

**Kaleida Health – Global Heart and Vascular Institute
University at Buffalo – CTRC/Incubator**

Buffalo, New York

Technical Report #2



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Structural Option
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Technical Report #2

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Executive Summary

The following document is the second technical report of senior thesis and includes information regarding the structural floor system of the Kaleida Health and University at Buffalo, Global Heart and Vascular Institute. This project will be referred to throughout this report simply as GHVI. This report includes the design and analysis of three alternative floor systems for a typical 31'-6" by 31'-6" interior bay on the fourth floor. These alternative floor systems include:

- Flat Slab with Drop Panels
- Pre-Cast Hollow Core Plank
- One-Way Joist and Beam

The design of the flat slab system resulted in a 10" slab with 6¼" drop panels which extend 6' in each direction from the column center line. Reinforcing was designed for the column strip and middle strip positive and negative moments, and one-way and two-way punching shear was checked. This strategy resulted in an efficient and cost effective design, and the most viable alternative option.

A pre-cast hollow core plank design with 4'-0" by 10" planks with 2" topping was used for the second alternative system. Using the PCI Design Handbook, it was determined that these planks would contain 6 - 8/16" diameter straight strands. Although this resulted in an easily constructed building and viable alternative, it required that the bay sizes be changed to 32' by 32' to avoid high priced custom planks.

For the third alternative system, a one-way joist and beam design was considered. Under this design, a 4½" slab with a 72" pan joist module was chosen. This module consists of 66" pans and 6" ribs which are 16" deep, resulting in a total thickness of 20½". Top and bottom reinforcement was sized, and shear was checked for an interior span. After the slab and joists were designed, the assumed girder width of 3' was checked and confirmed. It can be said that this is a viable option, but it requires that the bays be modified by almost 2' in each direction.

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Introduction

GHVI is a state-of-the-art medical facility and a fundamental component in a joint undertaking between Kaleida Health Systems and the University at Buffalo School of Medicine. The building spans ten levels and includes exam rooms, classrooms, offices, a café, a wellness center and library, and a research facility. It is intended to bring patients, surgeons, and researchers together to collaborate in an unprecedented way.

Key themes considered throughout the design were collaboration, flexibility, and comfort. Kaleida Health Systems sought a structure that would link clinical and research work and combine all vascular disciplines. A spirit of collaboration was the driving force behind bringing both Kaleida and the University at Buffalo together in a single structure. Keeping this in mind, the design team developed the facility with a “collaborative core” which enables interaction among those working within the facility. This collaborative learning environment brings together research, ideas, and solutions and results in better patient care.

A universal grid design increases the flexibility of space and achieves measurable advantage in initial capital cost, speed to market, operating economy, and future adaptability. The universal grid is comprised of three 10’-6” building modules and forms a 31’-6” x 31’-6” structural grid capable of integrating the building’s diverse functions. When combined with an 18’ floor-to-floor height, the flexible grid creates an open plan capable of adapting to present and future healthcare needs. The building will be able to incorporate unknown, but rapidly changing technological developments within the industry, also giving it longevity through its adaptability.

With comfort in mind, a separate “hotel” level was designed on the second floor and separated from the procedural floors. Functionally, the “hotel” is comprised of private patient rooms and a small lounge area. Other family lounges are also provided and the perimeter of the building is shaped to bring in as much natural daylight as possible. The vision of GHVI is to create an atmosphere that is more than a simple hospital, but instead a facility for world-class treatment and state-of-the-art technology.

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Structural System Overview

Foundation

Based on the recommendations of the October 2008 Geotechnical Report by Empire Geo-Services, Inc., the foundation of GHVI consists of grade beams and pile caps placed on top of steel helical piles.

The helical piles are HP12x74 sections with an allowable axial capacity of 342 kips (171 tons) which are driven to absolute refusal on limestone bedrock 82 to 87 feet below the sub-basement finish level. Grade beams and pile caps have a concrete strength of 4000 psi, and it should be noted that the width of the grade beams equals that of the pile caps at the foundations of the braced frames. The grade beams provide resistance to lateral column base movement, and the pile caps link the steel helical piles and the structural steel columns of the superstructure.

