TECHNICAL REPORT 2

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CityFlatsHotel – Holland, Michigan

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Executive Summary

The following technical report compares the existing floor system of CityFlatsHotel as well as three optional floor systems. All four systems were designed, analyzed, and compared in order to determine which system(s) were practical for the building and viable for further study. Currently, the floor system of CityFlatsHotel is precast hollow-core concrete plank, which is adequately designed to withstand the building load criteria, as previously determined. In order to properly compare each floor system, a typical floor bay of the building was taken into consideration. The following alternate floor systems were examined for the CityFlatsHotel:

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

The existing 8" hollow-core concrete plank system is supported by exterior masonry shear walls, as well as interior steel frames with additional masonry shear walls. This system 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 throughout the interior of the building. The precast hollow-core plank on steel framing was designed using the PCI Design Handbook to determine a 8" concrete slab without topping. The W12x50 steel girders that support the plank were designed with the AISC Steel Manual, by checking the live load and total load deflections. The composite steel deck system was designed using the Vulcraft Deck Catalog and the AISC Manual. The preliminary design consists of a 2VLI22 deck with a slab depth of 4.5" and a topping of 2.5". The supporting beams and girders are W10x12 (6) and W16x31 (8) respectively. The final alternative system is a one-way joist system, which consists of 6" wide joists spaced at 66" on center with a pan depth of 14". The slab designed is 4.5" and 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 promising system for further investigation. The only disadvantage of this system would be its increased floor system depth, which is not a concern for CityFlatsHotel since its current height is below the maximum height restrictions of Holland Michigan. Each of these alternative systems as a whole can be seen through detailed descriptions and diagrams. All calculations as well as building plans are provided in an Appendix at the end of the report.

Introduction: CityFlatsHotel

CityFlatsHotel is the latest eco-boutique hotel located at 61 East 7th Street in Holland Michigan. This environmentally friendly hotel has been awarded LEED Gold and is only the third eco-boutique hotel to achieve such status in the United States and is the first of its kind to earn such recognition in the Midwest. Located on the outskirts of downtown Holland, which was named the second happiest place in America in 2009, the 56-guest room hotel is a unique place to stay. Not only are the hotel rooms decorated in a variety of ways, so that no two rooms are alike, this 5-story hotel offers many additional features to keep visitors satisfied. Accommodations include guest rooms, junior suites, master suites and more. Coupled with being located close to top of the line shopping, fine dining and extravagant art venues CityFlatsHotel is the place to stay when visiting Holland and its surrounding unique attractions.

The ground floor houses the main lobby for the hotel, a fitness suite and the CitySen Lounge. Also available is office space, high-tech conference rooms, and a digital theater for those who may want to conduct business meetings or private get-togethers. The remaining floors of the building are occupied by the various hotel rooms, with the top floor mostly reserved for CityVu Bistro restaurant and City Bru bar. The views from the restaurant of downtown Holland and Lake Macatawa are spectacular, which go well with the diverse fresh entrees served at CityVu Bistro.

The exterior of CityFlatsHotel consists of multiple materials. Mainly covered in glass, other features including brick accents, metal panels, and terra cotta finishing make up the building seen at the intersection of College Ave and 7th Street. The contrast in simple materials leaves an appealing building image and gives it a sense of modernity, which is continued throughout the entire hotel. Accompanying the exterior image and fascinating interior design, efficient features can be found in every room. Such features include but are not limited to cork flooring, occupancy sensors, low flow toilets and faucets, fluorescent lighting, Cradle-to-Cradle countertops, and low VOC products.

CityFlatsHotel's structural system will be described throughout this report by taking a closer look at the structural concepts and existing conditions. To understand how the various structural components work, detailed descriptions of the foundation, floor system, lateral system, and gravity system are provided.

Structural Systems

Foundation

Soils & Structures Inc. completed the geotechnical engineering study for the CityFlatsHotel on July 16, 1998. A series of five test borings were drilled in the locations shown in the proposed plan (Figure 1.1). Each test boring was drilled to a depth of 25 feet in order to reveal the types of soil consistent with the location of the site. The results showed that the soil profile consisted of compact light brown fine sand to a depth of 13.0 to 18.0 feet over very compact coarse sand and compact fine silt. In test boring two a small seam of very stiff clay was discovered at 20.0 feet. Groundwater was encountered at a depth of 14.0 feet. From these findings it was recommended that a bearing value of 4000 psf be used for design of rectangular or square spread foundations and a value of 3000 psf be used for strip foundations. Since the test boring was performed in a relatively dry period, it was noted that the water table might rise by as much as 2.0 to 3.0 feet during excessive wet periods.

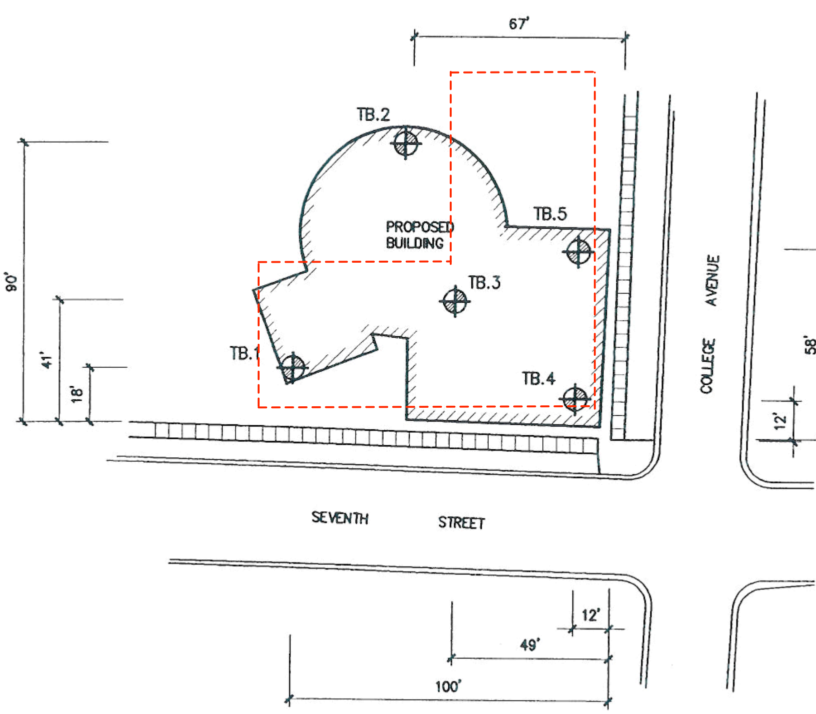


FIGURE 1.1: This is a plan view of the Five Test Boring Locations
Note: The layout of the building here was the proposed shape. The actual building takes on an L-shape as can be seen later in Figure 5.1

Based on the conclusion from the geotechnical report it was decided to have all sand and/or sand fill be compacted to a density of 95 percent of its maximum density as determined by ASTM D1557. By compacting the soil through methods of vibration allowed the soil bearing capacity to be set at 8000 psf for footings. The basement floor consists of 4” concrete slab on grade that has a concrete compressive strength of 3000 psi and is reinforced with 6x6 W2.9xW2.9 welded wire fabric. Examples of the foundation and footings can be seen in Figures 1.2 and 1.3 respectively. This typical layout is consistent throughout the entire foundation system.

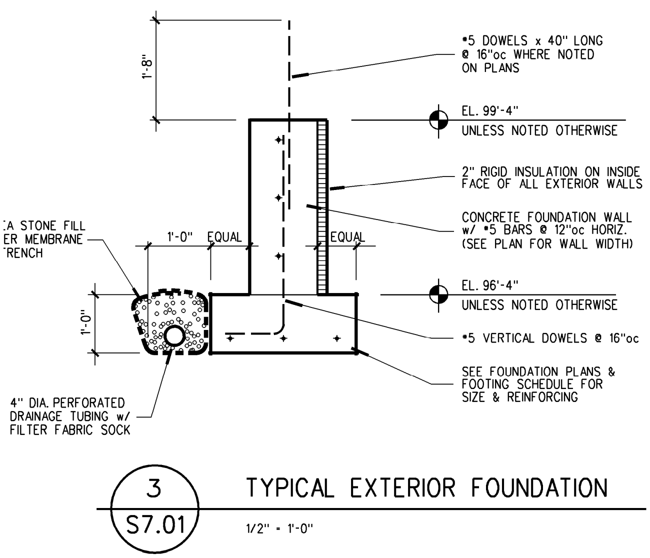


Figure 1.2: Typical Exterior Foundation

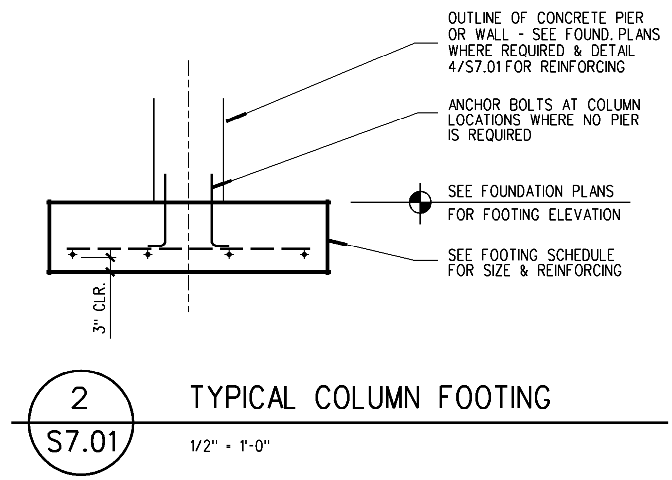


Figure 1.3: Typical Column Footing

Superstructure

Due to the relatively “L” shape of CityFlatsHotel, the buildings framing system is able to follow a simple grid pattern. The overall building is split into two rectangular shapes that consist of 6 and 7 bays. The typical grid size is between 18’-0” to 18’-8” wide and 22’-6” to 30’-2” long. The main floor system used is an 8” precast planking deck with 2” non-composite concrete topping. The concrete topping is normal weight concrete and has a compressive strength of 4000 psi. The floor system is then supported by steel beams, which range in size and include W30x173’s for exterior bays and W8x24’s for interior corridors. Details for these two beam connections can be seen in Figure 1.4 below.

The precast plank allows for quicker erection, longer spans, and open interior spaces. The use of precast plank is typical for all floors other than the basement floor and specific areas of the ground floor, which utilizes slab on grade. All floor slabs on grade are 4" thick except for radiant heat areas, which require the slab to be 5" thick. Both of these slabs are reinforced with 6x6 W2.9x2.9 welded wire fabric.

Masonry walls are also used throughout the building layout to hold up the precast concrete plank floors. Refer to Appendix A for wall locations. These walls simply consist of concrete masonry units that are reinforced with #5 bars vertically spaced at 16" o.c. and extend the full height of the wall (Figure 1.5). In order to connect the precast planks with the masonry block, 4" dowels, typically 3'-0" long spaced at 48" o.c., are grouted into keyways and used to connect the two members together (Figure 1.6).

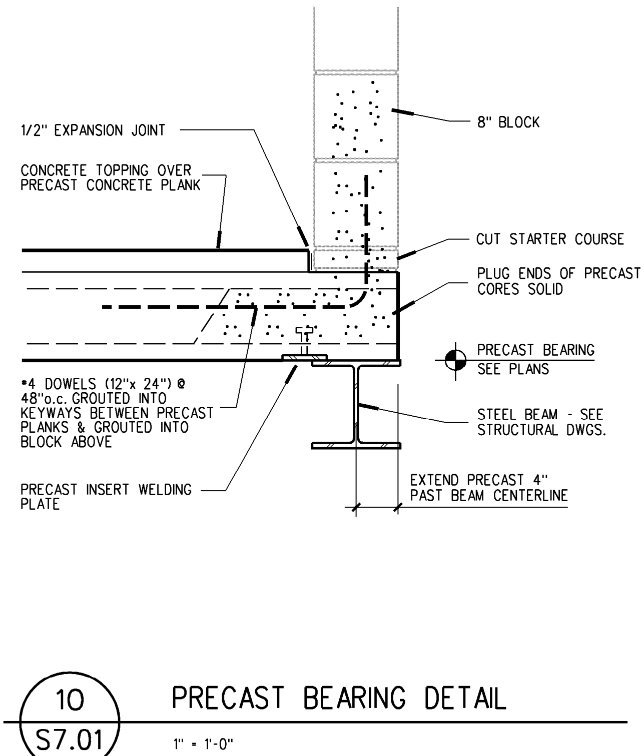


Figure 1.4: Typical Steel Beam Support Detail

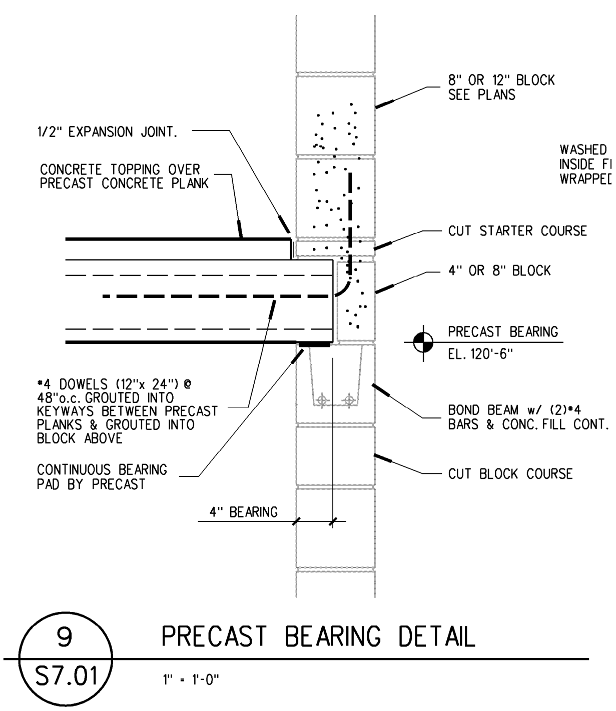


Figure 1.5: Typical Masonry Wall Reinforcing

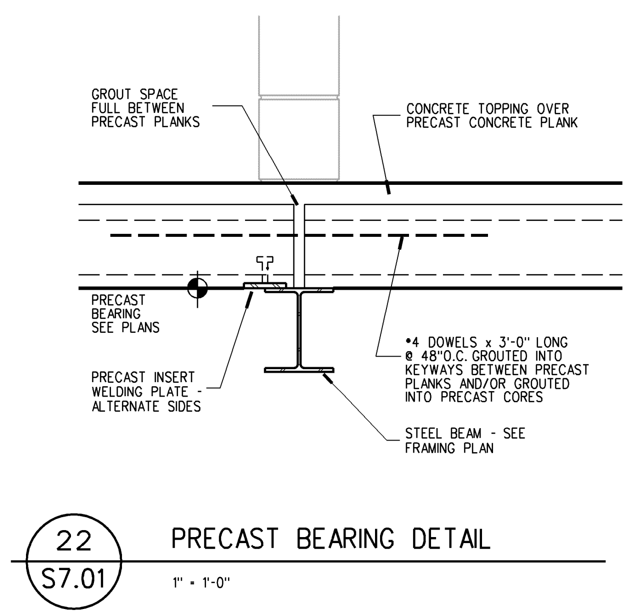


Figure 1.6: Typical Member Connection Detail

Columns add the final support and are typically HSS columns located around the perimeter of the building as well as along the corridors of the hotel. Refer to Appendix A for plans with column locations. HSS 8x8x3/8” columns were typically used on the exterior and HSS 8x8x1/2” columns were used in the interior. HSS 12x12x5/8” were used in order to support the larger beams and greater tributary areas. All load bearing masonry walls and steel beams will take the reaction load from the precast concrete plank flooring, as well as any additional loads from upper levels, and transfer the loads thru the columns and exterior walls thru to the foundation system.

