[Helios Plaza]

Houston, Texas

Kevin Zinsmeister
Structural Option
Adviser: Dr. Linda Hanagan

[TECHNICAL REPORT II:]

Pro-Con Structural Study of Alternate Floor Systems

Table of Contents

Executive Summary

The purpose of this report is to discuss potential alternative flooring systems for a typical bay in Helios Plaza. As part of the report, the existing one-way pan joist floor system is analyzed in addition to the preliminary design of three alternate floor systems:

1. Non-Composite Steel with Composite Steel Deck
2. Composite Steel with Composite Steel Deck
3. Two-way Concrete Flat Plate

The typical bay chosen for design and analysis measures 27' x 30' and is interiorly located. This bay is the same area of interest that was chosen in Technical Report I and is ideal for design because it occurs at every level. Another contributing factor for this bay's selection is that it remains as a one-way pan joist at every level.

Loads used for analysis in this report are still based upon the loads determined in Tech I, but are reduced where appropriate according to the ACSE 7-10 guidelines. The material choices for the floor systems' design are based on the strengths specified by the design structural firm to make the systems comparable to each other.

The existing one-way pan joist system was found to have the smallest calculable deflection without the aid of computer software and the second cheapest square foot cost. It also has the smallest structural depth, excluding the flat plate system, and requires no fireproofing to be applied. The deflection may be the lowest in part because the members are also designed to resist the lateral loads in the building. Negatively, this system is the second heaviest and is roughly twenty-five pounds heavier per square foot than the steel floor systems.

Both steel systems are similar statistically. Both have minimally larger structural depths than the existing system and similar maximum deflections. These deflections border the limit allowed by code because deflection was the controlling case in both designs. They differ mainly in weight and cost; the non-composite system is considerably more expensive and is 4.7 pounds per square foot heavier than the composite system. The increase in cost for both these systems is partially due to the need for fireproofing.

Of all the systems, the two-way flat plate exhibits the most polar behavior. It is the heaviest by far and because of this, it is the only system that requires an increase in the foundation design. Positively, it is the thinnest as well as the cheapest system. The square foot cost is incredibly rough in nature because the RSMeans tables do not provide information for a bay sized as large as the one that is designed. In an attempt to normalize cost, a factor of 1.1 was superimposed.

Viable systems for further consideration are both of the steel systems. Their minimal increase in floor thickness and drastic decrease in floor weight make them candidates for continued investigation. The major problem with these systems is the cost and construction lead time. Their reduced weight could lead to savings on the foundations potentially.

The report follows this general order of the executive summary after a brief overview of the existing floor systems. The bay of interest can be seen in the following Figure 1.

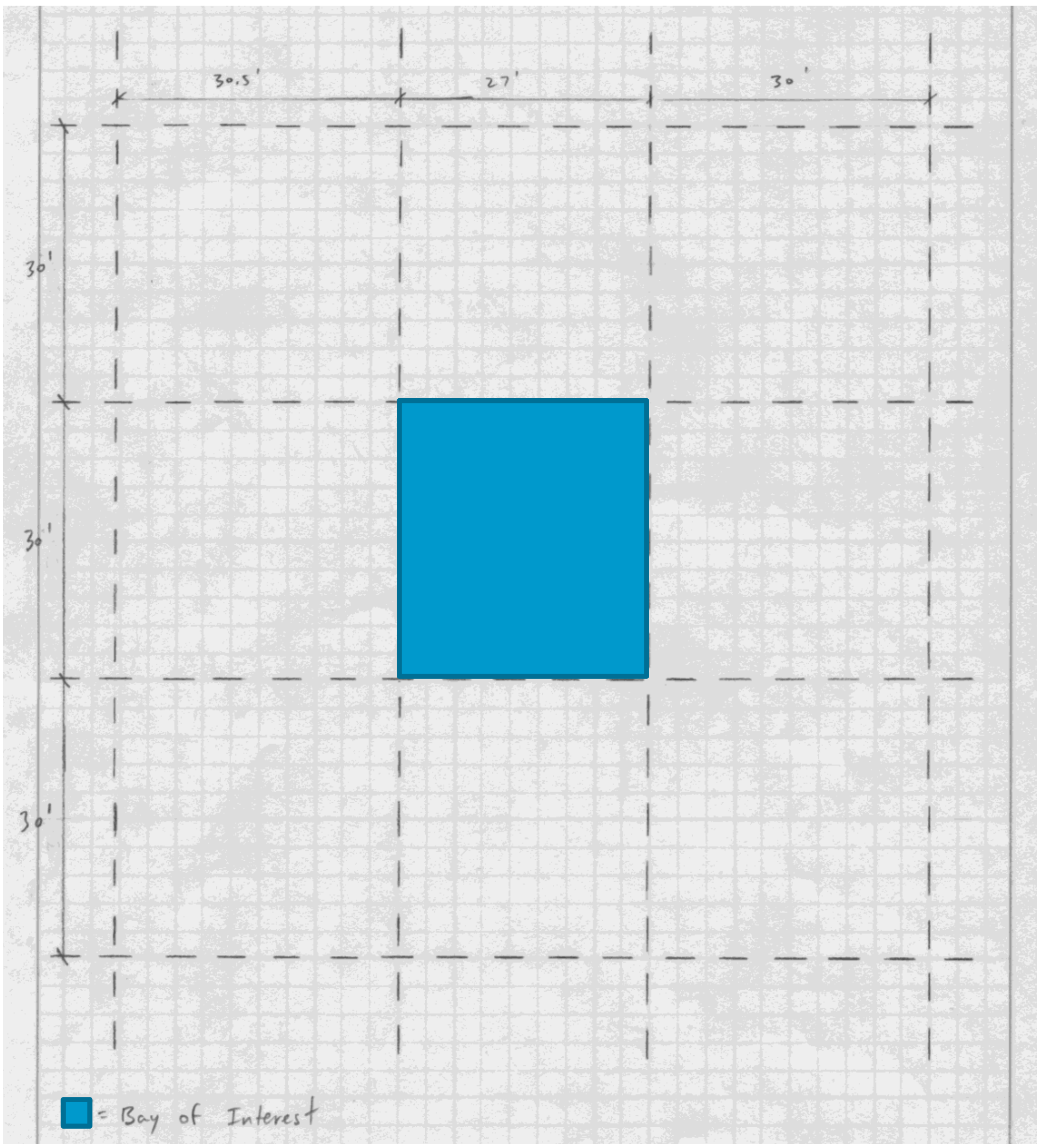


Figure 1: Typical Spacing & Bay of Interest

Introduction

Helios Plaza is a corporate campus that comprises of three main structures. The first structure, which is the focus of this report, is a six-story IST building. In addition to the IST building, there is a 1,909 car capacity parking deck and a five megawatt combined heat and power plant housed in its own structure. The IST building will be referred to as Helios Plaza throughout the rest of this document.

Helios Plaza is 423,500 gross square feet with an overall building height of 113 feet, the typical floor to floor height being 15 feet. After the second level, the floors systems split between concrete and composite deck to allow for double-story trading floors. From story three upward, a u-shaped concrete floor repeats at every level until level six leaving a rectangular space open for the composite deck system. This rectangular composite deck only occurs at levels four and six to create a total of three double-story trading floors for the building occupant. Refer to Appendix A for additional floor plans and elevations.

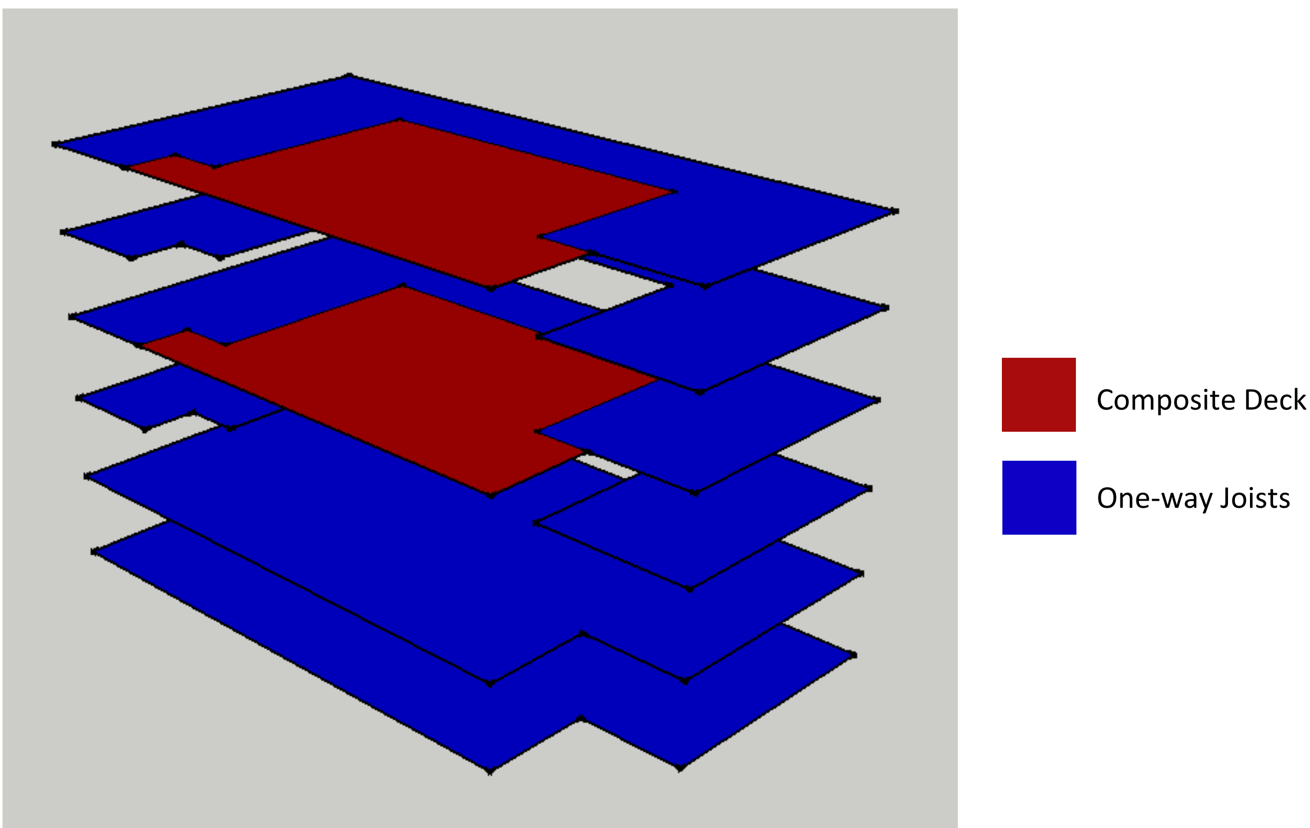


Figure 2: Simplified Floor Systems Diagram

Structural System Overview

The main structural system of Helios Plaza is framed in reinforced concrete. Gravity loads are handled largely by square concrete columns, although concrete filled HSS columns are used for aesthetics in larger spaces. For shorter spans, averaging thirty feet, concrete girders in combination with pan beams are used. For larger spans of the magnitude of forty-five feet post tensioned girders are employed. Finally, for spans of sixty feet, castellated wide flanges shapes are used to reduce the weight span ratio while maintaining strength.

The floor is mainly a concrete one-way system that uses 66/6 skip joists typically. In mechanical rooms, two-way slabs are used to distribute the larger loads more evenly to the supporting members. Composite decking with lightweight concrete is used over the long span steel members in the trading rooms.

To resists lateral loads, the building relies on the typical framing members to perform as concrete moment frames. Large HSS members are used in the trading floors at the skip levels to transfer loads horizontally into the concrete adjacent and vertically to the floors above.

Foundation

The site had to be extensively dewatered prior to the excavation for the project because of the porosity of the soil in Houston. Also, the soil has a high clay content which required the delivery of soils with better bearing capacity to the site.

Spread concrete footings are placed at the base of all grade level columns. The typical depth of the footings is three feet below the member that they are supporting. Their sizes range from 4' x 4' x 15" to 17' x 17' x 57".