Spanning the grade beams is the sub-basement floor, a 5" slab-on-grade. Due to the slope of the site, part of this sub-basement is below grade, and therefore a one foot thick foundation wall slopes along the west elevation of the sub-basement.

Floor System

The floors of GHVI consist of 3" composite metal deck with a total slab thickness ranging from 4" to 7½". The metal deck is 18-gage galvanized steel sheets resting on various different beam and girder sizes. These sizes change throughout the structure because of the various functions of the spaces. The bay sizes through the building are mostly 31'-6" by 31'-6", with beams spaced at 10'-6". As was discussed in the introduction, this universal grid design increases the future flexibility of the space. A slight variation in the floor can be seen on Levels 6-8. On these levels, part of the floor structure is left open to provide for the collaborative atrium that was designed to bring the various disciplines together.

Gravity System

Steel columns are used throughout the building to transmit the gravity load to the foundation. All of the columns in the building are W14s, but they range in weight from 68 lb/ft to 370 lb/ft, and they are typically spliced every 36 feet. These columns provide an 18' floor-to-floor height, which also contributes to the universal grid and future flexibility of the space.

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Lateral System

The lateral system of GHVI utilizes braced frames located near the perimeter of the building, all of which are HSS sections. A braced frame system is ideal in steel buildings because of its low cost compared to moment connection frames. There are moment connections in some parts of this structure, but they are used to support the small amount of slab overhang that is cantilevered. These moment connections may actually add some stiffness to the lateral system, but they cannot be included in the lateral system design. Figure A depicts the location of the braced frames on the outer part of the structure.