Lateral System

The main lateral system for the CityFlatsHotel consists of the concrete masonry shear walls. The exterior as well as the interior walls are constructed with 8” concrete masonry, which extend the entire height of the building. The core shear walls are located around the staircases and elevator shafts. The average spacing between these walls are 18’-6” and they extend between 22’-6” to 25’-6” in length. In addition to the masonry walls there are steel moment connections in the southeast corner of the building similar to (Figure 1.7), which allows for additional lateral support of the two-story entrance atrium. Moment connections are also utilized on the top floor again similar to (Figure 1.7). This is in order to support the large amounts of glazing that is present, as an architectural feature for the restaurant located there. On floors three to five there are lateral braces used again in the southeast corner of the building that help with resisting the lateral load, which is prominent in the North/South direction. This will be expressed later when calculating wind loads.

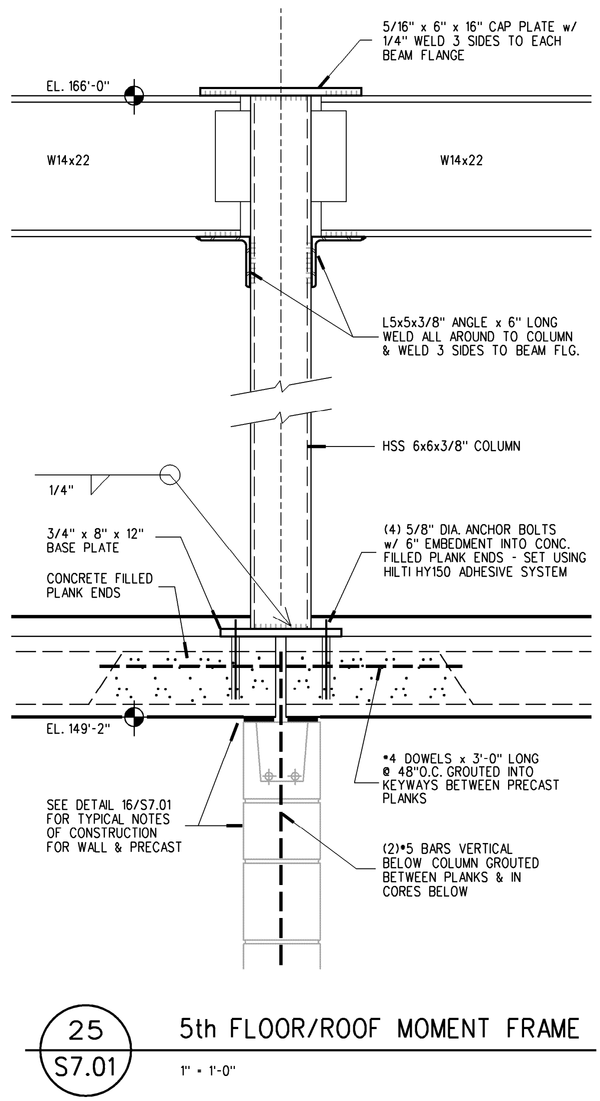


Figure 1.7: Typical Moment Frame Connection

Roof System

The roof framing system like the floor framing system is laid out in a rectangular grid. It consists of 1.5B 20-gauge metal decking supported by K-series joists. The typical joists that are used range between 12K1 an 20K5, which have depths of 12” and 20” respectively. These K-series joists span between 16’-6” to 30’-8”. The roof deck spans longitudinally, which is perpendicular to the K-series joists. The joists are spaced no further than 5’-0” apart and typically no shorter than 4’-0”.

Codes and References

Codes Used in the Original Design

- 2003 Michigan Building Code
- ASCE 7-05, Minimum Design Loads for Buildings
- ACI 318-05, Building Code Requirements for Structural Concrete
- Specifications for Structural Steel Buildings (AISC)
- International Building Code (IBC), 2006

Codes Used in Analysis

- ASCE 7-05, Minimum Design Loads for Buildings
- ACI 318-05, Building Code Requirements for Structural Concrete
- Specifications for Structural Steel Buildings (AISC), 13th Edition
- International Building Code (IBC), 2009
- PCI Design Handbook, 7th Edition
- RS Means Assemblies Cost Data, 2010
- RS Means Facilities Construction Cost Data, 2010
- PCA
- VULCRAFT Deck Catalog

Materials

Reinforced Concrete

Footings	$f'_c = 3000$ psi
Slab On Grade	$f'_c = 4000$ psi
Precast	$f'_c = 5000$ psi
Precast Topping Slab	$f'_c = 4000$ psi

Reinforcement Steel

Deformed Bars	ASTM A615
Welded Wire Fabric	ASTM A185

Structural Steel

Structural W Shapes	ASTM A992
Steel Tubes (HSS Shapes)	ASTM A500
Angles & Plates	ASTM A36
Bolts, Fasteners, & Hardware	ASTM A153

Masonry

8" CMU	$f'_m = 2000$ PSI
Grout	$f'_c = 3000$ PSI

Design Load Summary

All of the design loads that are used during the analysis of CityFlatsHotel are listed in Table 4.1 below.

Live Loads (LL)		
Area	**GMB Design Loads (PSF)**	**ASCE 7-05 Load (PSF)**
Private Guest Rooms	40	40
Public Spaces	100	100
Corridors	100	40 (Private Corridor) / 100 (Public Corridor)
Lobbies	100	100
Stairs	100	100
Storage/Mechanical	125	125 (Light)
Theater (Fixed)	60	60
Restaurant/Bar	100	100
Patio (Exterior)	100	100

Dead Loads (DL)		
Material	**GMB Design Loads (PSF)**	**ASCE 7-05 Load (PSF)**
8" Precast w/2" Topping	80	Section 3.1
10" Precast w/2" Topping	92	
8" Masonry Wall, Full Grout w/Rein. @ 16" o.c.	-	
MEP	10	
Partition	25	
Finishes/Miscellaneous	-	
Roof	15	

Snow Load (SL)		
Area	**GMB Design Loads (PSF)**	**ASCE 7-05 (PSF)**
Flat Roof	35	35
Ground	50	50

Table 4.1: Summary of Design Loads

Typical Span

The typical bay used in the analysis of the existing and alternative floor systems is defined in Figure 5.1

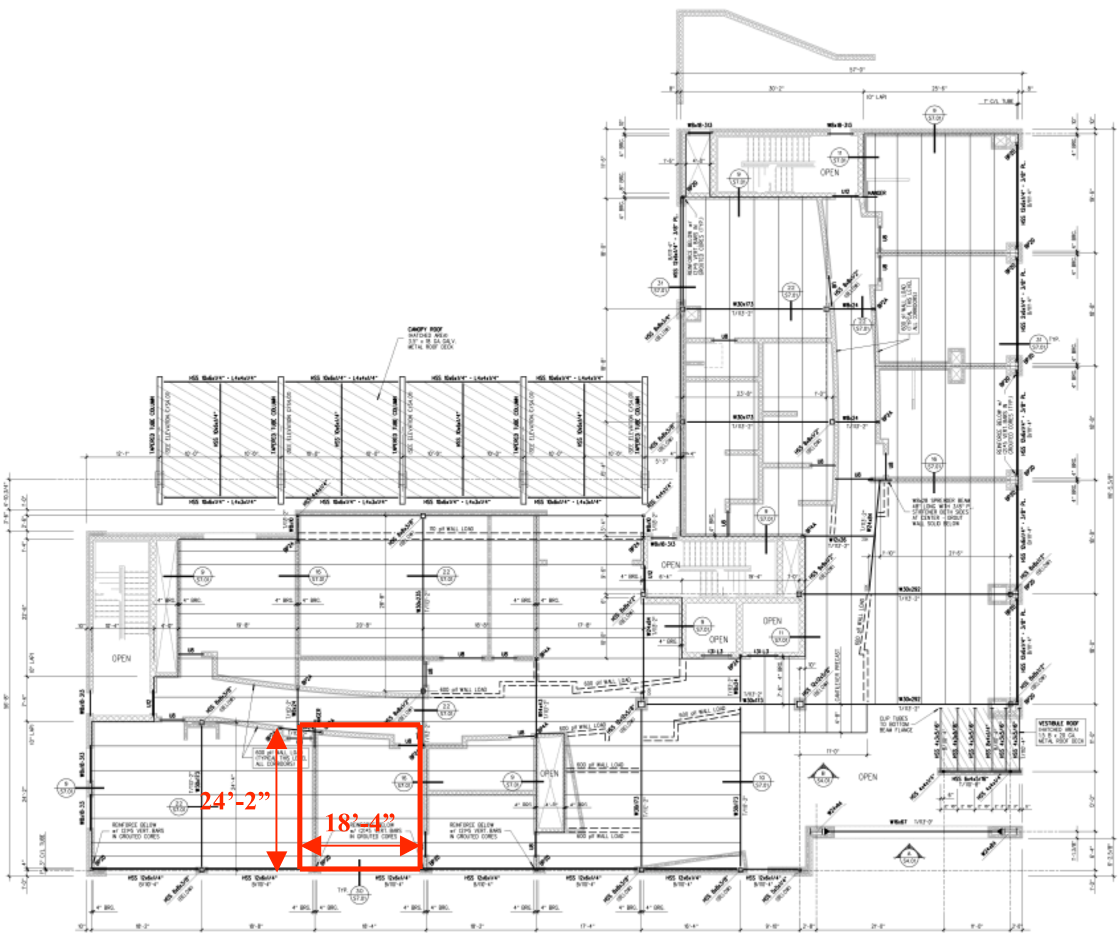


Figure 5.1: Typical Bay Used in the Analysis of Existing and Alternate Floor Systems

Floor Systems

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

<u>Material Properties</u>

Concrete: 8" x 4'-0" w/2" Topping

f'c = 5,000 psi

Tendons: 66-S

f_{pu} = 270,000 psi

Loadings: Dead (Self Weight) = 81 psf

Live = 40 psf

Superimposed = 35 psf

Descrpition

The hollow core precast concrete plank system spans a maximum distance of 18'-4" for the particular section of the building shown in Figure 6.1. The 4'-0" wide planks run the entire length of the floor. For the analysis of this floor system, a typical bay of 18'-4" x 24'-2" was used can be seen 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.

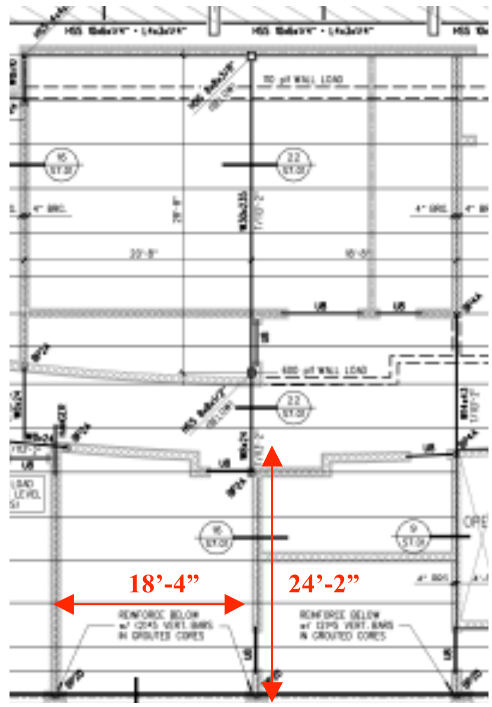


Figure 6.1: Existing Hollow-Core Plank

The planks that were designed for the building are 8" thick planks with 2" topping and come in 4' wide sections. The design method for the planks used by the manufacturer was unknown, so it was assumed that the planks were designed using the PCI Design Handbook. In order to achieve the maximum span of 18'-4", 66-S strands were used within the hollow core panel. This relates to the designation of the number of strands (6), the diameter of the strands in 16^{th} (6), and that the strands are to be straight throughout the panel. The assembly of this panel can hold a max service load of 224 psf, which exceeds the total un-factored load of 90 psf. Reducing the number of strands can be a way to have the plank support only the 90 psf load required. The total un-factored load is a combination of hotel room live loads, superimposed dead loads, and an additional 15 psf for the 2" topping. Supporting calculations may be found in Appendix B.

Advantages

The main advantage of precast hollow-core concrete planks is the low cost and time efficient construction process. The precast plank floor has the lowest cost compared to all the floor systems investigated in this report. Since precast concrete does not require the curing time that cast-in-place concrete requires, the installation process is much quicker. The reason behind this is due to the fact that precast planks are constructed in a plant where curing can take place year round under controlled conditions. The overall effect is faster construction schedules and ultimately a lower overall project cost. Typical spans of hollow-core systems tend to be greater, resulting in open floor plans and greater structural grid sizes. Hollow-core planks can span up to 33' before the amount of loading allowed greatly decreases. This can be a result of the general use of higher strength concrete, such as 5000 PSI. Along with the longer span, the floor depth of the hollow core-planks is much shallower than the alternative floor systems, except where supported on beams, allowing for the most efficient floor-to-floor heights. Building height restrictions could be a main reason to use hollow-core plank to decrease floor-to-floor height, which reduced the overall building height. Due to the majority of this floor system consisting of concrete, sound and heat transmission is greatly reduced. Plus 2 hour-fire rating can be achieved with minimal fireproofing required for only the few interior steel frames. Finally, even though the amount of concrete used increases the building weight, the voids in the planks lead to minimal increases to the overall building weight.

Disadvantages

The most relevant disadvantage using the hollow-core precast system is that precast concrete requires more upfront planning. Thus, the design phase of the project could potentially prolong the construction schedule. Lead-time becomes a concern since the concrete planks may have to be transported via oversized trucks from the manufacturer. Plus the speed is set by how fast the masonry walls are erected, and the planks need to be threaded between the framing columns and beams, which requires a lot of coordination of floor to floor construction. Also there are more members that need to be picked up by the crane for this system, again slowing the process down. An additional concern is that the architectural design can be limited as this system works best with square or rectangular bays since precast planks are not good for curved or angled edges.

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

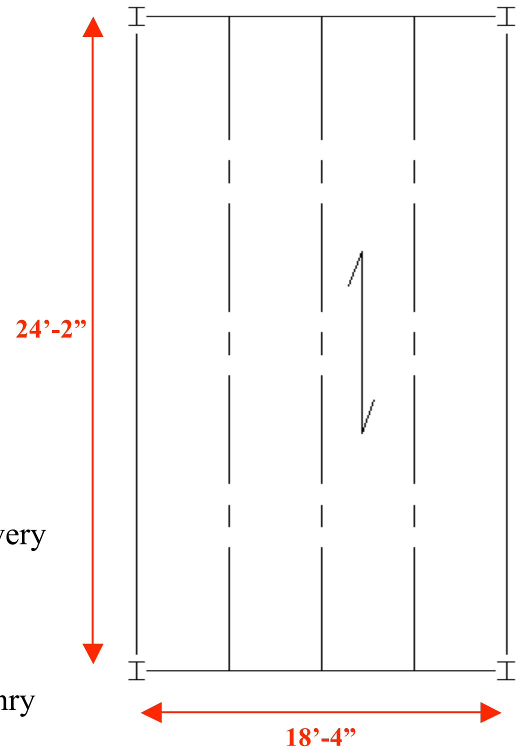


Figure 6.2: Hollow Core Plank on Steel

<u>Material Properties:</u>

Concrete: 8" x 4'-0" Untopped
$f'c$ = 5,000 psi

Tendons: 66-S
f_{pu} = 270,000 psi

Loadings: Dead (Self Weight) = 56 psf
Live = 40 psf
Superimposed = 35 psf

Description

The precast hollow-core concrete plank on steel system is very similar to the existing precast plank system utilized by the CityFlatsHotel. However, this system would utilize steel columns/beams and replace the exterior load bearing masonry walls. For this report, the steel columns that support the precast plank system were not analyzed, as they will be further investigated at a later time.