Retaining walls are only used in the southeast corner of the building where there is a sub-grade basement with access to the adjacent parking structure via a tunnel.

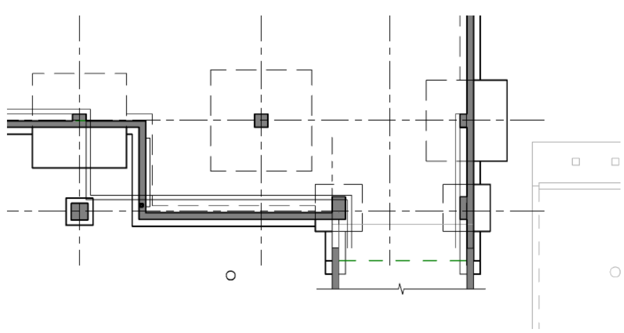


Figure 3: Basement Tunnel Entrance to Parking Structure

At level one, the floor is a slab on grade with thickness ranging from 5" to 12". Grade beams are also implemented at level one sized at 42" x 30".

Columns

Rectangular concrete columns are the predominant system used in Helios Plaza. For the most part these normal weight columns are 24" x 24" in size at all floors except level one where there is an increase in size to 30" x 30". The concrete strength decreases as the levels increase from 6000 psi at the basement level and level one to 5000 psi at levels two and three to 4000 psi for levels four through six.

In addition to the rectangular concrete columns, concrete filled HSS columns are used in the double story trading spaces. These columns are 24Ø and are fillet welded to a metal plate at the base. This plate is then tied to the floor or foundations with anchor rods. The same concrete strengths apply to these HSS columns as the rectangular columns listed above.

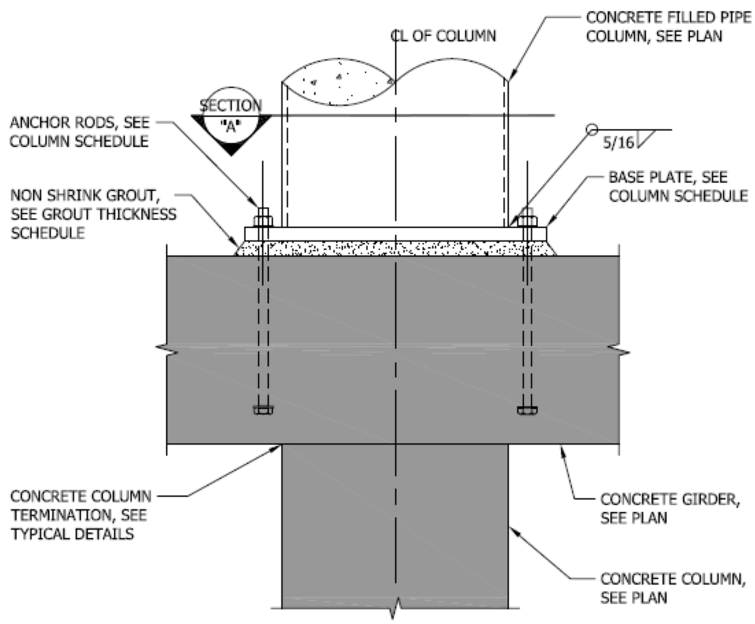


Figure 4: Typical Detail of Concrete Filled HSS Anchorage

Floor Systems

As with the rest of the structural systems in Helios Plaza, the floor system is split into two main categories, one-way pan joists and composite deck. The one-way pan joist system has a WWR, 4" slab that rests on 16" deep pan typically. The one-way system frames into girders that range from 20" to 33" deep with a width ranging from 24" to 36". Girders also span in the same direction as the one-way joist system, but these are there to create concrete moment frames to resist lateral loads.

In the corner bays of the building, a large pan (typically 33" x 30") is placed to transfer load from the exterior stairwells' framing members. A large pan extends from the exterior stairwells' wall perpendicular to the enlarged pan from above and ties into it for load transfer. This is done to reduce torsion that would otherwise be placed on the edge girder of the main building.

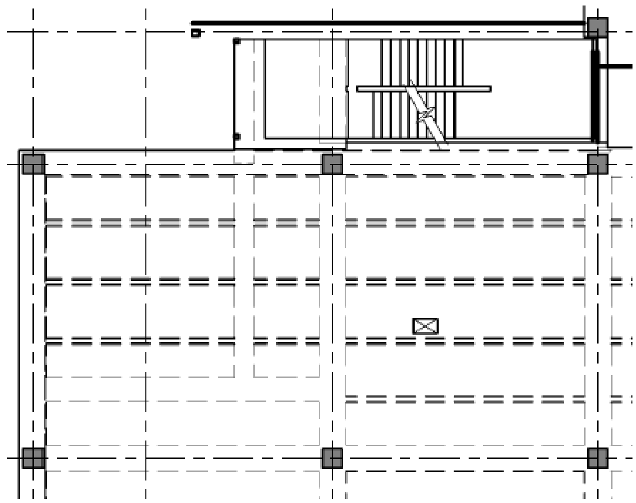


Figure 5: Plan of Enlarged Corner Pan Joists

Post-tensioned girders are used all along the south face of the building that span in the North-South direction. This is necessary to meet the strength requirements for the 45' distance that these members span. The tendons are typically bundled in groups of four and the minimum final post-tension force is 351 kips.

Two-way slabs are implemented in areas where mechanical equipment is housed on every floor. The slabs are typically 10" thick, but in some cases they can reach 12" in thickness. These slabs are also used when bathrooms are placed over top.

The second floor system used in Helios Plaza is a composite deck on w-shapes. The change occurs because of the move to long span castellated beams to accommodate open, double story spaces for the trading floors. Spans of 60' dominate these spaces and the castellated beams vary between CB24x100 and CB30x44/62. In addition to the weight saving caused by punching out parts of the web, the beams are cambered 1.5" and 1.75" to meet deflection limits. The composite decking used is typically 3 1/2" light weight concrete over 2" composite deck. The concrete is reinforced with additional WWR.

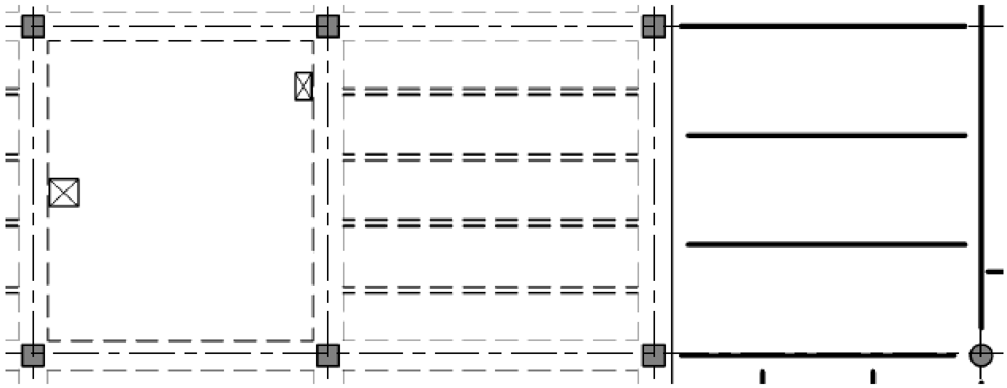


Figure 6: All Three Floor Systems in Adjacent Bays

Lateral Systems

Lateral forces are resisted in Helios Plaza by concrete moment frames. As mentioned before, girders run in the same direction as the one-way joist system to make up the frames in the East-West direction. In the North-South direction the same system is in place, however, the moment frame to building width ratio is much smaller due to the double story spaces. When a double story occurs, the floor that gets cut out is no longer there to distribute lateral forces from the building's enclosure to the moment frames. The force is instead transferred perpendicularly by horizontal circular HSS members to the one-way joists or to the floors above and below by the columns.

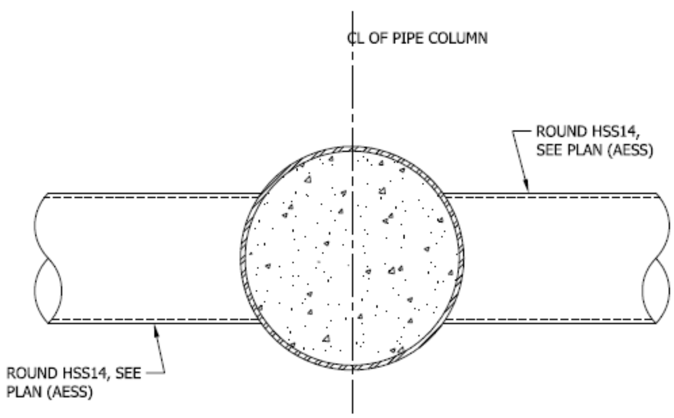


Figure 7: Round HSS Members Framing Into Each Other

Codes and References

Original Design Codes

- National Model Code:
    - 2003 International Building Code with City of Houston Amendments
- Design Codes:
    - Texas Architectural Barrier Act Standard
    - ANSI/AWS Structural Welding Code
- Structural Standards:
    - American Society of Civil Engineers, SEI/ASCE 7-02, Minimum Design Loads for Buildings and Other Structures

Thesis Design Codes

- National Model Code:
    - 2009 International Building Code
- Design Codes:
    - Steel Construction Manual 13^{th} edition, AISC
    - ACI 318-08, Building Code Requirements for Structural Concrete
- Structural Standards:
    - American Society of Civil Engineers, SEI/ASCE 7-10, Minimum Design Loads for Buildings and Other Structures

Materials

Concrete		f'c (psi)
Spread Footings		4000
Basement Walls		6000
Slabs	On-Grade	3500
	Level 2	5000
	Level 3-6	4000
	Metal Deck	3500
Columns	Basement	6000
	Level 1	6000
	Levels 2-3	5000
	Levels 4-6	4000
Beams		Same As Columns
Girders		Same As Columns
Reinforcement		**Fy (ksi)**
Rebar	#7 to #18	75
	All Other Sizes	60
Welded Wire	Smooth	65
	Deformed	75
Post-Tensioning Steel		**fs (ksi)**
Tendons		270
Concrete Masonry		**f'm (psi)**
All Types		1500
Structural Steel		**Fy (ksi)**
Wide Flange Shapes		50
Edge Angles/Bent Plates		36
HSS		42
Baseplates		36

Table 1: Material Strengths

Load Determinations

Dead Loads

For the analysis of the dead loads, several assumptions were made. Although depth of metal deck and topping was specified, a specific deck type was not mentioned, so decks were chosen from the Vulcraft catalog. The weight of lighting, electrical, and plumbing equipment was also not specified. A summary of the dead loads is tabulated below.

Floor Dead Load	
Load Source	Design Load
Normal Weight Concrete	150 PCF
Composite Decking	44 PSF
MEP	15 PSF
Roof Dead Load	
Load Source	Design Load
Roof Decking	23 PSF
Roof Cladding	5PSF

Table 2: Dead Loads

Live Loads

Since Helios Plaza is an IST and trading office, many of the loads used are not prescribed directly in the ASCE 7-10 Code. The following table shows the comparison of the ASCE 7-10 live loads and the loads used by the designer.

Live Load		
Load Source	Design Load	ASCE 7-10 Load
First Floor Corridors	100 PSF	100 PSF
Corridors Above First Floor	80 PSF	80 PSF
Lobbies	100 PSF	100 PSF
Office	80 PSF	50 PSF
Server Rooms	100 PSF	-
Mechanical Rooms	100 PSF	-
Roof	20 PSF	20 PSF

Table 3: Live Loads

The loadings previously used in Tech Report I proved too extreme for the existing structural system to meet flexural and shear strength requirements. With this in mind, the loads were reduced for analysis and design in this technical report. Utilizing ASCE 7-10 live load reduction guidelines, the 20 PSF partition load was allowed to be neglected and uniform live loads were reduced as per ASCE 4.7.2. All designs were made assuming office live loads were assessed.

Floor System Analysis & Design

One-Way Pan Joist System (Existing Condition)

This typical floor system consists of 66/6 skip joists that have single pieces rebar in the bottom of the pan beams. The distance between the pan beams and the parallel girders is 53" in most situations to make up a centerline distance between girders of 30'. The slab thickness is 4" and most members are 20" deep, slab inclusive. In certain places, deeper members are used to make up strength.

