]

\[ S_{bop} = \frac{I}{4} \quad 4.5 \text{ in} \]

SET DESIGN PARAMETERS → SEE N-S DIRECTION

TENDON PROFILE → AS RECOMMENDED IN MAY 2003 ARTICLE IN CONCRETE INTERNATIONAL BY AALAMI.

\[ a_{min} = \frac{13 - 9.5}{3} \left( 18'' \right) + 9.5'' = 9.5'' \]

\[ a_{int} = 13'' - 10'' = 12'' \]

---

**Technical Report 2**

Kathryn Gromowski | Structural Option

October 27th, 2010 | University Sciences Building | Northeast USA
REQUIRED PRESTRESS

\[
W_D = 0.7 \times (102 \text{ psf}) \times (21 \text{ ft}) = 1500 \text{ psf} = 115 \text{ kif}
\]

\[
P_{\text{end}} = \frac{W_D \times L^2}{8 \times a_{\text{end}}} = \frac{1.5 \text{ kif} \times (36 \times 33)^2}{8 \times (\frac{a_{\text{end}}}{2})} = 313 \text{ k}
\]

\[
# \text{ OF TENDONS} \rightarrow \frac{313 \text{ k}}{26.6 \text{ k/Tendon}} = 11.8 \text{ TENDONS} \rightarrow \text{USE 15 TENDONS}
\]

\[
P_{\text{act}} = 15 \times 26.6 = 399 \text{ k}
\]

\[
\frac{K}{K_{\text{ref}}} = \frac{29.9}{29.5} \times 1000 = 194 \text{ psi} > 125 \text{ OK} \quad < 300 \text{ OK}
\]

\[
P_{\text{int}} = \frac{W_D \times L^2}{8 \times a_{\text{int}}} = \frac{1.5 \times (17)^2}{8 \times (4.5)} = 54.2 \text{ k} \quad \text{(LESS PRESTRESS REQUIRED)}
\]

\[
W_{D,\text{int}} = \frac{399 \times (8)}{(17)^2} = 11.0 \text{ kif}
\]

\[
\frac{W_D}{W_{D,\text{int}}} = 516\% \quad \text{NOT GOOD} \rightarrow \text{REDUCE DRAPE (DUE TO LARGE DISCREPANCY)}
\]

\[
\frac{a_{\text{int, ref}}}{a_{\text{int}}} = \frac{173 \times (0.152) \times (21)}{399 \times (8)} \times 12 = 4.07 \text{ in} \rightarrow \text{USE 4.0 in}
\]

\[
W_0,\text{END} = \frac{399 \times (8)}{36.33^2} = 1.91 \text{ kif} \quad \text{(90% W_D)} \quad \text{OK}
\]

\[
W_{D,\text{int}} = \frac{399 \times (8)}{17^2} = 3.68 \text{ kif} \quad \text{(175% W_D)} \quad \text{OK}
\]

\[
P_{\text{eff}} = 399 \text{ k}
\]

CHECK SLAB STRESSES

\[
W_D = (0.102 \text{ ksf}) + 0.015 \text{ ksf} \times (21 \text{ ft}) = 2.46 \text{ kif}
\]

\[
W_L = 0.79 \times (0.08 \text{ ksf}) \times (21 \text{ ft}) = 1.33 \text{ kif}
\]
POST-TENSIONING

TENSION @ BOTTOM STRESSES:

MIDSPAN ON SPAN 1-2

\[ f_{\text{top}} = \frac{-2(265 + 142 + 303)12000}{4612 \text{ in}^3} = -194 \text{ psi} < 0.45f'c \text{ OK} \]

\[ f_{\text{bot}} = \frac{(265 + 142 + 303)12000}{2184 \text{ in}^3} = 419 \text{ psi} < 0.45f'c \text{ OK} \]

SUPPORT @ 3

\[ f_{\text{top}} = \frac{-92.4 + 50.3)12000}{4612 \text{ in}^3} = -173 \text{ psi} < 0.6f'c \text{ OK} \]

\[ f_{\text{bot}} = \frac{(92.4 + 50.3)12000}{2184 \text{ in}^3} = 237 \text{ psi} < 0.6f'c \text{ OK} \]

TENSION @ TOP STRESSES:

SUPPORT @ 2

\[ f_{\text{top}} = \frac{(312 - 355)12000}{4612 \text{ in}^3} = -306 \text{ psi} < 0.6f'c \text{ OK} \]

\[ f_{\text{bot}} = \frac{(312 + 355)12000}{2184 \text{ in}^3} = 42.3 \text{ psi} < 3f'c \text{ OK} \]