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 18'-4" x 24'-2" as shown in Figure 5.1. However, the concrete planks will span in the 24'-2" direction rather than the 18'-4" direction of the current system, as seen in Figure 6.2. The 4' wide planks run the entire length of the floor. In order to decrease the precast plank self weight and still withstand the total floor load, a plank depth of 8" with no topping was selected using PCI Design Handbook. To achieve the span, strands of 66-S were used within the hollow-core panel. This designates that there are 6 strands with diameter of 6/16" running straight throughout the panel. This plank system design has a capacity of 98 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 and superimposed dead loads. If the plank is topped an additional 10 psf would need to be added, but the plank capacity still exceeds this amount as well. Supporting calculations may be found in Appendix C.

The steel members that support the precast concrete planks were design using the American Institute of Steel Construction manual (AISC). Girders were determined to be W18x35 members. Additional options include W12x50 members and W16x36 members. These options are in place in order to reduce the overall system depth by decreasing the flange depth, however these options are less economical due to the increase in flange weight.

Advantages

There are many benefits of using precast hollow-core concrete plank on steel. Structurally, hollow-core planks provide the efficiency of a pre-stressed member. This allows for larger load capacity, a great span range, and deflection control. Since the precast hollow-core concrete planks are produced and cured in a control environment, the result is a product with greater strength and durability, which allows for increased floor load capacity. Future costs aren't an issue, as this system requires very little maintenance. Again precast planks lead 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. Additionally this system as a whole is recognized as a LEED rated system, which is a main component for the CityFlatsHotel. Other advantages consist of naturally sound-resistant material and reduced building weight.

Disadvantages

Unfortunately, with advantages come disadvantages. The main downside is the decrease in floor-to-floor height, or inevitably the increase in overall building height. The reduction is due to the deeper floor system caused by the W12x50 steel girders that support the concrete planks. The floor system depth would increase from 10" (existing floor system with topping) to 20.25" (the 12.25" depth of the girder + the 8" depth of the precast plank). This presents a problem in areas where the total overall height of the building is limited. The lead-time would also increase as the fabrication, detailing, and transportation of the steel become factors. Lastly, all steel members require spray fireproofing to obtain the appropriate fire rating. These factors can be anticipated to increase the overall project cost.

Feasibility

In Holland Michigan, the building height limit is 11 stories. Since CityFlatsHotel is currently only 5 stories above grade, this system could be implemented and keep the building within the code limitations of its current location. For this system to be considered as a potential candidate, a further investigation would have to be conducted to verify if this system would actually impact the pace of construction as well as the overall budget. The money saved through a faster construction schedule could account for the increased costs and leave it as a viable option. Due to the fact that there is less needed coordination of multiple trades, and the cold weather becomes less of an issue if the building becomes all steel versus a mix of steel and masonry. The final check that would have to be completed would be the effect the increase in building height would have on the structural system as a whole, recalculating seismic and wind loads.

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:	2VLI22 – 3 Span
Loadings:	Dead = 45 psf
	Live = 40 psf
	Superimposed = 35 psi

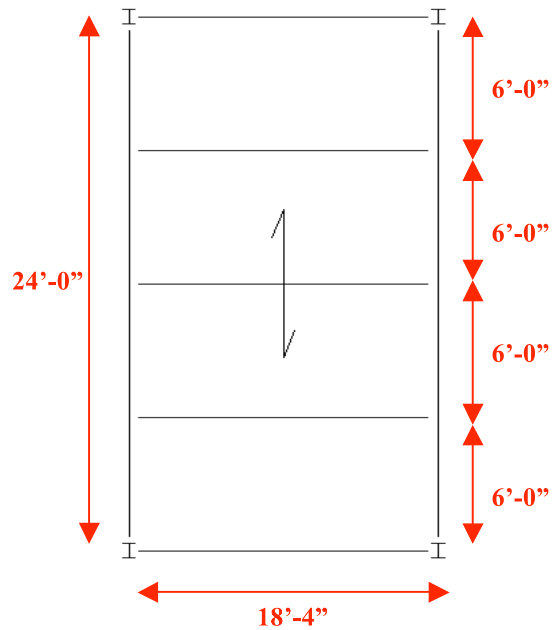


Figure 6.3: Composite Steel Deck

Description

The typical bay size used to design a composite steel deck system is 24'-0" x 18'-4" as shown in Figure 6.3. This was chosen to maintain a fair comparison between alternate and existing floor systems and allow for intermediate beams to be spaced at 6'-0". This slight change does not alter the building layout in a drastic manner, which allows for the column spacing to remain the same. Note that the columns for this floor assembly were not designed for this report, although due to changes in framing structure the column sizes would most likely change.

To comply with the typical bay and loadings, a 2VLI22 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 is able to span 9'-4" unshored given a 3 span condition. This exceeds the 6'-0" 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). The size of the members designed as well as slab thickness satisfies the load and deflection limits of the entire system. 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 concrete dominate systems. This results in a reduced gravity load on the foundation, which reduces the required size of columns and foundation. This minimizes the costs associated with the overall structural system. Since a composite steel deck is a quick erection system the construction process is simplified. This is partly due to the fact that no shoring is required for the 6'-0" spans. Also, steel erection takes less time since there is less forming (metal deck serves as the formwork), placing, and curing concrete. The overall result is a fast construction schedule, cheaper budget, and less waste material. Additional advantages include a fire rating of 2-hours and a relatively shallow system depth of 20.4" (15.9" depth of girder +4.5" slab depth) that will leave sufficient space and flexibility for mechanical ducts and plumbing in the ceiling.

Disadvantages

Once again, the main disadvantage is the floor system depth of 20.4". The girder size designed is a W16x31, which increases the floor depth drastically. 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 rating for the building. Other concerns with a steel frame building is additional lead time as a result of the steel needing to be fabricated, shipped, and the extra detailing that is required. An additional disadvantage to the composite deck system is the poor sound-insulating property of steel. This may be of concern since CityFlatsHotel has a large concern for noise transferring between walls and floors, which may require additional soundproofing and lead to an increased cost.

Feasibility

Ultimately, after weighing the advantages and disadvantages of the composite system, it seems like the disadvantages outweigh the advantages. Even with a low system cost the negative factors, which include a decrease in floor-to-floor height and poor sound-insulating materials, are too overwhelming for a hotel design. Therefore, use of this system for CityFlatsHotel is not likely, and further investigation is not necessary.

Alternative #3: One-Way Joist System

<u>Material Properties:</u>

Concrete:	4.5" Slab
	66" / 6" Pan Joists
	f'c = 3000 psi
Reinforcement:	f'y = 60,000 psi
Loadings:	Dead = 56.25 psf
	Live = 40 psf
	Superimposed = 35 psf

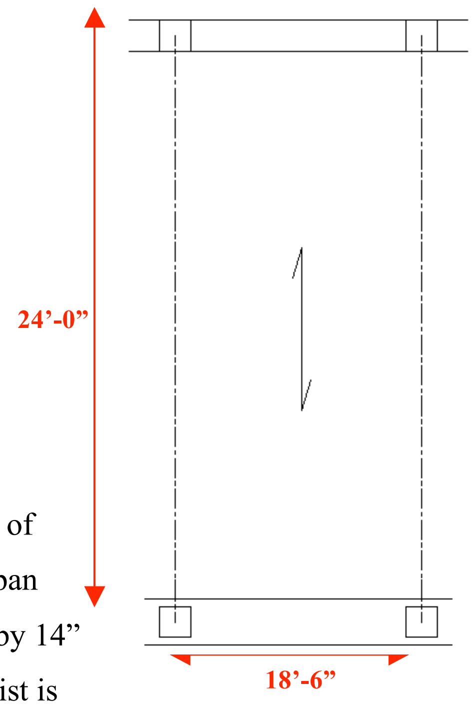


Figure 6.4: One-Way Joist

Description

The one-way joist system was designed using a typical bay of 24'-0" x 18'-6" as show in Figure 6.4. It was designed to span in the 24'-0" direction. A 4.5" slab was used with 6" wide by 14" deep joists spaced at 66" on center. The depth of the pan joist is 14", which is adequate for deflection control, in accordance with PCA requirements. The minimum reinforcement for the slab is (1) #3 bar spaced at 12" on center. In order to prevent flexural failure, reinforcement was designed for the joists. Reinforcement for the negative moment is (2) # 6 bars (top reinforcement) and reinforcement for the positive moment is (1) #8 bar (bottom reinforcement). Shear reinforcement includes #3 bars with 8" spacing.

Both exterior and interior girders were designed to span in the 18'-6" direction, which is perpendicular to the joist ribs. The exterior girder and interior girder were both designed at 24" wide in order to match the assumed column dimensions, which is a 24" square column. These dimensions provide for better constructability. For the interior girder the required top reinforcement is (3) #8 bars, while the required bottom reinforcement is (2) #8 bars. For the exterior edge girder the required top reinforcement is (3) #6 bars, while the required bottom reinforcement is (2) #6 bars. Supporting calculation may be found in Appendix E.

Advantages

The one-way joist system is the most economical concrete systems for long spans with heavy loads, which is why it was chosen as an alternative. The 6"/66" joist system designed is considered a "skip" joist, since the pans are spaced further apart. The longer spans result in wider column spacing that allows for a more open floor plan, a desirable feature for hotels. One-way joist systems also have inherent vibration resistance, reduced dead load due to pan voids, and easier placement of electrical and mechanical equipment between pan joists. Another advantage to owners is the simplicity of future renovations, reducing costs. Plus, this system is capable of a 2-hour fire rating without additional fireproofing. Overall with the longer spans and inherent vibration resistance a one-way joist system is an attractive alternative floor assembly for hotels.

Disadvantages

One disadvantage of the one-way joist system is the self-weight, which is larger than the self-weight of the other alternative floor systems due to the amount of concrete used. 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 due to the necessary framework that is required in order to build this system. Another slight disadvantage is the depth of the system, which is larger than the existing system. However, electrical and mechanical equipment can potentially be run between the pan joists, except for at each column line where the equipment would hit the girder. This eliminates the need for additional floor depth in order to accommodate this equipment.

Feasibility

The one-way joist system may be worthwhile to examine in the future and 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. Since there is potential that the cheaper cost of the one-way joist system could outweigh the effects of the increased self-weight, the one-way joist system is a feasible alternative and may require additional study. Luckily, the increase in floor depth is not of concern, since the building, which resides in Holland Michigan, has overall building height flexibility before reaching the maximum allowable height of the area. However, increasing the overall height does become a cost comparison issue.

Overall System Comparison

Comparison Criteria	Precast Plank on Load Bearing Walls and Steel Frame	Precast Plank on Steel Framing	Composite Steel Deck System	One-Way Joist System
Slab Self Weight	81 PSF	56 PSF	45 PSF	80 PSF
Slab Depth	8"	8"	4.5"	4.5"
System Depth	10" (8"+2"Topping)	20.25"	20.4"	18.5"
Deflection	0.77" < 0.91"	0.71" < 0.92"	0.66" < 0.8"	0.20" < 0.92"
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
Constructibility	Easy	Easy	Easy	Average
System Cost*	$12.21/SF	$22.22/SF	$14.79/SF	$14.83/SF
Feasibility	Yes	Yes	No	Yes

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

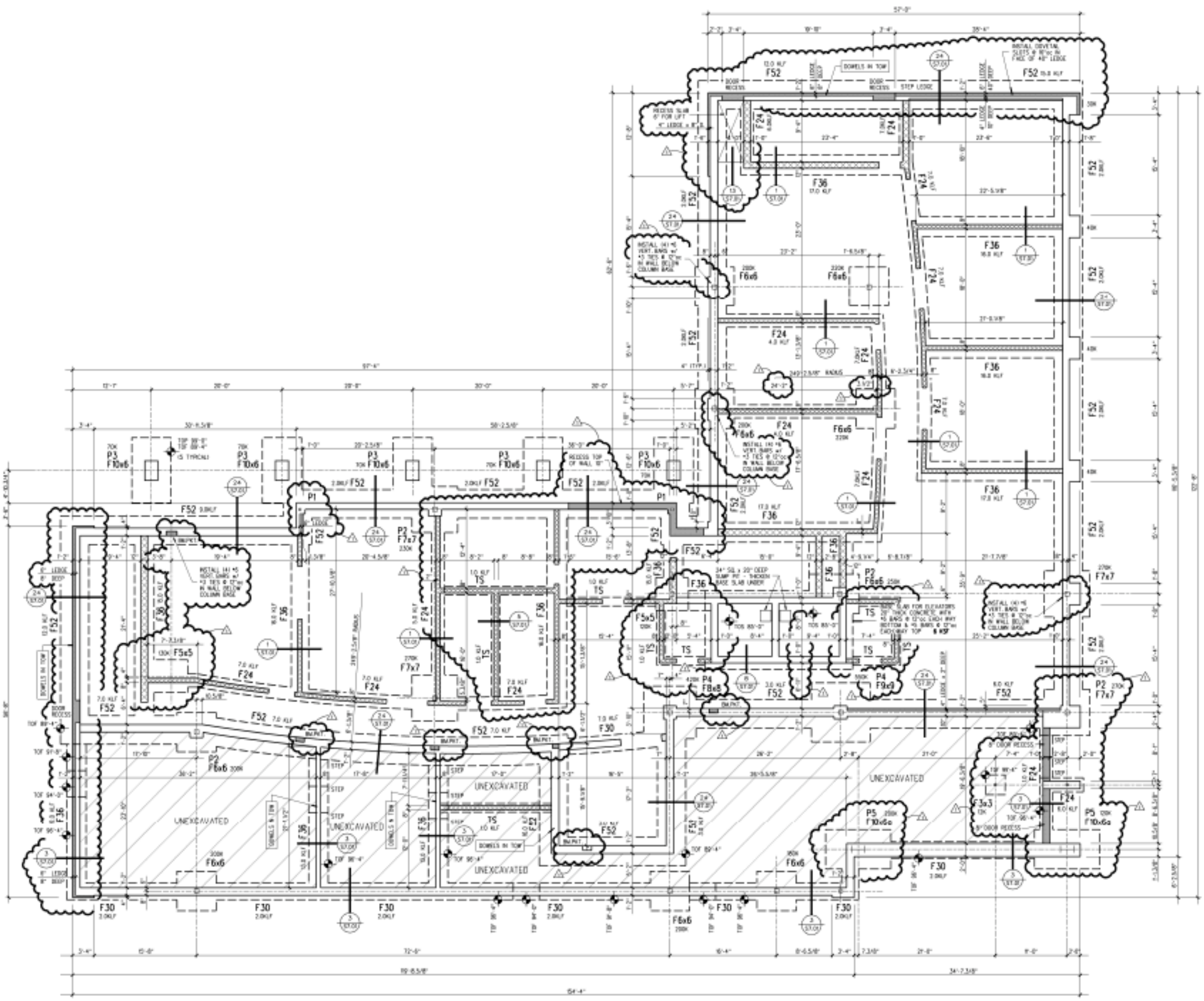
Conclusion

In analyzing alternative floor systems for CityFlatsHotel, a better understanding of the impacts of various design decisions was formed. Each alternative system was designed using a typical bay size, and 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, which bears on exterior load bearing masonry walls and an interior steel frame. The alternative floor systems include a precast hollow-core concrete plank on steel framing system, a composite steel deck system, and a one-way joist system. The major comparisons factors for this report were system depth, self-weight, cost, and constructability.