The typical bay analyzed was 27' x 30', but standard bay sizes can be 30' x 30' and 30.5' x 30'. In long-span situations, post-tensioned girders are utilized to carry the 45' distance that have 66/6 skip joists running between them.

After the loads were reduced from Tech Report I, the girder that had previously failed in flexure and shear adequately met the required capacities. Moving forward from this, the allowable deflections were checked and the beam and girder were found to meet these minimums.

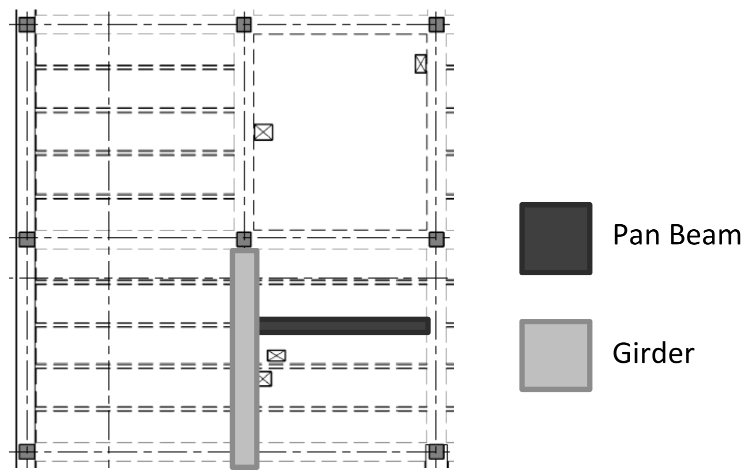


Figure 8: Typical Bay Members Analyzed

The thickness of the floor is not a major architectural concern in Helios Plaza. The total structural thickness is predominantly 20", but in many places raised access floors are installed on top of the concrete slab making the overall floor thickness several inches higher. These raised access floors help with the bundling of cables and also serve as under floor mechanical and electrical spaces. In addition to the under floor routing of services, suspending cable trays are placed in the IST office areas.

One-way pan joists are used primarily to achieve longer spans and create larger spacing between columns. The architectural goals of the building are to open up the spaces and create an airy feel. Views and day lighting are particularly important to the design of Helios Plaza.

With these bigger spans, more space is opened internally for the occupants to interact with the curtain wall façade.

The core problem with this concrete system is its weight. The site has soil with poor bearing capacity and soils were imported to deal with this issue, but this addition may still not stem the underlying problems of settlement. Underneath the fill, the water table is quite high.

On the positive side, the concrete system achieves the equivalent of a two-hour fire rating since the spaces are sprinklered. The concrete inherently provides a fire rating of 1.5 hours based upon its thickness. Another benefit of using concrete in Houston is its relative cost compared to the national average. Based upon historical Portland Cement Association data, the cost of concrete in Houston is roughly 80% of the national average. Concrete construction is also labor intensive, but Houston has a relatively inexpensive workforce. According to 2011 RSMeans values, installation costs in Houston are 78.1% of the national average for superstructures.

Advantages	Disadvantages
Cheap Construction Cost ($16.99/SF)	Heavy Structure Leads to Large Foundations
Inherent Fire Protection	Labor Intensive Construction
Short Lead Time	Formwork Required
Integral with Lateral Systems	

Table 4: Existing System Pros & Cons

Non-Composite Steel System

The design of this system was based upon a spacing that allowed for unshored construction spans of the composite decking. From the Vulcraft catalog, a non-composite deck was chosen that carried the dead load and spanned a distance that the bay could be broken down into even increments. As a result of this, a 1.3C20 deck with 3 ½" light weight concrete topping was chosen that could span the unshored construction clear span of 6'. This layout can be seen in Figure 9 on the next page.

In choosing the supporting beams and girders, deflection was the dominant factor in design. Since the moment of inertia was the controlling property in member selection, the depths of the beams increased largely as compared to the member needed to meet flexural and shear capacities only. The overall depth of the non-composite system for the bay in questions is 21.1", which is only 1.1" more than the existing system. The span between beams is even similar in this system.

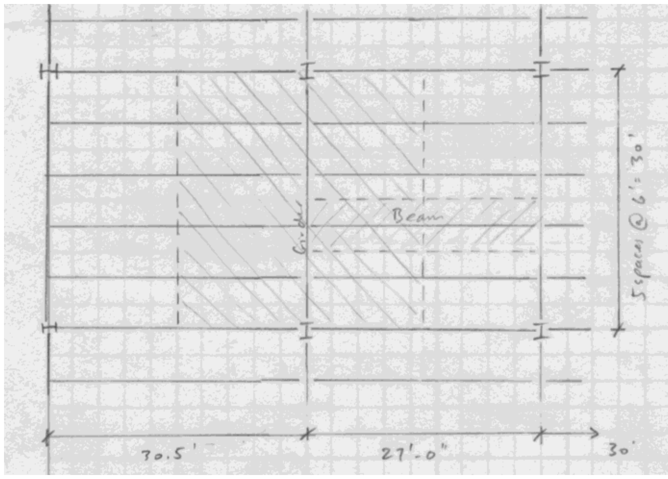


Figure 9: Non-Composite Floor Layout

Similarities between the two systems stop there though. The steel system is over twenty-five pounds per square foot lighter than the pan-joist system, which will result in significant reduction to the building mass. Potentially, this reduction in mass could result in smaller footings and basement walls. Other construction perks are the elimination of formwork and decrease in labor intensity. Unfortunately, steel construction means that there is a longer lead time for material procurement, ultimately affecting the scheduling of the project.

Cost wise, this structure is the most expensive of all the systems. Additionally, this floor system requires the application of fire proofing to meet the two hour fire rating requirement. To help offset the increased cost of construction, the potential foundation reductions may make up for the more expensive steel assembly.

Architecturally, this system offers little difference from the existing one-way system's floor plan. 14" inch deep beams project down from a 4.8" non-composite deck as opposed to 16" pans from a 4" slab. There will be a decrease in the column dimensions with the change from concrete to steel, but the smaller columns will not substantially increase the useable floor area of the building. What these changes will do is completely negate the existing lateral system. Because of this, lateral stiffness needs to be developed in this new construction and steel bracing members will probably run between the previously unhindered bays. This has potential to obstruct the views sought by the architect and client and even impede the circulation of peoples within the building.

Advantages	Disadvantages
Light Structure	Most Expensive System ($24.95/SF)
Ease of Construction	Lateral System Incompatible
No Formwork Required	Fireproofing Required
Potential Foundation Reduction	Long Lead Time

Table 5: Non-Composite System Pros & Cons

Composite Steel System

Utilizing the larger unshored construction spans, the distance between beams was chosen to be 10' for the composite steel flooring. From the Vulcraft catalog, a 1.5VL17 composite deck with a 3.25" topping was chosen. With the composite deck laterally bracing the beams for their entire length, the members were now able to be designed for their full plastic moment capacity. Unfortunately, member deflections controlled the design once more. After solving for the proper moment of inertia and selecting appropriate shapes, the thickness of the assembly was found to be 22.75".

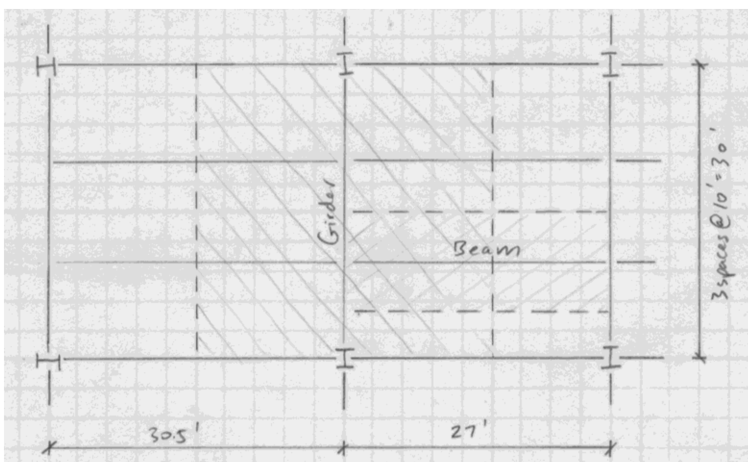


Figure 10: Composite Floor Layout

Akin to the non-composite system, the composite system shares all of the same advantages. What the composite does have over the non-composite system is the cost and weight advantage. The square foot cost is roughly $1.50 more than the one-way pan beam system and the weight benefits are substantial; the structural weight is more than thirty pounds per square foot less than the existing system.

With the thickest floors, this system has potential to make alterations to the architectural layout. Despite having raised access floors, the ceiling assembly may need to drop to help locate MEP services in the ceiling plenum similarly to their existing arrangements. As mentioned earlier, the elimination of the concrete moment frames means that a new lateral force resisting system needs to be put in place with this change. Steel moment frames are a possibility to ensure that the existing bays are not impeded, but the connections involved in this type of bracing are expensive and time intensive.

An added benefit of altering the typical bay to composite steel is an elimination of many torsional irregularities that exist in Helios Plaza. With the different floor systems in tandem now, there is a discontinuity in the diaphragm stiffness. The move to an all composite steel deck system will ensure a unified diaphragm and remove this particular irregularity.

Advantages	Disadvantages
Lightest Structure	Thickest Floor
Ease of Construction	Lateral System Incompatible
No Formwork Required	Fireproofing Required
Potential Foundation Reduction	Long Lead Time

Table 6: Composite System Pros & Cons

Two-Way Flat Plate System

Before design could begin on the flat plate system, seven stipulations had to be met to ensure that the direct design method was applicable. These checks can be found in Appendix E. Having met these conditions, the design followed the ASCE code for two-way slab design for slab thickness and reinforcing. Based on the ASCE guidelines, a 9.5" slab was determined with deflection limitations in mind and rebar was sized accordingly. All of the rebar used was #6 or less to ensure a uniform tensile strength of 60 ksi as per the building materials specifications.

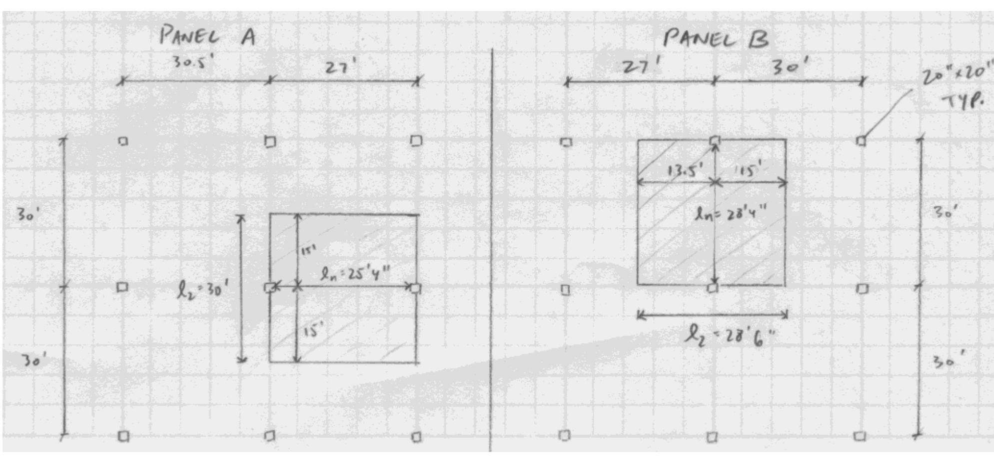


Figure 11: Flat Plate Floor Layout

The design was entered into with the intention of adding drop panels around the columns. After the shear calculations were compiled, it was discovered that the increased capacity was not needed and the proposed flat slab system stayed a flat plate system.

The starkest differences between the one-way slab and the two-way slab are in regards to thickness and weight. The two-way slab is less than half of the thickness of the original concrete system and inversely to this decrease in thickness, a major increase in structure weight occurs. The two-way slab is nearly fifty pounds per square foot heavier than the one-way slab and is roughly three times as heavy as the two steel systems.