STRESS IN SERVICE

TENSION @ BOTTOM STRESSES:

MIDSPAN ON SPAN 1-2

\[ f_{\text{top}} = \frac{-2(265 - 142 + 303)12000}{4612 \text{ in}^3} = -194 = -465 \text{ psi} < 0.45f'c \text{ OK} \]

\[ f_{\text{bot}} = \frac{(265 + 142 - 303)12000}{2184 \text{ in}^3} = 377 \text{ psi} < 6f'c \text{ OK} \]

SUPPORT @ 3

\[ f_{\text{top}} = \frac{-92.4 - 19.2 + 50.3)12000}{4612 \text{ in}^3} = -223 \text{ psi} < 0.45f'c \text{ OK} \]

\[ f_{\text{bot}} = \frac{(92.4 + 19.2 - 50.3)12000}{2184 \text{ in}^3} = 132 \text{ psi} < 0.45f'c \text{ OK} \]

TENSION @ TOP STRESSES:

SUPPORT @ 2

\[ f_{\text{top}} = \frac{(312 + 171 - 355)12000}{4612 \text{ in}^3} = -139 \text{ psi} < 6f'c \text{ OK} \]

\[ f_{\text{bot}} = \frac{(-312 - 171 + 355)12000}{2184 \text{ in}^3} = -897 \text{ psi} < 0.45f'c \text{ OK} \]

ALL STRESSES ACCEPTABLE
ULTIMATE STRENGTH

\[ M_i = \frac{P_e}{e} \]
\[ e = 0 \text{ in} \ @ \text{Ext Support} \]
\[ e = 13' - 9.5'' = 3.5 \text{ in} \ @ \text{Int Support} \]

\[ M_i = 399 \text{ k}(\frac{3\pi}{12}) = 116.4 \text{ k-ft} \]

\[ M_{sec} = M_{bal} - M_i \quad (\text{VARIES LINEARLY BETWEEN SUPPORTS}) \]

\[ M_U = 1.2 M_{DL} + 1.6 M_{LL} + 1.0 M_{sec} \]

BONDED REINFORCEMENT

POSITIVE MOMENT REGION:

\[ \text{MINSPAN ON SPAN 1-2} = 377 \text{ psi} \]

\[ f_{tens} = 6.27 \text{ in} \quad f_{tens} \quad f_{tens} \]

\[ M_U = 1.2(325) + 1.6(142) + 1.0(0.5)(239-0) = 708 \text{ k-ft} \]

\[ \gamma = \frac{f_{tens}}{f_{tens} + f_{tens}} = \frac{377}{377 + 455} \]

\[ N = \frac{(216 + 142)}{(48 + 218)} \]

\[ A_{min} = 568 \text{ k} \quad 18.9 \text{ in}^2 \rightarrow \text{MUST GO IN BEAM} \]

USE (13) # 11 USE A = 20.28

\[ S = (13)(1.14) - 2(0.75) = 1.85 \text{ in} \]

\[ A_{20} = 15 \quad 0.153 \text{ in}^2 \text{TENSION} = 2.30 \text{ in}^2 \]

\[ f_{tens} = 184 + 2.442 \text{ ksi} \]

\[ d = 14'' - 1/8'' - 0.5(1.41\text{ in}) = 11.8 \text{ in} \quad \text{f} \text{pl} = 215.2 \text{ ksi} \]

\[ q = 20.28 \text{ in}^2(60 \text{ ksi}) + 2.30 \text{ in}^2(215.2 \text{ ksi}) = 1.60 \text{ in} \]

\[ D = 0.9(20.28(60) + 2.30(215.2)) \]

\[ 11.8 - \frac{1.42}{2} = 1412 \text{ k-ft} > M_U \text{ OK} \]

USE (13) # 11
POST-TENSIONING

Support @ 3

No tension in service loads. Provide min. reinforcement.

Use (13) #11

Negative Moment Region:

Support @ 2

R PE limit does not apply at negative moment.

\[ M_0 = 1.2(-312) + 1.6(-171) + 1.0(239) = -409 \text{ k-ft} \]

Minimum reinforcement

\[ A_{RF} = \max \left\{ \frac{22.33 + 12}{21} \text{ ft} \right\} \geq \frac{20.7 \text{ ft}}{21 \text{ ft}} = \frac{20.7 \text{ ft}}{17.8 \text{ in}}(12 \text{ in/ft}) = 4486 \text{ in}^2 \]

\[ A_{S,\text{min}} = 0.00075(4486) = 3.36 \text{ in}^2 \]

