Cambria Suites Hotel
Pittsburgh, PA

Technical Report 2
Structural Study of Alternate Floor Systems

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# TABLE OF CONTENTS

Table of Contents .................................................................................................................. 2

Executive Summary ................................................................................................................. 3

Introduction: Cambria Suites Hotel ..................................................................................... 4

Structural System Summary ................................................................................................... 5
  Foundation System ................................................................................................................. 5
  Superstructure System ........................................................................................................... 7
  Lateral System ...................................................................................................................... 9

Code & Design Requirements ................................................................................................. 10

Materials ................................................................................................................................ 11

Design Load Summary ........................................................................................................... 12

Typical Span .......................................................................................................................... 13

Floor Systems ......................................................................................................................... 14
  Precast Concrete Plank Floor – Existing ........................................................................... 14
  Precast Concrete Plank on Steel ......................................................................................... 16
  Composite Steel Deck System ............................................................................................. 18
  One-Way Joist System ......................................................................................................... 20

Overall Floor System Comparison .......................................................................................... 22

Conclusion ............................................................................................................................... 23

Appendix A: Building Layout .................................................................................................. 24

Appendix B: Existing Precast Concrete Floor System Calculations ...................................... 27

Appendix C: Precast Concrete Plank on Steel Calculations .................................................. 30

Appendix D: Composite Steel Deck Calculations .................................................................... 33

Appendix E: One-Way Joist Calculations .............................................................................. 39

Appendix F: Cost Analysis Calculations .................................................................................. 49
Executive Summary

The following technical report compares the existing floor system of Cambria Suites Hotel with three proposed alternate floor systems. The existing system, as well as the three alternate floors systems were designed, analyzed, and then compared to determine which system(s) were practical for the building and were possible to be further studied in the future. The current floor system of Cambria Suites Hotel is precast hollow-core concrete plank on load bearing masonry walls and interior steel framing which is adequately designed to withstand the building load criteria. In order to properly compare each floor system, a typical floor section of the building was taken into consideration. The following alternate floor systems were examined:

- Precast Hollow-Core Concrete Plank on Steel Framing
- Composite Steel Deck System
- One-Way Joist System

The existing 10” hollow-core concrete plank system bears on exterior masonry walls, as well as an interior steel frame. The design of the precast planks is assumed to be designed by the PCI Design Handbook. The system self-weight is fairly heavy, compared to the other alternative floor systems, but takes advantage of using larger spans with minimal steel columns located through the middle of the building. The precast hollow-core plank on steel framing was designed using the PCI Design Handbook to determine a 10” concrete slab without topping. To span the 38’-0”, a 10” slab had to be used again to achieve the loading capacity. However, the use of fewer tendons and no topping decreased the system self-weight from the existing floor assembly. W14x82 steel girders which support the plank were designed by the AISC Steel Manual. The composite steel deck system was designed using the Vulcraft Deck Catalog and the AISC Steel Manual. A 2VLI20 deck was designed with a slab depth of 4.5” and topping of 2.5”. The supporting beams are W10x12 (6) and the girders are W21x44 (12). The final alternate system was a one-way joist system. It consists of 6” wide joists spaced at 66” on center with an 18” pan depth. The slab designed is 4.5” which has a 2-hour fire rating.

The advantages and disadvantages are discussed for each floor system, and ultimately the existing precast concrete plank is the best choice for this type of construction. However, through comparison of the designed alternative floor systems it was determined that the one-way joist system may be the most feasible system under further investigation. The only disadvantage of this system would be its increased floor system depth, but this is not a concern for the building location. This is because Cambria Suites Hotel rises 102’-2” above grade and is allowed to reach a maximum of 160’ in Pittsburgh.
Introduction: Cambria Suites Hotel

Cambria Suites Hotel is the newest, upscale, contemporary all-suite hotel located at 1320 Center Avenue in Pittsburgh, Pennsylvania. This luxury hotel is built adjacent to the new CONSOL Energy Center, home to the Pittsburgh Penguins hockey team, and numerous concerts and special events. The 142-suite hotel contains a total of 7 levels above grade and was built on a quite challenging site. The hotel will have a variety of room suites, such as the double/queen suite, king suite, one bedroom suite, deluxe tower king suite, and hospitality suite.

The Plaza Floor level will mainly consist of a few bistro-style restaurants which open to an outdoor terrace which will overlook the city of Pittsburgh and the CONSOL Energy Center. At the Hotel Floor level, guests will be greeted by an airy two-story lobby where they can take part in a state-of-the-art fitness center or the relaxing indoor pool and spa. There are also two meeting rooms and a board room for guest use, as well as, a large kitchen/bar off of the lobby entrance. At the North end of the Second Floor level, a steel Porte Cochere will be cantilevered to cover part of the main entrance. In addition, the property will feature an 1800 square foot presidential suite with one of a kind skyline view of downtown Pittsburgh and a 7th floor concierge lounge that will offer a wet bar and lounge space for guests to use and enjoy.

The hotel is fully landscaped and will also have an exclusive 143 space onsite parking garage with access to the CONSOL Energy Center for event patrons staying at the property. The Hotel Floor level will have a precast concrete pedestrian bridge leading to the top level of the parking garage. The bridge is supported by the hotel and the garage. The South end of the bridge will be supported by the garage on slide-bearings to allow for differential lateral movement between the two structures. The exterior of Cambria Suites Hotel is mainly brick and cast-stone veneer, architectural decisions made to resemble the bordering CONSOL Energy Center. A lighter color brick is used from the 2nd Roof Floor levels, with the addition of a cast-stone band at Floor levels 2 and 7. The darker color brick is used from the 2nd Floor level and below, as well as vertical strips to separate window pairings and to accent building corners.

The following report will take a closer look into the existing floor system of the Cambria Suites Hotel. Alternate floor systems were also designed and analyzed to fit the existing building conditions, followed by a comparison of each floor system to determine which floor system is best suited for the building’s structural system.
Structural System

Foundation

The geotechnical engineering study for the Cambria Suites Hotel was completed by GeoMechanics Inc. on December 29, 2008. In the study, the site of Cambria Suites Hotel is underlain by sedimentary rocks of the Conemaugh group of rocks of Pennsylvania age. The Conemaugh group is predominantly comprised of clay stones and sandstones interbedded with thin limestone units and thin coal beds. The soil zone conditions consisted of materials of three distinct geologic origins: man-made fill, alluvial deposits, and residual soils. The fill in the hotel test borings was placed in conjunction with the recent demolition and regarding of St. Francis Hospital in order to build Cambria Suites and CONSOL Energy Center. Ground water exists locally as a series of perched water tables located throughout the sol zone and new the upper bedrock surface. Excavations in soils and bedrock can be expected to encounter perched water. The volume of inflow into excavations should be relatively minor, should diminish with time and should be able to be removed by standard pump collection/pumping techniques. The report also states that the most economical deep foundation solution for Cambria Suites included a system of drilled-in, cast-in-place concrete caissons with grade beams spanning between adjacent caissons to support the anticipated column and wall loads of the structure. With varying types of bedrock on site, the allowable end bearing pressure ranges from 8, 16, and 30 KSF. As for the floor slab, GeoMechanics Inc. recommended to place a ground floor slab on a minimum six-inch thick granular base and to provide expansion joints between the ground floor slab and any foundation walls and/or columns. This is done to permit independent movement of the two support systems.