With a decrease in the floor thickness of 10.5" per floor, the overall building height would only drop by 4'-4.5". This drop does not affect the wind loading significantly, but has potential to lessen the seismic distribution of forces. Despite lowering the distribution of seismic forces to the upper stories, the increased building mass would drastically increase the amount of seismic load on the building completely offsetting this decrease. Architecturally, this taller ceiling height would be attractive, but it is not necessary considering that the ceiling is already proportionately high at roughly 12'.

Supporting the implementation of the flat plate system is its cheap construction cost. It is the cheapest system that was investigated, but this value is highly speculative. The RSMeans does not provide costing information of flat plate systems for bay sizes of 30' x 30'. In an attempt to rectify this estimating gap, an adjustment factor of 1.1 was applied to the most expensive of the 25' x 25' bay total costs.

Even though two-way slabs are considered concrete moment frames, this system does not distribute loads in remotely the same way as the one-way slab. With the addition of lateral loads, the flat plate could see the need for an increase in slab thickness that would even further exacerbate the building mass problem. One benefit of the thickness of the slab is that fire protection is more than handled due to the inherent fire resistance of concrete.

Advantages	Disadvantages
Thinnest Structure	Heavy Structure Leads to Large Foundations
Cheapest Construction Cost ($15.38/SF)	Labor Intensive Construction
Inherent Fire Protection	Formwork Required
Short Lead Time	

Table 7: Flat Plate System Pros & Cons

Conclusions

From the analysis performed for this technical report, several conclusions can be drawn:

- Both steel systems are viable for further consideration
- Despite multiple advantages, the two-way flat plate is not viable
- The existing structure is efficiently designed

Both of the steel systems are worthy of further investigation because both significantly reduce the building weight. This building weight reduction will in turn decrease the foundations' strength requirements. The reason that the non-composite system is still in the running is due to its smaller structural depth, which is nearer to the existing systems. The composite deck has more potential since it is capable of longer unshored construction spans and will be able to cope with more diverse floor loadings and geometries. Both systems would also make the entirety of the building into steel which would eliminate torsional irregularities and create a unified floor diaphragm. An area of concern moving forward with steel construction involves the elimination of the 45' post tensioned girders. Cambering and castellation of the members may be the only way to accomplish these spans. These measures will negatively affect the building cost.

As mentioned in the bulleted point above, despite the many advantages of the two-way flat plate, its viability is in question. The sheer weight of this system would require extensive increases to the building foundation and would more than likely increase the seismic loading on the building. The decrease in floor thickness is impressive with this floor system, but decreasing the floor thickness is not a priority in the buildings function or design. The inherent fire proofing nature of the concrete is a further pro. Even with its low cost, which is very approximate, the weight disadvantages control its omission as a continued candidate.

Not surprisingly, the existing floor system is efficiently designed and hard to overlook. The system is very good in deflection resistance and it incorporates the lateral resistance into its design. Its weight is not excessive, and minimal fire proofing measures were needed to make this system safe by Underwriters Lab standards. The cheap cost of concrete in Houston in parallel with its inexpensive labor force makes concrete a very enticing option. Match these factors with the short lead time needed for concrete construction and the building is nearly ready to begin construction.

A summary of the main factors influencing viable floor system choice is tabulated on the next page.

Floor System Comparisons				
Floor System	**One-Way Pan Joists**	**Non-Composite Steel with Composite Steel Deck**	**Composite Steel with Composite Steel Deck**	**Two-Way Concrete Flat Plate**
Structural Weight (psf)	72.2	46.7	42.0	119
Slab Depth	4"	4.8"	4.75"	9.5"
Structural Depth	20"	21.1"	22.75"	9.5"
Square Foot Cost	$16.99	$24.95	$18.54	$15.38
Fireproofing	No	Yes	Yes	No
Labor Intensity	High	Moderate	Moderate	High
Formwork	Yes	No	No	Yes
Lead Time	Short	Long	Long	Short
Max Deflection (D+L)	0.746"	1.004"	1.050"	Omitted
Foundation Impact	-	Potential For Reduction	Potential For Reduction	Increase Necessary
Lateral System Impact	-	Yes	Yes	Yes
Viable System	-	Yes	Yes	No

