Cambria Suites Hotel

Pittsburgh, PA

Technical Report 2

Structural Study of Alternate Floor Systems

Adam Kaczmarek | Structural
Professor Linda Hanagan
October 27, 2010

Table of Contents

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 7^{th} 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 2^{nd}-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 2^{nd} 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.

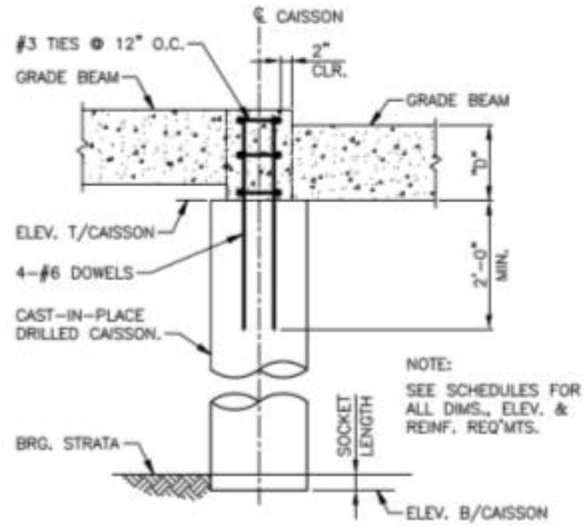
Structural System

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.

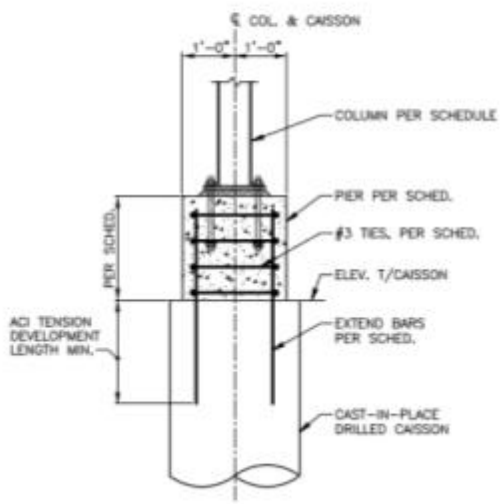
As a result of GeoMechanic's geotechnical study, the foundation of Cambria Suites Hotel incorporates a drilled cast-in-place concrete caissons and grade beams designed to support the load bearing walls and columns. The ground floor is comprised of a 4" concrete slab on grade, as well as, 10" precast 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*)

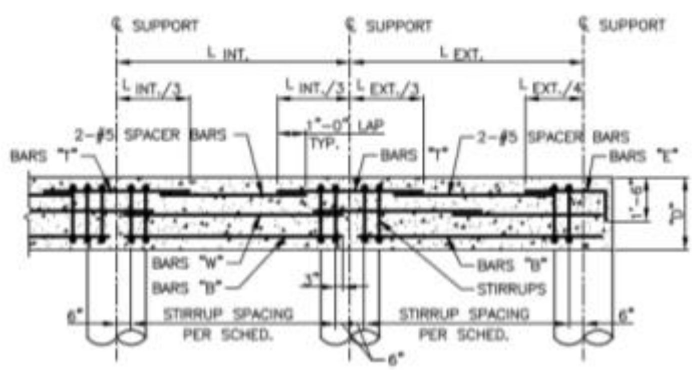


Typical Caisson & Grade Beam Detail
Figure 1.1

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*)



Typical Caisson Cap Detail
Figure 1.2

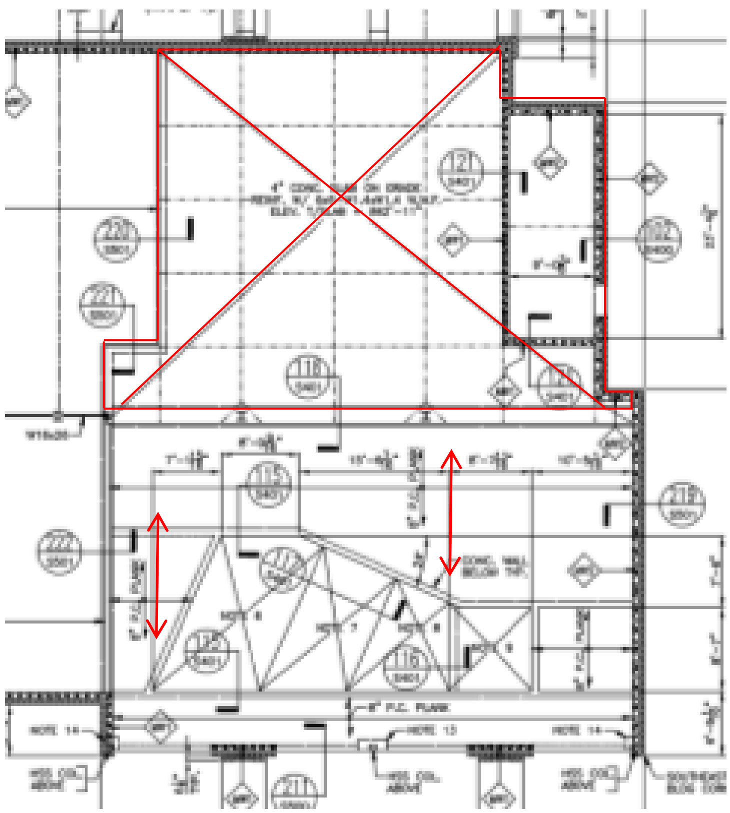


Typical Grade Beam Reinforcing Detail
Figure 1.3

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.

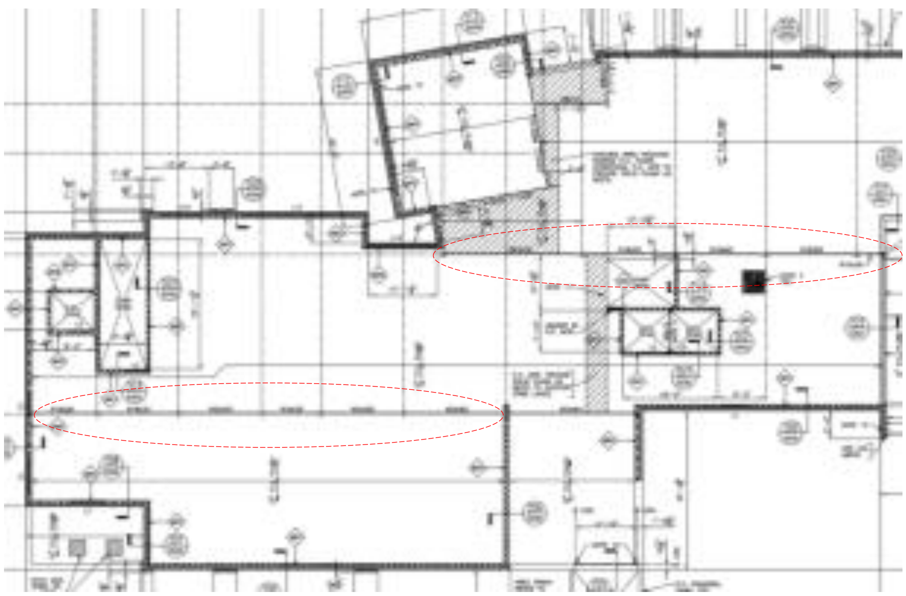
The Plaza level floor system is a combination of 10" precast concrete plank, 8" precast concrete plank and 4" slab on grade. Since there is no basement in the North-East section of the hotel due to the fitness center and pool, the site was excavated properly in order to place the 4" slab on grade and 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*)



Partial Hotel Level Floor Slab

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
Figure 2.2

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.

<u>Lateral Shear Wall System</u>

Figure 3.1

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 PSI
Grade Beam Foundations	f'_c = 3000 PSI
Slabs on Grade	f'_c = 4000 PSI
Walls	f'_c = 4000 PSI
Exterior Bar or Wire Reinforcement Slabs	f'_c = 5000 PSI

Reinforcement Steel

Deformed Bars	ASTM A615, Grade 60
Welded Wire Fabric	ASTM A185

Structural Steel

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

Masonry

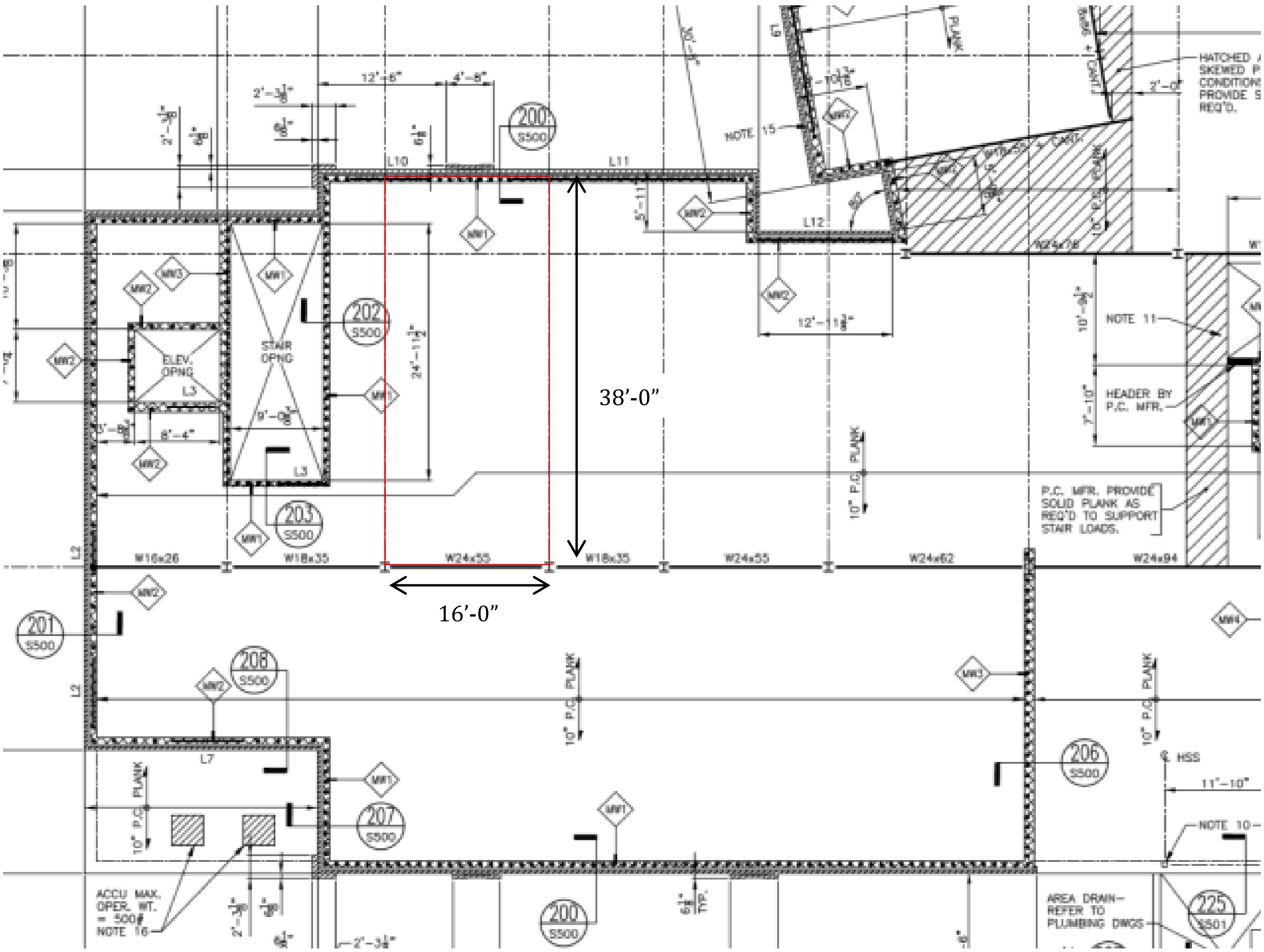
8” & 12” CMU	f'_m = 2000 PSI
Grout	f'_c = 3000 PSI