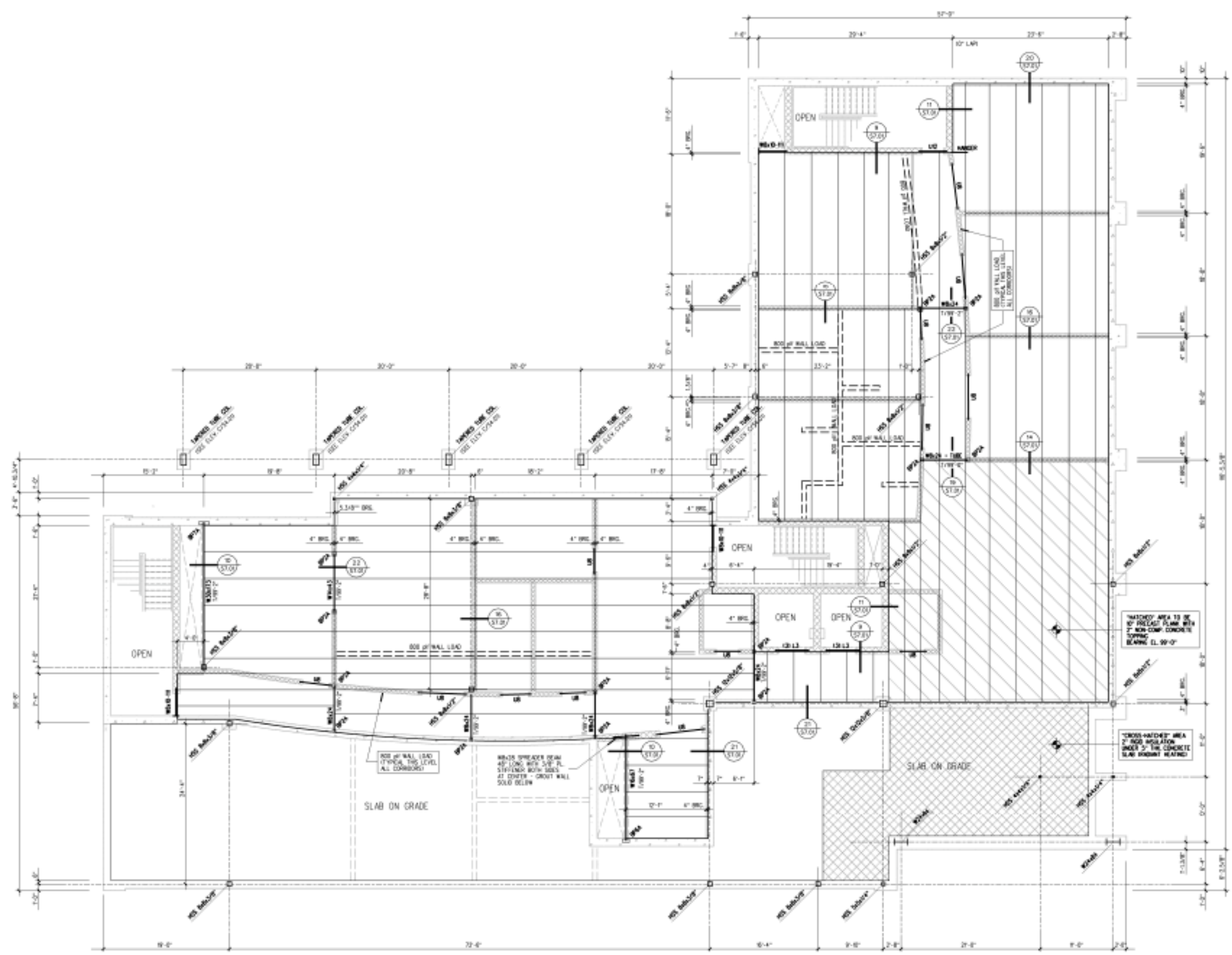
After comparing each alternative floor system with the existing system, it was concluded that the existing floor system is the most efficient due to its cost, system depth, and acoustic properties. However, a few of the alternative systems may be a realistic solution for the building as well. The precast hollow-core plank on steel frame offers a design consistent with the existing system, but eliminates the exterior load bearing masonry walls. Although it is a lightweight system that is time efficient, the additional steel sacrifices cost and floor-to-floor height or overall building height. A one-way joist system incorporates a deeper system and is a heavier system (self-weight), but is the most economical concrete system for long span conditions. The composite steel deck system is arguably the least feasible for the CityFlatsHotel. Even though the total cost per square foot is lower than other alternative floor assemblies, but has the largest floor system depth and poor sound-insulating properties, which is a priority for hotels.

The most likely alternative system for the CityFlatsHotel, besides its existing system, is the one-way pan joist system. This system created the second thinnest overall floor system depth, as well as one of the cheaper systems per square foot. Being the most economical concrete system for long span conditions CityFlatsHotel could utilize this alternative system with wider column spacing, reduced dead load due to pan voids, and easier placement of electrical and mechanical equipment in the pan joists. Another upside is the natural sound-insulating properties as well as fireproofing the concrete system provides, which is a common system for hotels. Therefore it is logical that this system is feasible for the CityFlatsHotel.