Table 8: Floor System Comparisons

Appendix

Appendix A: Typical Floor Plans

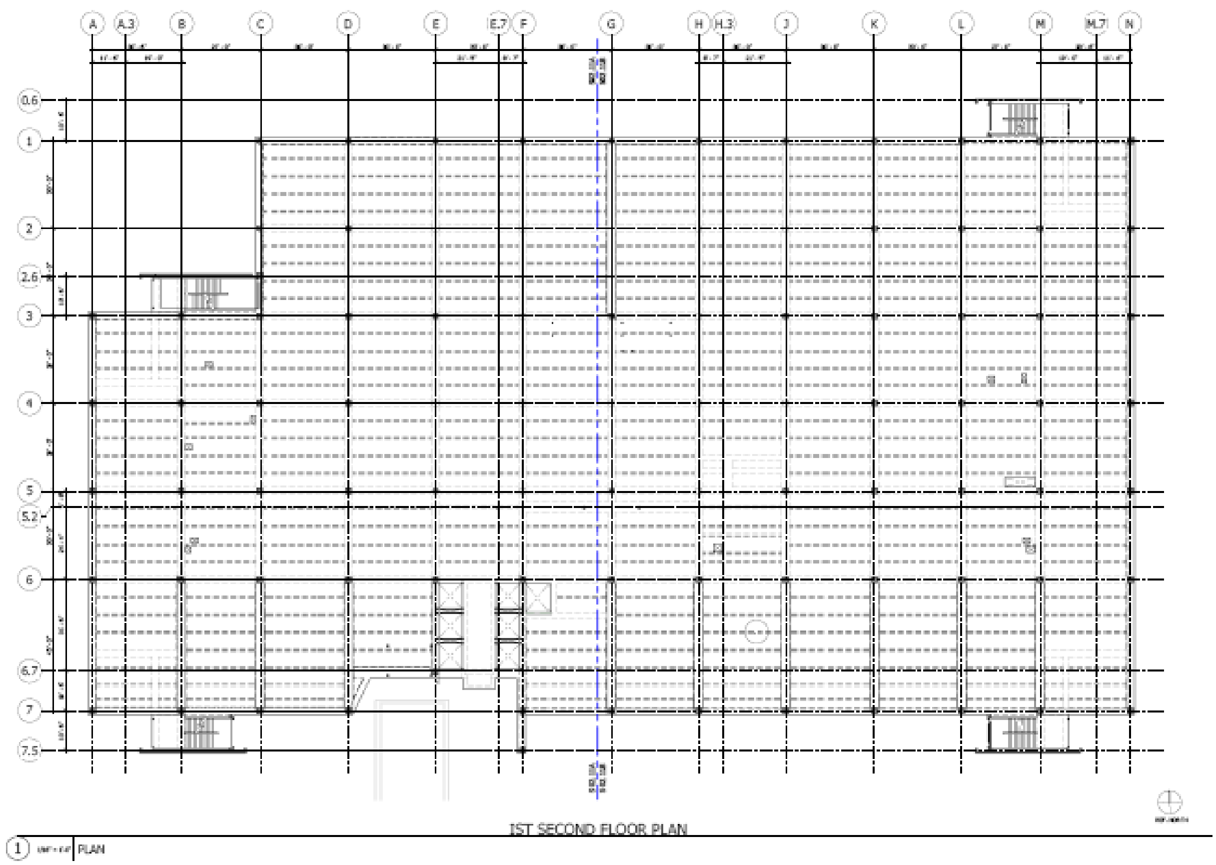


Figure 12: Second Floor Plan

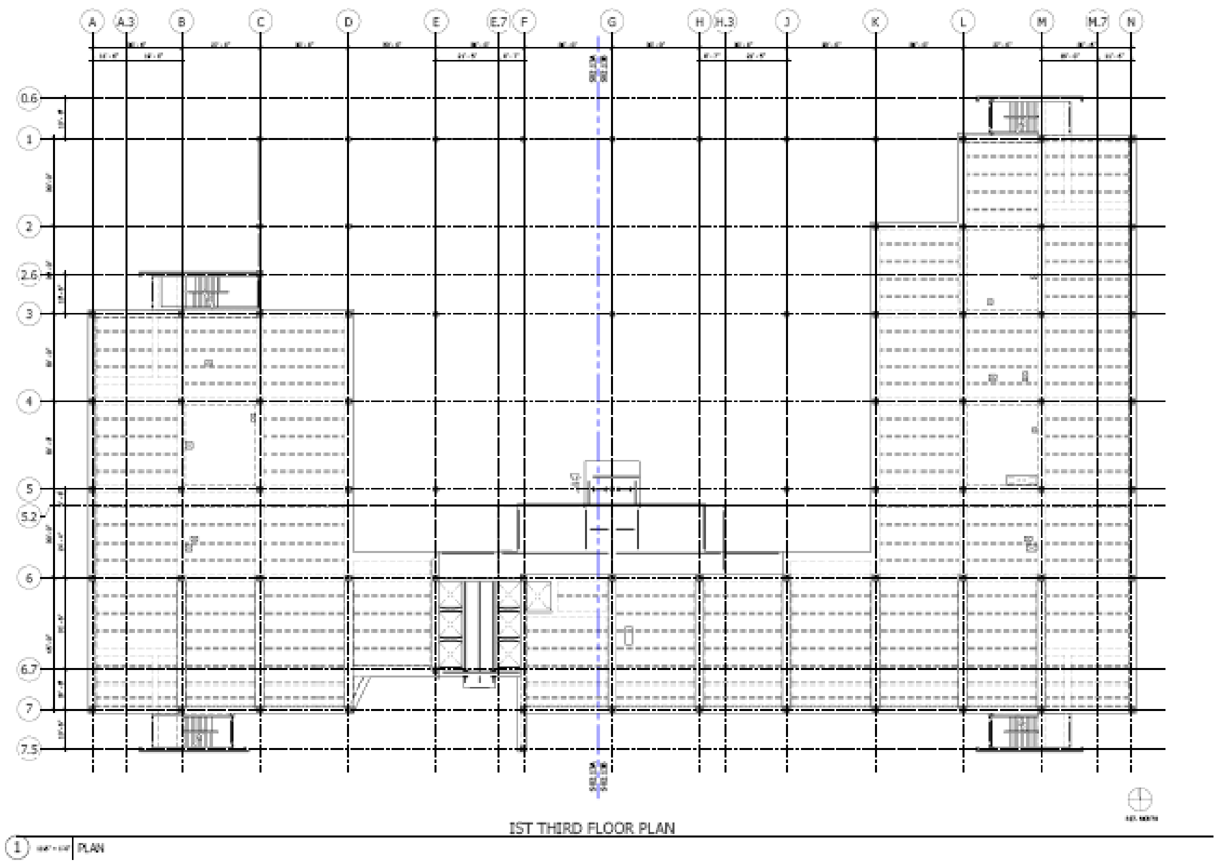


Figure 13: Third Floor Plan

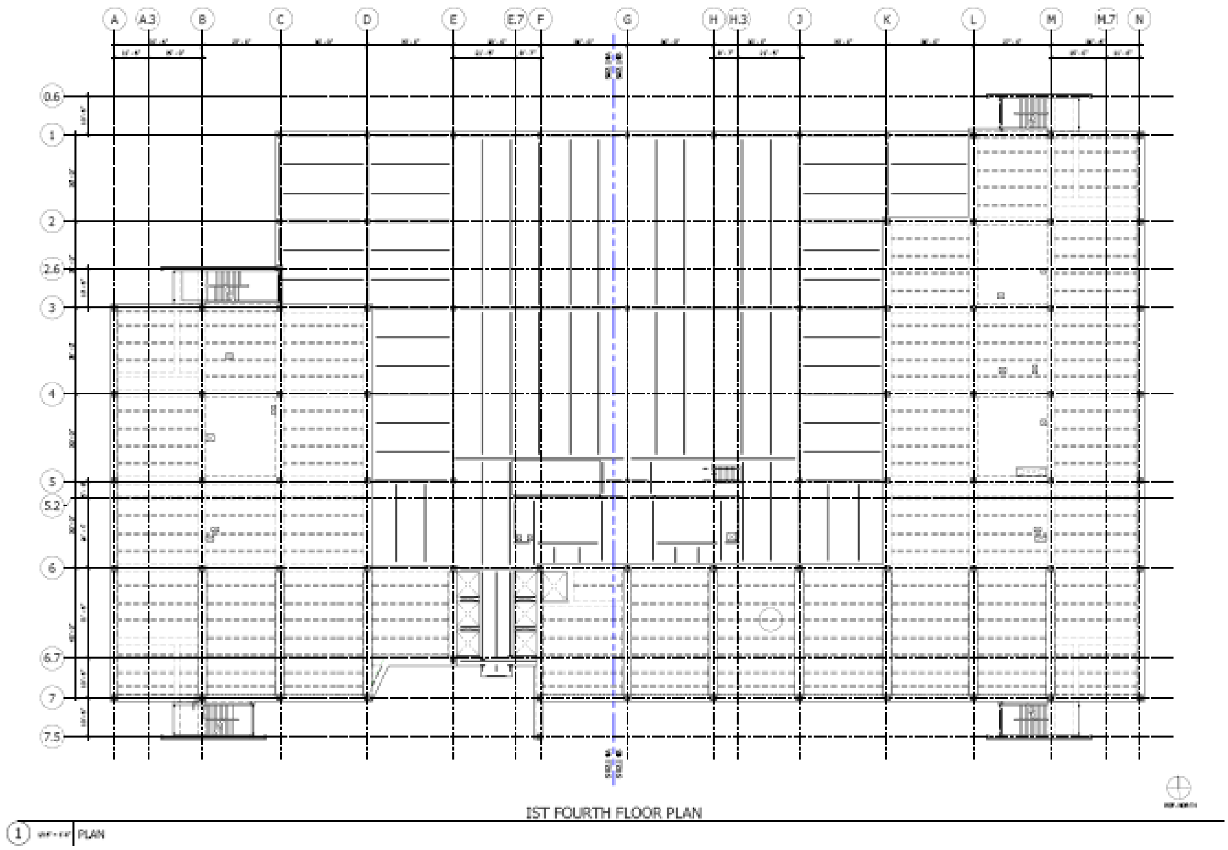
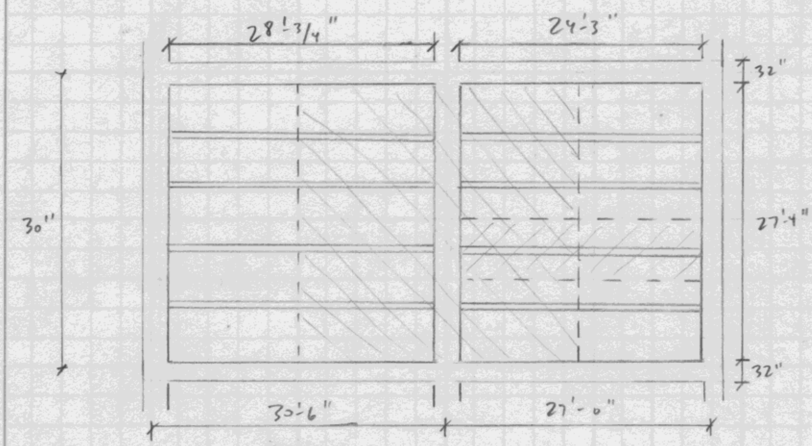


Figure 14: Fourth Floor Plan

Appendix B: Existing One-Way Pan Joist System Analysis

Kevin Zinsmeister | Tech Report 2 | Pan Joist System | 1



Pan Check:

$DL = \frac{6(20)}{144}150 + \frac{4(66)}{144}150 + \overset{SDL}{15}\left(\frac{70}{12}\right) = 488 \text{ plf}$

$LL = 80\left(\frac{70}{12}\right) = 467 \text{ plf} \Rightarrow w_u = 1.2(488) + 1.6(467) = 1.33 \text{ klf}$

Max neg. Moment $= \frac{w_u l_n^2}{9} = \frac{1.33(28.0625)^2}{9} = 116.4'^{k}$

Max pos. Moment $= \frac{w_u l_n^2}{14} = \frac{1.33(27.0625)^2}{14} = 74.8'^{k}$

Top reinforcing: $F_y = 60$ ksi $A_s = 5(0.44) = 2.2 \text{ in}^2$, $f'_c = 5000$ psi

$d = 16 + 1.5 = 17.5"$, $a = \frac{A_s f_y}{0.85 f'_c b} = \frac{2.2(60)}{0.85(5)6} = 5.18"$ $\therefore c = \frac{5.18}{0.8} = 6.475"$

$\varepsilon_s = \frac{0.003}{6.475}(17.5 - 6.475) = 0.0051 > 0.005$ $\therefore$ beam is tension controlled

$\phi M_n = 0.9 A_s f_y (d - a/2) = 0.9(2.2)60\left(17.5 - \frac{5.18}{2}\right) = 147.6'^{k} > 116.4'^{k}$ $\therefore$ okay

Bottom Reinforcing: $F_y = 60$ ksi $A_s = 0.6(2) = 1.2 \text{ in}^2$ $f'_c = 5000$ psi

$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{1.2(60)}{0.85(5)6} = 2.82"$, $c = \frac{2.82}{0.8} = 3.53"$

$\varepsilon_s = \frac{0.003}{3.53}(17.5 - 3.53) = 0.012 > 0.005$ $\therefore$ beam is tension controlled

$\phi M_n = 0.9(1.2)60\left(17.5 - \frac{2.82}{2}\right) = 86.9'^{k} > 74.8'^{k}$ $\therefore$ okay

Kevin Zinsmeister | Tech Report 2 | Pan Joist System | 2

Joist Shear: $A_v = 0.11\ in^2$

$V_u = \frac{1.15(1.33)28.0625}{2} = 21.5^k$

$\phi(V_c+V_s) = 0.75\left[2\sqrt{5000}(6)17.5 + 0.11(60)\frac{17.5}{4}\right] = 32.8^k > 21.5^k \therefore$ okay

Girder Check:

$LL = 80\left(0.25 + \frac{15}{\sqrt{2(27.3)28.75}}\right) = 50.3\ psf$

$w_{u\ floor} = \left[1.6(50.3)\frac{20}{12} + 488(1.2)\right]\left(\frac{12}{66}\right) = 191.8\ psf = 0.192\ ksf$

self weight $w_u = 1.2(33)20\left(\frac{150}{144}\right) = 0.825\ klf$

$w_{u\ girder} = \frac{33}{12}(1.2(0.015) + 1.6(0.05)) = 0.270\ klf$

$w_u = 0.192\left(26.2 - \frac{33}{12}\right) + 0.825 + 0.270 = 5.60\ klf$

Max. Neg. Moment $= \frac{w_u l_n^2}{11} = \frac{5.6(27.\bar{3})^2}{11} = 380.4^{'k}$

Max Pos. Moment $= \frac{w_u l_n^2}{14} = \frac{5.6(27.\bar{3})^2}{14} = 298.8^{'k}$

Top Reinforcing: $F_y = 75\ ksi$, $A_s = 6(0.79) = 4.74\ in^2$ $f'_c = 5000\ psi$

$d = 16 + 1.5 = 17.5"$, $a = \frac{A_s f_y}{0.85 f'_c b} = \frac{4.74(75)}{0.85(5)33} = 2.53"$ $\therefore c = \frac{2.53}{0.8} = 3.16"$

$\varepsilon_s = \frac{0.003}{3.16}(17.5 - 3.16) = 0.0136 > 0.005$ $\therefore$ tension controlled

$\phi M_n = 0.9 A_s F_y (d - a/2) = 0.9(4.74)75\left(17.5 - \frac{2.53}{2}\right) = 432.9^{'k} > 380.4^{'k}$ $\therefore$ okay

Bottom Reinforcing: $F_y = 75\ ksi$, $A_s = 6(0.6) = 3.6\ in^2$, $f'_c = 5000\ psi$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{3.6(75)}{0.85(5)33} = 1.93"$, $c = \frac{1.93}{0.8} = 2.41"$

$\varepsilon_s = \frac{0.003}{2.41}(17.5 - 2.41) = 0.0188 > 0.005$ $\therefore$ tension controlled

$\phi M_n = 0.9(3.6)75\left(17.5 - \frac{1.93}{2}\right) = 334.8^{'k} > 298.8^{'k}$ $\therefore$ okay

Shear:

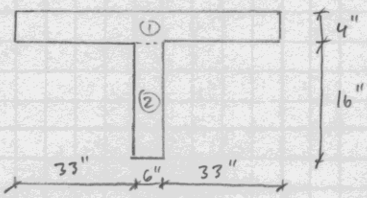
$V_u = \frac{w_u l_n}{2} = \frac{5.6(27.\bar{3})}{2} = 76.5^k$

$\phi(V_c+V_s) = 0.75\left[2\sqrt{5000}(33)17.5 + 0.11(60)\frac{17.5}{3}\right] = 90.1^k > 76.5^k$ $\therefore$ okay

Kevin Zinsmeister	Tech Report 2	Pan Joist System	3

Check Deflections:

Joist $b_{eff} = 6 + \min \begin{cases} (span/8)2 = 24.25(12)/4 = 72.75" \\ (\ell/2)2 = 66" \leftarrow \text{controls} \end{cases} = 6 + 66 = 70"$



$\bar{y} = \dfrac{4(72)(16+2) + 16(6)8}{4(72) + 16(6)} = 15.5"$

$I_1 = \dfrac{72(4)^3}{12} + 2.5^2(72)4 = 2184 \text{ in}^4$

$I_2 = \dfrac{6(16)^3}{12} + 7.5^2(6)16 = 7448 \text{ in}^4$

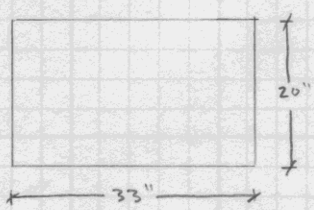
$I = 2184 + 7448 = 9632 \text{ in}^4$

$E = 33\sqrt{f'_c}\,(w_c)^{1.5} = 33\sqrt{5000}\,(150)^{1.5} = 4286.8 \text{ ksi} \approx 4287 \text{ ksi}$

$\Delta_{tot} = \dfrac{5(1.33)24.25^4(1728)}{384(4287)9632} = 0.251" < \ell/240 = 24.25(12)/240 = 1.21" \therefore \text{okay}$

$\Delta_{LL} = \dfrac{5(0.467)24.25^4(1728)}{384(4287)9632} = 0.088" < \ell/360 = 27(12)/360 = 0.9" \therefore \text{okay}$

Girder:



$I = \dfrac{33(20)^3}{12} = 22000 \text{ in}^2$

$E = 4287 \text{ ksi}$

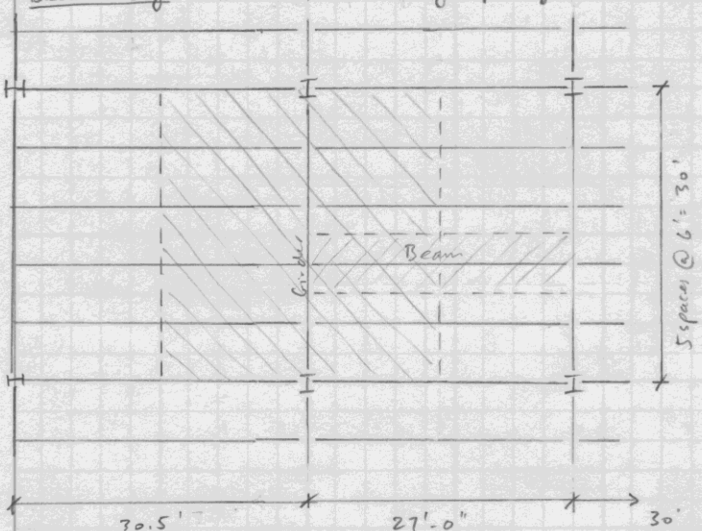
$\Delta_{TOT} = \dfrac{5(5.6)27.\bar{3}^4(1728)}{384(4287)22000} = 0.746" < \ell/240 = 27.\bar{3}(12)/240 = 1.367" \therefore \text{okay}$