Design Load Summary

Live Loads (LL)			
Area	**AES Design Load (PSF)**	**ASCE 7-05 Load (PSF)**	**Design Load (PSF)**
Public Areas	100	100	100
Lobbies	100	100	100
First Floor Corridors	100	100	100
Corridors above First Floor	40	40	40
Private Hotel Rooms	40	40	40
Partitions	15	≥15	15
Mechanical	150	150	150
Stairs	100	100	100
Roof	20	20	20
Dead Loads (DL)			
Material	**AES Design Load (PSF)**	**ASCE 7-05 Load (PSF)**	**Design Load (PSF)**
10" Concrete Plank	Unknown	Section 3.1	91
8" Masonry Wall (Fully Grouted)	Unknown		91
8" Masonry Wall (Partially Grouted w/ Reinf. @ 24" O.C.)	Unknown		69
8" Masonry Wall (Partially Grouted w/ Reinf. @ 48" O.C.)	Unknown		60
Steel	Unknown		varies
Partitions	Unknown		15
MEP	Unknown		10
Finishes & Miscellaneous	Unknown		5
Roof	Unknown		20
***Snow Load (SL)**			
Area	**AES Design Load (PSF)**	**ASCE 7-05 Load (PSF)**	**Design Load (PSF)**
Flat Roof	21	21	21
*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*.



Typical Bay Used in the Analysis of the Existing and Alternate Floor Systems

Figure 4

Floor Systems

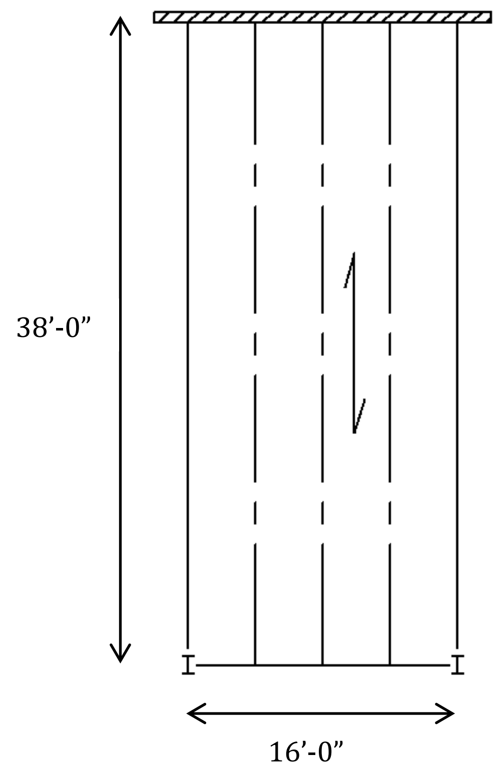
Existing: Precast Hollow-Core Concrete Plank on Load Bearing Masonry & Steel Interior

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



Existing Hollow-Core Plank
Figure 5.1

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") ∅, 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.

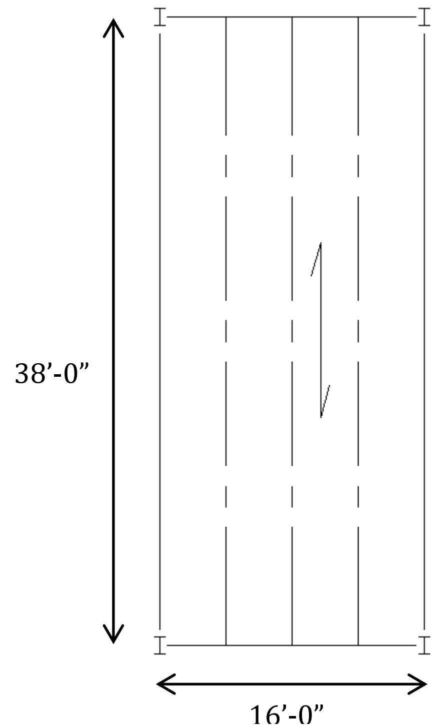
Alternative #1: Precast Hollow-Core Concrete Plank on Steel Framing

Material Properties

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

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

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



Hollow-Core Plank on Steel
Figure 5.2

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") Ø, 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 you 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.

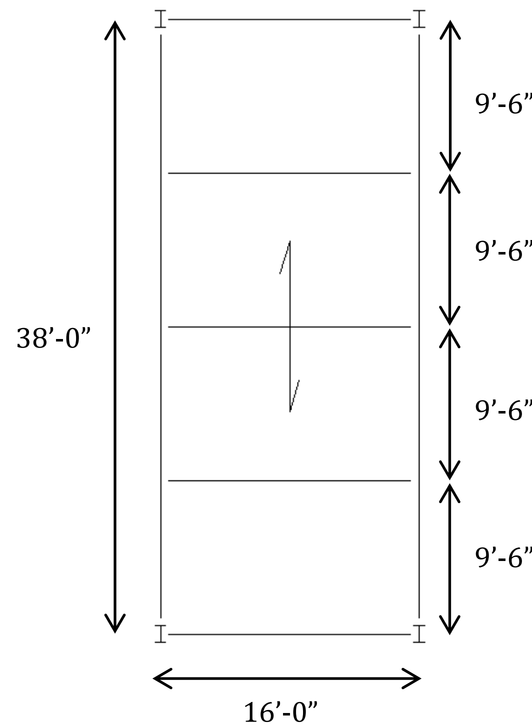
Feasibility

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



Composite Steel Deck
Figure 5.3

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

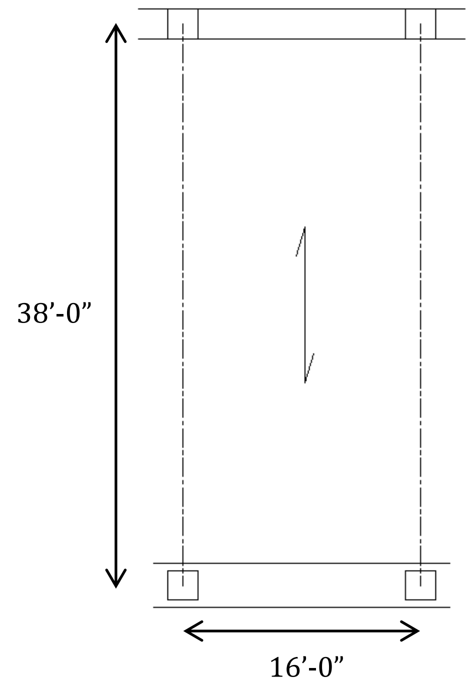
Although this system is the lightest self-weight and is efficient to construct, it still has several disadvantages. One disadvantage might be the floor system depth of 25.2” (20.7” depth of girder + 4.5” slab). This system depth would either adjust the entire height of the building, adding additional 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	
Concrete:	4.5” slab 66”/6” pan joists f'_c = 3000 PSI
Reinforcement:	f'_y = 60,000 PSI
Loadings:	DL =91.5 PSF LL = 40 PSF SDL = 25 PSF



One-Way Joist
Figure 5.4

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 quit 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

Comparison Criteria	Precast Plank on Load Bearing Walls & Steel Frame	Precast Plank on Steel Framing	Composite Steel Deck System	One-Way Joist System
Slab Self Weight	93 PSF	75 PSF	45 PSF	91.5 PSF
Slab Depth	10”	10”	4.5”	4.5”
System Depth	12” (10”+2” topping)	24.3”	25.2”	22.5”
Deflection (LL + DL)	Adequate with camber	0.193” < 0.8”	1.22” < 1.26”	.061” < 0.8”
Vibration	Average	Below Average	Good	Exceptional
Fire-Rating	2 Hour	2 Hour	1.5 – 2 Hour	2 Hour
Fire Protection	None	Minimal Spray	Spray	None
Impact on Building Design	Existing	Reduced floor-to-ceiling height	Reduced floor-to-ceiling height	Reduced floor-to-ceiling height
Constructability	Easy	Easy	Easy	Average
System Cost*	$14.01/SF	$25.34/SF	$16.95/SF	$17.00/SF
Feasibility	Yes	Yes	No	Yes

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

Conclusion

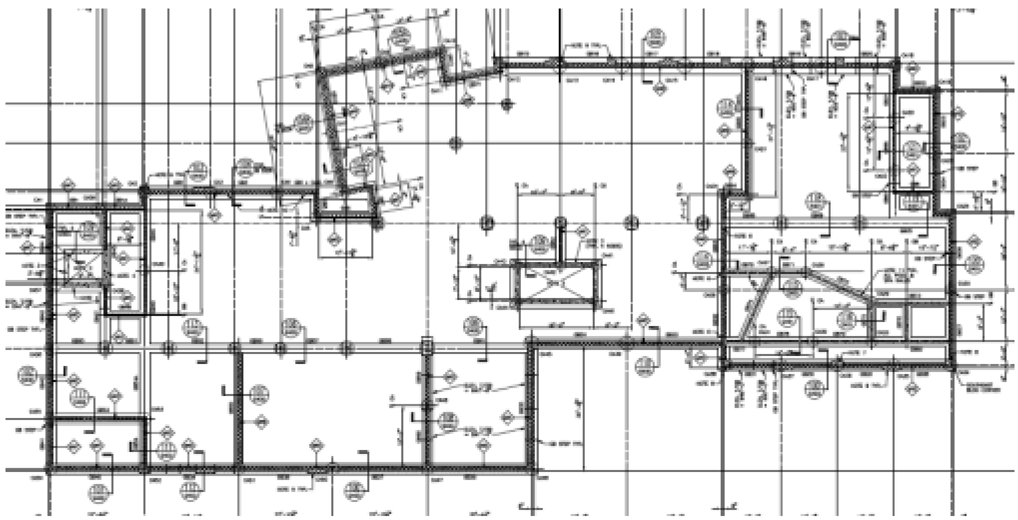
In the second technical report regarding the construction of the Cambria Suites Hotel, alternative floor systems were designed for a typical bay of 38’-0”x16’-0”. Each alternative floor system was compared to each other, as well as to the existing floor assembly. The existing floor system is a precast 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.