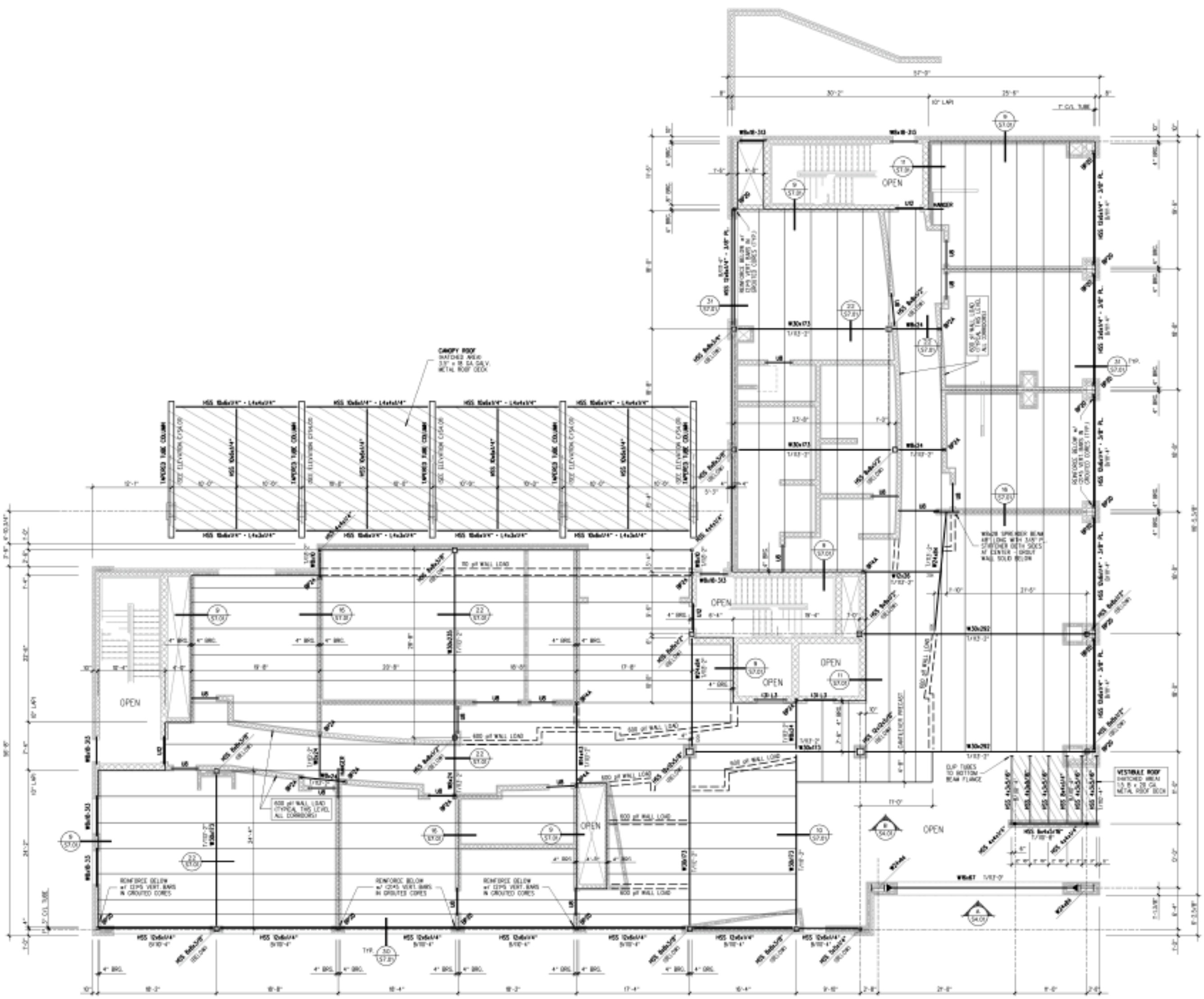
Appendix A: Plans



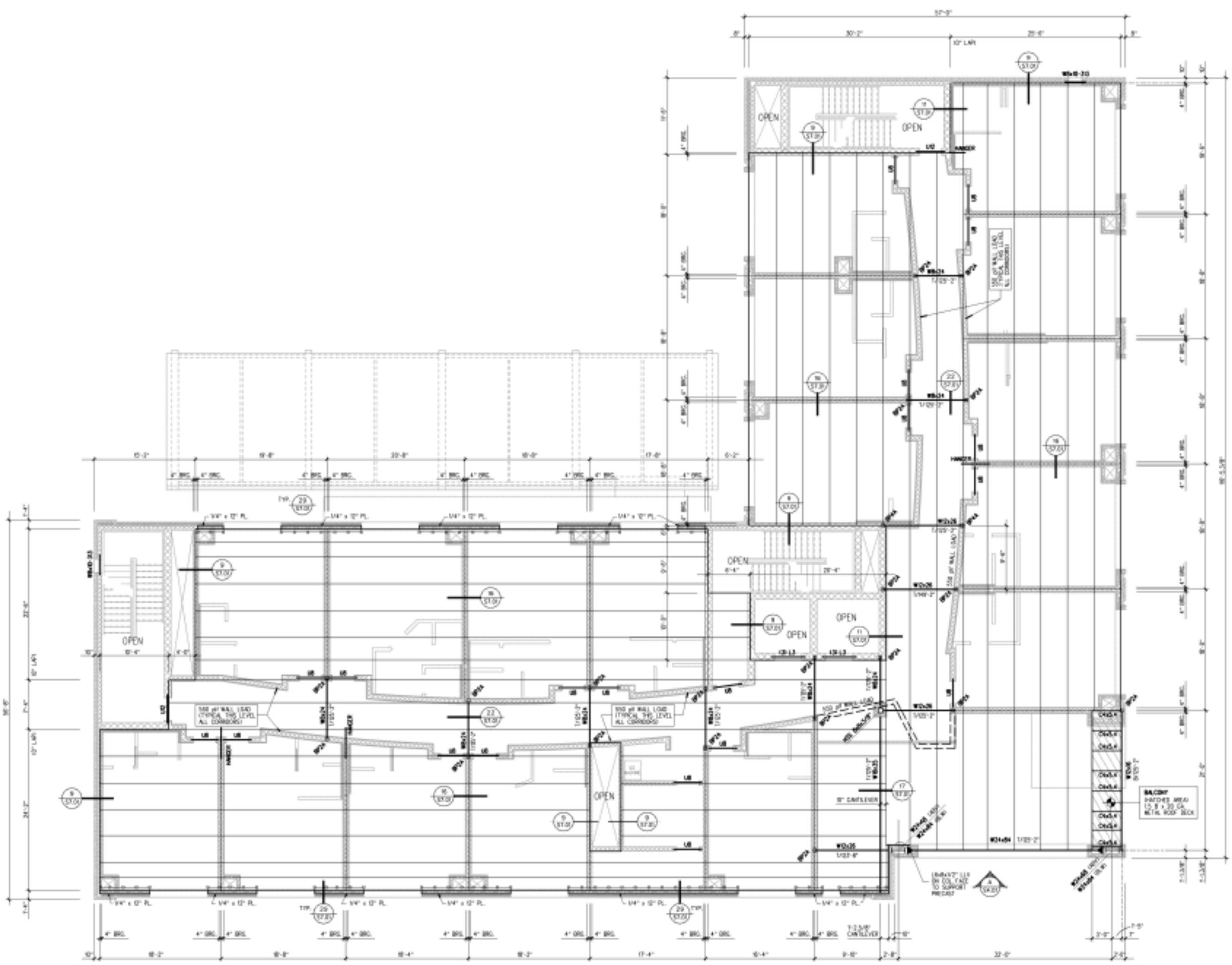
Foundation Plan



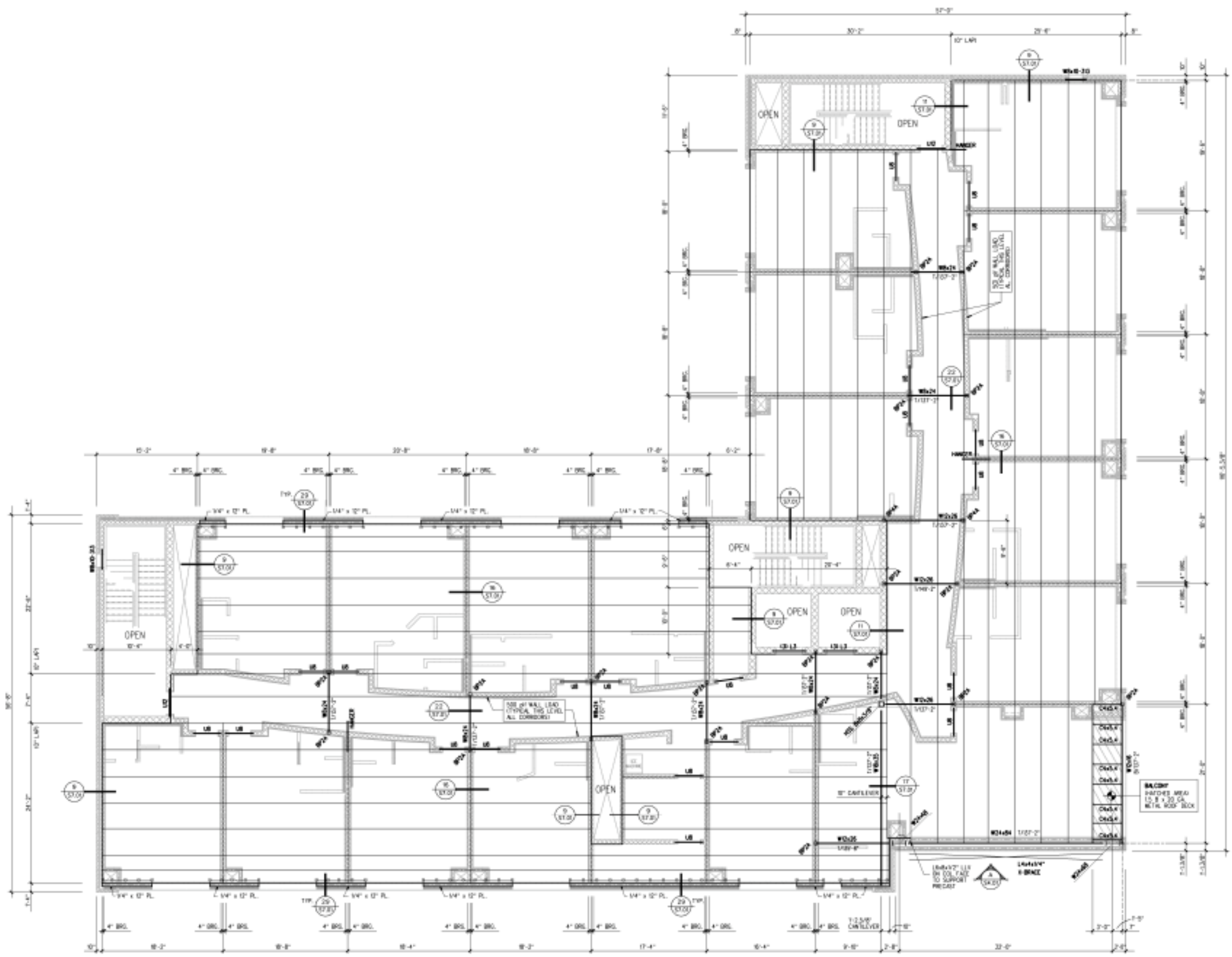
First Level Framing Plan



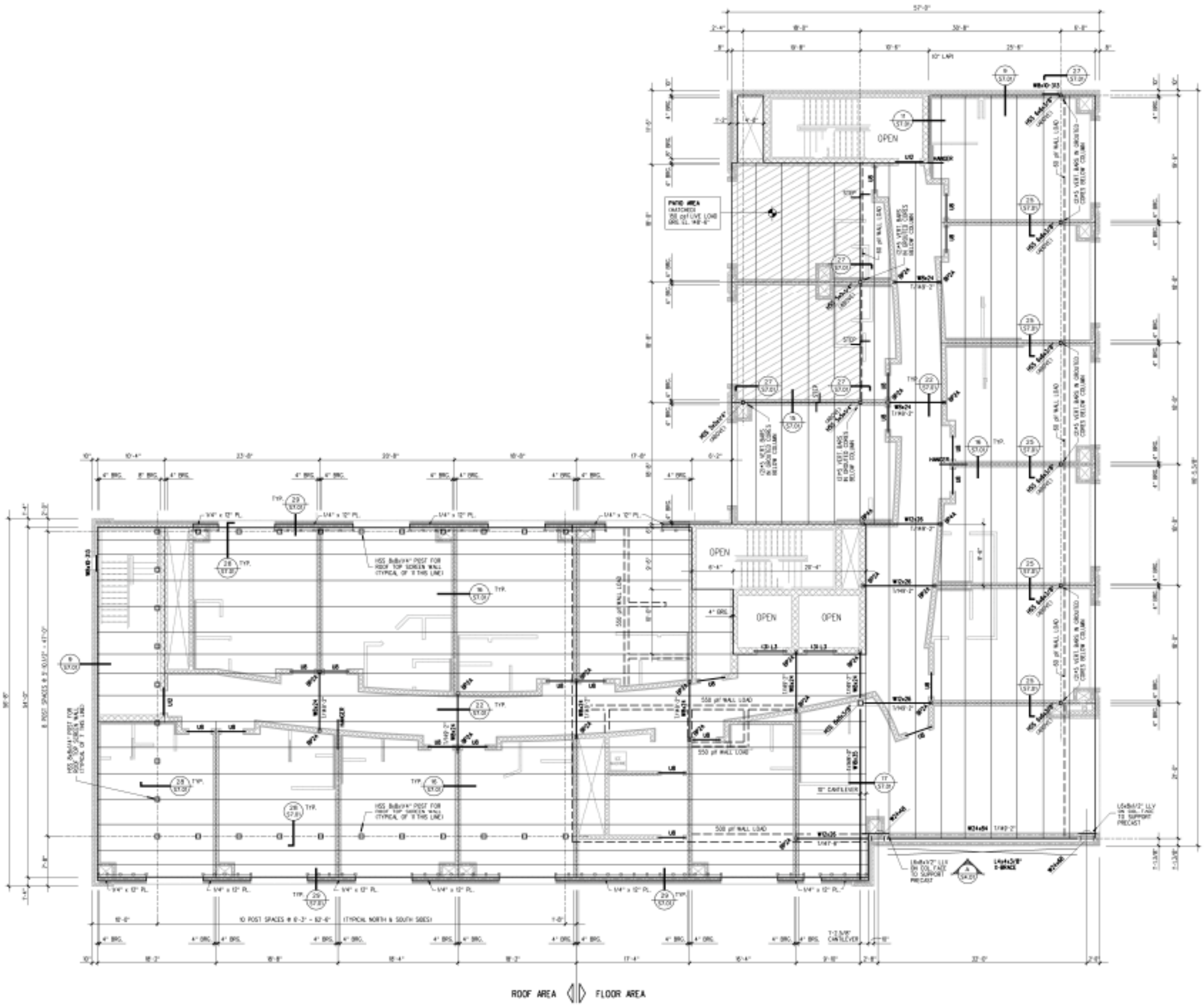
Second Level Framing Plan



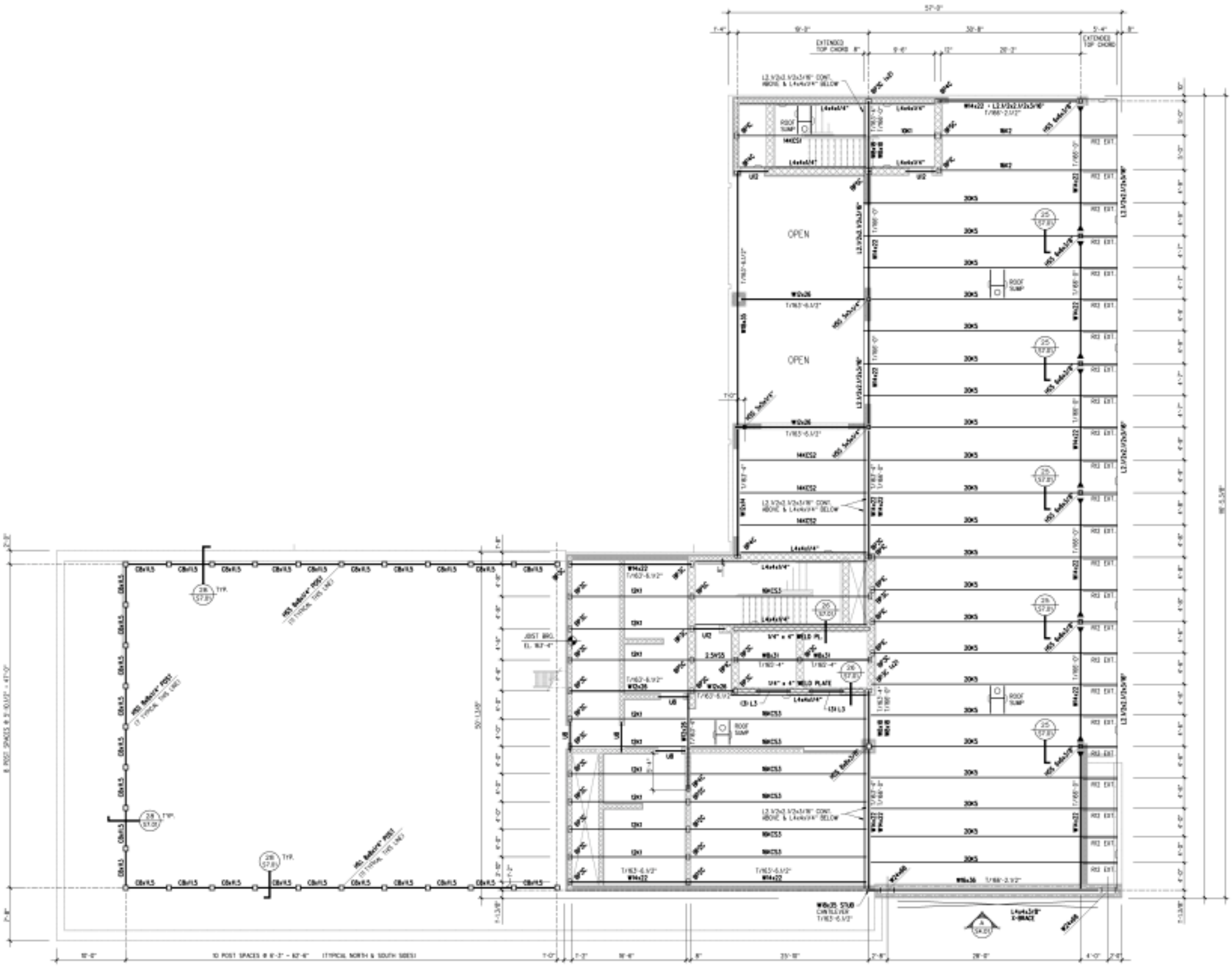
Third Level Framing Plan



Fourth Level Framing Plan



Fifth Level Framing Plan



Sixth Level (Upper Roof) Framing Plan

Appendix B: Existing Floor System

CHAPTER 3 PRELIMINARY DESIGN OF PRECAST / PRESTRESSED CONCRETE STRUCTURES

3

3.6 Hollow-Core Load Tables (cont.)

Strand Pattern Designation
76-S

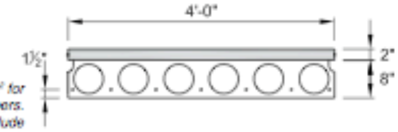
S = straight
Diameter of strand in 16ths
Number of strand (7)

Safe loads shown include dead load of 10 lb/ft² for untopped members and 15 lb/ft² 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
385 – Safe superimposed service load, lb/ft²
0.1 – Estimated camber at erection, in.
0.2 – Estimated long-time camber, in.

4'-0" x 8"
Normalweight Concrete



f'_c = 5000 psi
f_{pu} = 270,000 psi

Section Properties

		No Topping	2 in. topping
A	=	215 in.²	–
I	=	1666 in.⁴	3071 in.⁴
y_b	=	4.00 in.	5.29 in.
y_t	=	4.00 in.	4.71 in.
S_b	=	417 in.³	581 in.³
S_t	=	417 in.³	652 in.³
wt	=	224 lb/ft	324 lb/ft
DL	=	56 lb/ft²	81 lb/ft²
V/S	=	1.92 in.	

4HC8

Table of safe superimposed service load, lb/ft², and cambers, in. **No Topping**

Strand designation code	Span, ft																													
	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
66-S	385	345	313	283	260	240	223	204	179	158	140	124	110	98	87	77	69	61	54	48	43	38	33	29						
	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.2	0.1	0.0	0.0	–0.1	–0.2	–0.3	–0.5	–0.6						
	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.3	–0.5	–0.7	–0.9		–1.4						
76-S	449	407	367	334	309	285	263	242	213	188	167	149	133	119	106	95	86	77	69	62	55	50	44	39	35	31	26			
	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.3	0.3	0.3	0.3	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.5	–0.7	–0.9			
	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.6	–0.8	–1.1	–1.4	–1.7	–2.0			
58-S	422	380	346	316	290	267	247	231	216	202	190	179	169	160	144	130	118	107	97	88	80	72	66	60	54	48	42	37	32	28
	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.5	0.4	0.3	0.2	0.1	0.0	–0.4	–0.3	–0.5	–0.7	–0.9
	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.4	0.3	0.2	0.0	–0.2	–0.4	–0.6	–0.9	–1.2	–1.6	–2.0	–2.4
68-S	476	430	393	361	332	309	286	269	253	235	223	209	200	180	165	153	142	132	121	110	101	92	84	77	70	63	56	51	45	40
	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.2	0.1	–0.1	–0.3
	0.3	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.0	0.9	0.9	0.9	0.8	0.7	0.6	0.4	0.2	0.0	–0.2	–0.5	–0.8	–1.1	–1.5
78-S	488	442	402	370	341	318	295	275	259	241	229	215	203	195	180	168	157	144	135	126	118	110	101	92	84	77	70	64	58	52
	0.3	0.3	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.0	0.9	0.8	0.7	0.6	0.5	0.3
	0.4	0.5	0.5	0.6	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.2	1.3	1.3	1.3	1.3	1.3	1.2	1.2	1.1	1.0	0.8	0.7	0.5	0.3	0.0	–0.3	–0.7

4HC8 + 2

Table of safe superimposed service load, lb/ft², and cambers, in. **2 in. Normalweight Topping**

Strand designation code	Span, ft																											
	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
66-S	400	365	333	308	282	256	224	197	173	153	135	119	105	93	82	68	56	45	36	26								
	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.2	0.1	0.0	–0.0	–0.1	–0.2	–0.3								
	0.2	0.2	0.2	0.2	0.2	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	–1.2	–1.4								
76-S	474	435	396	366	340	304	267	235	208	184	164	146	130	116	103	88	74	62	51	41	31							
	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.3	0.3	0.3	0.3	0.2	0.2	0.1	–0.0	–0.1	–0.2							
	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.5	–0.7	–0.9	–1.2	–1.4							
58-S	445	405	374	342	318	298	275	260	243	228	217	196	177	159	143	126	110	95	82	70	59	49	40	32				
	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.5	0.1	0.3	0.2	0.1	0.0	–0.1				
	0.3	0.3	0.4	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.3	0.2	0.1	–0.1	–0.2	–0.4	–0.6	–0.9	–1.2	–1.5	–1.8				
68-S		463	426	393	366	342	319	299	282	267	251	239	216	195	177	158	140	124	110	97	84	73	62	53	44	36	28	
		0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.2	0.1	–0.1	
		0.4	0.5	0.5	0.6	0.6	0.6	0.6	0.7	0.7	0.7	0.6	0.6	0.6	0.5	0.4	0.3	0.2	0.0	–0.2	–0.4	–0.6	–0.9	–1.2	–1.6	–2.0	–2.4	
78-S		472	435	402	375	348	325	305	288	273	257	245	232	220	207	186	167	149	133	119	106	94	83	73	64	55	46	38
		0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.0	0.9	0.9	0.7	0.6	0.5	0.3
		0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.8	0.9	0.9	0.9	0.9	0.8	0.8	0.7	0.7	0.6	0.4	0.3	0.1	–0.1	–0.3	–0.6	–0.9	–1.3	–1.7	–2.2

Strength is based on strain compatibility; bottom tension is limited to $7.5\sqrt{f'_c}$; see pages 3–8 through 3–11 for explanation.
See item 3, note 4, Section 3.3.2 for explanation of vertical line.