Try (8) #6 with \( A_S = 3.52 \text{ in}^2 \)

\[ d = \frac{14'' - 3/4'' - 1/4''}{13''} = 13 \text{ in} \]

\[ f_{y} = 184 + 2.642(13) = 218.3 \text{ ksi} \]

\[ q = \frac{3.52 \text{ in}^2 (218.3 \text{ ksi}) + 2.30 \text{ in}^3 (218.3 \text{ ksi})}{0.85 (218.3 \text{ ksi}) (21.5 \times 12.75''/4)} = 0.67 \text{ in}^3 \]

\[ f_{y} = 0.9 (3.52(60) + 2.30(218.3)) (13 - \frac{0.67}{2}) = 678 \text{ k-ft} > M_0 \text{ OK} \]

Use (8) #6

Exterior Supports

No Moment. Use minimum reinforcing.

\[ A_{RF} = \max \left\{ \frac{22.33}{21 \text{ ft}} \right\} \geq \frac{21 \text{ ft}}{21 \text{ ft}} = \frac{21 \text{ ft}}{(12 \text{ in/ft})} = 3528 \text{ in}^2 \]

\[ A_{S,\text{min}} = 0.00075(3528 \text{ in}^3) = 2.64 \text{ in}^3 \]

Use (6) #6 with \( A_S = 2.16 \text{ in}^2 \)

SUMMARY

- (6) #6
- (13) #11
- (9) #6
- (13) #6

Bonded reinforcing

Tendon profile uses 15 banded tendons.
Appendix E: Precast Hollow core/Steel Girder Calculations

LOADS: DL = S.W. (ASSUME 10 psf FOR FRAMING)
SD = 15 psf
LL = 80 psf (REDUCE AS ALLOWED)

CHOOSE A PLANK → USE NUTTER HOUSE
→ MINIMIZE FLOOR DEPTH/WEIGHT
→ MAINTAIN 2 HR FIRE RATING

TRY 6” HOLLOW CORE W/ 2” TOPPING
CHOOSE STRAND PATTERN:
SUPERIMPOSED LOAD = 15 psf + 80 psf = 95 psf

USE (4) 1/2” X PATTERN
CHECK STRESSES

\[ M_u = \frac{(15 + 48.75 + 25 + 80)(4\text{ ft})(2.5\text{ ft})}{8} = 37.2 \text{ k-ft} \]
\[ P_{eff} = 0.4(270 \text{ kfs})(0.153 \text{ in}^2) = 24.8 \text{ k/lb/strand} \]

\[ f_T = \frac{446.5 \text{ k-in}}{253} = 1.79 \text{ ksi} \]
\[ f_b = \frac{446.5 \text{ k-in}}{255} = 1.7\text{ ksi} \]

\[ f = \frac{446.5 \text{ k-in}}{255} - 24.8(4) = 24.8(4) \]

\[ \Delta u = \frac{5(0.08 \text{ ksi})(4.5)(21.4)^2 (1728)}{284 (29000 \text{ kfs})(1519 \text{ in}^2)} = 0.051 \text{ in} \leq \Delta_{ul, \text{max}} = 0.71 \text{ in} \]

\[ \Delta_{TL} = \frac{5(1.2(0.015 + 0.04875 + 0.025) + 1.6(0.08))(4\text{ ft})(21\text{ ft})^2 (1728)}{3.84 (24000 \text{ kfs})(1519 \text{ in}^2)} = 0.093 \text{ in} \leq \Delta_{TL, \text{max}} = 1.05 \text{ in} \]

\[ M_u = \frac{1.2(0.015 + 0.04875 + 0.025) + 1.6(0.08)}{2}(21\text{ ft})^2 = 51.7 \text{ k-ft} \leq M_0 = 67.9 \text{ k-ft} \]

6” HOLLOW CORE W/ 2” TOPPING & (4) 1/2” STRANDS IS OK
CHOOSE BEAM

\[ W_0 = (15 + 48.75 + 25 + 10) (21 + 1) = 2074 \text{ plf} = 2.07 \text{ klf} \]

\[ L_{Lr} = 0.25 + \frac{15}{(20)(86.33)} = 0.634 > 0.5 \text{ OK} \]

\[ W_L = 0.634 (80) (21 + 1) = 1065 \text{ plf} = 1.07 \text{ klf} \]

\[ W_D = 1.2 (2.07) + 1.6 (1.07) = 4.20 \text{ klf} \]

\[ I_{req, LL} = \frac{5 \left[ 1.6 (1.07) \right] (36.33 \text{ ft})^4 (1728)}{384 (20000 \text{ ksi}) (36.33 \text{ in})} = 1911 \text{ in}^4 \]

\[ I_{req, TL} = \frac{5 \left[ 4.20 \right] (36.33 \text{ ft})^4 (1728)}{384 (20000 \text{ ksi}) (36.33 \text{ in})} = 3119 \text{ in}^4 \]