$\Delta_{LL} = \dfrac{5(0.05)28.2(27.\bar{3})^4 1728}{384(4287)22000} = 0.188" < \ell/360 = 30(12)/360 = 1" \therefore \text{okay}$

Appendix C: Non-Composite Steel System Design

Kevin Zinsmeister | Tech Report 2 | Non-Composite System | 1

Deck Design: Assume the following spacings



$K_{LL} A_T = 1(6)27 = 162\ ft^2 < 400\ ft^2$ ∴ Live load reduction not allowed

$w_u = \overset{SDL}{15} + \overset{LL}{80} + \overset{DL}{40} = 135\ psf$

∴ Pick 1.3C20 w/ 3½ LWC topping (4.8" thickness total)

Check Flexural Strength: 164 > 135 ∴ okay

Check live load deflection: 80 ≥ 80 ∴ okay

Check Unshored construction clear span (3 spans): 8'-4" > 6'-0" ∴ okay

Check W1: 107 > 40 ∴ okay

Beam Design

$K_{LL} A_T = 2(6)27 = 324\ ft^2 < 400\ ft^2$ ∴ Live load reduction not allowed

$w_u = 1.2(50) + 1.6(80) = 182\ psf \Rightarrow w_u = 182(6) = 1.09\ klf$

Note: 5 psf allowance for beam self weight

$V_u = \frac{1.09(27)}{2} = 14.7^k \qquad M_u = \frac{1.09(27)^2}{8} = 99.3'^k$

$\Delta_{max} = \frac{5wL^4}{384EI}$ ∴ $I_{min} = \frac{5wL^4}{384E\Delta_{max}} = \frac{5(1.09)27^4 1728}{384(29000)1.35} = 332.9\ in^4$

$\Delta_{max} = l/240 = 27(12)/240 = 1.35"$

$\phi V_n = 0.6 F_y A_w$ ∴ $A_{w,min} = \frac{V_u}{\phi 0.6 F_y} = \frac{14.7}{0.9(0.6)50} = 0.544\ in^2$

Kevin Zinsmeister | Tech Report 2 | Non-Composite System | (2)

$M_p = F_y Z \therefore Z_{min} = \frac{M_p}{F_y} = \frac{99.3(12)}{50} = 23.83 \text{ in}^3$

Selecting by I_x, either W18×35, W16×40, W12×65 or W10×88 will work

∴ Pick W16×40 for comparable floor thickness (20.8")

W16×40, $I_x = 518 \text{ in}^4 > 332.9 \text{ in}^4$ ∴ okay, $Z_x = 73.0 \text{ in}^3 > 23.83 \text{ in}^3$ ∴ okay

$A_w = t_w \cdot T = 0.305(13\,5/8) = 4.16 \text{ in}^2 > 0.54 \text{ in}^2$ ∴ okay

Check unbraced length:

$\frac{L_b}{r_y} \leq 1.76\sqrt{\frac{E}{F_y}} \Rightarrow \frac{27(12)}{1.57} \leq 1.76\sqrt{\frac{29}{50}} = 206 \nleq 1.34$ ∴ Use Table 3-10 from Steel Manual

From table 3-10, W14×43 ⇒ first shape that works

Pick $\boxed{W14\times43}$ w/ $\phi M_n = 106^{'k}$, $I_x = 428 \text{ in}^4$, $Z_x = 69.6 \text{ in}^3$

$106^{'k} > 99.3^{'k}$ ∴ okay in flexure, $I_x = 428 \text{ in}^4 > 332.9 \text{ in}^4$ ∴ okay in deflection

$A_w = t_w \cdot T = 0.305(10\,7/8) = 3.32 \text{ in}^2 > 0.54 \text{ in}^2$ ∴ okay in shear

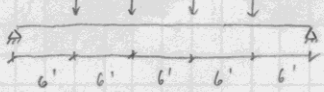
Girder Design

$K_{LL} A_T = 2\left(\frac{30.5}{2} + \frac{27}{2}\right)30 = 1725 \text{ ft}^2 > 400 \text{ ft}^2$ ∴ Live load can be reduced

$LL = 80\left(0.25 + \frac{15}{\sqrt{1725}}\right) = 80(0.611) = 48.9 \text{ psf}$ > 0.5 ∴ okay

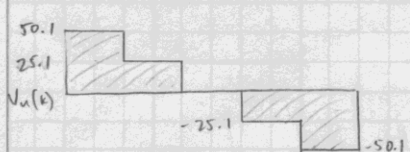
Assume ×43 on both sides

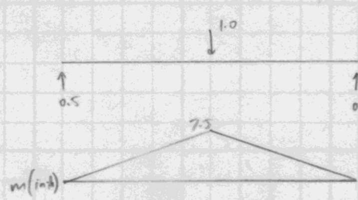
$w_u = 1.2(50) + 1.6(48.9) = 138.2 \text{ psf}$, $P_u = 138.2(6)\left(\frac{27}{2} + \frac{30.5}{2}\right) + \left(\frac{27}{2} + \frac{30.5}{2}\right)43 = 25.1^k$

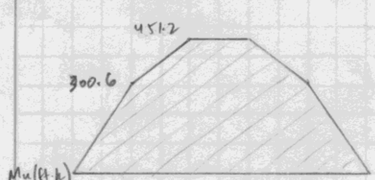


∴ $V_u = 50.1^k$, $M_u = 451.2^{'k}$

Virtual Work for Δ_{max}:







$\Delta_{max} = \left[\frac{9(901.8)}{EI} + \frac{27(2255.4)}{EI} + \frac{20.25(1353.6)}{EI}\right]2$ ← symmetry

$\Delta_{max} = \frac{192844.8 \text{ ft}^2 k^2 \, 144}{(29000)I}$, $\Delta_{max} = \frac{30(12)}{240} = 1.5"$

$I_{min} = \frac{192844.8(144)}{29000(1.5)} = 638.4 \text{ in}^4$

Kevin Zinsmeister | Tech Report 2 | Non-Composite | 3

Selecting by I_x, Pick W16x50

Check unbraced length:

$\frac{L_b}{r_y} \le 1.76\sqrt{\frac{E}{F_y}} \Rightarrow \frac{6}{1.59} \le 1.76\sqrt{\frac{29}{50}} \Rightarrow 3.77 \not\le 1.34$ ∴ Use Table 3-10 from steel manual

From Table 3-10, Pick W16x67 w/ $\phi M_n = 487^{'k} > 451.2^{'k}$ ∴ okay in flexure

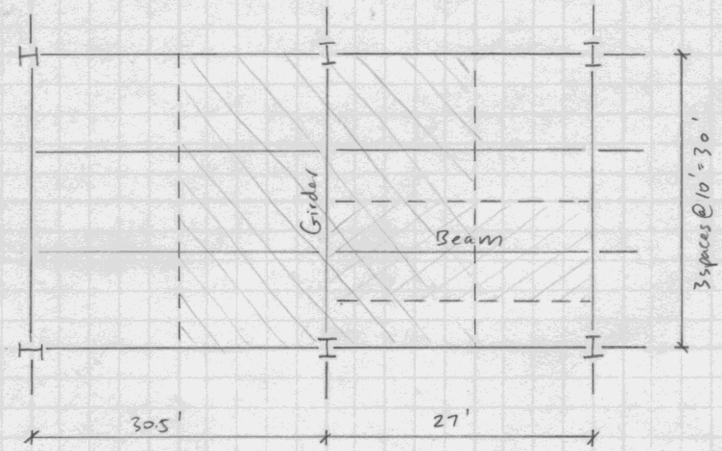
$A_{w_{min}} = \frac{V_u}{\phi 0.6 F_y} = \frac{50.1}{0.9(0.6)50} = 1.86 \text{ in}^2$, $A_w = t_w \cdot T = 0.395(13\,1/4) = 5.23 \text{ in}^2 > 1.86 \text{ in}^2$ ∴ okay in shear

$I_x = 954 \text{ in}^4 > 638.4 \text{ in}^4$ ∴ okay in deflection

Appendix D: Composite Steel System Design

Kevin Zinsmeister | Tech Report 2 | Composite Deck System | 1

Deck Design: Assume the following spacings



$K_{LL} A_T = 1(10)27 = 270\ ft^2 < 400\ ft^2$ ∴ live load reduction not allowed

$w_u = 15 + 80 + 37 = 132$ psf

∴ Pick 1.5 VL 17 w/ 3 1/4" LWC topping (4 3/4" thickness total)

Check Unshored Clear Span (3 spans): 10'-6" > 10'-0" ∴ okay

Check Load Capacity: 184 psf > 132 psf ∴ okay

Beam Design

$K_{LL} A_T = 2(10)27 = 540\ ft^2 > 400\ ft^2$ ∴ live load reduction allowed

$LL = 80\left(0.25 + \frac{15}{\sqrt{540}}\right) = 71.6$ psf

$w_u = 1.2(15+37+5) + 1.6(71.6) = 183.0$ psf $\Rightarrow w_u = 10(183) = 1.83$ klf

$M_u = \frac{wl^2}{8} = \frac{1.83(27)^2}{8} = 166.8^{'k}$

conservative ✓

Assume a Y2 value using a = 1.0 in, $Y2 = 3.75 - \frac{1.0}{2} = 3.25$ in. ∴ round down to 3 in

From Table 3-19 pick W10×19 w/ $M_u = 171^{'k}$; $\Sigma Q_n = 281^k$

$b_{eff} = \min \begin{cases} 2(\text{span}/8) = 27(12)/4 = 81'' \Leftarrow \text{controls} \\ 2(\ell/2) = 10(12) = 120'' \end{cases}$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{281}{0.85(5)81} = 0.816$ in. < 1.5 in = assumed a value ∴ selection is conservative

Kevin Zinsmeister | Tech Report 2 | Composite Deck System | 2

Check bare steel strength: Use DL & construction live load

$w_u = 1.2(15+37+5)+1.6(20) = 100.4$ psf $\Rightarrow w_u = 100.4(10) = 1.00$ klf $\Rightarrow M_u = \frac{1.00(27)^2}{8} = 91.2^{'k}$

W10x19 has a $\phi M_n = 81.0^{'k} < 91.2^{'k}$ ∴ not okay

∴ Pick W10x22 w/ $\phi M_n = 97.5^{'k} > 91.2^{'k}$ ∴ okay

From Table 3-19 W10x22 w/ $M_u = 173^{'k}$, $\Sigma Q_n = 221^k$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{221}{0.85(5)81} = 0.642$ in < 1.0 in assumed a value ∴ selection is conservative

Design Shear Studs:

$Q = 0.5 A_{sc}\sqrt{f'_c E_c} \le R_g R_p A_{sc} F_u \Rightarrow 0.5(0.442)\sqrt{5000(2.58)} = 25.1^k \le 0.442(65)0.6(0.85) = 14.6^k$ controls

$A_{sc} = \pi(\frac{0.75}{2})^2 = 0.442$ in², $E_c = w_c^{1.5}\sqrt{f'_c} = 0.11^{1.5}\sqrt{5000} = 2.58$, $R_p = 0.6$ for weak position

$w_r/h_r = 1.75/1.5 = 1.167 < 1.5$ ∴ $R_g = 0.85$

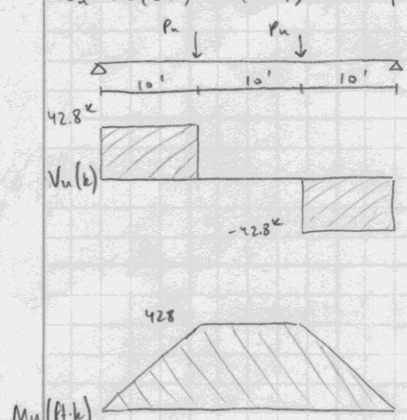
$V'_g = \Sigma Q_n = 221^k$ controls, $V'_c = 0.85 f'_c b_{eff} t = 0.85(5)81(3.25) = 1118.8^k$

studs $= \frac{221}{14.6} = 15.13$ stud ∴ 32 studs for the full beam span

Girder Design

$LL = 80(0.25 + \frac{15}{\sqrt{2(28.75)30}}) = 48.9$ psf

$w_u = 1.2(57)+1.6(48.9) = 146.6$ psf, $P_u = 146.6(10)(28.75) + 22(28.75) = 42.8^k$