As a result of GeoMechanic’s geotechnical study, the foundation of Cambria Suites Hotel incorporates a drilled cast-in-place concrete caissons and grade beams designed to support the load bearing walls and columns. The ground floor is comprised of a 4” concrete slab on grade, as well as, 10” precast concrete plank in the Southern portion of the building. The 4” concrete slab on grade is reinforced with 6x6-W1.4 welded wire fabric and has 4000 PSI normal weight concrete. The slab increases to 8” in thickness with #5 @ 16” O.C. in the South-West corner of the building, and increases to 24” with #5 @ 12” O.C. within the core shear walls where the elevator shaft is located. For the majority of the slab on grade, the slab depth is 14′-0” below finish grade.
The drilled cast-in-place caissons extend anywhere from 20-30 feet deep below grade and are socketed at least 3' into sound bedrock. Caisson end bearing capacity is 30 KSF (15 ton/SF) on Birmingham Sandstone bedrock. The caissons are designed with a compressive strength of 4000 PSI, range from 30-42 inches in diameter, and are spaced approximately between 15' and 30' apart (refer to Appendix A). Typical caissons terminate at the Plaza level and are tied into a grade beam with #3 ties @ 12" O.C. (horizontal reinforcement) and 4-#6 dowels (vertical reinforcement) embedded at least two feet into the drilled caisson. Where steel columns are located, a pier is poured integrally with the grade beam and reinforced with 8-#8 vertical bars and #3 @ 8" O.C. horizontal ties. (As shown in Figures 1.1 & 1.2)

The grade beams have a compressive strength of 3000 PSI and range from 30-48 inches in width and 36-48 inches in depth. Each grade beam is reinforced with top and bottom bars which vary according to the size of the beam. Grade beams span between drilled cast-in-place caissons which transfer the loads from bearing walls, shear walls, and columns into the caissons. From the caissons, the loads are then transferred to bedrock. (As shown in Figures 1.1 & 1.3)
**Superstructure System**

The typical floor system of Cambria Suites Hotel consists of 10” precast hollow-core concrete plank with 1” leveling topping. The precast plank allowed for quicker erection, longer spans, open interior spaces, and serves as an immediate work deck for other trades. Concrete compressive strength for precast plank floors is 5000 PSI and uses normal weight concrete. The typical spans of the plank floors range from 30’-0” to 40’-0”. The floor system is supported by exterior load bearing concrete masonry walls, as well as, interior steel beams and columns.

The Plaza level floor system is a combination of 10” precast concrete plank, 8” precast concrete plank and 4” slab on grade. Since there is no basement in the North-East section of the hotel due to the fitness center and pool, the site was excavated properly in order to place the 4” slab on grade and 8” precast concrete plank. The 4” slab on grade will be for the fitness center where as the 8” concrete plank will surround the pool area. (As shown in Figure 2.1)
Since the masonry bearing walls are typically located on the perimeter of the hotel, steel beams and columns were needed thru the center of the building to support the precast concrete plank floors. Steel beam sizes range from W16x26 to W24x94, and steel column sizes range from W8x58 to W18x175. Each column connects into concrete piers within the grade beams via base plates which vary in size. Base plates use either a 4-bolt or 8-bolt connection, typically using 1” A325 anchor bolts which extend 12” or 18” respectively into the concrete pier. The steel beams vary in length from 13'-0" to 19'-0" and typically span in the East-West direction. Exterior bearing masonry walls and the steel beams will take a reaction load from the precast concrete plank flooring, as well as other loads from levels above, which will then transfer thru steel columns and exterior bearing walls and thus transferring all loads to the foundation system. (As shown in Figure 2.2)

![Typical Partial Floor Plan](image)

The roof structural system at both the Second level and main Roof level uses untopped 10” precast concrete plank. Reinforced concrete masonry extends passed the Roof level to support a light gauge cornice which wraps the entire building. A high roof is constructed for hotel identification purposes and uses 10”-16 GA light gauge roof joists @ 16” O.C., supported by 8”-20 GA light gauge wall framing below. W8x21 hoist beams support the top of the elevator shaft which rest on ½”x7”x7” base plates. There are a total of eight drains located on the roof for the drainage system. (As shown in Appendix A)
Lateral System

The lateral system for the Cambria Suites hotel consists of reinforced concrete masonry shear walls. The exterior shear walls, as well as the core interior shear walls, are constructed of 8” concrete masonry, with the exception of a few 12” concrete masonry walls on the lower floor levels. All shear walls are solid concrete masonry walls which extend the entire height of the structure without openings for windows or doors. The core shear walls are located around the staircases and elevator shafts. The exterior shear walls are scattered around the perimeter of the building. (As shown in Figure 3.1) Shear walls supporting the Plaza level to the Third Floor level have a compressive strength of 2000 PSI. All other shear walls support a compressive strength of 1500 PSI. In addition, all concrete masonry shall be grouted with a minimum compressive strength of 3000 PSI. Typical reinforcement for all shear walls is comprised with #5 bars at either 8” O.C. or 24” O.C.

Wind and seismic loads, as well as gravity loads, are transferred to the foundation by first traveling thru the rigid diaphragm; the precast concrete plank floor system. Loads are then transferred to the concrete masonry shear walls. From there, loads are transferred down to the preceding floor system and/or transferred the entire way to the grade beam foundation, finally travelling thru the concrete caissons which are embedded in bedrock.

![Lateral Shear Wall System](figure3.1)
Codes and Requirements

- International Building Code (IBC), 2006
  *(As amended by the City of Pittsburgh)*

- Minimum Design Loads for Buildings and Other Structures (ASCE 7-05), American Society of Civil Engineers

- Building Code Requirements for Structural Concrete (ACI 318-05), American Concrete Institute
- Specifications for Structural Concrete (ACI 301-05), American Concrete Institute

- PCI Design Handbook – Precast/Prestressed Concrete Institute

- The Building Code Requirements for Masonry Structures (ACI 530), American Concrete Institute
- Specifications for Masonry Structures (ACI 530.1), American Concrete Institute

- Specifications for Structural Steel Buildings – Allowable Stress Design and Plastic Design (AISC), American Institute of Steel Construction

- RS Means Assemblies Cost Data

- PCA

- VULCRAFT Deck Catalog

- Pittsburgh Flexicore P.C. Plank Specifications
Materials

Reinforced Concrete

Caissons & Piers \( f'_c = 4000 \text{ PSI} \)
Grade Beam Foundations \( f'_c = 3000 \text{ PSI} \)
Slabs on Grade \( f'_c = 4000 \text{ PSI} \)
Walls \( f'_c = 4000 \text{ PSI} \)
Exterior Bar or Wire Reinforcement Slabs \( f'_c = 5000 \text{ PSI} \)

Reinforcement Steel

Deformed Bars \( \text{ASTM A615, Grade 60} \)
Welded Wire Fabric \( \text{ASTM A185} \)

Structural Steel

Structural W Shapes \( \text{ASTM A992} \)
Channels \( \text{ASTM A572, Grade 50} \)
Steel Tubes (HSS Shapes) \( \text{ASTM A500, Grade B} \)
Steel Pipe (Round HSSS) \( \text{ASTM A500, Grade B} \)
Angles & Plates \( \text{ASTM A36} \)
Structural Shapes & Rods \( \text{ASTM A123} \)
Bolts, Fasteners, & Hardware \( \text{ASTM A153} \)

Masonry

8” & 12” CMU \( f'_m = 2000 \text{ PSI} \)
Grout \( f'_c = 3000 \text{ PSI} \)
# Design Load Summary

## Live Loads (LL)

<table>
<thead>
<tr>
<th>Area</th>
<th>AES Design Load (PSF)</th>
<th>ASCE 7-05 Load (PSF)</th>
<th>Design Load (PSF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Public Areas</td>
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</tr>
<tr>
<td>Lobbies</td>
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<td>100</td>
</tr>
<tr>
<td>First Floor Corridors</td>
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<tr>
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<td>Stairs</td>
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<tr>
<td>Roof</td>
<td>20</td>
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</table>

## Dead Loads (DL)

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<thead>
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<th>Material</th>
<th>AES Design Load (PSF)</th>
<th>ASCE 7-05 Load (PSF)</th>
<th>Design Load (PSF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10&quot; Concrete Plank</td>
<td>Unknown</td>
<td></td>
<td>91</td>
</tr>
<tr>
<td>8&quot; Masonry Wall (Fully Grouted)</td>
<td>Unknown</td>
<td></td>
<td>91</td>
</tr>
<tr>
<td>8&quot; Masonry Wall (Partially Grouted w/ Reinf. @ 24&quot; O.C.)</td>
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<td>Section 3.1</td>
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<tr>
<td>8&quot; Masonry Wall (Partially Grouted w/ Reinf. @ 48&quot; O.C.)</td>
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<td>60</td>
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<tr>
<td>Steel</td>
<td>Unknown</td>
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<tr>
<td>Partitions</td>
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</tr>
<tr>
<td>MEP</td>
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<tr>
<td>Finishes &amp; Miscellaneous</td>
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<td>5</td>
</tr>
<tr>
<td>Roof</td>
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<td>20</td>
</tr>
</tbody>
</table>

## Snow Load (SL)

<table>
<thead>
<tr>
<th>Area</th>
<th>AES Design Load (PSF)</th>
<th>ASCE 7-05 Load (PSF)</th>
<th>Design Load (PSF)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flat Roof</td>
<td>21</td>
<td>21</td>
<td>21</td>
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</tbody>
</table>

*Refer to Appendix B for Snow Analysis*
Typical Span

The typical bay used in the analysis of the existing and alternative floor systems is defined in Figure 4.
Floor Systems

*Existing: Precast Hollow-Core Concrete Plank on Load Bearing Masonry & Steel Interior*

**Material Properties**

Concrete: 4'-0"x10" w/ 2" topping  
$f_c = 5,000$ PSI

Tendons: T10S108  
$f_{pu} = 270,000$ PSI

Loadings:  
DL = 93 PSF  
LL = 40 PSF  
SDL = 25 PSF

**Description**

The precast hollow-core concrete plank system spans a maximum distance of 38'-0" for the particular section of the building shown in Figure 5.1. For the analysis of this floor system, a typical bay of 38'-0" x 16'-0" was used as shown in Figure 5.1. The weight of the hollow-core plank is distributed evenly to the exterior load bearing masonry wall, as well as the interior steel frame.