PRECAST HOLLOW CORE CONCRETE PLANKS
ON LOAD BEARING MASONRY & STEEL INTERIOR

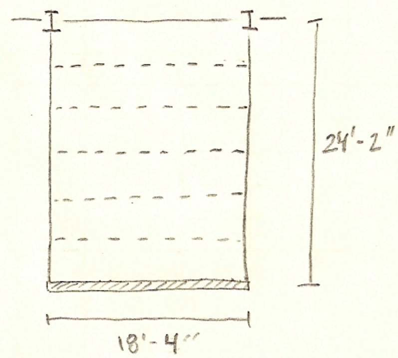
• LOADS:

LIVE LOADS = 40 PSF (GUEST ROOMS)
SUPERIMPOSED DEAD LOADS = 35 PSF
DEAD LOAD = 15 PSF (2" TOPPING FROM PCI HANDBOOK)

TOTAL LOAD = 40 + 35 + 15 = 90 PSF

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

SPAN = 18'-4"



• DESIGNED FOR 8" - W/ TOPPING
4'-0" - 8" NORMAL WEIGHT CONCRETE (4HC8+2)

• FROM PCI DESIGN HANDBOOK:

66-S CARRYING 224 PSF CAPACITY @ 19'-0"
6 STRANDS @ 6/16" ϕ - STRAIGHT

SELF WEIGHT OF SLAB = 81 PSF

• LOAD TO MASONRY BEARING WALLS

$w_u = 1.2(35 + 81) + 1.6(40) \Rightarrow w_u = 203.2$ PSF

$M_u = \dfrac{203.2 \text{ PSF}(24'-2'')(18'-4'')^2}{8} = 206.3$ ft-K => 207 ft-K

A_{ps} = 6 STRANDS @ 6/16" ϕ = 6(0.375) = 2.25 IN^2

$f_{ps} \cong 270$ KSI   b = 4'-0" (12) = 48"

d_p = 10" - 1½" CLR = 8.5"

$a = \dfrac{A_{ps} f_{ps}}{0.85 f'_c b} = \dfrac{(2.25 \text{ in}^2)(270 \text{ ksi})}{0.85(5 \text{ ksi})(48 \text{ in})} = 2.98$ IN

$\phi M_N = \phi [A_{ps} f_{ps} (d_p - a/2)] = 0.9[2.25(270)(8.5 - 2.98/2)] = 3833$ IN-K

ϕM_N = 319 ft-K > M_u = 207 ft-K ∴ OK DESIGN

HOLLOW-CORE PLANK (CONTINUED)

- DEFLECTION

$E_c = 57000\sqrt{f'_c} = 57000\sqrt{5000} = 4030 \text{ KSI}$

$I = 3071 \text{ IN}^4$ (2" TOPPING)

$\Delta_{LL} = \ell/360 = (18'-4")(12)/360 = 0.61 \text{ IN}$

$\Delta_{LL} = \dfrac{5(40)(24'-2")(18'-4")^4}{384(4030000)(3071)} \times 1728 = 0.20 \text{ IN}$

$0.20 \text{ IN} < 0.61 \text{ IN} \therefore \text{OK}$

$\Delta_{TL} = \ell/240 = (18'-2")(12)/240 = 0.91 \text{ IN}$

$\Delta_{TL} = \dfrac{5(40+35+81)(24'-2")(18'-4")^4}{384(4030000)(3071)} \times 1728 = 0.77 \text{ IN}$

$0.77 \text{ IN} < 0.91 \text{ IN} \therefore \text{OK}$

EXISTING DESIGN EFFICIENT IN CARRYING LOADS

Appendix C: Alternative System #1

Precast Hollow-Core concrete Plank on Steel Framing

CHAPTER 3 PRELIMINARY DESIGN OF PRECAST / PRESTRESSED CONCRETE STRUCTURES

3.6 Hollow-Core Load Tables (cont.)

3

Strand Pattern Designation

76-S

- S = straight
- Diameter of strand in 16ths
- Number of strand (7)

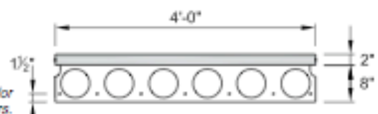
Safe loads shown include dead load of 10 lb/ft² for untopped members and 15 lb/ft² for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

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

Key

385 – Safe superimposed service load, lb/ft²
0.1 – Estimated camber at erection, in.
0.2 – Estimated long-time camber, in.

4'-0" × 8"
Normalweight Concrete



f'_c = 5000 psi
f_{pu} = 270,000 psi

Section Properties

	No Topping	2 in. topping
A =	215 in.²	-
I =	1666 in.⁴	3071 in.⁴
y_b =	4.00 in.	5.29 in.
y_t =	4.00 in.	4.71 in.
S_b =	417 in.³	581 in.³
S_t =	417 in.³	652 in.³
wt =	224 lb/ft	324 lb/ft
DL =	56 lb/ft²	81 lb/ft²
V/S =	1.92 in.	

4HC8

Table of safe superimposed service load, lb/ft², and cambers, in. — No Topping

Strand designation code	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
66-S	385	345	313	283	260	240	223	204	179	158	140	124	110	98	87	77	69	61	54	48	43	38	33	29						
	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.2	0.1	0.0	0.0	−0.1	−0.2	−0.3	−0.5	−0.6						
	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.1	0.0	−0.1	−0.2	−0.3	−0.5	−0.7	−0.9		−1.4						
76-S	449	407	367	334	309	285	263	242	213	188	167	149	133	119	106	95	86	77	69	62	55	50	44	39	35	31	26			
	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.3	0.3	0.3	0.3	0.2	0.2	0.1	0.0	−0.1	−0.2	−0.4	−0.5	−0.7	−0.9			
	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	0.0	−0.1	−0.2	−0.4	−0.6	−0.8	−1.1	−1.4	−1.7	−2.0			
58-S	422	380	346	316	290	267	247	231	216	202	190	179	169	160	144	130	118	107	97	88	80	72	66	60	54	48	42	37	32	28
	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.5	0.4	0.3	0.2	0.1	0.0	−0.4	−0.3	−0.5	−0.7	−0.9
	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.4	0.3	0.2	0.0	−0.2	−0.4	−0.6	−0.9	−1.2	−1.6	−2.0	−2.4
68-S	476	430	393	361	332	309	286	269	253	235	223	209	200	180	165	153	142	132	121	110	101	92	84	77	70	63	56	51	45	40
	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.2	0.1	−0.1	−0.3
	0.3	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.0	0.9	0.9	0.9	0.8	0.7	0.6	0.4	0.2	0.0	−0.2	−0.5	−0.8	−1.1	−1.5
78-S	488	442	402	370	341	318	295	275	259	241	229	215	203	195	180	168	157	144	135	126	118	110	101	92	84	77	70	64	58	52
	0.3	0.3	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.0	0.9	0.8	0.7	0.6	0.5	0.3
	0.4	0.5	0.5	0.6	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.2	1.3	1.3	1.3	1.3	1.3	1.2	1.2	1.1	1.0	0.8	0.7	0.5	0.3	0.0	−0.3	−0.7

4HC8 + 2

Table of safe superimposed service load, lb/ft², and cambers, in. — 2 in. Normalweight Topping

Strand designation code	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
66-S	400	365	333	308	282	256	224	197	173	153	135	119	105	93	82	68	56	45	36	26								
	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.2	0.1	0.0	−0.0	−0.1	−0.2	−0.3								
	0.2	0.2	0.2	0.2	0.2	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	−1.2	−1.4								
76-S	474	435	396	366	340	304	267	235	208	184	164	146	130	116	103	88	74	62	51	41	31							
	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.3	0.3	0.3	0.3	0.2	0.2	0.1	−0.0	−0.1	−0.2							
	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.1	0.0	−0.1	−0.2	−0.4	−0.5	−0.7	−0.9	−1.2	−1.4							
58-S	445	405	374	342	318	298	275	260	243	228	217	196	177	159	143	126	110	95	82	70	59	49	40	32				
	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.5	0.1	0.3	0.2	0.1	0.0	−0.1				
	0.3	0.3	0.4	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.3	0.2	0.1	−0.1	−0.2	−0.4	−0.6	−0.9	−1.2	−1.5	−1.8				
68-S		463	426	393	366	342	319	299	282	267	251	239	216	195	177	158	140	124	110	97	84	73	62	53	44	36	28	
		0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.2	0.1	−0.1	
		0.4	0.5	0.5	0.6	0.6	0.6	0.6	0.7	0.7	0.7	0.6	0.6	0.6	0.5	0.4	0.3	0.2	0.0	−0.2	−0.4	−0.6	−0.9	−1.2	−1.6	−2.0	−2.4	
78-S		472	435	402	375	348	325	305	288	273	257	245	232	220	207	186	167	149	133	119	106	94	83	73	64	55	46	38
		0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.0	0.9	0.9	0.7	0.6	0.5	0.3
		0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.8	0.9	0.9	0.9	0.9	0.8	0.8	0.7	0.7	0.6	0.4	0.3	0.1	−0.1	−0.3	−0.6	−0.9	−1.3	−1.7	−2.2

Strength is based on strain compatibility; bottom tension is limited to $7.5\sqrt{f'_c}$; see pages 3–8 through 3–11 for explanation.
See item 3, note 4, Section 3.3.2 for explanation of vertical line.

PRECAST HOLLOW-CORE CONCRETE PLANKS
ON STEEL FRAMING

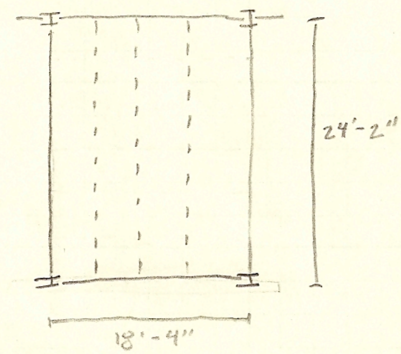
• LOADS:

LIVE LOADS = 40 PSF (GUEST ROOMS)
SUPERIMPOSED DEAD LOADS = 35 PSF
DEAD LOAD = 10 PSF (NO TOPPING FROM PCI HANDBOOK)

TOTAL LOAD = 40+35+10 = 85 PSF

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

SPAN = 18'-4"



• DESIGNED FOR 8" W/O TOPPING
4'-0" x 8" NORMAL WEIGHT CONCRETE (4HC6)

• FROM PCI DESIGN HANDBOOK:

66-S CARRYING 98 PSF CAPACITY @ 24'-0"
6 STRANDS @ 9/16" ϕ - STRAIGHT

SELF WEIGHT OF SLAB = 56 PSF

• LOAD TO GIRDERS

$w_u = 1.2(35+56) + 1.6(40) = 173.2$ PSF

$M_u = \frac{173.2 \text{ PSF} (18'-4")(24'-2")^2}{8} = 231.8$ ft-k => 232 ft-k

• USE W18x35 (AISC TABLE 3-2)

$\phi M_n = 249$ ft-k > $M_u = 232$ ft-k ∴ OK FOR W18x35

$\Delta_{LL} = \ell/360 = (18'-4")(12)/360 = 0.61$ IN

$0.61 = \frac{5(40)(24'-2")(18'-4")^4(1728)}{384(29000)I_x(1000)}$ => $I_x = 138.9 \text{ in}^4 < 510 \text{ in}^4$ FOR W18x30 ∴ OK

$\Delta_{TL} = \frac{5(40+35+56)(24'-2")(18'-4")^4(1728)}{384(29000)(510)(1000)} = 0.54$ IN

$\Delta_{TL} = 0.54 \text{ IN} < \ell/240 = (18'-4")(12)/240 = 0.92$ IN ∴ OK FOR W18x35

HOLLOW-CORE PLANK ON STEEL (CONTINUED)

* IN ORDER TO OBTAIN A LESS DEEP SYSTEM, A WIDE FLANGE WITH A SMALLER DEPTH CAN BE USED.

- USE W12 × 50 (LESS ECONOMICAL DUE TO GREATER WEIGHT)

$\phi M_N = 270 \text{ ft-K} > M_U = 232 \text{ ft-K} \quad \therefore \text{OK}$

$\Delta_{LL} = \ell/360 = (18'-4'')(12)/360 = 0.61 \text{ IN}$

$$0.61 = \frac{5(40)(24'-2'')(18'-4'')^4(1728)}{384(29000)\, I_x\, (1000)} \Rightarrow I_x = 138.9 \text{ IN}^4 < 391 \text{ IN}^4$$

$\therefore$ OK FOR W12×50

$$\Delta_{TL} = \frac{5(40+35+56)(24'-2'')(18'-4'')^4(1728)}{384(29000)(391)(1000)} = 0.71 \text{ IN}$$

$\Delta_{TL} = 0.71 \text{ IN} < \ell/240 = (18'-4'')(12)/240 = 0.92 \text{ IN} \quad \therefore$ OK FOR W12×50

COULD ALSO USE A W16×36 TO BE MORE ECONOMICAL THAN THE W12×50 AND STILL BE LESS DEEP THAN THE W18×35

FOR A W16×36

$\phi M_N = 240 \text{ ft-K} > M_U = 232 \text{ ft-K}$

$I_x = 448 \text{ IN}^4 > 138.9 \text{ IN}^4$ NEEDED FOR LIVE LOAD

$\Delta_{TL} = 0.62 \text{ IN} < 0.92 \text{ IN}$ MAX FOR TOTAL LOAD DEFLECTION

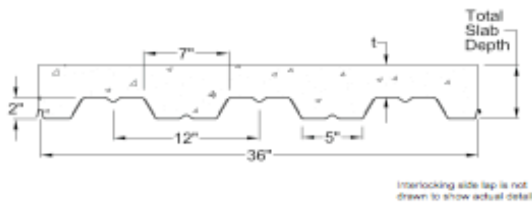
Appendix D: Alternative System #2

Composite Steel Deck System

VULCRAFT

2 VLI
No Studs

Maximum Sheet Length 42'-0"
Extra charge for lengths under 6'-0"
ICBO Approved (No. 3415)



STEEL SECTION PROPERTIES

Deck type	Design thickness (in.)	Weight psf	Section Properties I_p in⁴/ft	S_p in³/ft	I_n in⁴/ft	S_n in³/ft	V_a lbs/ft	F_y ksi
2VLI22	0.0295	1.82	0.324	0.263	0.321	0.288	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	2.81	0.559	0.495	0.558	0.504	3808	50
2VLI16	0.0598	3.29	0.704	0.653	0.704	0.653	3818	40

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

2VLI - NO STUDS

Total Slab Depth	Deck Type	SDI Max. Unshored Clear Span			Superimposed Live Load (PSF) Clear Span (ft.-in.)														
		1 Span	2 Span	3 Span	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
4.50 (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
5.00 (t=3.00) 51 PSF	2VLI22	6'-7	8'-7	8'-11	384	317	279	217	192	171	153	138	125	113	103	94	86	79	72
	2VLI20	7'-9	9'-10	10'-2	400	356	313	278	249	193	173	156	141	128	116	106	97	89	82
	2VLI19	8'-9	10'-11	11'-3	400	394	345	306	273	247	224	172	156	141	128	117	107	99	91
	2VLI18	9'-7	11'-10	11'-11	400	400	377	336	301	273	249	228	210	162	148	136	126	116	107
	2VLI16	9'-11	12'-0	12'-4	400	400	400	386	346	312	283	259	238	187	171	157	144	133	123
5.50 (t=3.50) 57 PSF	2VLI22	6'-4	8'-0	8'-6	400	355	278	244	216	192	172	155	140	127	116	106	97	89	81
	2VLI20	7'-5	9'-5	9'-9	400	400	351	312	244	217	194	175	158	143	131	119	109	100	92
	2VLI19	8'-4	10'-5	10'-9	400	400	388	343	307	277	215	193	175	159	144	132	121	111	102
	2VLI18	9'-2	11'-4	11'-7	400	400	400	377	338	306	279	256	199	182	167	153	141	130	121
	2VLI16	9'-5	11'-6	11'-10	400	400	400	400	388	350	318	290	230	210	192	176	162	150	138
6.00 (t=4.00) 63 PSF	2VLI22	6'-1	7'-5	8'-2	400	394	308	270	239	213	191	172	156	141	129	118	108	99	90
	2VLI20	7'-1	9'-1	9'-4	400	400	390	346	271	241	215	194	175	159	145	132	121	111	102
	2VLI19	8'-0	10'-1	10'-5	400	400	400	381	340	307	239	215	194	176	160	146	134	123	113
	2VLI18	8'-10	10'-11	11'-3	400	400	400	400	375	339	309	243	221	202	185	170	157	145	134
	2VLI16	9'-1	11'-1	11'-5	400	400	400	400	400	388	352	322	255	233	213	195	180	166	154
6.50 (t=4.50) 69 PSF	2VLI22	5'-11	6'-11	7'-11	400	390	339	297	263	234	210	189	171	155	141	129	118	108	99
	2VLI20	6'-11	8'-9	9'-0	400	400	400	337	297	264	237	213	193	175	159	145	133	122	112
	2VLI19	7'-10	9'-8	10'-0	400	400	400	400	374	293	262	236	213	193	176	161	147	135	124
	2VLI18	8'-7	10'-6	10'-11	400	400	400	400	400	373	340	268	243	222	203	187	172	159	147
	2VLI16	8'-10	10'-8	11'-0	400	400	400	400	400	400	387	309	280	256	234	215	198	183	169

Notes:
1. Minimum exterior bearing length required is 2.00 inches. Minimum interior bearing length required is 4.00 inches. If these minimum lengths are not provided, web crippling must be checked.
2. Always contact Vulcraft when using loads in excess of 200 psf. Such loads often result from concentrated, dynamic, or long term load cases for which reductions due to bond breakage, concrete creep, etc. should be evaluated.
3. All fire rated assemblies are subject to an upper live load limit of 250 psf.
4. 3/4 in. diameter welded shear stud utilized for calculations.
5. Refer to AISC for further stud material and installation requirements.