Assume $Y2 = 3$ ∴ From Table 3-19 pick W18x35 w/ $M_u = 428$ & $\Sigma Q_n = 387$

Kevin Zinsmeister | Tech Report 2 | Composite Deck System | 3

b'_{min}: $span/8 = 30(12)/8 = 45"$ ← controls; $(\ell_2/2) = 30.5(12)/2 = 183"$; $(\ell_1/2) = 27(12)/2 = 162"$ $\therefore b_{eff} = 2(45") = 90"$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{387}{0.85(5)90} = 1.01$ in < 1.5 in $\therefore$ choice is conservative

Check bare steel strength: use DL & Construction live load

$w_u = 1.2(57) + 1.6(20) = 100.4$ psf, $P_u = 100.4(10)28.75 = 28.8^k$ $\therefore M_u = 28.8(10) = 288^{1k}$

W18x35 has $\phi M_p = 249^{1k} < 288^{1k}$ $\therefore$ does not work

$\therefore$ Pick W16x45 w/ $\phi M_p = 309^{1k} > 288^{1k}$ $\therefore$ okay, $M_n = 437^{1k} > 428^{1k}$ $\therefore$ okay

$\Sigma Q = 216^k$

$a = \frac{216}{0.85(5)90} = 0.56 < 1.5$ $\therefore$ choice is conservative

Design Shear Studs:

$Q_n = 14.6$, $V'_g = 216^k$ (controls), $V'_c = 0.85 f'_c b_{eff} t = 0.85(5)90(3.25) = 1243.1^k$

\# studs $\frac{216}{14.6} = 14.8$ $\therefore$ 30 stud for the full beam span

Deflection Checks

Δ_{max} Beam $= \frac{5(1.83)27^4 1728}{384(290)29000} = 2.60" > 1.35"$ $\therefore$ not okay

$\therefore I_{LBmin} = \frac{5(1.83)27^4 1728}{384(29000)1.35} = 558.9$ in^4

$\therefore$ Pick W16x31 w/ $I_{LB} = 593$ in$^4 > 558.9$ in^4 $\therefore$ okay

$M_u = 279^{1k} > 166.8^{1k}$ $\therefore$ okay, $\phi M_p = 203^{1k} > 91.2^{1k}$ $\therefore$ okay

$\Sigma Q_n = 114^k$ $\therefore$ # studs $= 114/14.6 = 7.8$ $\therefore$ 16 studs for the full beam span

Δ_{max} Girder $= \frac{P\ell^3}{28EI} = \frac{42.8(27)^3 1728}{28(29000)985} = 1.82" > \frac{\ell}{240} = \frac{30(12)}{240} = 1.5"$ $\therefore$ not okay

$\therefore I_{LBmin} = \frac{42.8(27)^3 1728}{28(29000)1.5} = 1195.2$ in^4

$\therefore$ Pick W18x50 w/ $I_{LB} = 1220$ in$^4 > 1195.2$ in^4 $\therefore$ okay, $\phi M_p = 379^{1k} > 288^{1k}$ $\therefore$ okay

$M_u = 507^{1k} > 428^{1k}$ $\therefore$ okay, $\Sigma Q_n = 183^k$ $\therefore$ # studs $= \frac{183}{14.6} = 12.5$ $\therefore$ 13 studs per first 10' from support each side

Appendix E: Two-Way Flat Plate Design

Kevin Zinsmeister | Tech Report 2 | Two-Way Flat Plate | 1

Requirements of Direct-Design Method:

1. Minimum of three continuous spans in each direction ∴ okay.
2. Span ratio < 2 => $\frac{l_1}{l_2} = \frac{30}{27} = 1.11 < 2$ ∴ okay
3. Successive span lengths $\geq 2/3\, l$, $30.5'(\frac{2}{3}) = 20.\bar{3}' < 27'$ ∴ okay
4. Columns w/in 0.1 times span parallel to offset, no offsets ∴ okay
5. All loads due to gravity ∴ okay
6. Service live load ≤ 2 × service dead load ∴ Assume DL will be comparable for now
7. since no beams, does not apply ∴ okay

Pick Slab Thickness

Assume rebar < #7 ∴ 60 ksi steel, Assume Edge beams used

From ACI Table 9.5(c), $\frac{l_n}{36}$ for interior panels w/ drop panels

$l_n = 30 - (\frac{24}{12}) = 28'$ => $\frac{l_n}{36} = \frac{28}{36} = 0.\bar{7}\,ft = 9.\bar{3}\,in$ => Round up to 9.5" thick slab

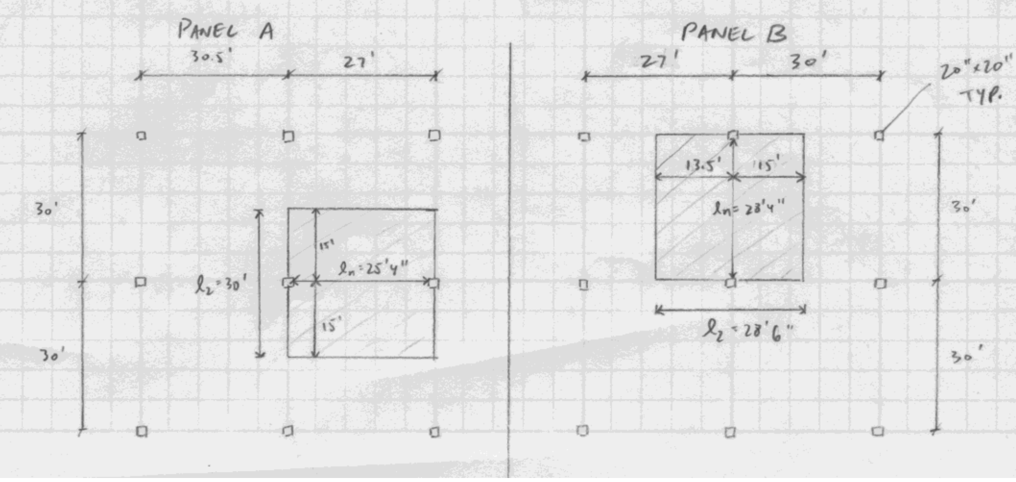
Load determination: As per ASCE 4.7.2

$LL = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right) = 80\left(0.25 + \frac{15}{\sqrt{1(810)}}\right) = 80(0.777) = 62.2\,psf > 80(0.5) = 40\,psf$ ∴ okay

$A_T = 27'(30') = 810 > 400\,ft^2$ ∴ okay $K_{LL} = 1$ for 2-way slabs

$q_u = 1.2\left(\frac{9.5}{12}\right)150 + 1.6(62.2) = 142.5 + 99.52 = 242.02\,psf = 0.242\,ksf$

Note: As per ASCE 4.3.2 partition load are neglected for uniform live loads greater than 80 psf



Kevin Zinsmeister	Tech Report 2	Two-Way Flat Plate	2

Moment Determination

$M_A = \frac{q_u l_2 l_n^2}{8} = \frac{1}{8}(0.242)30(25.33)^2 = 582.4\ ^{'k}$

$M_B = \frac{q_u l_2 l_n^2}{8} = \frac{1}{8}(0.242)28.5(28.33)^2 = 692.1\ ^{'k}$

From ACI 13.6.3.2 $M_A^- = 0.65(582.4) = 378.6\ ^{'k}$ $\quad M_A^+ = 0.35(582.4) = 203.8\ ^{'k}$

From ACI 13.6.3.2 $M_B^- = 0.65(692.1) = 449.9\ ^{'k}$ $\quad M_B^+ = 0.35(692.1) = 242.2\ ^{'k}$

Column & Middle Strips

$CS_A = 2\left(\frac{27}{4}\right) = 13.5'$ wide $\quad MS_A = 30 - 13.5 = 16.5'$ wide

$CS_B = \frac{30}{4} + \frac{27}{4} = 14.25'$ wide $\quad MS_{BL} = 27 - 13.5 = 13.5'$ wide $\quad MS_{BR} = 30 - 15 = 15'$ wide

From ACI 13.6.4.1 $CS_A^- = 0.75(378.6) = 284.0\ ^{'k}$ $\quad MS_A^- = 0.25(378.6) = 94.7\ ^{'k}$

$CS_B^- = 0.75(449.9) = 337.4\ ^{'k}$ $\quad MS_B^- = 0.25(449.9) = 112.5\ ^{'k}$

From ACI 13.6.4.4 $CS_A^+ = 0.6(203.8) = 112.3\ ^{'k}$ $\quad MS_A^+ = 0.4(203.8) = 81.5\ ^{'k}$

$CS_B^+ = 0.6(242.2) = 145.3\ ^{'k}$ $\quad MS_B^+ = 0.4(242.2) = 96.9\ ^{'k}$

Steel Calculations

$A_s = \frac{M_u}{\phi f_y jd}$

$jd = d - a/2$, but assume $jd = 0.95d$ where $d_A = h - 1.9 = 9.5 - 1.9 = 7.6''$ & $d_B = 9.5 - 1.15 = 8.35''$

Column Strip Panel A Negative Reinforcement

$A_s = \frac{284(12000)}{0.9(60000)0.95(7.6)} = \frac{3408000}{389880} = 8.74\ in^2 \Rightarrow a = \frac{A_s f_y}{0.85 f'_c b} = \frac{8.74(60000)}{0.85(5000)13.5(12)} = 0.76\ in$

$c = a/\beta_1 = 0.76/0.8 = 0.95''$ $\quad \varepsilon_s = \frac{0.003}{0.95}(7.6 - 0.95) = 0.021 > 0.005$ ∴ tension controlled ∴ $\phi = 0.9$

$jd = d - a/2 = 7.6 - 0.76/2 = 7.22 = 0.95d$ ∴ no change from before

From ACI 13.3.1 $A_{smin} = 0.0018bh = 0.0018(13.5)12(9.5) = 2.77\ in^2 < 8.74\ in^2$ ∴ $8.74\ in^2$ controls

From ACI 13.3.2 $s_{max} = 2h = 2(9.5) = 19'' > 18''$ From ACI 7.12.2.2 ∴ 18" controls

Minimum number of bar spaces $= \frac{13.5'(12)}{18} = 9$ ∴ 10 bars minimum

$\frac{8.74}{0.44} = 19.9$ ∴ pick 20-#6 bars @ 8" w/ $A_s = 8.8\ in^2$

8" < 18" ∴ okay for max spacing, $8.8\ in^2 > 8.74\ in^2$ ∴ okay for A_{smin}

Kevin Zinsmeister | Tech Report 2 | Two-Way Flat Plate | (3)

Middle Strip Panel A Negative Reinforcement

$A_s = \frac{94.7(12000)}{0.9(60000)0.95(7.6)} = 2.92\ in^2 \Rightarrow a = \frac{A_s f_y}{0.85 f'_c b} = \frac{2.92(60000)}{0.85(5000)16.5(12)} = 0.21\ in$

$c = a/\beta_1 = 0.21/0.8 = 0.26"$, $\varepsilon_s = \frac{0.003}{0.26}(7.6-0.26) = 0.085 > 0.005$ ∴ tension controlled ∴ $\phi = 0.9$

$jd = d - a/2 = 7.6 - 0.21/2 = 7.50" > 7.22"$ ∴ A_s is conservative ∴ okay

From ACI 13.3.1 $A_{smin} = 0.0018bh = 0.0018(16.5)12(9.5) = 3.39\ in^2 > 2.92\ in^2$ ∴ $3.39\ in^2$ controls

Minimum number of bar spaces $= \frac{16.5(12)}{18} = 11$ ∴ 12 bars minimum

$\frac{3.39}{0.31} = 10.94$ ∴ 11 - #5 bars, but need 12 for minimum for max spacing