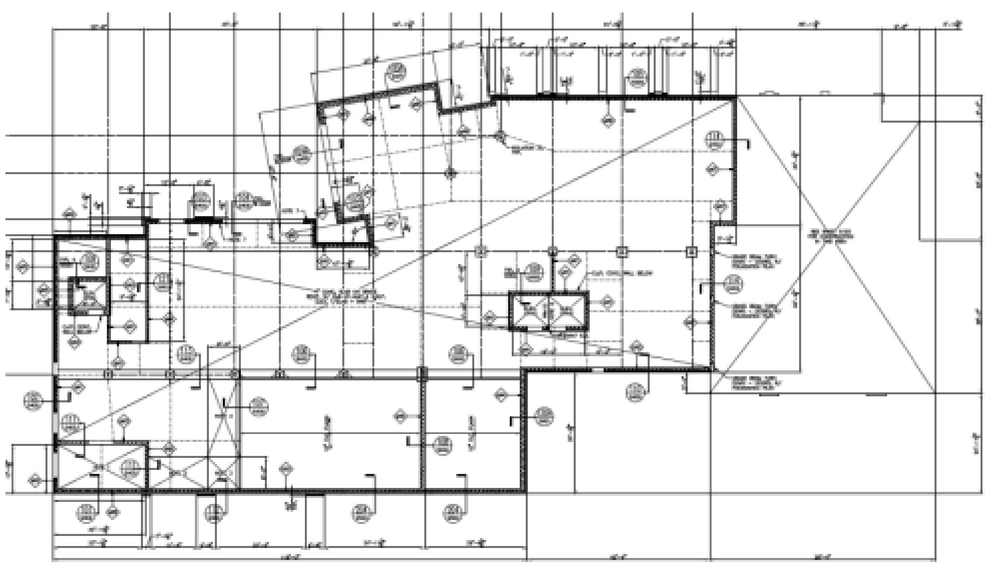
The most likely alternative system for the Cambria Suites Hotel, besides its existing system, is the one-way pan joist system. This system created the second thinnest overall floor system depth, as well as a fairly cheap system per square foot. One-way joist systems are the most economical concrete system 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.