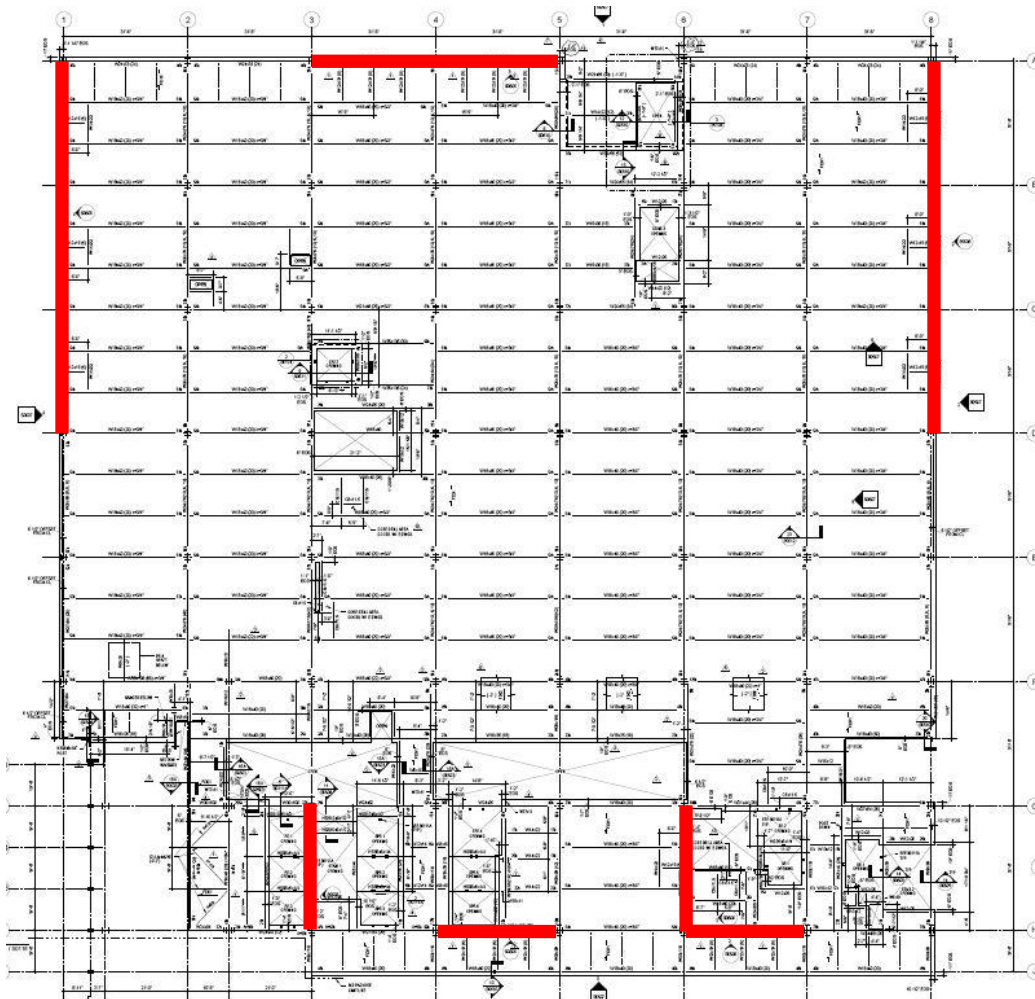


Figure A – Level Two Framing Plan with Braced Frames Highlighted (Canon Design)

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Codes and References

Original Design Codes

- National Model Building Code:
Building Code of New York State 2007
- Design Codes:
"Load and Resistance Factor Design Specification for Structural Steel Buildings," AISC

"Code of Standard Practice for Steel Buildings and Bridges", AISC

"Manual of Steel Construction - Load and Resistance Factor Design," AISC

ACI 318-05, Building Code Requirements for Structural Concrete

American Society of Civil Engineers, ASCE/SEI 7-02,
Minimum Design Loads for Buildings and Other Structures

Thesis Design Codes

- National Model Building Code:
2009 International Building Code
- Design Codes:
Steel Construction Manual 13th edition, AISC

ACI 318-05, Building Code Requirements for Structural Concrete

PCI Design Handbook, 6th Edition

RSMMeans Building Construction Cost Data 2011 Book

American Society of Civil Engineers, ASCE/SEI 7-10,
Minimum Design Loads for Buildings and Other Structures

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Materials

Structure Steel:

Type	Standard	Grade
Wide Flange Shapes, WT's	ASTM A-992	
Channels & Angles	ASTM A-36	
Pipe	ASTM A-53	Grade B
Hollow Structural Sections (Rectangular & Round)	ASTM A-500	Grade B
Base Plates	ASTM A-572	Grade 42
All Other Steel Members	ASTM A-36	

Concrete:

Type	f'c (psi)	Unit Weight (pcf)
Pile Caps	4000	150
Grade Beams	4000	150
All Other Concrete	4000	150
Slabs-On-Grade	3000	150
Foundation Walls	4000	150

Reinforcing:

Type	Standard	Grade
Typical Bars	ASTM A-615	60
Welded Bars	ASTM A-706	60
Welded Wire Fabric	ASTM A-185	
Steel Fibers	ASTM A-820	Type 1
Bars Noted To Be Field Bent	ASTM A-615	40

Connectors:

Type	Standard
High Strength Bolts, Nuts, & Washers	ASTM A-325 or A-490 (min. 3/4 Diameter)
Anchor Rods	ASTM F1554
Welding Electrode	E70XX
Steel Deck Welding Electrode	E60XX min.

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Gravity Loads

Design Floor Dead Loads

The dead loads shown below are a combination of information obtained from Cannon Design and values determined from ASCE 7-10.