The concrete planks designed are 10" thick planks with 2" topping and come in 4' wide sections. The manufacturer of the planks was Pittsburgh Flexicore, but the actual design method used by Pittsburgh Flexicore to make the planks is unknown. Therefore, a design assumption was made that the planks were designed using the PCI Design Handbook. However, since the PCI Handbook did not have the actual strand designation used for the design, the safe superimposed service load was taken from Pittsburgh Flexicore’s specifications for 10" hollow-core plank for the specific strand designation. Section properties were also taken from Pittsburgh Flexicore specifications. In order to obtain the camber, values were estimated from the PCI Handbook. To achieve the 38'-0" span, T10S108 strands were used within the hollow-core panel. This designates that there are 10 strands with a diameter of $(8/16") \phi$, and are to be straight throughout the panel. The assembly of the plank section can withstand a service load of 120 PSF which exceeds the
total un-factored load of 80 PSF. The total un-factored load is a combination of hotel room live loads, superimposed dead loads, and an additional 15 PSF for 2” topping. Supporting calculations may be found in Appendix B.

Advantages

The main advantages of precast hollow-core concrete planks are the low cost and efficient construction process. The precast plank floor has the lowest cost compared to all the floor systems investigated in this report. Precast concrete does not require the curing time that cast-in-place concrete requires, allowing it to be installed and constructed much quicker. This is because precast planks are constructed in a plant where curing can take place year round under controlled conditions. This leads to a faster construction schedule and ultimately a lower overall project cost. Another advantage is the option to span greater distances, resulting in open floor plans and greater structural grid sizes. Hollow-core planks can span up to 40’ and still withstand large loadings. Along with the longer span, the floor depth of the hollow-core planks is much thinner than alternative floor systems allowing for the most efficient use of floor-to-floor heights. Building height restrictions could be a main reason to use hollow-core plank to decrease floor-to-floor height, and ultimately total building height. Since the majority of this floor system is load bearing masonry walls and precast concrete, the system reduces sound and heat transmission. Another advantage is the 2-hour fire rating with minimal fireproofing for the interior steel frame. Other benefits would be reduced building weight due to voids in the planks, as well as flat soffits.

Disadvantages

The most relevant disadvantage using the hollow-core precast plank system is that precast concrete requires more upfront planning. Therefore, the design phase of the project could potentially delay the construction schedule. Longer lead time is also of concern since the concrete planks will have to be transported via oversized trucks from the manufacturer. Lastly, this system works best with square or rectangular bays since the precast planks are not good for curved or angled edges.
Alternative #1: Precast Hollow-Core Concrete Plank on Steel Framing

Material Properties

Concrete: 4'-0"x10" w/ 2" topping  
\[ f_c = 5,000 \text{ PSI} \]

Tendons: T10S108  
\[ f_{pu} = 270,000 \text{ PSI} \]

Loadings: DL = 93 PSF  
LL = 40 PSF  
SDL = 25 PSF

Description

The precast hollow-core concrete plank on steel system is very similar to the existing precast plank system of building. However, this system would dismiss the use of the exterior load bearing masonry walls and use steel columns/beams instead. For this report, the steel columns that support the precast plank system are not analyzed or designed, as they will be discussed in a more in-depth report at a later time.

The concrete planks will span the typical 38'-0" and come in 4' wide sections. To maintain a fair comparison of the alternate and existing floor assemblies, this system will continue to be analyzed for the typical bay size of 38'-0" x 16'-0" as shown in Figure 5.2. In order to decrease the precast plank self-weight, span 38"-0", and still withstand the total floor load, a plank depth of 10" with no topping was selected using the PCI Design Handbook. To achieve the span, strands of 78-S were used with the hollow-core panel. This designates that there are 7 strands with a diameter of \((8/16")\), and are to be straight throughout the panel. The design of this plank system is capable of holding a capacity of 96 PSF which exceeds the value of the total un-factored load of 75 PSF. The total un-factored load was determined using the hotel room live loads, superimposed dead loads, and an additional 10 PSF for untopped planks. Supporting calculations may be found in Appendix C.
Advantages

The precast hollow-core concrete plank on steel has numerous benefits. Structurally, hollow-core planks provide the efficiency of a pre-stressed member for large load capacity, span range, and deflection control. Due to hollow-core’s strength and durability, it allows for increased floor load capacity. It also provides a longer life span for your investment because precast is produced and cured in a controlled factory environment, which means a more dense and durable product. This ultimately leads to a faster construction schedule and cheaper overall project cost. Hollow-core installation is fast and efficient due to the fact that time-consuming actions of cast-in-place concrete are virtually eliminated. Other benefits consist of natural channels for conduits, naturally sound-resistant, and reduced building weight.

Disadvantages

Unfortunately, there are rather large disadvantages to the precast hollow-core plank system. The main drawback is the decrease in floor-to-floor height or the increase in overall building height. The decrease is due to the deeper floor system caused by the W14x82 steel girders that support the concrete planks. The floor system depth would increase from 12” (existing floor system w/ topping) to 24.3” (the 14.3” depth of girder + 10” precast plank depth). This would present a problem in areas where the total overall height of the building is limited. In addition to the lead time in the design phase and transportation of the precast planks, the steel girders and columns will also need to be planned and designed which will increase the overall lead time. Lastly, all steel members will require spray fireproofing to obtain the appropriate fire rating, which will increase overall building cost.

Feasibility

The City of Pittsburgh currently has a building height limit of 11 stories or 160 feet. Cambria Suites Hotel occupies 7 stories above grade; therefore this system could still exist within the boundary conditions at its current location of Pittsburgh. For this system to be considered as a potential candidate, a further investigation would have to be conducted to verify this system could actually impact the pace of the construction process.
Alternative #2: Composite Steel Deck System

Material Properties

Concrete: 4.5" slab
2.5" topping
f'_c = 3000 PSI

Steel: f'_y = 50,000 PSI

Reinforcement: f'_y = 60,000 PSI

Metal Deck: 2VLI20 – 3 Span Condition

Loadings: DL = 45 PSF
LL = 40 PSF
SDL = 25 PSF

Description

The typical bay size used to design a composite steel deck system is 38'-0"x16'-0" as shown in Figure 5.3. This was chosen to maintain a fair comparison between alternate and existing floor systems. Note that the columns for this floor assembly were not designed for this report.

To comply with the typical bay and loadings, a 2VLI20 composite deck was selected using the Vulcraft Deck Catalog. This deck will support a 4.5" normal weight concrete slab with a 2.5" topping, which will be able to span 10'-7" unshored giving a 3 span condition. This well exceeds the 9'-6" spacing used for this design. The size of the steel beams and girders were designed in accordance with the American Institute of Steel Construction (AISC). Supporting calculations may be found in Appendix D.

Advantages

Advantages of the composite steel deck system include its low self-weight and constructability. The system self-weight of 45 PSF is significantly lower than the self-weight of the other alternative floor assemblies investigated in this report. This will result in a reduced gravity load to the foundation, thus reducing the costs associated with the columns and foundation. The construction will be simplified since it requires no shoring for the 9'-6" spans. Typically, steel erection takes less time than forming, placing, and
curing concrete. This will then result in a faster construction schedule. Composite metal deck allows for a very efficient construction process since the metal decking will serve as the formwork for the concrete slab, thus cutting down on time, cost, and waste material. Additional advantages include a fire rating of 2-hours and a relatively shallow system depth of 25.2”. This will leave sufficient space for mechanical ducts and pipes in the ceiling.