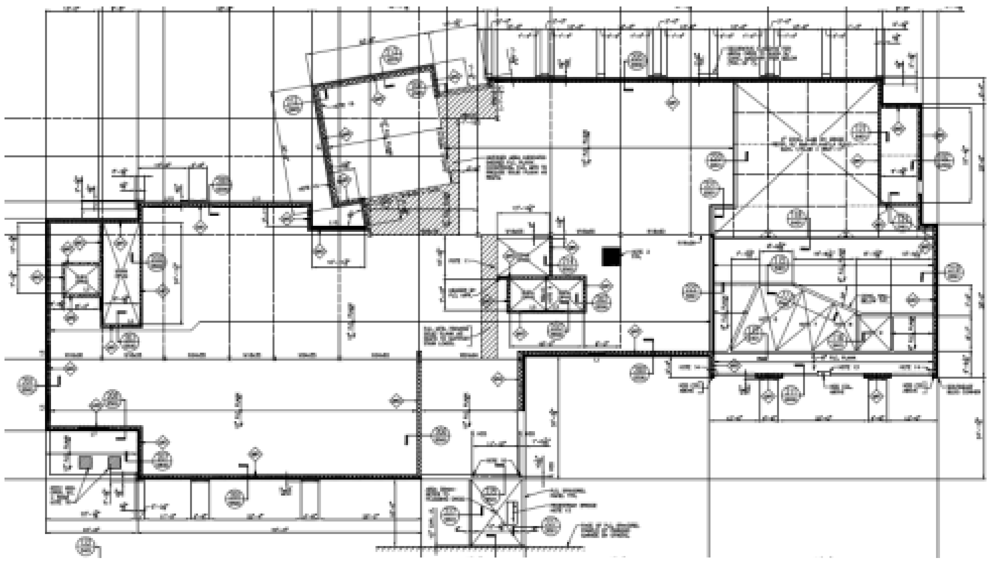
Appendix A: Building Layout



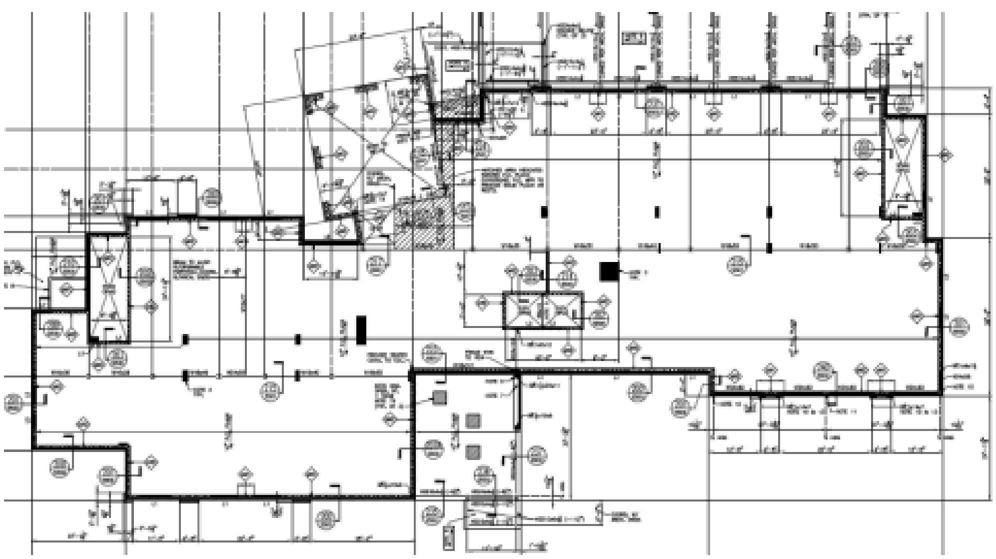
Foundation Plan



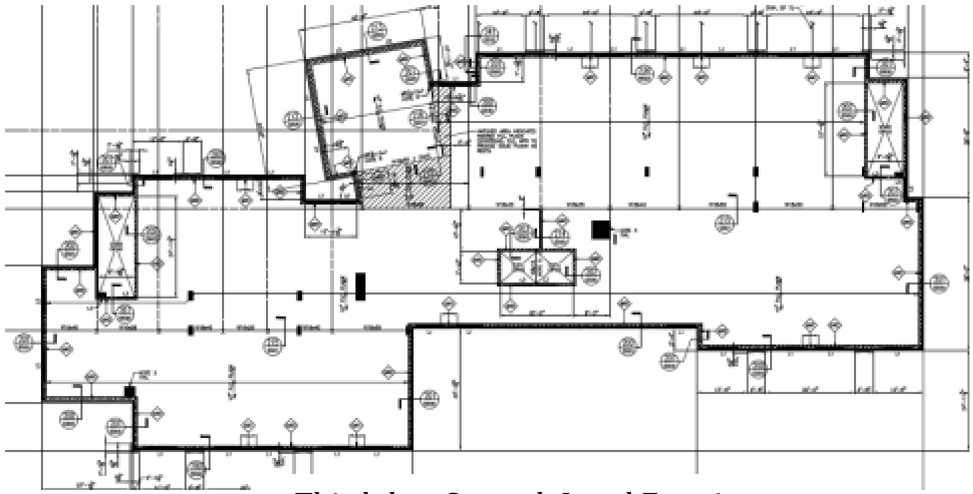
Plaza Level Framing Plan



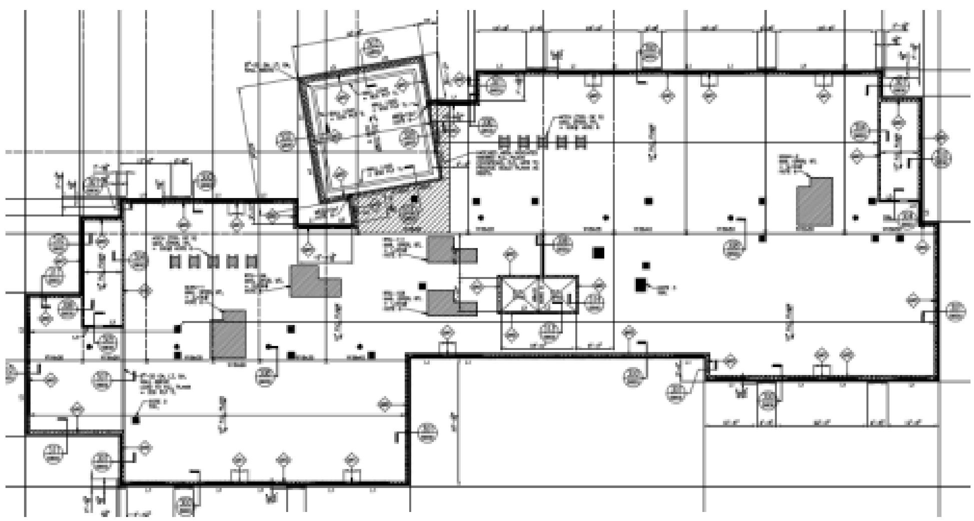
Hotel Level Framing Plan



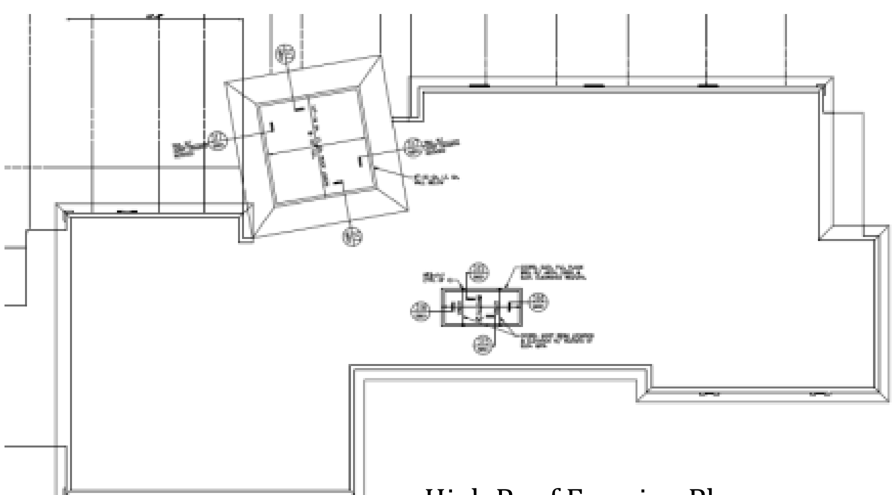
Second Level Framing Plan



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

Strand Pattern Designation
48-S

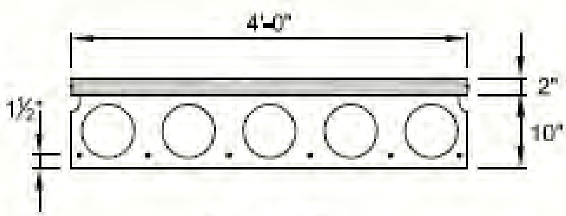
- S = straight
- Diameter of strand in 16ths
- No. of Strand (4)

Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Capacity of sections of other configurations are similar. For precise values, see local hollow-core manufacturer.

Key
258 – Safe superimposed service load, psf
0.3 – Estimated camber at erection, in.
0.4 – Estimated long-time camber, in.

HOLLOW-CORE
4'-0" x 10"
Normal Weight Concrete



$f'_c = 5{,}000$ psi
$f_{pu} = 270{,}000$ psi

Section Properties

		Untopped		Topped	
A	=	259	in.^{2}	355	in.^{2}
I	=	3,223	in.^{4}	5,328	in.^{4}
y_b	=	5.00	in.	6.34	in.
y_t	=	5.00	in.	5.66	in.
S_b	=	645	in.^{3}	840	in.^{3}
S_t	=	645	in.^{3}	941	in.^{3}
wt	=	270	plf	370	plf
DL	=	68	psf	93	psf
V/S	=	2.23	in.		

10" x 48" Hollowcore (2" Concrete Topping)
CLEAR SPAN IN FEET

Designation	14'	16'	18'	20'	22'	24'	26'	28'	30'	32'	34'	36'	38'	40'	42'	44'	46'	48'
T10S68-1.75	585	498	431	378	328	263	213	172	140	113	90	67	48	31	X	X	X	X
T10S78-1.75	601	512	443	389	344	308	256	210	173	142	114	89	67	49	33	X	X	X
T10S88-1.75	615	524	454	398	353	316	285	247	205	170	137	109	86	66	49	34	X	X
T10S98-1.75	630	537	465	409	363	325	292	265	235	193	158	129	104	82	64	48	34	X
T10S108-1.75	647	552	478	420	373	334	301	273	249	216	178	147	120	97	78	61	46	34

<u>Pittsburgh Flexicore Specifications</u>

TOPPED 2"

Ix	=	5576 IN^4
Sxtop	=	1013 IN^3
Sxbot	=	859 IN^3
AREA	=	345 IN^2
yb	=	6.5 IN
bw	=	12 IN

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650 Smithfield Street ▪ Suite 1200
Pittsburgh ▪ Pennsylvania 15222
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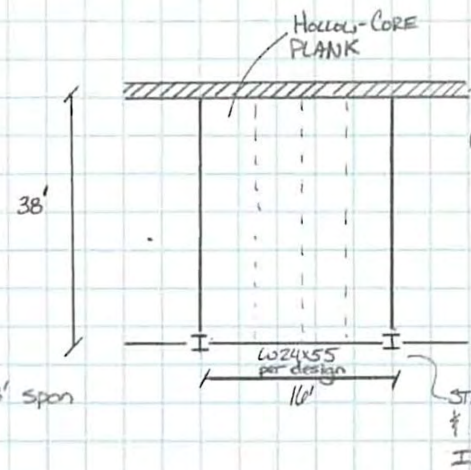
PRECAST HOLLOW-CORE CONCRETE PLANKS
ON LOAD BEARING MASONRY & STEEL INTERIOR

- LOADS
  - LL = 40 PSF (HOTEL ROOMS)
  - SDL = 25 PSF (MEP, PARTITIONS, FINISHES)
  - DL = 15 PSF (2" TOPPING ⇒ PCI HANDBOOK, 2-33)

TOTAL LOAD = 40 + 25 + 15 = 80 PSF

$f'_c = 5{,}000$ PSI
$f_{pu} = 270{,}000$ PSI
SPAN = 38'



- DESIGNED FOR 10" - w/ TOPPING
  4'-0" x 10" NWC (4HC10 + 2)

- FROM PITTSBURGH FLEXICORE
  T10S108 carrying 120 PSF capacity @ 38' span

10 strands @ 8/16" ϕ - straight → TAKEN FROM PCI, 2-33
SELF WEIGHT OF SLAB = 93 PSF → AES used 91 PSF to account for 1" topping

- LOAD TO MASONRY WALLS

$$w_u = 1.2(25 + 93) + 1.6(40) = 205.6 \text{ PSF}$$

$$M_u = \frac{205.6 \text{ PSF}(16'\text{-}0'')(38')^2}{8} = 593.8 \text{ ft·k} \Rightarrow 594 \text{ ft·k}$$

HOLLOW - CORE PLANK (cont.)

$A_{ps} = 10$ strands @ $8/16"$ ϕ
$= 10(0.5)$
$= 5.0 \text{ in}^2$

$f_{ps} = 270$ ksi
$b = 4'\text{-}0"(12) = 48"$
$d_p = 12" - 1\frac{1}{2}"$ CLR $= 10.5"$

$$a = \frac{A_{ps} f_{ps}}{0.85 f'_c b} = \frac{5.0(270 \text{ ksi})}{0.85(5 \text{ ksi})(48")} = 6.62 \text{ in.}$$

$$\phi M_n = \phi\left[A_{ps} f_{ps}\left(d_p - a/2\right)\right]$$
$$= 0.9\left[5.0(270)\left(10.5 - 6.62/2\right)\right]$$
$$= 8736 \text{ in.k} = 728 \text{ ft.k}$$

$\phi M_n = 728$ ft.k > 594 ft.k $= M_u$. $\therefore$ OK DESIGN

• DEFLECTION

$E_c = 57{,}000\sqrt{f'_c} = 57000\sqrt{5000}$
$E_c = 4030$ ksi
$I = 5576 \text{ in}^4$ (PITTSBURGH FLEXICORE)

$$\Delta_{LL} = l/360 = \frac{38'(12)}{360} = 1.267 \text{ in.}$$

$$\Delta_{LL} = \frac{5(40)(16')(38)^4}{384(4030000)(5576)} \times 1728 = 1.336 \text{ in.} > 1.267 \text{ in}$$

* However, P.C. Plank has an estimated camber of 1.2" (88-S in PCI), which will result in an acceptable deflection.

Appendix C: Alternative System #1

Precast Hollow-Core Concrete Plank on Steel Framing

Strand Pattern Designation
48-S

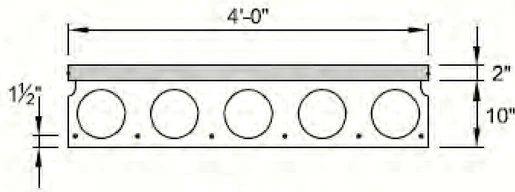
S = straight
Diameter of strand in 16ths
No. of Strand (4)

Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Capacity of sections of other configurations are similar. For precise values, see local hollow-core manufacturer.

Key
258 – Safe superimposed service load, psf
0.3 – Estimated camber at erection, in.
0.4 – Estimated long-time camber, in.

HOLLOW-CORE
4′-0″ x 10″
Normal Weight Concrete



f'_c = 5,000 psi
f_{pu} = 270,000 psi

Section Properties

	Untopped	Topped
A =	259 in.^{2}	355 in.^{2}
I =	3,223 in.^{4}	5,328 in.^{4}
y_b =	5.00 in.	6.34 in.
y_t =	5.00 in.	5.66 in.
S_b =	645 in.^{3}	840 in.^{3}
S_t =	645 in.^{3}	941 in.^{3}
wt =	270 plf	370 plf
DL =	68 psf	93 psf
V/S =	2.23 in.	