Typical Floor

Steel Deck and 7.5" Slab	75.0 psf
Steel Beams	12.0 psf
Total	87.0 psf

Typical Roof

3' Steel Deck	4.5 psf
Adhered Membrane	2.0 psf
4" Rigid Insulation	6.0 psf
1/2" Protection Board	2.0 psf
Total	14.5 psf

Electrical and Mechanical Areas

Steel Deck and 7.5" Slab	75.0 psf
Steel Beams	12.0 psf
Concrete Pad	25.0 psf
Total	112.0 psf

Vivarium (Level 7)

Steel Deck and 7.5" Slab	75.0 psf
Membrane and 6" LTWT Topping	65.0 psf
Steel Beams	12.0 psf
Masonry Partitions	73.0 psf
Total	225.0 psf

Superimposed Dead Load

MEP	15.0 psf
Ceiling	5.0 psf
Leveling Concrete for Deflection	5.0 psf
Total	25.0 psf

Exterior Curtain Wall – 15.0 psf

Partitions – 10.0 psf

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Floor Live Loads

The live loads shown below are a combination of information obtained from Cannon Design and values determined from ASCE 7-10.

Occupancy or Use	Design (psf)	ASCE 7-10 (psf)
Vivarium	80	60
Hotel (Patient) Floor	125	40
Procedure and Lab Floors	125	60
Mechanical Floors	150	--
Mechanical Floors with Catwalks below	175	--
Electrical Floors	200	--
Mechanical Mezzanine (Low)	40	40
Storage	--	20
Lobby	--	100
Stairs	--	100
Corridors	--	100
Roof	--	20

It should be noted that there is a large difference between the live loads used by Cannon Design and the live loads referenced from ASCE 7-10. This difference can most likely be attributed to the fact that the building was designed to adapt to the ever changing needs of the healthcare industry. By over-designing the floors, it can be assured that they can be used for a variety of functions in the future without the need for redesign and renovation.

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Existing Floor System – Composite Steel