**Disadvantages**

Although this system is the lightest self-weight and is efficient to construct, it still has several disadvantages. One disadvantage might be the floor system depth of 25.2" (20.7” depth of girder + 4.5” slab). This system depth would either adjust the entire height of the building, adding additional costs, or it would reduce the ceiling heights. With an all-steel frame building, fireproofing would be required to obtain an approved fire rated building. Additional disadvantages would be poor sound-insulating materials since steel is not a good material to absorb sound. This may be of concern since Cambria Suites is a hotel and noise level transferring between walls and floors should be minimal.

**Feasibility**

In summary, after weighing the advantages and disadvantages of the composite system, it seems like the disadvantages outweigh the advantages. Even though the cost of the system is fairly low and that steel buildings are typically dominate in Pittsburgh, the negative factors with this system do not coincide with a hotel design. Therefore, use of this system for the Cambria Suites Hotel is not likely, due to the decrease floor-to-floor height, additional costs that may be present and poor sound-insulating materials.
Alternative #3: One-Way Joist System

Material Properties

<table>
<thead>
<tr>
<th>Material</th>
<th>Property</th>
<th>Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>4.5&quot; slab</td>
<td>4.5“ slab</td>
</tr>
<tr>
<td></td>
<td>66”/6” pan joists</td>
<td>66”/6” pan joists</td>
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<td>$f'_c = 3000$ PSI</td>
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<td></td>
<td>LL = 40 PSF</td>
<td>LL = 40 PSF</td>
</tr>
<tr>
<td></td>
<td>SDL = 25 PSF</td>
<td>SDL = 25 PSF</td>
</tr>
</tbody>
</table>

Description

The one-way joist system was designed to span the 38’-0” direction of the typical bay of 38’-0”x16’-0” as shown in Figure 5.4. A 4.5” slab was used in conjunction with 6” wide by 18” deep joists spaced at 66” on center. The depth of the pan joist is 18” which is adequate for deflection control. Minimum reinforcement for the slab is (1) #3 bar spaced at 12” on center. The flexural reinforcement required for the negative moment is (2) #9 bars (top reinforcement). Bottom reinforcement required for the positive moment is (1) #10 bar.

An exterior and interior girder was designed to span the 16’-0” perpendicular to the joist ribs. A 24” exterior girder was designed to match the assumed column dimensions to provide for better constructability. A 36” girder was designed for the interior. The top reinforcement required for the interior girder is (3) # 7 bars, and the required bottom reinforcement is (2) #6 bars. The top reinforcement required for the exterior edge girder is (4) #6 bars, and the required bottom reinforcement is (2) #6 bars. Supporting calculations may be found in Appendix E.

Advantages

The one-way joist system was chosen as an alternative because they are the most economical concrete system for long spans with heavy loads. This results in wider columns spacing, inherent vibration resistance, reduced dead load due to pan voids, easier future renovations, and easier placement of electrical and mechanical equipment between pan joists. The 6”/66” joist system designed is considered a “skip” joist, since the pans are
spaced further apart. This results in even longer spans and larger column spacing. The longer spans and inherent vibration resistance make this alternative floor assembly attractive for hotels. In addition, this system is capable of a 2-hour fire rating without additional fireproofing.

**Disadvantages**

Disadvantages of the one-way joist system include the self-weight which is substantially larger than the self-weight of the other alternative floor systems. This will add more weight to the building, thus resulting in more gravity load to the foundation. Also, the construction will not be as efficient as the existing system or other alternatives due to the necessary formwork. Another slight disadvantage is the depth of the system, which is quite larger than the existing system. However, electrical and mechanical equipment can be run between the pan joists which mean additional floor depth is not needed to accommodate this equipment.

**Feasibility**

It may be worthwhile in the future to compare the total cost of the building associated with the one-way joist system against the total cost of the building using the existing floor system. Due to the potential that the low floor system cost could outweigh the effects of the larger self-weight, it is determined that the one-way joist system is a feasible alternative that may require additional study. The increase in floor depth is not of concern, since the building resides in Pittsburgh and still has additional building height before reaching the maximum allowable height of 160 feet.
Overall System Comparison

<table>
<thead>
<tr>
<th>Comparison Criteria</th>
<th>Precast Plank on Load Bearing Walls &amp; Steel Frame</th>
<th>Precast Plank on Steel Framing</th>
<th>Composite Steel Deck System</th>
<th>One-Way Joist System</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab Self Weight</td>
<td>93 PSF</td>
<td>75 PSF</td>
<td>45 PSF</td>
<td>91.5 PSF</td>
</tr>
<tr>
<td>Slab Depth</td>
<td>10”</td>
<td>10”</td>
<td>4.5”</td>
<td>4.5”</td>
</tr>
<tr>
<td>System Depth</td>
<td>12” (10”+2” topping)</td>
<td>24.3”</td>
<td>25.2”</td>
<td>22.5”</td>
</tr>
<tr>
<td>Deflection (LL + DL)</td>
<td>Adequate with camber</td>
<td>0.193” &lt; 0.8”</td>
<td>1.22” &lt; 1.26”</td>
<td>.061” &lt; 0.8”</td>
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<tr>
<td>Impact on Building Design</td>
<td>Existing</td>
<td>Reduced floor-to-ceiling height</td>
<td>Reduced floor-to-ceiling height</td>
<td>Reduced floor-to-ceiling height</td>
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<tr>
<td>Constructability</td>
<td>Easy</td>
<td>Easy</td>
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<td>Average</td>
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<td>System Cost*</td>
<td>$14.01/SF</td>
<td>$25.34/SF</td>
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<td>$17.00/SF</td>
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<tr>
<td>Feasibility</td>
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<td>Yes</td>
<td>No</td>
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</tbody>
</table>

*System cost is estimated using RS Means Assemblies Cost Data and RS Means Facilities Construction Cost Data.
Conclusion

In the second technical report regarding the construction of the Cambria Suites Hotel, alternative floor systems were designed for a typical bay of 38’-0”x16’-0”. Each alternative floor system was compared to each other, as well as to the existing floor assembly. The existing floor system is a precast hollow-core concrete plank floor supporting normal weight concrete for a total depth of 10”. This system bears on exterior load bearing masonry walls and an interior steel frame. The major comparisons factors for this report were system depth, self-weight, cost, and constructability.

After comparing each alternative floor system with the existing system, it was concluded that the existing floor system is the most efficient in construction time, cost, and system depth for the Cambria Suites Hotel. However, a few of the alternate systems may be a realistic solution for the building as well. A one-way joist system incorporates a deeper system depth and is a slightly heavier system (self-weight), but is the most economical concrete system for the long span condition of the Cambria Suites Hotel. The precast hollow-core plank on steel offers a design that is consistent with the existing system, but eliminates the exterior load bearing masonry walls. Although it is a lighter system and is easily constructed, more total cost is added for the additional steel. The only downfall with this system is the total system depth increases due to the steel girders supporting the precast planks. The composite steel deck system presented in this report can be argued to be a feasible or non-feasible building. The total cost/square feet is lower than the other alternative floor assemblies, but has the largest floor system depth and poor sound-insulating properties which hotels try to avoid.

The most likely alternative system for the Cambria Suites Hotel, besides its existing system, is the one-way pan joist system. This system created the second thinnest overall floor system depth, as well as a fairly cheap system per square foot. One-way joist systems are the most economical concrete system for long span conditions and heavy loads, and with the 38’-0” maximum spans of Cambria Suites Hotel, this alternative system seems to be practical. Other benefits such as good sound-insulating properties, wider column spacing, reduced dead load due to pan voids, and easier placement of electrical and mechanical equipment in the pan joists, the one-way system seems to be a very feasible alternative floor assembly.