4HC10

Table of safe superimposed service load (psf) and cambers (in.) **No Topping**

Strand Designation Code	Span, ft																										
	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44	45	46
48-S	258	234	209	187	168	151	136	123	111	100	90	82	74	66	60	54	48	43	38	34	30	26					
	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.4	–0.6	–0.7	–0.9					
	0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.3	–0.5	–0.7	–0.8	–1.1	–1.3	–1.3	–1.9					
58-S	267	249	237	223	211	197	179	162	148	134	122	112	102	93	85	77	70	64	58	53	48	43	39	35	30	26	
	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.4	0.4	0.3	0.2	0.2	0.1	0.0	–0.1	–0.3	–0.4	–0.6	–0.7	–0.9	–1.2	
	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.4	0.3	0.2	0.1	0.0	–0.1	–0.3	–0.5	–0.7	–1.0	–1.2	–1.5	–1.8	–2.2	–2.6	
68-S	273	255	243	229	217	206	196	187	176	162	153	141	129	118	109	100	92	84	78	71	65	60	54	49	44	39	34
	0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.5	0.4	0.3	0.2	0.1	–0.1	–0.2	–0.4	–0.6	–0.8
	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.9	0.9	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.2	0.1	–0.1	–0.3	–0.6	–0.8	–1.1	–1.4	–1.8	–2.2
78-S	282	264	249	235	223	212	202	193	185	174	165	153	144	136	129	119	113	104	96	89	82	76	69	63	57	52	47
	0.6	0.7	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	0.9	1.0	1.0	1.0	0.9	0.9	0.9	0.8	0.8	0.7	0.6	0.5	0.4	0.3	0.1	0.0	–0.2
	0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.0	1.0	0.9	0.8	0.6	0.5	0.3	0.1	–0.1	–0.4	–0.7	–1.0	–1.3
88-S	288	270	255	241	229	218	208	199	188	180	174	165	153	145	135	128	122	115	106	101	96	91	84	77	71	65	59
	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.1	1.1	1.0	0.9	0.8	0.7	0.5	0.3
	1.0	1.0	1.1	1.2	1.2	1.3	1.3	1.4	1.4	1.4	1.4	1.5	1.5	1.4	1.4	1.4	1.3	1.2	1.2	1.0	0.9	0.7	0.6	0.3	0.1	–0.2	–0.5

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JOB TECH. REPORT 2 - CALCULATIONS
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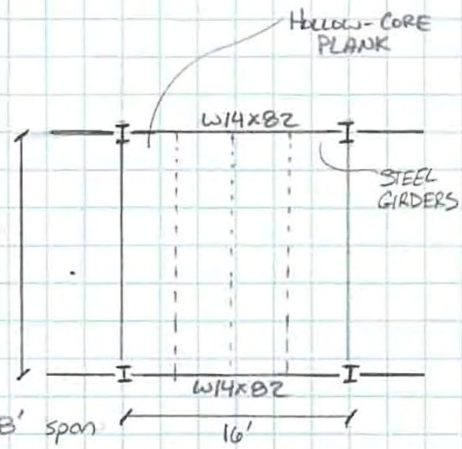
PRECAST HOLLOW-CORE CONCRETE PLANK
ON STEEL FRAMING.

- LOADS
  - LL = 40 PSF (HOTEL ROOMS)
  - SDL = 25 PSF (MEP, PARTITIONS, FINISHES)
  - DL = 10 PSF (w/o TOPPING ⇒ PCI HANDBOOK)

TOTAL LOAD = 40 + 25 + 10 = 75 PSF

$f'_c = 5000$ PSI
$f_{pu} = 270{,}000$ PSI
SPAN = 38'



- DESIGNED FOR 10" w/o TOPPING
  4'-0" × 10" NWC (4HC10)

- FROM PCI HANDBOOK
  - 78-S carrying 96 PSF capacity @ 38' span
  - 0.8" estimated camber at erection
  - 0.6" estimated long time camber
  - 7 strands @ 8/16" φ - straight
  - SELF WEIGHT OF SLAB = 68 PSF

- GIRDERS

$$w_u = 1.2(25 + 68) + 1.6(40) = 175.6 \text{ PSF}$$

$$M_u = \frac{175.6 \text{ PSF}(16'\text{-}0'')(38')^2}{8} = \underline{\underline{507 \text{ ft.k}}}$$

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HOLLOW-CORE PLANK ON STEEL (cont.)

• USE W21 x 62 (AISC TABLE 3-2)

$\phi M_n = 540$ ft.k > 507 ft.k = M_u ∴ OK

$\Delta_{LL} = \ell/360 = \frac{16'(12)}{360} = 0.53''$

$0.53 = \frac{5(40)(38')(16')^4(1728)}{384(29000)\,I_x\,(1000)} \Rightarrow I_x = 145.8\text{ in}^4 < 1330\text{ in}^4$ for W21x62 ∴ OK

$\Delta_{TL} = \frac{5(40+25+68)(38)(16)^4(1728)}{384(29000)(1330)(1000)} = 0.193''$

$\Delta_{TL} = 0.193'' < \ell/240 = \frac{16(12)}{240} = 0.80''$ ∴ OK

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

• USE W14 x 82 (less economical, but decreases system depth)

$\phi M_n = 521$ ft.k > 507 ft.k = M_u ∴ OK

$\Delta_{LL} = \ell/360 = \frac{16'(12)}{360} = 0.53''$

$0.53 = \frac{5(40)(38')(16')^4(1728)}{384(29000)(I_x)(1000)} \Rightarrow I_x = 145.8\text{ in}^4 < 881\text{ in}^4$ for W14x82 ∴ OK

$\Delta_{TL} = \frac{5(40+25+68)(38)(16)^4(1728)}{384(29000)(881)(1000)} = 0.292''$

$\Delta_{TL} = 0.292'' < \ell/240 = \frac{16(12)}{240} = 0.80''$ ∴ OK

USE W14 x 82

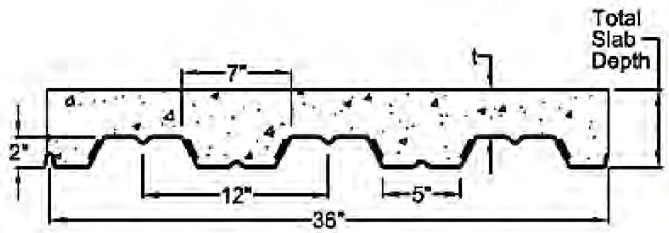
Appendix D: Alternative System #2

Composite Steel Deck System

VULCRAFT

2 VLI

Maximum Sheet Length 42'-0
Extra Charge for Lengths Under 6'-0
ICBO Approved (No. 3415)



Interlocking side lap is not drawn to show actual detail.

STEEL SECTION PROPERTIES

Deck Type	Design Thickness in	Deck Weight psf	Section Properties I_p in^4/ft	S_p in^3/ft	I_n in^4/ft	S_n in^3/ft	V_a lbs/ft	F_y ksi
2VLI22	0.0295	1.62	0.324	0.263	0.321	0.266	1832	50
2VLI20	0.0358	1.97	0.409	0.341	0.406	0.346	2698	50
2VLI19	0.0418	2.30	0.492	0.420	0.489	0.426	3190	50
2VLI18	0.0474	2.61	0.559	0.495	0.558	0.504	3608	50
2VLI16	0.0598	3.29	0.704	0.653	0.704	0.653	3618	40

(N=9.35) NORMAL WEIGHT CONCRETE (145 PCF)

COMPOSITE

TOTAL SLAB DEPTH	DECK TYPE	SDI Max. Unshored Clear Span 1 SPAN	2 SPAN	3 SPAN	Superimposed Live Load, PSF Clear Span (ft.-in.) 5'-6	6'-0	6'-6	7'-0	7'-6	8'-0	8'-6	9'-0	9'-6	10'-0	10'-6	11'-0	11'-6	12'-0	12'-6
4.00 (t=2.00) 39 PSF	2VLI22	7'-4	9'-6	9'-9	274	239	211	188	145	129	115	104	94	85	78	71	65	59	54
	2VLI20	8'-7	10'-10	11'-2	310	269	236	210	188	170	155	117	106	96	87	80	73	67	61
	2VLI19	9'-9	11'-11	12'-4	344	298	261	231	207	186	169	155	142	106	97	88	81	74	68
	2VLI18	10'-9	12'-9	12'-9	373	324	285	253	228	206	188	172	159	147	137	103	95	87	81
	2VLI16	11'-1	13'-2	13'-5	400	376	330	292	261	235	214	195	180	166	154	143	109	100	93
4.50 (t=2.50) 45 PSF	2VLI22	6'-11	9'-0	9'-4	319	278	245	190	168	150	134	121	109	99	90	83	76	69	63
	2VLI20	8'-2	10'-3	10'-7	361	313	275	244	219	198	152	136	123	112	102	93	85	78	72
	2VLI19	9'-2	11'-5	11'-9	400	346	303	268	240	216	196	180	136	124	113	103	94	86	79
	2VLI18	10'-2	12'-4	12'-4	400	376	331	295	264	239	218	200	184	171	130	119	110	102	94
	2VLI16	10'-5	12'-6	12'-11	400	400	383	339	303	274	248	227	209	193	150	137	126	117	108

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JOB TECH. REPORT 2 - CALCULATIONS
SHEET NO. 1 OF 5
CALCULATED BY A. KACZMAREK DATE
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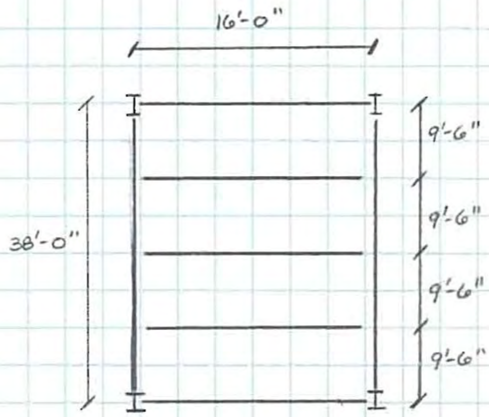
SCALE

COMPOSITE STEEL DECK

- LOADS
  - LL = 40 PSF (HOTEL ROOMS)
  - SDL = 25 PSF (PART., MEP., FINISHES)
  - DL = 45 PSF (DECK SELFWEIGHT)



- VULCRAFT DECK USED
  - slab depth = 4.5"
  - topping = 2.5"
  - N.W.C. (145 PCF), N = 9.35
  - 3 Span Condition
  - Use : 2VLI20 DECK
  - f'_c = 3000 PSI
  - $f_{y,STEEL}$ = 60,000 PSI

- TOTAL LOAD = LL + SDL + DL = 104 PSF

- DECK USED : 2VLI20 DECK, 3SPAN (VULCRAFT DECK CATALOG, pg. 52-53)
  - CLEAR SPAN = 9'-6"
  - 20 GAUGE
  - SUPERIMPOSED LL MAX. CAPACITY = 123 PSF > 104 PSF ∴ OK
  - F_b = 30,000

- BEAM :

$$\text{LOAD} = 1.2D + 1.6L$$
$$= 1.2(25+45) + 1.6(40)$$
$$= 148 \text{ PSF or } 0.148 \text{ KSF}$$