∴ pick 12 - #5 @ 18" w/ $A_s = 3.72\ in^2$

18" = 18" ∴ okay for max spacing, $3.72\ in^2 > 3.39\ in^2$ ∴ okay for A_{smin}

Column Strip Panel A Positive Reinforcement

$A_s = \frac{112.3(12000)}{0.9(60000)0.95(7.6)} = 3.46\ in^2 \Rightarrow a = \frac{3.46(60000)}{0.85(5000)13.5(12)} = 0.30\ in$

$c = a/\beta_1 = 0.3/0.8 = 0.38"$, $\varepsilon_s = \frac{0.003}{0.38}(7.6-0.38) = 0.057 > 0.005$ ∴ tension controlled ∴ $\phi = 0.9$

$jd = d - a/2 = 7.6 - 0.3/2 = 7.45" > 7.22"$ ∴ A_s is conservative ∴ okay

From ACI 13.3.1 $A_{smin} = 0.0018bh = 2.77\ in^2 < 3.46\ in^2$ ∴ $3.46\ in^2$ controls

Minimum number of bar spaces $= \frac{13.5(12)}{18} = 9$ ∴ 10 bars minimum

$\frac{3.46}{0.31} = 11.16$ ∴ Pick 12 - #5 @ 18 w/ $A_s = 3.72\ in^2$

18" = 18" ∴ okay for max spacing, $3.72\ in^2 > 3.46\ in^2$ ∴ okay for A_{smin}

Middle Strip Panel A Positive Reinforcement

$A_s = \frac{81.5(12000)}{0.9(60000)0.95(7.6)} = 2.51\ in^2 \Rightarrow a = \frac{2.51(60000)}{0.85(5000)16.5(12)} = 0.18\ in$ $c = a/\beta_1 = 0.18/0.8 = 0.23"$

$\varepsilon_s = \frac{0.003}{0.23}(7.6-0.23) = 0.023 > 0.005$ ∴ tension controlled ∴ $\phi = 0.9$, $jd = 7.51" > 7.22"$ ∴ A_s is conservative

From ACI 13.3.1 $A_{smin} = 0.0018bh = 3.39\ in^2 > 2.51\ in^2$ ∴ $3.39\ in^2$ controls

∴ Same as MS_A^- reinforcement ∴ 12 - #5 @ 18" w/ $A_s = 3.72\ in^2$

18" = 18" ∴ okay for max spacing, $3.72\ in^2 > 3.39\ in^2$ ∴ okay for A_{smin}

Kevin Zinsmeister | Tech Report 2 | Two-Way Flat Plate | (4)

Column Strip Panel B Negative Reinforcement

$A_s = \frac{337.4(12000)}{0.9(60000)\,0.95(8.35)} = 9.45\text{ in}^2 \Rightarrow a = \frac{9.45(60000)}{0.85(5000)\,14.25(12)} = 0.88\text{ in}$ $\quad c = \frac{a}{\beta_1} = \frac{0.88}{0.8} = 1.1\text{ in}$

$\varepsilon_s = \frac{0.003}{1.1}(8.35-1.1) = 0.020 > 0.005 \therefore$ tension controlled $\therefore \phi = 0.9$

$jd = d - a/2 = 8.35 - 0.88/2 = 7.91'' < 0.95(8.35) = 7.93'' \therefore A_s$ needs to be recalculated

$A_s = \frac{337.4(12000)}{0.9(60000)\,7.91} = 9.48\text{ in}^2$

From ACI 13.3.1 $A_{s\,min} = 0.0018bh = 0.0018(14.25)12(9.5) = 2.92\text{ in}^2 < 9.48\text{ in}^2 \therefore 9.48\text{ in}^2$ controls

Minimum number of bar spaces $= \frac{14.25(12)}{18} = 9.5 \therefore$ 10 bars minimum

$\frac{9.48}{0.44} = 21.55 \therefore$ Pick 24 - #6 @ 7" w/ $A_s = 10.56\text{ in}^2$

$7'' < 18'' \therefore$ okay for max spacing, $10.56\text{ in}^2 > 9.48\text{ in}^2 \therefore$ okay for $A_{s\,min}$

Middle Strip Panel B Negative Reinforcement

$A_s = \frac{112.5(12000)}{0.9(60000)\,0.95(8.35)} = 3.15\text{ in}^2 \Rightarrow a = \frac{3.15(60000)}{0.85(5000)\,13.5(12)} = 0.28\text{ in}$ $\quad c = \frac{a}{\beta_1} = \frac{0.28}{0.8} = 0.35\text{ in}$

$\varepsilon_s = \frac{0.003}{0.35}(8.35-0.35) = 0.069 > 0.005 \therefore$ tension controlled $\therefore \phi = 0.9$, $jd = 8.21'' > 7.93'' \therefore A_s$ conservative

From ACI 13.3.1 $A_{s\,min} = 0.0018bh = 2.77\text{ in}^2 < 3.15\text{ in}^2 \therefore 3.15\text{ in}^2$ controls

Minimum number of bar spaces $= \frac{13.5(12)}{18} = 9 \therefore$ 10 bars minimum

$\frac{3.15}{0.31} = 10.16 \therefore$ Pick 11 - #5 @ 15" w/ $A_s = 3.41\text{ in}^2$

$15'' < 18'' \therefore$ okay for max spacing, $3.41\text{ in}^2 > 3.15\text{ in}^2 \therefore$ okay for $A_{s\,min}$

Column Strip Panel B Positive Reinforcement

$A_s = \frac{145.3(12000)}{0.9(60000)\,0.95(8.35)} = 4.07\text{ in}^2 \Rightarrow a = \frac{4.07(60000)}{0.85(5000)\,14.25(12)} = 0.34\text{ in}$ $\quad c = \frac{a}{\beta_1} = \frac{0.34}{0.8} = 0.43\text{ in}$

$\varepsilon_s = \frac{0.003}{0.43}(8.35-0.43) = 0.055 > 0.005 \therefore$ tension controlled $\therefore \phi = 0.9$, $jd = 8.18'' > 7.93'' \therefore A_s$ conservative

From ACI 13.3.1 $A_{s\,min} = 0.0018bh = 2.92\text{ in}^2 < 4.07\text{ in}^2 \therefore 4.07\text{ in}^2$ controls

Minimum number of bar spaces $= \frac{14.25(12)}{18} = 9.5 \therefore$ 10 bars minimum

$\frac{4.07}{0.31} = 13.13 \therefore$ Pick 14 - #5 @ 12" w/ $A_s = 4.34\text{ in}^2$

$14'' < 18'' \therefore$ okay for max spacing, $4.34\text{ in}^2 > 4.07\text{ in}^2 \therefore$ okay for $A_{s\,min}$

Kevin Zinsmeister | Tech Report 2 | Two-Way Flat Plate | 5

Middle Strip Panel B Positive Reinforcement

$A_s = \frac{96.9(12000)}{0.9(60000)\,0.95(8.35)} = 2.72\ in^2 \Rightarrow a = \frac{2.72(60000)}{0.85(5000)13.5(12)} = 0.24 \quad c = \frac{a}{\beta_1} = \frac{0.24}{0.8} = 0.3\ in$

$\varepsilon_s = \frac{0.003}{0.3}(8.35-0.3) = 0.081 > 0.005$ ∴ tension controlled ∴ $\phi = 0.9$

$jd = d - a/2 = 8.35 - 0.24/2 = 8.23'' > 7.93''$ ∴ A_s is conservative

From ACI 13.3.1 $A_{smin} = 0.0018bh = 2.77\ in^2 > 2.72\ in^2$ ∴ $2.77\ in^2$ controls

Minimum number of bar spaces $= \frac{13.5(12)}{18} = 9$ ∴ 10 bars minimum

$\frac{2.77}{0.31} = 8.94$ ∴ 9-#5 bars, but 10 bars minimum for max spacing

∴ Pick 10-#5 @ 16" w/ $A_s = 3.1\ in^2$

$16'' < 18''$ ∴ okay for max spacing, $3.1\ in^2 > 2.77\ in^2$ ∴ okay for A_{smin}

Shear Design: One-way shear design

$d = 9.5 - \underset{cover}{0.75} - \underset{d_b}{0.75} = 8$

$V_{uA} = 0.242\left(\frac{27}{2} - \left(\frac{10+8}{12}\right)\right)30 = 87.1^k \quad V_{uB} = 0.242\left(\frac{30}{2} - \frac{18}{12}\right)28.5 = 93.1^k$

$\phi V_{cA} = 0.75(2\lambda\sqrt{f'_c}\,bd) = 0.75(2)1.0\sqrt{5000}\,(30)12(8) = 305470\ lb = 305.5^k > 87.1^k$ ∴ okay

$\phi V_{cB} = 0.75(2)1.0\sqrt{5000}\,(28.5)12(8) = 290197\ lb = 290.2^k > 93.1^k$ ∴ okay for one-way shear

Two-way Punching Shear:

$V_u = \left[\left(\frac{30.5}{2} + \frac{27}{2}\right)30 - \frac{28}{12}\left(\frac{28}{12}\right)\right]0.242 = 207.4^k \qquad b_o = 2(28+28) = 112\ in$

$V_c = \min \begin{cases} 4\lambda\sqrt{f'_c}\,b_o d = 4(1.0)\sqrt{5000}\,(112)8 = 253427\ lb = 253.4^k \leftarrow \text{controls} \\ \left(2 + \frac{4}{\beta}\right)\lambda\sqrt{f'_c}\,b_o d = \left(2 + \frac{4}{1}\right)1.0\sqrt{5000}\,(112)8 = 380141\ lb = 380.1^k \\ \left(\frac{\alpha_s d}{b_o} + 2\right)\lambda\sqrt{f'_c}\,b_o d = \frac{40(8)}{112}(1.0)\sqrt{5000}\,(112)8 = 307733\ lb = 307.7^k \end{cases}$

$\beta = \frac{20}{20} = 1$, $\alpha_s = 40$ for interior columns

$253.4^k > 207.4^k$ ∴ okay for punching shear

Deflection Checks

By ACI 9.5.3, having met minimum slab thickness requirements, deflections meet ACI Standards and do not need to be calculated.

Appendix F: Floor Comparison Calculations

Floor Systems Cost Calculations				
Floor System	Square Foot Cost	Location Factor	Size Factor	Adjusted Square Foot Cost
One-Way Pan Joist	$18.65	0.911	1.00	$16.99
Non-Composite	$27.39	0.911	1.00	$24.95
Composite	$20.35	0.911	1.00	$18.54
Two-Way Flat Plate	$15.35	0.911	1.10	$15.38

Table 9: Floor System Cost Calculations

Non Composite Weight		
Beams & Girders		
linear length (ft)	linear weight (lb/ft)	total weight (lb)
135	43	5805
30	67	2010
Deck & Slab		
area (sf)	weight (psf)	total weight (lb)
810	40	32400
	Sum (lb)=	40215
	Sum (k)=	40.215
	Unit Weight (psf)=	49.648

Table 10: Non-Composite Weight Calculations

Composite Weight		
Beams & Girders		
linear length (ft)	linear weight (lb/ft)	total weight (lb)
81	31	2511
30	50	1500
Deck & Slab		
area (sf)	weight (psf)	total weight (lb)
810	37	29970
	Sum (lb)=	33981
	Sum (k)=	33.981
	Unit Weight (psf)=	41.952

Table 11: Composite Weight Calculations

Pan Joist Weight				
Beams & Girders				
linear length (ft)	width (in)	height (in)	volume (ft^3)	total weight (lb)
97	6	16	64.7	9700
30	16.5	16	55	8250
	Slab			
	area (sf)	thickness (in)	volume (ft^3)	total weight (lb)
	810	4	270	40500
			Sum (lb)=	58450
			Sum (k)=	58.450
			Unit Weight (psf)=	72.160

Table 12: Pan Joist Weight Calculations

Flat Plate Weight			
Slab			
area (sf)	thickness (in)	volume (ft^3)	total weight (lb)
810	9.5	641.25	96187.5
		Sum (lb)=	96187.5
		Sum (k)=	96.188
		Unit Weight (psf)=	118.750

Table 13: Flat Plate Weight Calculations