Lastly, concrete systems are commonly used for midrise hotels. Therefore it is logical that a concrete system would be more applicable and feasible for the Cambria Suites Hotel. Please refer to the following appendices for detailed calculations and analysis for each floor system designed for the Cambria Suites Hotel.
Appendix A: Building Layout

Foundation Plan

Plaza Level Framing Plan
Third thru Seventh Level Framing

Roof Framing Plan

High Roof Framing Plan
Appendix B: Existing Floor System

Precast Hollow-Core Concrete Plank on Load Bearing Masonry & Steel Interior

Pittsburgh Flexicore Specifications
PRECAST HOLLOW-CORE CONCRETE PLANKS
ON LOAD BEARING MASONRY & STEEL INTERIOR

* LOADS
  \[ LL = 40 \text{ PSF} \quad \text{(Hotel Rooms)} \]
  \[ SDL = 25 \text{ PSF} \quad \text{(MEP, Partitions, Finishes)} \]
  \[ DL = 15 \text{ PSF} \quad \text{(2" Topping \Rightarrow PCI Handbook, 2-33)} \]

**Total Load** = 40 + 25 + 15 = 80 PSF

- \( f'_c = 5,000 \) PSI
- \( f'_{ck} = 2700 \) PSI
- **SPAN** = 38'

* Designed for 10" - 0" topping
  \[ 4' - 0" \times 10" \\text{SLAB} (4H10 + 2) \]

* From Pittsburgh Flexicore
 โทร5108 ค้gy 120 PSF capacity @ 38' span

- 10 strands @ 0.16" Ø - straight
- **Self Weight of Slab** = 93 PSF
- **AES used 91 PSF** to account for 1"
  **topping**

* **Load to Masonry Walls**
  \[
  W_m = 1.2 \left( 25 + 93 \right) + 1.4 \left( 40 \right) = 205.6 \text{ PSF} \\
  M_u = \frac{205.6 \text{ PSF} \times (10' - 0") \times (38')}{8} = 573.8 \text{ \#k} \Rightarrow 594 \text{ \#k}
  \]

\[ \text{Load BEARING MASONRY} \]

\[ \text{STEEL BEAM} \quad \text{(HUM)} \]

\[ \text{INTERIOR} \]
Hollow-Core Plank (cont.)

\[ A_{ps} = 10 \text{ strands} \times \frac{9}{16}'' = 10 \times (0.5) = 5.0 \text{ in}^2 \]
\[ f_{ps} = 270 \text{ ksi} \]
\[ d = 4\frac{5}{8}'' (12) = 48'' \]
\[ d_p = 12'' - 1\frac{1}{2}'' \text{ CLR} = 10.5'' \]
\[ A_p = \frac{5.0}{0.85} (270 \text{ ksi}) \]
\[ A_{ps} \frac{f_{ps}}{f} = \frac{5.0}{0.85} (5 \text{ ksi})(48'') = 6.62 \text{ in.} \]

\[ f_{Mn} = \frac{f_{ps} d_p}{2} \left( d_p - \frac{9}{16}'' \right) \]
\[ = 0.9 \left[ 5.0(270)(10.5 - 6.62) \right] \]
\[ = 8736 \text{ in. \cdot k} = 728 \text{ ft.k} \]

\[ f_{Mn} = 728 \text{ ft.k} > 594 \text{ ft.k} = M_u \cdot \Rightarrow \text{OK Design} \]

*Deflection*

\[ E = 57,000 \text{ ksi} \]
\[ E_t = 4030 \text{ ksi} \]
\[ I = 5576 \text{ in}^4 \] (PITTSBURGH FLEXICORE)
\[ \Delta L = \frac{360}{360} = 1.267 \text{ in.} \]
\[ \Delta L = \frac{5(40)(16)(38)^4}{384(403000)(5576)} \times 1728 = 1.334 \text{ in.} > 1.267 \text{ in.} \]

*However, P.E. Plank has an estimated camber of 1.2" (88-5 in PC1), which will result in an acceptable deflection.*
Appendix C: Alternative System #1

Precast Hollow-Core Concrete Plank on Steel Framing

![Diagram of HOLLOW-CORE plank]

**Table of safe superimposed service load (psf) and cambers (in.)**

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<th>21</th>
<th>22</th>
<th>23</th>
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<td>73</td>
<td>67</td>
<td>61</td>
<td>56</td>
<td>50</td>
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</tbody>
</table>

Key:
- 258 = Safe superimposed service load, psf
- 0.3 = Estimated camber at erection, in.
- 0.4 = Estimated long-term camber, in.

- $f_c' = 5,000$ psi
- $f_{pu} = 270,000$ psi

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</tr>
<tr>
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<td>$S_z = 645$ in.$^3$</td>
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<tr>
<td>$w_t = 270$ psf</td>
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<td>$DL = 68$ psf</td>
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<td>$DL = 70$ psf</td>
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<td>$W_S = 2.23$ in.</td>
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4HC10
Precast Hollow-Core Concrete Plank
on Steel Framing.

* Loads
  LL = 40 PSF  (Hotel Rooms)
  DL = 25 PSF  (MEP, Partitions, Finishes)
  DL = 10 PSF  (W/O Topping ⇒ PCI Handbook)

Total Load = 40 + 25 + 10 = 75 PSF

f_c = 5000 PSI
f_mu = 27000 PSI
Span = 38'

* Designed for 10' W/O Topping
  4' - 0" x 10" NUC (4HC10)

* From PCI Handbook
  7B-5 carrying 90 PSF capacity @ 38’ span
  0.8” estimated camber at erection
  0.4” estimated long-time camber
  7 strands @ 9/16” → Straight
  Self Weight of Slab = 68 PSF

* Girders
  \[ w_u = 1.2(25 + 68) + 1.6(40) = 175.6 \text{ PSF} \]
  \[ M_u = \frac{175.6 \times 68(16-0")(38")^2}{8} = 507 \text{ kN} \]
Hollow-Core Plank on Steel (cont.)

- USE L121 x 62 (AISC TABLE 3-2)

\[ \phi M_0 = 540 ft \cdot k > 507 ft \cdot k = M_u \quad \therefore \text{OK} \]

\[ A_{ul} = \frac{1}{360} = \frac{14'(12)}{3600} = 0.53'' \]

\[ 0.53 = \frac{5(40)(38')(16)'(1728)}{384(29000)} \Rightarrow I_x = 145.8 \text{ in}^4 \leq 1330 \text{ in}^4 \]

\[ \frac{5(40+25+68)(38')(16)'(1728)}{384(29000)(1330)(1000)} = 0.193'' \quad \therefore \text{OK} \]

\[ \Delta T_L = 0.193'' < \frac{14'(12)}{240} = 0.60'' \quad \therefore \text{OK} \]

* In order to achieve a thinner system depth, use a wide flange with a smaller depth.

- USE L14 x 82 (less economical, but decreases system depth)

\[ \phi M_0 = 521 ft \cdot k > 507 ft \cdot k = M_u \quad \therefore \text{OK} \]

\[ A_{ul} = \frac{1}{360} = \frac{14'(12)}{3600} = 0.53'' \]

\[ 0.53 = \frac{5(40)(38')(16)'(1728)}{384(29000)} \Rightarrow I_x = 145.8 \text{ in}^4 \leq 881 \text{ in}^4 \text{ for} \]

\[ \frac{5(40+25+68)(38')(16)'(1728)}{384(29000)(881)(1000)} = 0.292'' \quad \therefore \text{OK} \]

\[ \Delta T_L = 0.292'' < \frac{14'(12)}{240} = 0.60'' \quad \therefore \text{OK} \]

USE L14 x 82
Appendix D: Alternative System #2

*Composite Steel Deck System*

![Composite Steel Deck System Diagram]

---

### VULCRAFT

**2 VLI**

Maximum Sheet Length 42'-0" Extra Charge for Lengths Under 6'-0"

ICBO Approved (No. 3415)

---

#### STEEL SECTION PROPERTIES

<table>
<thead>
<tr>
<th>Deck</th>
<th>Design Thickness</th>
<th>Weight</th>
<th>Depth</th>
<th>Spacing</th>
<th>Section Properties</th>
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(N=9.35) NORMAL WEIGHT CONCRETE (145 PCF)

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<th>SDI Max. Unshored</th>
<th>Superimposed Live Load, PSF</th>
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<td>Type</td>
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<td>9-6</td>
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<td>8-7</td>
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<td></td>
<td>VLI16</td>
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**COMPOSITE**

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<td>VLI14</td>
<td>10-5</td>
<td>12-6</td>
</tr>
</tbody>
</table>