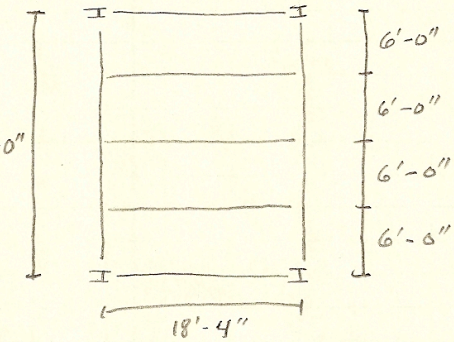
COMPOSITE STEEL DECK

• LOADS

LIVE LOADS = 40 PSF (GUEST ROOMS)
SUPERIMPOSED DEAD LOADS = 35 PSF
DEAD LOAD = 45 PSF (SLAB SELF WEIGHT)

• VULCRAFT DECK USED

SLAB DEPTH = 4.5"
TOPPING = 2.5"
NORMAL WEIGHT CONCRETE (145 PCF)
3 SPAN CONDITION
USE: 2VLI22 DECK
f'_c = 3000 PSI
$F_{y,steel}$ = 50,000 PSI



• TOTAL LOAD = 40 + 35 + 45 = 120 PSF

• DECK USED = 2VLI22 DECK, 3 SPAN
CLEAR SPAN = 6'-0"
22 GAUGE
SUPERIMPOSED LL MAX. CAPACITY = 278 PSF > 120 PSF ∴ OK
F_b = 30,000

• BEAM:

LOAD = 1.2D + 1.6L = 1.2(35 + 45) + 1.6(40) = 160 PSF = 0.16 KSF
TRIB LENGTH = 6'-0"

w_u = 6'(0.16 KSF) = 0.96 KLF

$V_u = \frac{0.96(18'-4")}{2} = 8.8^k$ $M_u = \frac{0.96(18'-4")^2}{8} = 40.3$ ft-k

b_{eff} = MIN
| $2\left(\frac{SPAN}{8}\right) = 2\left(\frac{(18'-4")(12)}{8}\right) = 27.5"$ ⇒ CONTROLS
| $2(\frac{1}{2})$ SPACING = 6'(12) = 72"

ASSUME a = 1.0
$Y_2 = t_{SLAB} - a/2 = 4.5" - \frac{1}{2} = 4"$

FOR 3/4" φ STUDS AND 3000 PSI NORMAL WEIGHT CONCRETE

$Q_N = 17.2^k$ (DECK PERPENDICULAR)

COMPOSITE STEEL DECK (CONTINUED)

TRY W10×12

$\phi M_N = 73.2$ ft-k $\phi M_p = 47.3$ ft-k PNA = 7 $\Sigma Q_N = 44.2^k$

$$a = \frac{\Sigma Q_N}{0.85 f'_c\, b_{eff}} = \frac{44.2}{0.85(3)(27.5)} = 0.63'' < 1'' \therefore \text{OK}$$

$$Y_2 = 4.5 - 0.63/2 = 4.19'' > 4.0'' \therefore \text{CONSERVATIVE}$$

SHEAR STUDS => $\frac{\Sigma Q_N}{Q_N} = \frac{44.2}{17.2} = 2.57 \rightarrow 3$ STUDS/HALF = 6 STUDS

CHECK UNSHORED STRENGTH

C_{LL} = 20 PSF (6') = 0.120 KLF
$w_{LL} = C_{LL}$ = 0.120 KLF
w_{DL} = (45 PSF)(6') + 12 PLF = 0.282 KLF
$w_U = 1.2w_D + 1.6w_L = 1.2(0.282) + 1.6(0.120) = 0.5304$ KLF

$$M_U = \frac{w_U l^2}{8} = \frac{0.5304(18'-4'')^2}{8} = 22.3 \text{ ft-k} < \phi M_p = 47.3 \text{ ft-k} \therefore \text{OK}$$

CHECK MEMBER STRENGTH

$\phi M_N = 73.2$ ft-k > $M_U = 40.3$ ft-k ∴ OK

$\phi V_N = 56.3$ k >> $V_U = 8.8^k$ ∴ OK

CHECK LIVE LOAD DEFLECTION

w_{LL} = (40 PSF)(6') = 0.240 KLF $I_{LB} = 110$ IN^4

$$\Delta_{LL} = \frac{5wl^4}{384EI} = \frac{5(0.240)(18'-4'')^4(1728)}{384(29000)(110)} = 0.191''$$

$$l/360 = (18'-4'')(12)/360 = 0.611'' > 0.191'' \therefore \text{OK}$$

CHECK WET CONCRETE DEFLECTION

w_{WC} = 45 PSF (6') + 12 PLF = 0.282 KLF $I_s = 53.8$ IN^4

$$\Delta_{WC} = \frac{5wl^4}{384EI} = \frac{5(0.282)(18'-4'')^4(1728)}{384(29000)(53.8)} = 0.459''$$

$$\Delta_{WC,MAX} = l/240 = (18'-4'')(12)/240 = 0.917'' > 0.459'' \therefore \text{OK}$$

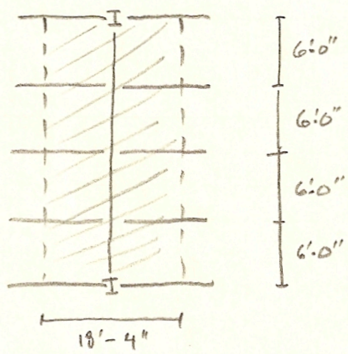
USE W10×12 (6)

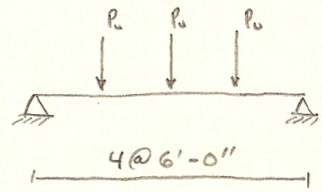
COMPOSITE STEEL DECK (CONTINUED)

• GIRDERS

TOTAL DEAD LOAD = (45 PSF + 35 PSF)(6') = 0.480 KLF
TOTAL LIVE LOAD = (40 PSF)(6') = 0.240 KLF

$\text{TOTAL DEAD LOAD}_{\text{, GIRDERS}}$ = (0.480 KLF)(24') = 11.52^{K}
$\text{TOTAL LIVE LOAD}_{\text{, GIRDERS}}$ = (0.240 KLF)(24') = 5.76^{K}





$P_U = 1.2D + 1.6L = 1.2(11.52) + 1.6(5.76) = 23.04^{K}$

$V_U = P_U = 23.04^{K}$

$M_U = 6'(23.04^{K}) = 138.24$ ft-K

b_{eff} = MIN | $2\left(\frac{\text{SPAN}}{8}\right) = 2\cdot\left(\frac{24(12)}{8}\right) = 72''$ → CONTROLS
| $2(1/2)$ SPACING = (18'-4")(12) = 220"

ASSUME $a = 1.0$
$Y_2 = 4.5 - 1/2 = 4''$

FOR 3/4" ϕ STUDS AND 3000 PSI NORMAL WEIGHT CONCRETE

$Q_N = 21.0^{K}$ (DECK PARALLEL)

TRY W12×22 – $\phi M_N = 162$ ft-k $\phi M_p = 110$ ft-k PNA=7 $\Sigma Q_N = 81.0^{K}$

$a = \frac{81}{0.85(3)(72)} = 0.441'' < 1.0''$ ∴ OK

$Y_2 = 4.5 - 0.441/2 = 4.30'' > 4.0''$ ∴ CONSERVATIVE

SHEAR STUDS ⇒ $\frac{81.0}{21.0} = 3.86$ → 4 STUDS/HALF = 8 STUDS

COMPOSITE STEEL DECK (CONTINUED)

CHECK UNSHORED STRENGTH

$w_U = 1.2D + 1.6L = 1.2(0.022) = 0.0264$ KLF
$C_{LL} = 20$ PSF $(6')(18'-4'') = 2.2^K$
$C_{DL} = [45$ PSF $(6') + 12$ PLF$](18'-4'') = 5.17^K$

$P_U = 1.2D + 1.6L = 1.2(5.17) + 1.6(2.2) = 9.724^K$
$V_U = P_U = 9.724^K$

$$M_U = \frac{w_U l^2}{8} + \frac{P_U l}{4} = \frac{0.0264(24)^2}{8} + \frac{9.724(24)}{4} = 60.2 \text{ ft-K}$$

$M_U = 60.2$ ft-K $< \phi M_p = 110$ ft-K ∴ OK

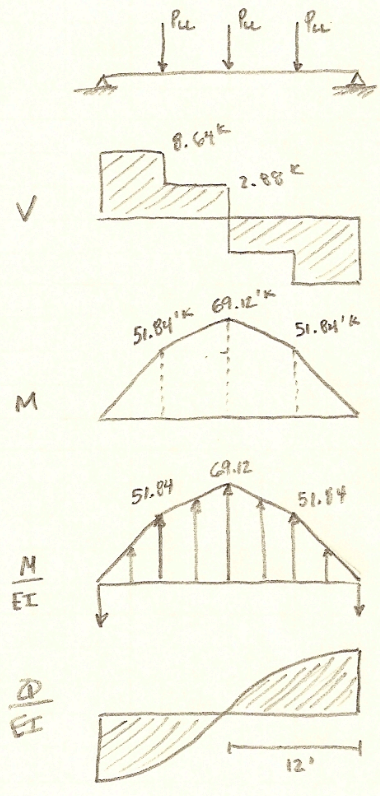
CHECK MEMBER STRENGTH

$\phi M_N = 162$ ft-K $> M_U = 138.24$ ft-K ∴ OK

$\phi V_N = 96^K >> V_U = 9.724^K$

CHECK LIVE LOAD DEFLECTION

$P_{LL} = 5.76^K$ $I_{LB} = 290$ in^4



155.52 51.94 311.04

$$\frac{51.84(6)}{2} + \frac{(69.12 - 51.84)(6)}{2} + 51.84(6) = QA$$

$$\frac{QA}{EI} = \frac{518.2}{EI}$$

$$\Delta_{LL} = \frac{(518.2)(12)(2/3)(1728)}{(29000)(290)} = 0.852''$$

$$\Delta_{LL,MAX} = l/360 = 24'(12)/360 = 0.8''$$

∴ NO GOOD

$$I_{REQ} = \frac{(518.2)(12)(2/3)(1728)}{(29000)(0.8'')} = 308.8 \text{ in}^4$$

∴ **USE W16×31 (8)** $\phi M_p = 203$ ft-k
$I_x = 375$ in^4

Appendix E: Alternative System #3

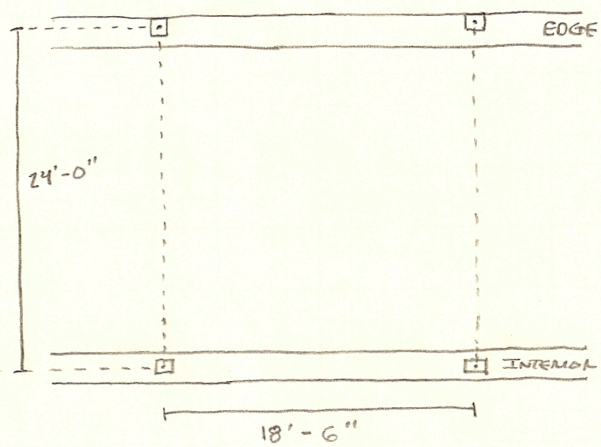
One-Way Joist System

ONE-WAY JOIST SYSTEM

ASSUME: NORMAL WEIGHT CONCRETE (150 PCF)
$f'_c = 4$ KSI
$f_y = 60$ KSI

EDGE BEAM WIDTH = 24"
INTERIOR BEAM WIDTH = 24"

6" WIDE JOISTS SPACED 66" O.C.