TRIB. LENGTH = 9'-6" or 9.5'

$$w_u = 9.5'(0.148 \text{ KSF}) = 1.406 \text{ klf}$$

$$V_u = \frac{1.406(16')}{2} = 11.3^k$$

$$M_u = \frac{1.406(16')^2}{8} = 45.0 \text{ ft·k}$$

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JOB TECH. REPORT 2 - CALCULATIONS
SHEET NO. 2 OF 5
CALCULATED BY A. KACZMAREK DATE
CHECKED BY DATE
SCALE

COMPOSITE STEEL DECK (cont.)

$$b_{eff.} = \min \begin{cases} 2\left(\frac{SPAN}{8}\right) = 2\left(\frac{16'(12)}{8}\right) = \boxed{48''} \\ 2\left(\frac{1}{2}\right) SPACING = 9.5'(12) = 114'' \end{cases}$$

ASSUME $a = 1.0$

$Y_2 = t_{slab} - a/2 = 4.5'' - 1/2 = 4.0''$

$Q_n = 17.2\ k$ for 3 ksi NWC w/ deck perpendicular

TRY W10 x 12 - $\phi M_n = 73.2$ ft·k, $\phi_b M_p = 47.3$ ft·k, PNA = 7, $\Sigma Q_n = 44.2\ k$

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff.}} = \frac{44.2}{0.85(3)(48)} = 0.361'' < 1'' \quad \therefore OK$$

$$Y_2 = 4.5 - \frac{0.361}{2} = 4.32'' > 4.0'' \quad \therefore \text{CONSERVATIVE}$$

$$\text{SHEAR STUDS} \Rightarrow \frac{44.2}{17.2} = 2.57 \rightarrow 3\ \text{STUDS/HALF} = 6\ \text{STUDS}$$

CHECK UNSHORED STRENGTH

$C_{LL} = 20\ PSF(9.5') = 0.190\ klf$

$W_{LL} = C_{LL} = 0.190\ klf$

$W_{DL} = (45\ PSF)(9.5') + 12\ plf = 0.4395\ klf$

$W_u = 1.2(0.4395) + 1.6(0.19) = 0.8314\ klf$

$$M_u = \frac{W_u l^2}{8} = \frac{0.8314(16')^2}{8} = 22.6\ ft{\cdot}k < 47.3\ ft{\cdot}k = \phi_b M_p$$

CHECK MEMBER STRENGTH

$\phi M_n = 73.2\ ft{\cdot}k > 45\ ft{\cdot}k = M_u$

$\phi V_n = 56.3\ k > 11.3\ k = V_u$

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JOB TECH, REPORT 2 – CALCULATIONS
SHEET NO. 3 OF 5
CALCULATED BY A. KACZMAREK DATE
CHECKED BY DATE
SCALE

COMPOSITE STEEL DECK (cont.)

CHECK LL DEFLECTION

$w_{LL} = 40\text{ PSF}(9.5') = 0.38\text{ klf}$

$I_{LB} = 117\text{ in}^4$

$$\Delta_{LL} = \frac{5wl^4}{384EI} = \frac{5(0.38)(16^4)(1728)}{384(29000)(117)} = 0.165''$$

$$l/360 = \frac{16'(12)}{360} = 0.533'' > 0.165'' \therefore \text{OK}$$

CHECK WET CONCRETE DEFLECTION

$w_{wc} = 45\text{ PSF}(9.5') + 12\text{ PLF} = 0.4395\text{ klf}$

$I_x = 53.8\text{ in}^4$

$$\Delta_{wc} = \frac{5wl^4}{384EI} = \frac{5(0.4395)(16)^4(1728)}{384(29000)(53.8)} = 0.415''$$

$$\Delta_{wc,max} = l/240 = 16'(12)/240 = 0.8'' > 0.415'' \therefore \text{OK}$$

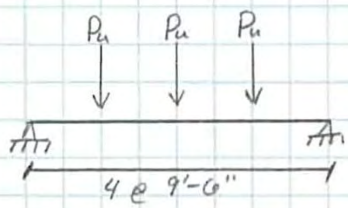
$\boxed{\text{USE } W10\times12\ (6)}$

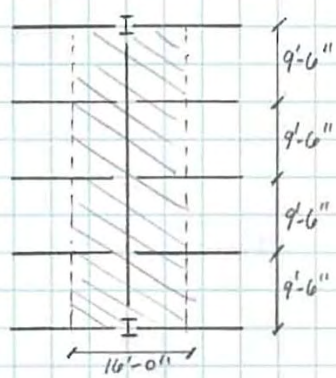
• GIRDERS

TOTAL DL $= (45\text{ PSF} + 25\text{ PSF})(9.5') = 0.665\text{ klf}$ } FROM BEAMS
TOTAL LL $= 40\text{ PSF}(9.5') = 0.38\text{ klf}$

TOTAL $DL_{,GIRDER} = 0.665\text{ klf}(16') = 10.64^{k}$

TOTAL $LL_{,GIRDER} = 0.38\text{ klf}(16') = 6.08^{k}$





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SHEET NO. 4 OF 5
CALCULATED BY A. KACZMAREK DATE
CHECKED BY DATE
SCALE

COMPOSITE STEEL DECK (cont.)

$P_u = 1.2D + 1.6L = 1.2(10.64) + 1.6(6.08) = 22.496^k$
$V_u = P_u = 22.496\ k$
$M_u = 9.5'(22.496^k) = 213.7\ ft\cdot k$

$b_{eff} = 2\left(\frac{SPAN}{8}\right) = 2\left(\frac{38(12)}{8}\right) = \boxed{114''}$

$2(\tfrac{1}{2})\ SPACING = 16'(12) = 192''$

ASSUME $a = 1.0$
$Y_2 = 4.5 - \tfrac{1}{2} = 4.0''$
$Q_n = 21.0^k$ for 3 ksi NWC w/ deck parallel

TRY W12x30 — $\phi M_n = 228\ ft\cdot k$, $\phi_b M_p = 162\ ft\cdot k$, PNA=7, $\Sigma Q_n = 110^k$

$a = \frac{110^k}{0.85(3)(114)} = 0.378'' < 1.0''$ ∴ OK

$Y_2 = 4.5 - \frac{0.378}{2} = 4.311'' > 4.0''$ ∴ CONSERVATIVE

SHEAR STUDS ⇒ $\frac{110}{21.0} = 5.24 \rightarrow 6$ STUDS/HALF ⇒ 12 STUDS

CHECK UNSHORED STRENGTH
steel weight
$w_u = 1.2D + 1.6L = 1.2(0.03) = 0.036\ klf$
$C_{LL} = 20PSF(9.5')(16') = 3.04^k$
$C_{DL} = [45PSF(9.5') + 12\ PIF](16') = 7.032^k$

$P_u = 1.2D + 1.6L = 1.2(7.032) + 1.6(3.04) = 13.3^k$
$V_u = P_u = 13.3^k$
$M_u = \frac{w_u l^2}{8} + \frac{P_u l}{4} = \frac{0.036(38')^2}{8} + \frac{13.3(38')}{4} = 132.8\ ft\cdot k$

$M_u = 132.8\ ftk < 162\ ft\cdot k = \phi_b M_p$ ∴ OK

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SHEET NO. 5 OF 5
CALCULATED BY A. KACZMAREK DATE
CHECKED BY DATE
SCALE

COMPOSITE STEEL DECK (cont.)

CHECK MEMBER STRENGTH

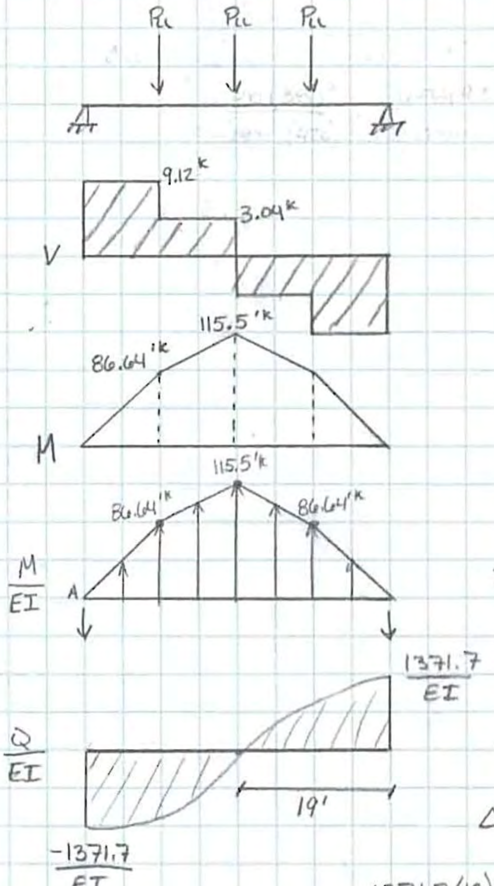
$\phi M_n = 228 \text{ ft.k} > 213.7 \text{ ft.k} = M_u$

$\phi V_n = 96.3 \text{ k} > 13.3 \text{ k} = V_u$

CHECK LL DEFLECTION

$P_{LL} = 6.08 \text{ k}$

$I_{LB} = 420 \text{ in}^4$



$$\frac{(86.64)(9.5)}{2} + \frac{(115.5 - 86.64)(9.5)}{2} + 86.64(9.5)$$

$$\frac{Q_A}{EI} = 411.54 + 137.1 + 823.1 = \frac{1371.7}{EI}$$

$$\Delta_{LL} = \frac{(1371.7)(19')(2/3)(1728)}{29000(420)} = 2.46''$$

$$\Delta_{LL,MAX} = l/360 = \frac{38'(12)}{360} = 1.26'' \quad \therefore \text{NOT OK}$$

$$I_{REQ.} = \frac{1371.7(19)(2/3)(1728)}{(29000)(1.26'')} = 821.7 \text{ in}^4$$

$\therefore$ $\boxed{\text{USE W21x44 (12)}}$, $\phi_b M_p = 398 \text{ ft-k}$, $I_x = 843 \text{ in}^4$

Appendix E: Alternative System #3

One-Way Joist System

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SHEET NO. 1 OF 10
CALCULATED BY A.KACZMAREK DATE
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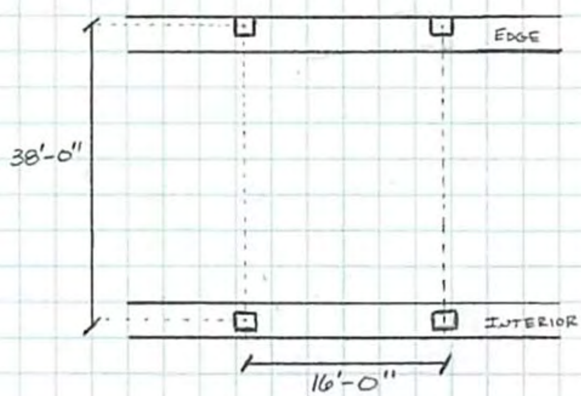
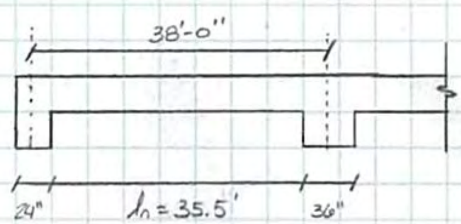
ONE-WAY JOIST SYSTEM

ASSUME: NWC (150 PCF)
$f'_c = 4$ ksi
$f_y = 60$ ksi



Edge beam width = 24"
Interior beam width = 36"

6" wide joists spaced 66" o.c.