---

Page | 33
### Composite Steel Deck

**Loads**
- LL = 40 PSF (Hotel Rooms)
- SDL = 25 PSF (Plnt., MEP, Finishes)
- DL = 45 PSF (Deck Self-Weight)

**Vulcraft Deck Used**
- Slab depth = 9.5"
- Topping = 2.5"
- N.W.C. (145 RF), N = 9.35
- 3 Span Condition
- Use: 2ULIZ20 Deck
  - f'c = 5000 PSI
  - f_y, Steel = 60,000 PSI

**Total Load** = LL + SDL + DL = 104 PSF

**Deck Used**: 2ULIZ20 Deck, 3 Span
- Clear Span = 9'6"
- 20 Gauge
- Superimposed LL Max. Capacity = 123 PSF > 104 PSF \( \therefore \) OK
  - FL = 30,000

**Beam**
- Load = 1.2D + 1.6L
  = 1.2(25+95) + 1.6(40)
  = 148 PSF or \( 0.148 \text{ ksf} \)
- Trib. Length = 9'6" or 9.5'
- \( W_u = 9.5' \left( 0.148 \text{ ksf} \right) = 1,406 \text{ klf} \)
- \( W_u = \frac{1,406 \left( 11' \right)}{2} = 11.3 \text{ k} \)
- \( M_u = \frac{1,406 \left( 11' \right)^2}{8} = 45.0 \text{ ft.k} \)
Composite Steel Deck (cont.)

\[
\begin{align*}
\text{Deflection} \: & = \: Z \left( \frac{\text{Span}}{B} \right) = Z \left( \frac{16}{8} \right) = 4.8'' \\
& \left( \frac{L}{2} \right) \text{-spacing} = 9.5'' \left( 12 \right) = 114''
\end{align*}
\]

Assume \( a = 1.0 \)

\[
\frac{V_2}{2} = \frac{0.085}{4} = 9.5'' - \frac{L}{2} = 4.0''
\]

\( Q_0 = 17.7 \: k \) for 3 ksi nuc \( w/ \) deck perpendicular

Try \( W_{10} \times 12 \) - \( q_{Mn} = 7.32 \: ft \cdot k \), \( q_{Mn} = 7.32 \: ft \cdot k \)

\[
\begin{align*}
q_{En} & = \frac{44.2}{0.85} \: \text{def.} = \frac{44.2}{0.85(12)} = 0.361'' \leq 1'' \: \text{OK}
\end{align*}
\]

\[
\frac{q}{2} = 4.5 - \frac{0.361}{2} = 4.13'' \geq 4.0'' \: \text{CONSERVATIVE}
\]

Shear Studs \( \geq \frac{44.2}{17.2} = 2.57 \geq 3 \) Studs / H.E.A.L.E = 6 Studs

Check Unshored Strength

\[
\begin{align*}
\text{C}_{LL} & = 20 \: \text{psf} \left( 9.5'' \right) = 0.190 \: \text{kif} \\
\text{WLL} & = \text{C}_{LL} = 0.190 \: \text{kif} \\
\text{W} & = \left( 45 \: \text{psf} \right) \left( 9.5'' \right) + 12 \: \text{psf} = 0.4395 \: \text{kif} \\
\text{W} & = 1.12 \left( 0.4395 \right) + 1.4 \left( 0.19 \right) = 0.8344 \: \text{kif} \\
\text{M} & = \frac{22.6}{B} = \frac{0.8344(16)^2}{B} = 22.6 \: \text{Ft} \cdot \text{k} \leq 47.3 \: \text{Ft} \cdot \text{k} = \phi_{Mn}
\end{align*}
\]

Check Member Strength

\[
\begin{align*}
\phi_{Ma} & = 73.2 \: \text{A} \cdot \text{k} > 45 \: \text{Ft} \cdot \text{k} = M_m \\
\phi_{Vn} & = 54.8 \: \text{k} > 11.3 \: \text{k} = V_n
\end{align*}
\]
Composite Steel Deck (cont.)

Check LL Deflection

\[ W_{LL} = 45 \text{ PSF} (9.5') = 0.38 \text{ klf} \]
\[ I_{LB} = 117 \text{ in}^4 \]
\[ \Delta_{LL} = \frac{5Lw^4}{384EI_{LB}} = \frac{5(0.38)(16^4)(1728)}{384(29,000)(117)} = 0.145'' \]
\[ \frac{L}{300} = \frac{16'(12')}{300} = 0.533'' > 0.165'' \text{ - OK} \]

Check Wet Concrete Deflection

\[ W_{wc} = 45 \text{ PSF} (9.5') + 12 \text{ PLF} = 0.4395 \text{ klf} \]
\[ I_{X} = 53.8 \text{ in}^4 \]
\[ \Delta_{wc} = \frac{5Lw_{wc}^4}{384EI_{X}} = \frac{5(0.4395)(16^4)(1728)}{384(29,000)(53.8)} = 0.415'' \]
\[ \Delta_{wc,max} = \frac{L}{240} = \frac{16'(12')}{240} = 0.8'' > 0.415'' \text{ - OK} \]

Use W10 x 12 (w)

Girders

Total DL = (45 PSF + 25 PSF) (9.5') = 0.665 klf
Total LL = 40 PSF (9.5') = 0.38 klf

Total DL, Girders = 0.665 klf (16') = 10.64 k
Total LL, Girders = 0.38 klf (16') = 6.08 k
Composite Steel Deck (cont.)

\[ P_u = 1.2D + 1.6L = 1.2(10.04) + 1.6(4.08) = 22.496 \text{ k} \]
\[ V_u = P_u = 22.496 \text{ k} \]
\[ M_u = 9.5'(22.496\text{ k}) = 213.7 \text{ ft-k} \]

Defl. = \[ 2 \left( \frac{span}{8} \right) = 2 \left( \frac{36(12)}{8} \right) = \frac{114}{2} \]
\[ 2 (1/2) \text{ SPACING} = 16'(12) = 192'' \]

Assume \( a = 1.0 \)
\[ \gamma_c = 4.15 - \frac{1}{2}a = 4.10'' \]
\[ Q_o = 21.0 \text{ k} \text{ for 3 ksi NOC w/ deck parallel} \]

Try \( W/2 \times 30 \) - \( \phi M_u = 228 \text{ ft-k}, \phi b M_p = 102 \text{ ft-k}, p = 7', \phi Q_o = 110 \text{ k} \)
\[ q = \frac{225(3)(114)}{120} = 0.378'' < 1.0'' \quad \therefore \text{OK} \]
\[ \gamma_c = 4.15 - 0.378 - 4.311'' > 4.0'' \quad \therefore \text{CONSERVATIVE} \]

Shear Studs \( \phi \)
\[ \frac{110}{21.0} = 5.24 \rightarrow 6 \text{ STUDS/HALF} \rightarrow 12 \text{ STUDS} \]

Check Unshored Strength
\[ w_u = 1.2D + 1.6L = 1.2(0.63) = 0.756 \text{ klf} \]
\[ C_u = 20(PSF)(9.5')(10') = 3.04 \text{ k} \]
\[ C_L = [45\text{PSF} (9.5') + 12 \text{ PIF}] (10') = 7.032 \text{ k} \]
\[ P_u = 1.2D + 1.6L = 1.2(7.032) + 1.6(3.04) = 13.3 \text{ k} \]
\[ V_u = P_u = 13.3 \text{ k} \]
\[ M_u = \frac{w_u d^2}{8} + \frac{P_u L}{4} = \frac{0.756(38')^2}{8} + \frac{13.3(38')}{4} = 132.8 \text{ ft-k} \]
\[ M_u = 132.8 \text{ ft-k} < 142 \text{ ft-k} = \phi b M_p \quad \therefore \text{OK} \]
COMPOSITE STEEL DECK (cont.)