FROM AISC TABLE 3-3, OPTIONS ARE:

\[ \begin{aligned}
& \text{1) } W30 \times 90 \quad W/ I = 3610 \text{ in}^4 \\
& \text{2) } W27 \times 94 \quad W/ I = 3270 \text{ in}^4 \\
& \text{3) } W24 \times 117 \quad W/ I = 3540 \text{ in}^4 \\
& \text{4) } W21 \times 132 \quad W/ I = 3320 \text{ in}^4 \\
& \text{5) } W18 \times 175 \quad W/ I = 3450 \text{ in}^4 \\
& \text{6) } W14 \times 257 \quad W/ I = 3400 \text{ in}^4 \\
\end{aligned} \]

\[ \begin{aligned}
\text{To reduce floor depths, try } W18 \times 175 & \quad W/ d = 20.0 \text{ in} \\
\Phi M_M & = 1490 \text{ k-ft} \\
\Phi V_M & = 535 \text{ k} \\
\end{aligned} \]

\[ M_v = \frac{4.20 \text{ klf} (36.33 \text{ ft})^2}{6} = 693 \text{ k-ft} < \Phi M_M \text{ OK}\]

\[ V_v = 4.20 \text{ klf} (36.33 \text{ ft})(0.5) = 76.3 \text{ k} < \Phi V_M \text{ OK}\]

CHECK S.W. ASSUMPTION \[ \frac{1728}{21} = 8.33 \text{ psf} < 10 \text{ psf} \text{ OK}\]

USE \( W18 \times 175 \)

NOTE: IT MIGHT BE POSSIBLE TO REDUCE THIS FURTHER IF COMPOSITE ACTION WAS ACHIEVED BETWEEN THE BEAM AND THE PRECAST/TOPPING

CHOOSE COLUMN

\[ P_c = 1.5 (76.3 \text{ k}) = 115 \text{ k} \]

Use \( k = 2.0 \) (very conservative) \[ KL = 26 \text{ ft} \]

USE \( W12 \times 52 \)

\[ W/ \Phi P_c = 192 \text{ k} \]
Prestressed Concrete
6"x4'-0" Hollow Core Plank
2 Hour Fire Resistance Rating With 2" Topping

**PHYSICAL PROPERTIES**

<table>
<thead>
<tr>
<th>Component</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_c = 253 \text{ in}^2$</td>
<td>Precast $h_c = 16.13 \text{ in}$</td>
</tr>
<tr>
<td>$l_c = 1519 \text{ in}^2$</td>
<td>Precast $s_{c,k} = 370 \text{ in}^3$</td>
</tr>
<tr>
<td>$W_c = 4.10 \text{ in}$</td>
<td>Topping $s_{t,k} = 551 \text{ in}^3$</td>
</tr>
<tr>
<td>$W_t = 1.90 \text{ in}$</td>
<td>Precast $s_{t,k} = 799 \text{ in}^3$</td>
</tr>
<tr>
<td>$W_{tt} = 3.90 \text{ in}$</td>
<td>Precast Wt. = 195 PLF</td>
</tr>
<tr>
<td></td>
<td>Precast Wt. = 48.75 PSF</td>
</tr>
</tbody>
</table>

**DESIGN DATA**

1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø 270K Lo-Relaxation
5. Strand Height = 1.75 in
6. Ultimate moment capacity (when fully developed).
   - 4-1/2"Ø, 270K = 67.4 k-ft at 60% jacking force
   - 6-1/2"Ø, 270K = 92.6 k-ft at 60% jacking force
   - 7-1/2"Ø, 270K = 85.3 k-ft at 60% jacking force
7. Maximum bottom tensile stress is $10\sqrt{c} = 775$ PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is 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>SPAN (FEET)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>12</td>
</tr>
<tr>
<td>4-1/2&quot;Ø LOAD (PSF)</td>
<td>349</td>
</tr>
<tr>
<td>6-1/2&quot;Ø LOAD (PSF)</td>
<td>324</td>
</tr>
<tr>
<td>7-1/2&quot;Ø LOAD (PSF)</td>
<td>341</td>
</tr>
</tbody>
</table>

**NITTERHOUSE CONCRETE PRODUCTS**

2655 Molly Pitcher Hwy, South, Box N
Chambersburg, PA 17202-8023
717-267-4505 Fax 717-267-4518

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stair openings and narrow widths. The allowable loads shown in this table reflect a 2 Hour & 8 Minute fire resistance rating.