PRELIMINARY PAN DEPTH : 18" for a 20x40 bay size, 66" Pan (PCA Handout)

- SLAB
  - 4 ½" thick slab w/ 2-hr. fire rating
  - $w_{SDL} = 25$ PSF
  - $w_{LL} = 40$ PSF
  - $w_{DL} = (4.5/12)(150\ PCF) = 56.25$ PSF
  - $w_u = 1.2(56.25 + 25) + 1.6(40) = 161.5$ PSF
  - * For 1' strip ⇒ $w_u = (1')(161.5) = 161.5$ PLF or 0.1615 klf

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JOB: TECH. REPORT 2 - CALCULATIONS
SHEET NO. 2 OF 10
CALCULATED BY: A. KACZMAREK DATE
CHECKED BY DATE
SCALE

$M_u = \frac{w_u l_n^2}{10} = \frac{0.1615(66''/12)^2}{10} = 0.489$ k·ft / ft. of slab

MIN. REINFORCEMENT

$A_{s,req.} = 0.0018\,(4.5)(12) = 0.0972$ in²

Try #3 bars ⇒ $A_s = 0.11$ in²

(1) #3 = 0.11 in² > 0.0972 in²

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.11\,(60)}{0.85\,(4)(12'')} = 0.162''$

$\phi M_n = 0.9 A_s f_y (d - a/2) = 0.9\,(0.11)(60)\left(\frac{4.5}{2} - \frac{0.162}{2}\right) = 1.07$ ft·k / ft. slab
> 0.489 ft·k / ft. slab
∴ OK

SPACING

$3t = 3(4.5'') = 13.5''$ ⇒ USE 12''

$\boxed{\therefore (1)\ \#3\ @\ 12''\ \text{o.c.}}$

• JOIST

$w_{SDL} = 25\ \text{PSF}\,(6') = 150$ PLF

$w_{SLAB} = (4.5''/12)(150\ \text{PCF})(6') = 337.5$ PLF

$w_{SELF} = (18'')(6'')(150\ \text{PCF}) / 144 = 112.5$ PLF

$w_{LL} = 40\ \text{PSF}\,(6') = 240$ PLF

$w_u = 1.2\,(0.150 + 0.3375 + 0.1125) + 1.6\,(0.24)$
$= 1.07$ klf

$M^+_{max} = \frac{w_u l_n^2}{14} = \frac{1.07\,(35.5)^2}{14} = 96.3$ ft·k

$M^-_{max} = \frac{w_u l_n^2}{10} = \frac{1.07\,(35.5)^2}{10} = 134.8$ ft·k

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SHEET NO. 3 OF 10
CALCULATED BY A. KACZMAREK DATE
CHECKED BY DATE
SCALE

TOP REINFORCEMENT

$A_s = M_u/4d = 134.8/4(20.25) = 1.66\ in^2$ $\quad * d = 22.5 - \left(\frac{4.5''}{2}\right) = 20.25''$

Try (2) #9 $\Rightarrow A_s = 2(1.0) = 2.0\ in^2$

$\rho = A_s/bd = 2.0/6''(20.25) = 0.0165$

$a = A_s f_y / 0.85 f'_c b = 2.0(60)/0.85(4)(6) = 5.88''$

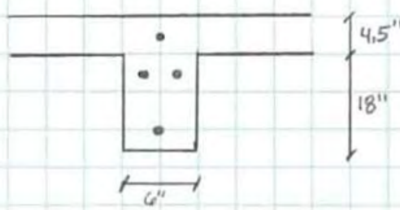
$c = a/\beta = 5.88/0.85 = 6.92''$

$\varepsilon_s = \frac{0.003}{c}(d-c) = \frac{0.003}{6.92}(20.25 - 6.92) = 0.00577 > 0.005$

$\therefore$ Tension Controlled, $\phi = 0.9$

$\phi M_n = \phi A_s f_y (d - a/2)$
$= \left[0.9(2.0)(60)\left(20.25 - \frac{5.88}{2}\right)\right] / 12$
$= 155.79\ ft{\cdot}k > 134.8\ ft{\cdot}k \quad \therefore$ OK

$\boxed{\text{USE (2) \# 9}}$



BOTTOM REINFORCEMENT

$d = 22.5'' - 1.5'' - 0.375'' - 0.635 = 19.99''$
COVER #3 STIRRUP #10 BAR

$A_s = M_u/4d = 96.3/4(19.99) = 1.2\ in^2$

Try (1) #10 $\Rightarrow A_s = 1.27\ in^2$

$\rho = A_s/bd = 1.27/6(19.99) = 0.0106$

$a = A_s f_y/0.85 f'_c b = 1.27(60)/0.85(4)(72) = 0.311$

$b_{eff} = $ | $\frac{1}{4}L = \frac{1}{4}(38)(12) = 114$
| $16h_s + b_w = 16(4.5) + 6 = 78$
MIN | $b_w + L_n = 6 + 66 = \boxed{72}$

$\therefore$ NA is in Flange

$c = 0.311/0.85 = 0.366$

$\varepsilon_s = \frac{0.003}{0.366}(20.25 - 0.366) = 0.163 > 0.005$ $\quad \therefore$ Tension Controlled, $\phi = 0.9$

$$\phi M_n = \phi A_s f_y (d - a/2)$$
$$= [0.9(1.2)(60)(20.25 - \tfrac{0.311}{2})] / 12$$
$$= 108 \text{ ft·k} > 96.3 \text{ ft·k} \quad \therefore \text{OK}$$

$$\boxed{\text{USE (1) \# 10}}$$

SHEAR DESIGN

$$V_u = 1.15 w_u l_n / 2 = 1.15(1.07)(35.5)/2 = 21.8 \text{ k}$$

$$\phi V_c = \phi 2 \sqrt{f'_c} b_w d = 0.75(2)\sqrt{4000}(6)(20.25'')/1000 = 11.5 \text{ k}$$

$$\phi V_s = V_u - \phi V_c = 21.8 - 11.5 = 10.3 \text{ k}$$

$$\phi V_s = 10.3 \text{ k} = \frac{\phi A_{sv} f_y d}{S_{max}}$$

$$10.3 = \frac{A_{sv}(60)(20.25)(0.75)}{8''}$$

$$* S_{max} = \min \begin{cases} d/2 = \frac{20.25}{2} = 10.125'' \\ 24'' \end{cases} \Rightarrow \boxed{\text{USE } 8''}$$

$$\Rightarrow A_{sv} = 0.09 \text{ in}^2$$

$$\boxed{\therefore \text{use \#3 @ } 8'' \text{ spacing}}$$

$$\phi V_c + \phi V_s = 11.5^k + \frac{0.75(0.11)(60)(20.25)}{6} = 28.2^k > 21.8^k = V_u$$

$\therefore$ OK

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JOB TECH. REPORT 2 – CALCULATIONS
SHEET NO. 5 OF 10
CALCULATED BY A. KACZMAREK DATE
CHECKED BY DATE
SCALE

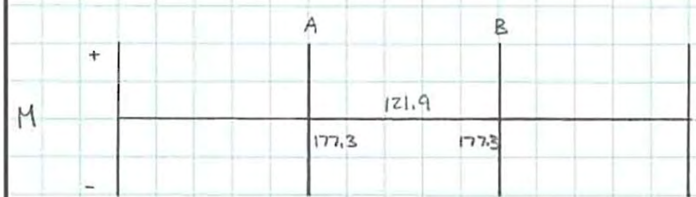
• GIRDER DESIGN (INTERIOR)

$w_{tr.} = [(1.07 \text{ klf})/6'](35.5') = 6.33 \text{ klf}$

$w_{girder} = 1.2(25 \text{ PSF})(3') + 1.6(40 \text{ PSF})(3') = 0.282 \text{ klf}$

$w_{self} = 1.2(150 \text{ PCF})(22.5'')(36'')/144 = 1.01 \text{ klf}$

$w_u = 6.33 + 0.282 + 1.01 = 7.62 \text{ klf.}$



ACI MOMENT COEFFICIENTS

$$M^-_{A,B} = \frac{w_u l_n^2}{11} = \frac{7.62(16')^2}{11} = 177.3 \text{ ft.k}$$

$$M^+ = \frac{w_u l_n^2}{16} = \frac{7.62(16')^2}{16} = 121.9 \text{ ft.k}$$

TOP REINFORCEMENT (INT. SPAN / INT. SUPPORT)

$$A_s = M_u/4d = \frac{177.3}{4(19.99'')} = 2.22 \text{ in}^2$$

TRY (4) #7 $\Rightarrow$ $A_s = 2.4 \text{ in}^2$

$$a = \frac{2.4(60)}{0.85(4)(36)} = 1.18''$$

$$c = 1.18/0.85 = 1.38''$$

$$\varepsilon_t = \frac{0.003}{1.38}(19.99 - 1.38) = 0.0405 > 0.0075$$ ∴ MOMENT can be reduced per ACI 8.4