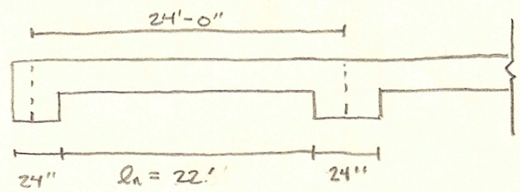


- SLAB

4.5" THICK w/ 2 HR FIRE RATING
$w_{SDL} = 35$ PSF
$w_{LL} = 40$ PSF
$w_{DL} = (4.5/12)(150) = 56.25$ PSF

$w_u = 1.2(35 + 56.25) + 1.6(40)$
$= 173.5$ PSF

FOR 1' STRIP
$w_u = (1')(173.5 \text{ PLF})$
$= 173.5$ PLF $= 0.1735$ KLF



$$M_u = \frac{w_u \ell_n^2}{10} = \frac{(.1735)(66/12)^2}{10}$$

$M_u = 0.525$ k-ft / ft OF SLAB

PRELIMINARY PAN DEPTH:
14" FOR A 20 × 25 BAY SIZE, 66" PAN

MINIMUM REINFORCEMENT

$A_{s,TOP} = 0.0018(4.5)(12) = 0.0972 \text{ IN}^2 \Rightarrow$ TRY #3 BARS $\rightarrow A_s = 0.11 \text{ IN}^2$

$$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{0.11(60)}{.85(4)(12)} = 0.162"$$

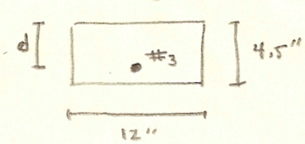
$$\phi M_n = 0.9 A_s F_y (d - a/2) = 0.9(0.11)(60)\left(\frac{4.5}{2} - \frac{0.162}{2}\right) = 1.07 \text{ ft-k / ft SLAB}$$

> 0.525 ft-k / ft SLAB
$\therefore$ OK

SPACING

$3t = 3(4.5) = 13.5" \Rightarrow$ USE 12"

$\therefore$ (1) #3 @ 12" O.C



ONE WAY JOIST (CONTINUED)

• JOIST

$w_{SDL} = 35\ PSF\ (6') = 210\ PLF$
$w_{SLAB} = (4.5''/12)(150\ PCF)(6') = 337.5\ PLF$
$w_{SELF} = (14'')(6'')(150\ PCF)/144 = 87.5\ PLF$
$w_{LL} = 40\ PSF\ (6') = 240\ PLF$

$w_u = 1.2(0.210 + 0.3375 + 0.0875) + 1.6(0.240) = 1.146\ KLF$

$M^+_{MAX} = \frac{w_u l_n^2}{14} = \frac{1.146(22)^2}{14} = 39.6\ ft\text{-}k$

$M^-_{MAX} = \frac{w_u l_n^2}{10} = \frac{1.146(22)^2}{10} = 55.5\ ft\text{-}k$

TOP REINFORCEMENT

$A_s = M_u/4d = 55.5/4(16.25) = 0.854\ in^2$ WHERE $d = 18.5 - \left(\frac{4.5''}{2}\right) = 16.25''$

TRY (2) #6 => $A_s = 2(0.44) = 0.88\ in^2$

$\rho = A_s/bd = 0.88/(6'')(16.25) = 0.009$

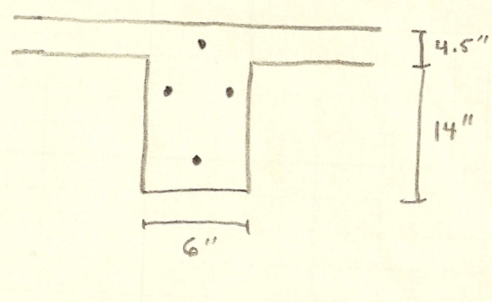
$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.88(60)}{.85(4)(6)} = 2.59''$ $c = a/\beta = 2.59/0.85 = 3.05''$

$\varepsilon_s = \frac{0.003}{c}(d-c) = \frac{0.003}{3.05}(16.25 - 3.05) = 0.013 > 0.005$

∴ TENSION CONTROLLED
$\phi = 0.9$

$\phi M_n = \phi A_s f_y (d - a/2)$
$= [0.9(0.88)(60)(16.25 - \frac{2.59}{2})]/12 = 59.2\ ft\text{-}k > 55.5\ ft\text{-}k$ ∴ OK

USE (2) #6



ONE-WAY JOIST SYSTEM (CONTINUED)

BOTTOM REINFORCEMENT

$d = 18.5'' - 1.5'' - 0.375'' - 0.5'' = 16.125''$

↳ COVER ↳ #3 STIRRUP ↳ #8 BAR

$A_s = M_u / 4d = 39.6 / 4(16.125) = 0.614 \text{ IN}^2$

$b_{eff} = \min \begin{cases} 1/4 L = 1/4 (24)(12) = 72 \\ 16h_f + b_w = 16(4.5) + 6 = 78 \\ b_w + L_x = 6 + 66 = \boxed{72} \end{cases}$

TRY (1) #8 $\Rightarrow A_s = 0.79 \text{ IN}^2$

$\rho = A_s / bd = 0.79 / 6(16.125) = 0.0082$

$a = A_s f_y / 0.85 f'_c b = 0.79(60) / .85(4)(72) = 0.194''$ ∴ NA IS IN FLANGE

$c = 0.194 / 0.85 = 0.228$

$\varepsilon_s = \frac{0.003}{0.228}(16.125 - 0.228) = 0.208 > 0.005$ ∴ TENSION CONTROLLED

$\phi = 0.9$

$\phi M_n = \phi A_s f_y (d - \frac{a}{2}) = [0.9(.79)(60)(16.125 - \frac{.194}{2})]/12 =$

$\phi M_n = 57.0$ ft-k > 37.1 ft-k ∴ OK

USE (1) #8

SHEAR DESIGN

$V_u = \frac{1.15 w_u l_n}{2} = \frac{1.15(1.146)(22)}{2} = 14.5^k$

$\phi V_c = \phi 2 \sqrt{f'_c}\, b_w d = 0.75(2)\sqrt{4000}(6)(16.125) = 9.2^k$

$\phi V_s = V_u - \phi V_c = 14.5^k - 9.2^k = 5.3^k$

$\phi V_s = 5.1^k = \frac{\phi A_{sv} f_y d}{S_{MAX}} \Rightarrow 5.1^k = \frac{A_{sv}(60)(16.125)(0.75)}{8''} \Rightarrow A_{sv} = 0.05 \text{ IN}^2$

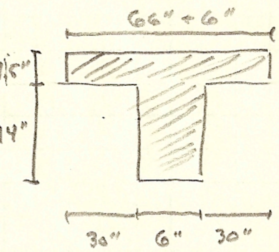
$* \; S_{MAX} = \min \begin{cases} d/2 = \frac{16.125}{2} = 8.0625'' \\ 24'' \end{cases} \Rightarrow$ USE 8''

∴ USE #3 @ 8'' SPACING

$\phi V_c + \phi V_s = 9.2^k + \frac{0.75(0.11)(60)(16.125)}{8} = 19.2^k > V_u = 14.5^k$

∴ OK

- JOIST DEFLECTION



$$\bar{y} = \frac{4.5(72)(14+2.25) + 14(6)(7)}{4.5(72) + 14(6)} = 14.35 \text{ IN}$$

$$I = \frac{72(4.5)^3}{12} + (72)(4.5)(1.9)^2 + \frac{6(14)^3}{12} + 6(14)(7.35)^2$$

$$= 546.75 + 1169.64 + 1372 + 4537.89 = 7626.28 \text{ IN}^4$$

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

$$w_{LL} = 40 \text{ PSF}(6') = 240 \text{ PLF}$$
$$w_{DL} = 150 \text{ PLF} + 337.5 \text{ PLF} + 87.5 \text{ PLF} = 575 \text{ PLF}$$

$$w_{TL} = 240 \text{ PLF} + 575 \text{ PLF} = 815 \text{ PLF}$$

$$\Delta_{LL} = \frac{5 w_{LL} \ell_n^4}{384 EI} = \frac{5(0.24)(22')^4(1728)}{384(3834)(7626.28)} = 0.04''$$

$$\Delta_{LL} = \ell/360 = 24'(12)/360 = 0.8'' \gg 0.04'' \therefore \text{OK}$$

$$\Delta_{TL} = \frac{5(.815)(22)^4(1728)}{384(3834)(7626.28)} = 0.15''$$

$$\Delta_{TL} = \ell/240 = 24(12)/240 = 1.2'' \gg 0.15'' \therefore \text{OK}$$

ONE-WAY JOIST SYSTEM (CONTINUED)

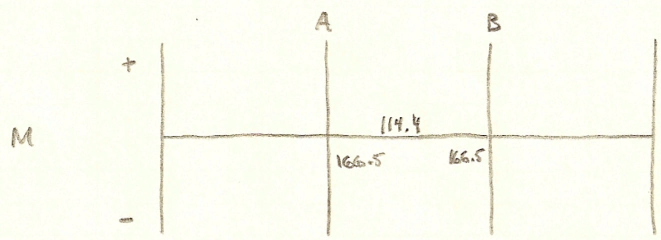
• GIRDER DESIGN (INTERIOR)

$w_{FLOOR} = [(1.146)(6')](22') = 4.202$ KLF

$w_{GIRDER} = 1.2(35)(3') + 1.6(40)(3) = 0.318$ KLF

$w_{SELF} = 1.2(150)(18.5'')(36'')/144 = 0.8325$ KLF

$w_U = 4.202$ KLF $+ 0.318$ KLF $+ 0.8325$ KLF $= 5.35$ KLF



ACI MOMENT COEFFICIENTS

$$M^-_{A,B} = \frac{w_U l_n^2}{11} = \frac{5.35(18.5')^2}{11} = 166.5 \text{ k-ft}$$

$$M^+ = \frac{w_U l_n^2}{16} = \frac{5.35(18.5')^2}{16} = 114.4 \text{ k-ft}$$

TOP REINFORCEMENT (INT SPAN / INT SUPPORT)

$A_s = M_U/4d = 166.5/4(16.125) = 2.58 \text{ in}^2 \rightarrow$ TRY (4) #8 $\Rightarrow A_s = 3.16 \text{ in}^2$

$a = \frac{3.16(60)}{0.85(4)(24)} = 2.32''$ $\quad c = 2.32''/0.85 = 2.73''$

$\varepsilon_t = \frac{0.003}{2.73}(16.125 - 2.73) = 0.0147 > 0.0075$ ∴ MOMENT CAN BE REDUCED PER ACI 8.4

MOMENT REDISTRIBUTION: $1000\,\varepsilon_t = 14.7 \rightarrow 14.7\%$ REDUCTION

$M_U = 166.5(1 - 0.147) = 142$ ft-k

$A_s = \frac{142}{4(16.125)} = 2.2 \text{ in}^2 \longrightarrow$ TRY (3) #8 $\Rightarrow A_s = 2.37 \text{ in}^2 \rightarrow a = 1.74$

$\phi M_n = [0.9(2.37)(60)(16.125 - \frac{1.74}{2})]/12 = 162.7$ ft-k > 142.0 ft-k ∴ OK

USE (3) #8 TOP REINFORCEMENT

ONE-WAY JOIST SYSTEM (CONTINUED)

BOTTOM REINFORCEMENT

$A_s = 114.4 / 4(16.125) = 1.77 \text{ in}^2 \rightarrow$ TRY (3) #7 $\Rightarrow A_s = 1.8 \text{ in}^2$

$a = \frac{1.8(60)}{0.85(4)(24)} = 1.32''$ $\quad c = 1.32/0.85 = 1.56''$

$\varepsilon_t = \frac{0.003}{1.56}(16.125 - 1.56) = 0.028 > 0.0075$ ∴ MOMENT CAN BE REDUCED PER ACI 8.4

MOMENT REDISTRIBUTION: $1000\,\varepsilon_t = 28 \rightarrow$ REDUCE BY 28 %

$M_u = 114.4(1 - 0.28) = 82.4$ ft-K

$A_s = \frac{82.4}{4(16.125)} = 1.28 \text{ in}^2$ TRY (2) #8 $\Rightarrow A_s = 1.58 \text{ in}^2$

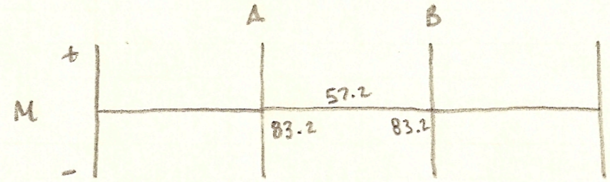
$a = \frac{1.58(60)}{0.85(4)(24)} = 1.16''$

$\phi M_n = \left[0.9(1.58)(60)\left(16.125 - \frac{1.16}{2}\right)\right] / 12 = 110.5$ ft-K > 82.4 ft-K ∴ OK

USE (2) #8 BOTTOM REINFORCEMENT

• EDGE GIRDER DESIGN (INTERIOR SPAN)

W_u = 5.35 KLF/2 = 2.675 KLF ⇒ HALF THE TRIBUTARY WIDTH OF THE INTERIOR GIRDER



ACI MOMENT COEFFICIENTS

$$M^-_{A,B} = \frac{w_u l_n^2}{11} = \frac{2.675(18.5')^2}{11} = 83.2 \text{ K-Ft}$$

$$M^+ = \frac{w_u l_n^2}{16} = \frac{2.675(18.5)^2}{16} = 57.2 \text{ K-Ft}$$

TOP REINFORCEMENT (INT SPAN, INTERIOR SUPPORT)

$A_s = M_u/4d = 83.2/4(16.125) = 1.29 \text{ in}^2$ → TRY (3) #6 ⇒ $A_s = 1.32 \text{ in}^2$

$a = \frac{1.32(60)}{.85(4)(24)} = 0.97''$ $c = 0.97''/0.85 = 1.14''$

$\varepsilon_t = \frac{0.003}{1.14}(16.125 - 1.14) = 0.039 > 0.005$ ∴ TENSION CONTROLLED, $\phi = 0.9$

$\phi M_n = [0.9(1.14)(60)(16.125 - \frac{.97}{2})]/12 = 80.2 \text{ ft-k} > 83.2 \text{ ft-k}$ ∴ OK

USE (3) #6 TOP REINFORCEMENT

BOTTOM REINFORCEMENT (INT SPAN)

$A_s = 57.2/4(16.125) = 0.88 \text{ in}^2$ ⇒ TRY (2) #6 ⇒ $A_s = 0.88 \text{ in}^2$

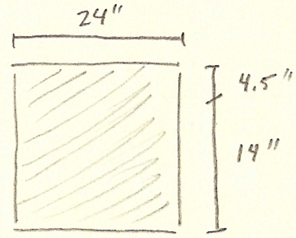
$a = \frac{0.88(60)}{0.85(4)(24)} = 0.647''$ $c = 0.647/0.85 = 0.761''$

$\varepsilon_t = \frac{0.003}{0.761}(16.125 - 0.761) = 0.061 > 0.005$ ∴ TENSION CONTROLLED, $\phi = 0.9$

$\phi M_n = [0.9(0.88)(60)(16.125 - \frac{0.647}{2})]/12 = 62.6 \text{ ft-k} > 57.2 \text{ ft-k}$
∴ OK

USE (2) #6 BOTTOM REINFORCEMENT

INTERIOR GIRDER DEFLECTION



$\bar{y} = 18.5''/2 = 9.25''$

$I = \dfrac{24(18.5)^3}{12} = 12663.25 \text{ in}^4$

$E = 3834 \text{ KSI}$

$w_{LL} = 40 \text{ PSF}(24') = 960 \text{ PLF} = 0.960 \text{ KLF}$

$w_{DL} = (35 \text{ PSF})(24') + (4.5/12)(150)(24') + (24'')(18.5'')(150)/144$

$= 840 + 1350 + 462.5 = 2.65 \text{ KLF}$

$w_{TL} = 0.960 + 2.65 = 3.61 \text{ KLF}$

$\Delta_{LL} = \dfrac{5 w_{LL} l_n^4}{384 EI} = \dfrac{5(.960)(18.5)^4(1728)}{384(3834)(12663.25)} = 0.05''$

$\Delta_{LL} = l/360 = (18.5)(12)/360 = 0.62'' > 0.05'' \therefore \text{OK}$

$\Delta_{TL} = \dfrac{5(3.61)(18.5)^4(1728)}{384(3834)(12663.25)} = 0.20$

$\Delta_{TL} = l/240 = 18.5(12)/240 = 0.925'' > 0.20'' \therefore \text{OK}$

Appendix F: Cost Analysis

	SYSTEM	FACTOR		MATERIAL $		LABOR $		TOTAL $
1)	PRECAST PLANK ON LOAD BEARING MASONRY AND STEEL FRAME	(0.88)	×	[($9.7/SF)	+	($4.18/SF)]	=	$12.21/SF
2)	PRECAST PLANK ON STEEL	(0.88)	×	[($8.80/SF)	+	($1.99/SF)]	=	$9.50/SF
				[($10.45/SF)	+	($4.00/SF)]	=	$12.72/SF
								$22.22/SF
3)	COMPOSITE STEEL DECK	(0.88)	×	[($11.15/SF)	+	($5.65/SF)]	=	$14.79/SF
4)	ONE-WAY JOIST SYSTEM	(0.88)	×	[($6.55/SF)	+	($10.30/SF)]	=	$14.83/SF