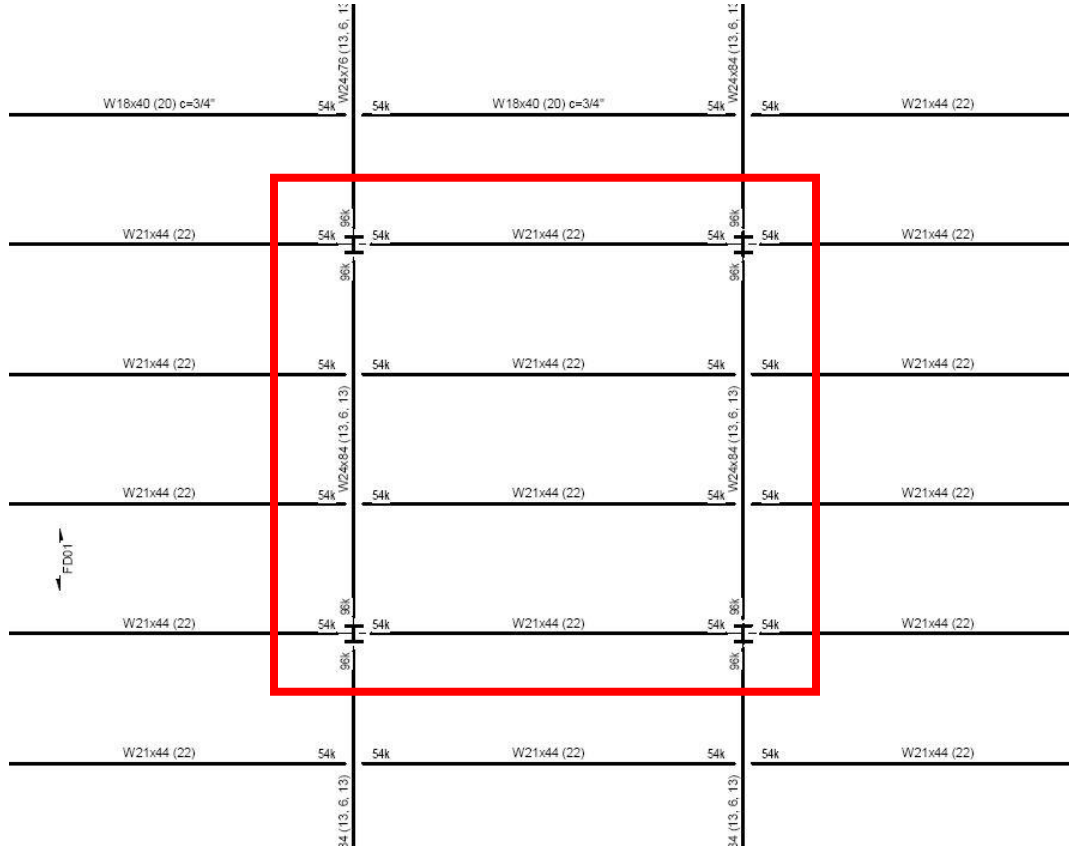


Figure B – Typical Bay on Level 4 (Canon Design)

Description

The existing floor system for GHVI is composite steel with 31'-6" by 31'-6" bays and beams spaced at 10'-6". A typical bay on the fourth level was used for the analysis of this system, and is shown in Figure B. The structure utilizes a 3" composite metal deck with a total slab thickness of 7½". The metal deck is 18-gage galvanized steel and rests on W21x44 beams, which frame into two W24x84 girders. Shear studs are used on both the beams and girders in order to create composite action with the slab. Refer to Appendix B for the design of the existing floor framing system.

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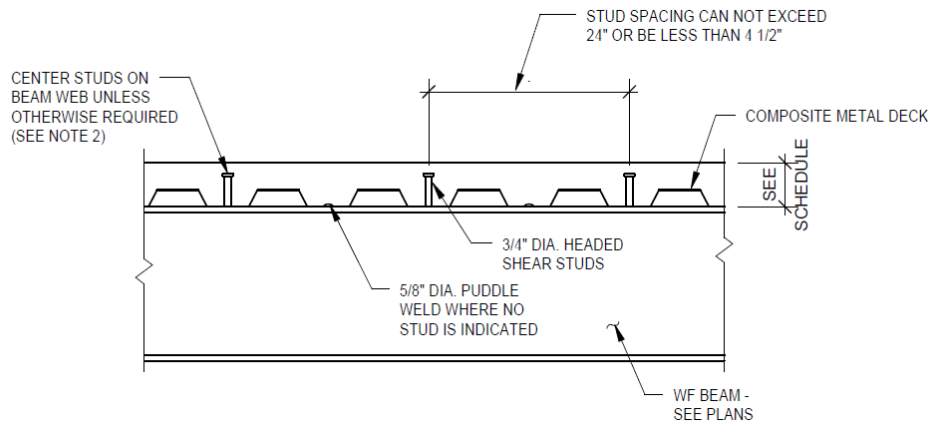


Figure C – Typical Composite Steel Construction (Canon Design)

Design Criteria

Bay Size = 31.5' x 31.5'
Slab Thickness = 7.5" (Total)
Normal Weight Concrete
$f'_c = 3000$ psi
18-Gage Galvanized Steel Deck
Beams: 2 - W21x44 (20)
Girders: 2 - W24x84 (30)
3/4" Shear Studs
Cost = \$29.31/ft ²

Advantages

- Reduced weight
- Shallow members
- Effective use of materials (concrete and steel)
- Good for long spans and heavy loads
- Easy and efficient to design
- Quick to construct

Disadvantages

- Requires spray on fireproofing
- Medium lead time required for steel members
- Added cost of shear connectors

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Alternative Floor System 1 – Flat Slab with Drop Panels