MOMENT REDISTRIBUTION : $1000\,\varepsilon_t = 40.5 \rightarrow$ reduce by 40.5%

$$M_u = 177.3(1 - 0.405) = 105.5 \text{ ft.k}$$

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JOB TECH. REPORT 2 – CALCULATIONS
SHEET NO. 6 OF 10
CALCULATED BY A. KACZMAREK DATE
CHECKED BY DATE
SCALE

$A_s = \frac{105.5}{4(19.99)} = 1.32 \text{ in}^2$

TRY (3) #7 ⇒ $A_s = 1.8 \text{ in}^2$

$a = 1.8(60) / 0.85(4)(36) = 0.88$

$\phi M_n = \left[0.9(1.8 \text{ in}^2)(60'')(19.99'' - \frac{0.88}{2})\right] / 12$

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

$\boxed{\text{USE (3) \#7 TOP REINF.}}$

BOTTOM REINFORCEMENT (INT. SPAN)

$A_s = 121.9 / 4(19.99) = 1.53 \text{ in}^2$

TRY (3) #7 ⇒ $A_s = 1.8 \text{ in}^2$

$a = \frac{1.8(60)}{0.85(4)(36)} = 0.88$

$c = 0.88/0.85 = 1.035$

$\varepsilon_t = \frac{0.003}{1.035}(19.99 - 1.035) = 0.055 > 0.0075$ ∴ MOMENT can be reduced per ACI 8.4

MOMENT REDISTRIBUTION: $1000\,\varepsilon_t = 54.94$ → reduce by 54.94%

$M_u = 121.9(1 - 0.55)$

$= 54.86 \text{ ft.k}$

$A_s = \frac{54.86}{4(19.99)} = 0.686 \text{ in}^2$

TRY (2) #6 ⇒ $A_s = 0.88 \text{ in}^2$

$a = \frac{0.88(60)}{0.85(4)(36)} = 0.431$

$\phi M_n = \left[0.9(0.88)(60)(19.99 - \frac{0.431}{2})\right] / 12$

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

$\boxed{\text{USE (2) \#6 BOT. REINF.}}$

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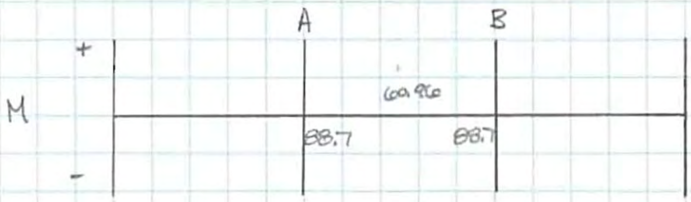
JOB: TECH. REPORT 2 – CALCULATIONS
SHEET NO. 7 OF 10
CALCULATED BY: A. KACZMAREK DATE:
CHECKED BY: DATE:
SCALE:

• EDGE GIRDER DESIGN (INTERIOR SPAN)

$\omega_u = 7.62 \text{ klf} / 2 = 3.81 \text{ klf}$ * Half the Tributary Width of the Interior Girder.



ACI MOMENT COEFFICIENTS

$$M_{AB}^{-} = \frac{\omega_u l_n^2}{11} = \frac{3.81(16')^2}{11} = 88.7 \text{ ft.k}$$

$$M^{+} = \frac{\omega_u l_n^2}{16} = \frac{3.81(16')^2}{16} = 60.96 \text{ ft.k}$$

TOP REINFORCEMENT (INT. SPAN, INT. SUPPORT)

$$A_s = M_u/4d = \frac{88.7}{4(19.99)} = 1.11 \text{ in}^2$$

TRY (4) #6 $\Rightarrow A_s = 1.76 \text{ in}^2$

$$a = \frac{1.76(60)}{0.85(4)(24)} = 1.29$$

$$c = 1.29/0.85 = 1.52$$

$$\varepsilon_t = \frac{0.003}{1.52}(19.99 - 1.52) = 0.036 > 0.005$$ ∴ Tension Controlled $\phi = 0.9$

$$\phi M_n = \left[0.9(1.76)(60)\left(19.99 - \frac{1.29}{2}\right)\right] / 12$$

$$= 153.2 \text{ ft.k} > 88.7 \text{ ft.k} \therefore \text{OK}$$

$\boxed{\text{USE (4) \#6 TOP REINF.}}$

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JOB TECH. REPORT 2 - CALCULATIONS
SHEET NO. 8 OF 10
CALCULATED BY A. KACZMAREK DATE
CHECKED BY DATE
SCALE

BOTTOM REINFORCEMENT (INT. SPAN)

$$A_s = \frac{60.96}{4(19.99)} = 0.76 \text{ in}^2$$

TRY (2) #6 $\Rightarrow$ $A_s = 0.88 \text{ in}^2$

$$a = \frac{0.88(60)}{0.85(4)(24)} = 0.647$$

$$c = 0.647/0.85 = 0.761$$

$$\varepsilon_t = \frac{0.003}{0.761}(19.99 - 0.761) = 0.0758 > 0.005$$ $\therefore$ Tension Controlled $\phi = 0.9$

$$\phi M_n = \left[0.9(0.88)(60)\left(19.99 - \frac{0.647}{2}\right)\right] / 12$$

$$= 76.57 \text{ ft.k} > 60.96 \text{ ft.k} \quad \therefore \text{ OK}$$

$\boxed{\text{USE (2) \#6 BOT. REINF.}}$

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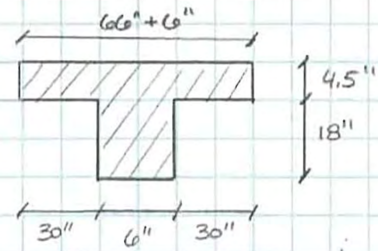
JOB TECH. REPORT 2 - CALCULATIONS
SHEET NO. 9 OF 10
CALCULATED BY A. KACZMAREK DATE
CHECKED BY DATE
SCALE

• JOIST DEFLECTION



$$\bar{y} = \frac{4.5(72)(18+2.25) + 18(6)(9)}{4.5(72) + 18(6)}$$
$$= 16.19 \text{ in.}$$

$$I = \frac{72(4.5)^3}{12} + (72)(4.5)(4.06)^2 + \frac{6(18)^3}{12} + 6(18)(7.19)^2$$
$$= 5887 + 8499$$
$$= 14386.2 \text{ in}^4$$

$$E = 33\sqrt{f'_c}\,(w_c)^{1.5} = 33\sqrt{4000}\,(150)^{1.5} = 3834 \text{ KSI}$$

$w_{LL} = 40 \text{ PSF}(6') = 240 \text{ PLF}$
$w_{DL} = 150 \text{ PLF} + 337.5 \text{ PLF} + 112.5 \text{ PLF}$
$= 600 \text{ PLF}$
$w_{TL} = 840 \text{ PLF}$

$$\Delta_{LL} = \frac{5 w_{LL} l_n^4}{384 EI} = \frac{5(0.24)(35.5')^4(1728)}{384(3834)(14386.2)} = 0.16''$$

$$\Delta_{LL} = \frac{l}{360} = \frac{38'(12)}{360} = 1.26'' > 0.16'' \quad \therefore \text{ OK}$$

$$\Delta_{TL} = \frac{5(0.84)(35.5)^4(1728)}{384(3834)(14386.2)} = 0.54''$$

$$\Delta_{TL} = \frac{l}{240} = \frac{38'(12)}{240} = 1.9'' > 0.54'' \quad \therefore \text{ OK}$$

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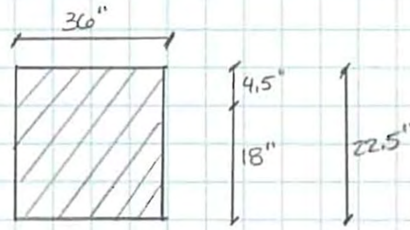
JOB: TECH. REPORT 2 - CALCULATIONS
SHEET NO. 10 OF 10
CALCULATED BY: A. KACZMAREK DATE:
CHECKED BY: DATE:
SCALE:

• INTERIOR GIRDER DEFLECTION



$\bar{y} = 22.5''/2 = 11.25''$

$I = \dfrac{36(22.5)^3}{12} = 34171.9 \text{ in}^4$

$E = 3834$ ksi

$\omega_{LL} = 40 \text{ PSF}(38') = 1.52$ klf

$\omega_{DL} = (25 \text{ PSF})(38') + (4.5/12)(150)(38') + (36'')(22.5'')(150)/144$

$= 3.93$ klf.

$\omega_{TL} = 3.93 + 1.52 = 5.45$ klf.

$\Delta_{LL} = \dfrac{5\omega_{LL} l_n^4}{384 EI} = \dfrac{5(1.52)(16)^4(1728)}{384(3834)(34171.9)} = 0.017''$

$\Delta_{LL} = l/360 = \dfrac{16'(12)}{360} = 0.533'' > 0.017''$ ∴ OK

$\Delta_{TL} = \dfrac{5\omega_{TL} l_n^4}{384 EI} = \dfrac{5(5.45)(16)^4(1728)}{384(3834)(34171.9)} = 0.061''$

$\Delta_{TL} = l/240 = \dfrac{16'(12)}{240} = 0.8'' > 0.061''$ ∴ OK

Appendix F: Cost Analysis

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JOB TECH. REPORT 2 - CALCULATIONS
SHEET NO. 1 OF 1
CALCULATED BY A. KACZMAREK DATE
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SYSTEM	FACTOR MATERIAL $ LABOR $ TOTAL $
1) PRECAST PLANK ON LOAD BEARING MASONRY & STEEL FRAME	(1.009) × [($9.7/SF) + ($4.18/SF)] = $\boxed{\$ 14.01 /SF}$
2) PRECAST PLANK ON STEEL	(1.009) × [($ 8.8/SF) + ($ 1.99/SF)] = $10.89/SF (1.009) × [($ 10.45/SF) + ($ 4 /SF)] = $14.45/SF + $\boxed{\$ 25.34 /SF}$
3) COMPOSITE STEEL DECK	(1.009) × [($11.15/SF) + ($5.65/SF)] = $\boxed{\$ 16.95/SF}$
4) ONE-WAY JOIST SYSTEM	(1.009) × [($ 6.55/SF) + ($ 10.30/SF)] = $\boxed{\$ 17.00/SF}$