CHECK MEMBER STRENGTH

\[ \phi M_a = 228 \text{ k} > 213.7 \text{ k} = M_a \]

\[ \phi V_a = 94.3 \text{ k} > 13.3 \text{ k} = V_a \]

CHECK LL DEFLECTION

\[ P_{ll} = 6.08 \text{ k} \]

\[ I_{xx} = 420 \text{ in}^4 \]

\[ \frac{Q}{E} = 411.54 + 137.1 + 823.1 = \frac{1371.7}{EI} \]

\[ \frac{Q}{E} = \frac{(80.44)(9.5)}{2} + \frac{(115.5 - 80.44)(9.5)}{2} + 80.44(9.5) \]

\[ \frac{Q}{E} = 411.54 + 137.1 + 823.1 = \frac{1371.7}{EI} \]

\[ \Delta L = \frac{(1371.7)(19')}{(920)} = 2.46'' \]

\[ \Delta L_{max} = \frac{4}{500} = \frac{38'(12)}{500} = 1.26'' \text{, NOT OK} \]

\[ I_{req.} = \frac{1371.7(19')(1728)}{(29000)(1.26'')} = 821.7 \text{ in}^4 \]

\[ I_{req.} = \frac{1371.7(19')(1728)}{(29000)(1.26'')} = 821.7 \text{ in}^4 \]

- USE \( W21 	imes 44(12) \), \( f_{flw} = 398 \text{ psi} \), \( I_x = 893 \text{ in}^4 \)
Appendix E: Alternative System #3

*One-Way Joist System*

---

**Assume:**
- $\text{NWC (150 RCF)}$
- $f'_c = 4 \text{ ksi}$
- $f_y = 60 \text{ ksi}$

**Edge beam width:** 24”
**Interior beam width:** 36”
**6” wide joists spaced 66” o.c.

---

**Preliminary Pan Depth:** 18” for a 20 x 60 bay size, 66” Pan (Per Handling)
\[ \mu = \frac{w_a h_n^2}{10} = 0.1615 \left(40"/12"\right)^2 = 0.0489 \text{ kips/ft of slab} \]

**MIN. REINFORCEMENT**

\[ A_{sf,req} = 0.001b \left(4\right) (12) = 0.0972 \text{ in}^2 \]

\( A_{sf} = 3 \text{ bars} \Rightarrow A_s = 0.11 \text{ in}^2 \)

\[ a_s = \frac{A_{sf}}{0.85(4)(12)} = 0.11 \text{ in} \]

\[ \phi M_n = 0.9 \cdot A_{sf} f_y \left(d - 0.4s\right) = 0.9 \cdot 0.11 \cdot (60) \left(\frac{4.5}{2} - \frac{0.11 \cdot 12}{2}\right) = 1.07 \text{ kips/ft of slab} > 0.489 \text{ kips/ft of slab} \]

**SPACING**

\[ 3d = 3(4.5") = 13.5" \Rightarrow \text{use 12"} \]

\[ \therefore (1) \# 3 @ 12" \text{ o.c.} \]

**JOINT**

\[ W_{scl} = 25 \text{ PSF} (12") = 150 \text{ PLF} \]

\[ W_{slab} = \left(4\frac{1}{2}"\right)(150 \text{ PSF})(6") = 337.5 \text{ PLF} \]

\[ W_{silw} = \left(18"\right)(6") \left(\frac{144}{144}\right) = 112.5 \text{ PLF} \]

\[ W_{ll} = 40 \text{ PSF}(6") = 240 \text{ PLF} \]

\[ L_a = 1.2 \left(0.150 + 0.3575 + 0.1125\right) + 1.6 (0.214) = 1.07 \text{ kips} \]

\[ M_{max} = \frac{w_a h_n^2}{14} = \frac{1.07(35.5)^2}{14} = 96.3 \text{ ft.k} \]

\[ M_{max} = \frac{w_a h_n^2}{10} = \frac{1.07(35.5)^2}{10} = 134.8 \text{ ft.k} \]
**Top Reinforcement**

\[ A_t = \frac{M}{2d} = \frac{134.8}{4(20.25)} = 2.02 \text{ in}^2 \]

\[ d = 22.5 - (4.5) = 22.5'' \]

\[ f_y = 210 \text{ ksi} \]

\[ d = \frac{A_t}{f_y} = \frac{2(1.2)}{210} = 0.0165 \]

\[ a = \frac{A_t}{f_y} / 0.85 \times 4 = 5.88'' \]

\[ c = \frac{a}{B} = \frac{5.88}{0.85} = 6.92'' \]

\[ \varepsilon = \frac{0.003}{c} (d - c) = \frac{0.003}{6.92} (20.25 - 0.92) = 0.00577 > 0.005 \]

\( \text{-- Tension Controlled, } \phi = 0.9 \)

\[ \phi = \frac{\phi_M}{f_y} \left( 1 - \frac{9}{8} \right) \]

\[ \phi = \frac{0.9 (210)(60)}{0.925} \approx 0.12 \]

\[ M_{tk} = 155.79 \text{ ft.k} > 134.18 \text{ ft.k} \]

\( \text{OK} \)

**Bottom Reinforcement**

\[ d = 22.5 - 1.5 = 0.375 = 0.035 = 19.99'' \]

\[ \text{Cover} \]

\[ A_s = \frac{M}{2d} = \frac{90.3}{4(19.99)} = 1.2 \text{ in}^2 \]

\[ f_y = 210 \text{ ksi} \]

\[ d = \frac{A_s}{f_y} = \frac{1.2}{210} = 0.106 \]

\[ a = A_s / 0.85 \times 4 = 0.31 \]

\[ c = \frac{a}{0.85} = 0.366 \]

\[ \varepsilon = \frac{0.003}{0.346} (20.25 - 0.366) = 0.163 > 0.005 \]

\( \text{-- Tension Controlled, } \phi = 0.9 \)
\[ \phi M_n = \phi A_s f_y (d - \frac{d}{2}) \]
= \[ \phi (0.9(1.5)(40 - 20.25 - \frac{20.25}{2})) \]
= \[ 108 \text{ ft.k} > 96.13 \text{ ft.k} \]
\[ \therefore \text{OK} \]

\text{USE (1) \# 10}

\text{SHEAR DESIGN}

\[ V_u = 1.15 \omega_u h_n \frac{d}{2} = 1.15 (1.07)(35.5)(20.25) \]
\[ \phi V_u = \phi 2.61\overline{f} b d = 0.75 \overline{f} \frac{(20.25)(4000)(4000)}{1025} = 11.5 \text{ k} \]
\[ \phi V_s = V_u - \phi V_c = 21.8 - 11.5 = 10.3 \text{ k} \]

\[ \phi V_s = 10.3 \text{ k} = \frac{\phi A_{sf} f_d}{S_{\text{min}}} \]
\[ 10.3 = \frac{A_{sf}(20.25)(0.75)}{8''} \]
\[ \Rightarrow A_{sf} = 0.09 \text{ in}^2 \]

\[ \therefore \text{USE 3 \# 8'' SPACING} \]

\[ \phi V_c + \phi V_s = 11.5 \text{ k} + \frac{0.75(2.11)(40)(20.25)}{C_6} = 20.2 \text{ k} > 21.0 \text{ k} = V_u \]
\[ \therefore \text{OK} \]
Girder Design (Interior)

\[ w_{G_{surf}} = \frac{(1.07 \text{ klf}')}{(12')}(13.5') = 6.33 \text{ klf} \]

\[ w_{G_{watt}} = 1.2(25 \text{ psf})(3') + 1.4(40 \text{ psf})(3') = 0.782 \text{ klf} \]

\[ w_{self} = 1.2(150 \text{pcf})(22.5\text{'})\left(\frac{3\text{'}/12\text{'}}{12\text{'}/12\text{'}}\right) = 1.01 \text{ klf} \]

\[ w_{u} = 6.33 + 0.782 + 1.01 = 8.12 \text{ klf} \]

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Aci Moment Coefficients

\[ M_{a,b} = \frac{w_{u}L_{u}^{2}}{12} = \frac{102(16)'}{12} = 177.3 \text{ ft-k} \]

\[ M^{*} = \frac{w_{u}L_{u}^{2}}{16} = \frac{742(16)'}{16} = 121.9 \text{ ft-k} \]

Top Reinforcement (Int. Span / Int. Support)

\[ A_{s} = \frac{M_{u}}{f_{d}} = \frac{9(19.89)}{16} = 2.22 \text{ in}^{2} \]

\[ \text{TRV}(\%) = 7 \Rightarrow A_{s} = 2.4 \text{ in}^{2} \]

\[ d = 2.4(60)'' = 1.18'' \]

\[ c = 1.18/0.65 = 1.80'' \]

\[ \varepsilon_{t} = \frac{0.003}{1.38}(19.97 - 1.38) = 0.0405 > 0.0075 \Rightarrow \text{Moment can be reduced per Aci B.4} \]