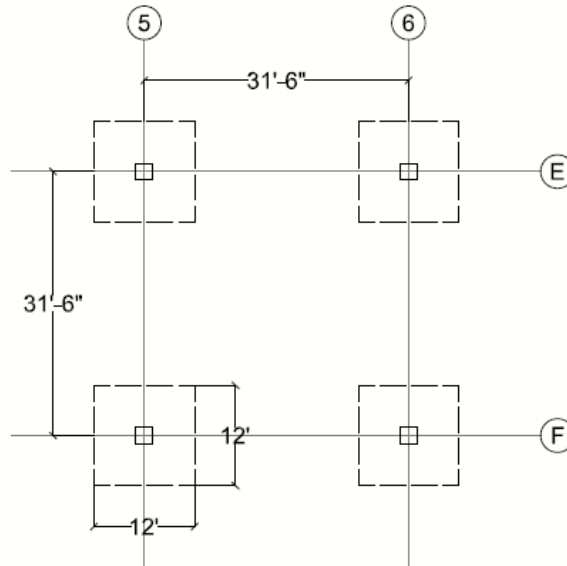


Figure D – Typical Flat Slab with Drop Panels Bay

Description

The first alternate system to be designed was a two-way flat slab with drop panels. The design initially started with a flat plate system, under the assumption that drop panels would be needed due to the large live load on the slab. A slab thickness of 10" was chosen to meet ACI 9.5.3.2, and it was determined that the direct design method could be used to design the slab. The column strip and middle strip reinforcing was designed, and can be found in Appendix C. Finally, one-way shear and two-way punching shear was checked for the slab at the columns. It was determined that the column would indeed punch through the slab if drop panels were not implemented, and so drop panels were designed. When designing the drop panels, ease of constructability was considered, and so a drop of 6 $\frac{1}{4}$ " was used. A detailed set of calculations for this system is located in Appendix C.

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Design Criteria

Bay Size = 31.5' x 31.5'
Slab Thickness = 10"
Normal Weight Concrete
$f'_c = 4000$ psi
Drop panel size = 12' x 12'
Drop panel depth = 6.25"
Cost = \$18.03/ft ²

Advantages

- Increased shear strength at critical section around columns
- Simple construction and formwork
- Low floor-to-floor heights (not necessary for this building)
- Relatively flat ceiling
- Used for moderate to heavy live loads

Disadvantages

- Increased self-weight
- Limited deflection control
- Future slab openings will be a problem with two-way reinforcing
- Possible lateral system redesign

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Alternative Floor System 2 – Pre-Cast Hollow Core Plank

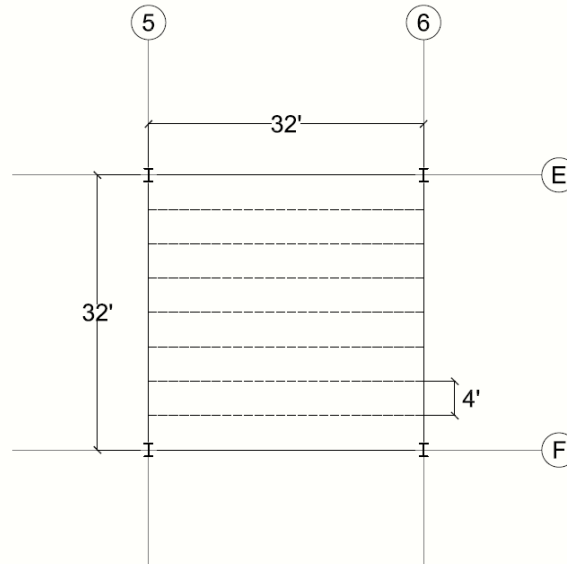


Figure E – Typical Pre-Cast Hollow Core Plank Bay

Description

The second alternative floor system designed for GHVI was a pre-cast hollow core plank system. Because the pre-cast planks come in 4' sections, the standard bay size of the building was altered from 31'-6" by 31'-6" to 32' by 32'. This change is minimal, would be easy to implement, and would have a lower cost than ordering specially designed pre-cast planks. The PCI Design Handbook was used to size the planks, and it was found that 4'-0" by 10" planks with 2" of topping would be sufficient. These planks would contain 6 - 8/16" diameter straight strands. Finally, these planks would rest on 2 W30x116 girders. A complete set of calculations for this system can be found in Appendix D.