Moment Redistribution: 1000 \( \varepsilon_{t} = 40.5 \Rightarrow \text{reduce by 40.5\%} \)

\[ M_{u} = 177.3(1 - 0.405) = 105.5 \text{ ft-k} \]
\[
A_s = \frac{105.5}{4(19.99)} = 1.32 \text{ m}^2
\]

Try (3) #7 \(A_s = 1.8 \text{ m}^2\)

\(a = \frac{1.8(40)}{0.85(4)(36)} = 0.88\)

\[
\Phi M_u = \left[0.9(1.8 \text{ in.}^2)(60^\circ)(19.99 - \frac{0.88}{2})\right] / 12
\]

\[
= 158.4 \text{ ft.k} > 105.5 \text{ ft.k} \quad \therefore \text{OK}
\]

**USE (3) #7 TOP REINFORCE.**

**BOTTOM REINFORCEMENT (INT. SPAN)**

\(A_s = \frac{121.9}{4(19.99)} = 1.53 \text{ m}^2\)

Try (3) #7 \(A_s = 1.8 \text{ m}^2\)

\(a = \frac{1.8(40)}{0.85(4)(36)} = 0.88\)

\(c = 0.88 / 0.85 = 1.035\)

\[
\epsilon_t = 0.003(19.99 - 1.035) = 0.055 > 0.0075 \quad \therefore \text{MOMENT can be reduced per ACI 18.4}
\]

**MOMENT REDISTRIBUTION:** \(1000 \epsilon_t = 54.94 \Rightarrow \text{reduce by 54.94 ft.k}\)

\(M_u = 121.9(1 - 0.55)\)

\[
= 54.86 \text{ ft.k}
\]

\(A_s = \frac{54.86}{4(19.99)} = 0.668 \text{ m}^2\)

Try (2) #6 \(A_s = 0.88 \text{ m}^2\)

\(a = \frac{0.88(40)}{0.85(4)(36)} = 0.431\)

\[
\Phi M_u = \left[0.9(0.88)(40)(19.99 - \frac{0.431}{2})\right] / 12
\]

\[
= 78.31 \text{ ft.k} > 54.86 \text{ ft.k} \quad \therefore \text{OK}
\]

**USE (2) #6 BOT. REINFORCE.**
* EDGE GIRDER DESIGN (INTERIOR SPAN)

\[ w_i = 7.62 \text{ klf} / 2 = 3.81 \text{ klf} \]

* Half the Tributary Width of the Interior Girder.

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\[ M = 88.7 \text{ ft.k} \]

ACI MOMENT COEFFICIENTS

\[ M_{ab} = \frac{w_i h^2}{12} = \frac{3.81(16)^2}{12} = 88.7 \text{ ft.k} \]

\[ M_\tau = \frac{w_i h^2}{16} = \frac{3.81(16)^2}{16} = 60.90 \text{ ft.k} \]

**Top REINFORCEMENT (INT. SPAN, INT. SUPPORT)**

\[ A_s = \frac{M_\tau}{f_y} = \frac{88.7}{60} = 1.48 \text{ in}^2 \]

TRY (4) #6 \( A_s = 1.74 \text{ in}^2 \)

\[ a = 0.85(4)(24) = 1.29 \]

\[ c = 1.29 \]

\[ \epsilon_t = \frac{0.003}{1.52} (19.99 - 1.52) = 0.003 > 0.005 \quad \text{:: Tension Controlled} \]

\[ f_{hm} = \left[ 0.9 \left( 1.74 \text{ in}^2 \right) (19.99 - \frac{1.29}{2}) \right] / 12 \]

\[ = 153.2 \text{ ft.k} > 88.7 \text{ ft.k} \quad \text{:: OK} \]

USE (4) #6 TOP REINF.
Bottom Reinforcement (Int. Span)

\[ A_s = \frac{60,000}{4(19.99)} = 0.76 \text{ in}^2 \]

Try (2) #6 \[ A_s = 0.88 \text{ in}^2 \]

\[ a = \frac{0.65(24)}{24} = 0.647 \]

\[ c = \frac{0.647}{0.85} = 0.741 \]

\[ \varepsilon_t = \frac{0.003}{0.741} (19.99 - 0.741) = 0.0758 > 0.005 \text{ - Tension Controlled} \]

\[ \phi M_n = [0.9(0.88)(24)(19.99 - \frac{0.647}{2})] / 12 \]

\[ = 74.57 \text{ ft.k} > 60.96 \text{ ft.k} \text{ - OK} \]

Use (2) #6 Bot. Reinforcement.
Joint Deflection

\[ J = \frac{9.5(72x16 + 2.25) + 18(6)(9)}{4.5(72) + 18(6)} \]
\[ J = \frac{14.19}{in.} \]

\[ I = \frac{72(4.5)^3}{12} + \left(72(4.5)(4.5)^2\right) \left(\frac{6(18)^3}{12} + 6(18)(7.14)^2\right) \]
\[ I = 5887 + 8499 \]
\[ I = 14386.2 \text{ in}^4 \]

\[ E = 33,522 \text{ ksi} \]
\[ 12 = 40 \text{ PSF} \]
\[ W_{DL} = 150 \text{ PLF} + 557.5 \text{ PLF} + 112.5 \text{ PLF} \]
\[ W_{DL} = 400 \text{ PLF} \]
\[ W_{TL} = 840 \text{ PLF} \]

\[ \Delta l = \frac{5(0.24)(35.5)\right)^4(728)}{384(384)(14386.2)} = 0.10^\prime \]
\[ \Delta l = \frac{5}{3600} = \frac{384(12)}{3600} = 1.26^\prime > 0.10^\prime \quad \text{OK} \]
\[ \Delta l = \frac{5(0.84)(35.5)\right)^4(728)}{384(384)(14386.2)} = 0.54^\prime \]
\[ \Delta l = \frac{5}{240} = \frac{384(12)}{240} = 1.9^\prime > 0.54^\prime \quad \text{OK} \]
Interior Girder Deflection

\[ \frac{y}{L} = \frac{22.5''}{2} = 11.25'' \]

\[ I = \frac{3\times (22.5)'^3}{12} = 34171.9 \text{ in}^4 \]

\[ E = 3834 \text{ ksi} \]

\[ W_{LU} = 40 \text{ PSF} (38') = 1.52 \text{ klf} \]
\[ W_{UL} = (25 \text{ PSF} (38')) + (\frac{4.5}{2} \times 100 (38')) + (24' (22.5'')(50))/144 = 3.93 \text{ klf} \]

\[ W_{TL} = 3.93 + 1.52 = 5.45 \text{ klf} \]

\[ \Delta_{UL} = \frac{5W_{UL}L^4}{384EI} = \frac{5(1.52)(10'')^4(1728)}{384(3834)(34171.9)} = 0.017'' \]

\[ \Delta_{UL} = \frac{5W_{UL}L^4}{384EI} = \frac{5(3.93)(10'')^4(1728)}{384(3834)(34171.9)} = 0.061'' \]

\[ \Delta_{TL} = \frac{5W_{TL}L^4}{384EI} = \frac{5(5.45)(10'')^4(1728)}{384(3834)(34171.9)} = 0.061'' \]

\[ \Delta_{TL} = \frac{16''(12)}{240} = 0.8'' > 0.061'' \therefore \text{OK} \]
## Appendix F: Cost Analysis

### 1. Precast Plank on Load Bearing Masonry & Steel Frame

\[
(1.009) \times [(\$9.7/\text{SF}) + (\$4.10/\text{SF})] = \text{\$14.01/\text{SF}}
\]

### 2. Precast Plank on Steel

\[
(1.009) \times [(\$8.8/\text{SF}) + (\$1.99/\text{SF})] = \text{\$10.89/\text{SF}}
\]
\[
(1.009) \times [(\$10.45/\text{SF}) + (\$4/\text{SF})] = \text{\$14.45/\text{SF}}
\]
\[
25.34/\text{SF}
\]

### 3. Composite Steel Deck

\[
(1.009) \times [(\$11.15/\text{SF}) + (\$5.05/\text{SF})] = \text{\$16.20/\text{SF}}
\]

### 4. One-Way Joist System

\[
(1.009) \times [(\$6.55/\text{SF}) + (\$10.30/\text{SF})] = \text{\$17.00/\text{SF}}
\]