Design Criteria

Bay Size = 32' x 32'
4' x 10" panels with 2" topping
6 - 8/16" ϕ strands (straight)
Slab Thickness = 10"
Normal Weight Concrete
$f'_c = 4000$ psi
Girders: 2 - W30x116
Cost = \$25.74/ft ²

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Advantages

- Reduced weight
- Reduced sound and heat transmission
- Flat ceilings
- Quick and simple construction
- Sustainable

Disadvantages

- Slight adjustment of column grid
- Large girders needed for support
- Long lead time
- Not exactly level; leveling compound may be necessary

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Alternative Floor System 3 – One-Way Joist and Beam

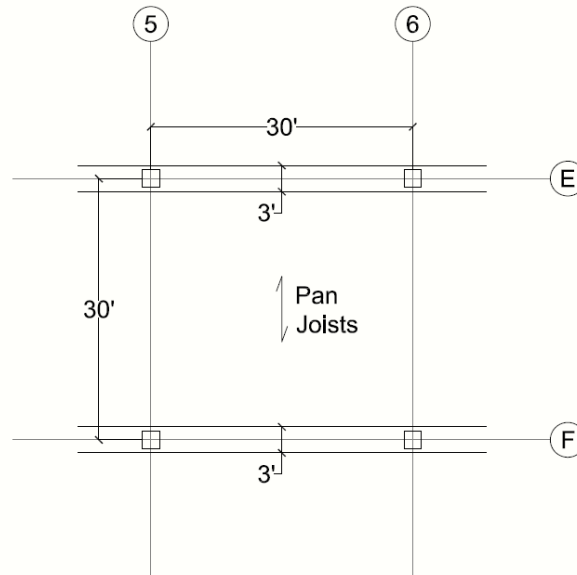


Figure F – Typical One-Way Joist and Beam Bay

Description

The third and final alternative system designed for this building was a one-way pan joist and beam system. This system was considered because it is normally adequate for long spans and heavy live loads. A 4½” slab was selected to meet a two hour fire rating, and a 72” pan joist module was chosen. This module consists of 66” pans and 6” ribs as prescribed by ACI requirements. The ribs are 16” deep, making the total thickness 20½”. Top and bottom reinforcement was sized, and shear was checked for an interior span. After the slab and joists were designed, the assumed girder width of 3’ was checked and confirmed. Detailed calculations for the one-way joist and beam system can be found in Appendix E.

Design Criteria

Bay Size = 30' x 30'
Slab Thickness = 4.5"
72" Pan module
66" Pan, 6" joist
Normal Weight Concrete
$f'_c = 4000$ psi
Girders: 3' wide
Cost = \$19.96/ft ²

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Advantages

- Long spans
- Heavy live loads
- Wider column spacing
- Inherent vibration resistance
- Reduced self-weight due to joist voids
- Possible to fit mechanical equipment in joist voids

Disadvantages

- Large self-weight, despite joist voids
- Adjustment of column grid
- More complex construction and formwork
- Possible lateral system redesign

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Systems Comparison

	System			
	Composite Steel	Flat Slab with Drop Panels	Pre-Cast Hollow Core Plank	One-Way Joist and Beam
Slab Thickness	7.5"	10"	10"	4.5"
Total Structure Thickness	28.5"	16.25"	42"	20.5"
Self-Weight	75 psf	125 psf	93 psf	73 psf
Column Grid Impact	N/A	No	Yes (minimal)	Yes
Fireproofing	2 hr (spray on)	2 hr	2 hr	2 hr
Constructability	Easy	Medium	Easy	Difficult
Lead Time	Medium	Short	Long	Short
Cost	\$29.31/ft ²	\$18.03/ft ²	\$25.74/ft ²	\$19.96/ft ²
Further Investigation?	Yes	Yes	Yes	Yes

Table 1 – Systems Comparison

Structure

Obviously, changing a major component of the structure, such as the floor system, will have an impact on the remainder of the building. The concrete systems are inherently heavier, and therefore would place a larger load on the building's foundation. The number of steel piles and the size of the pile caps may need to be reconsidered. Changing the diaphragm will also have an impact on the lateral system. Concrete systems will be stiffer and more capable of carrying the lateral load, but the current braced frame system would probably need to be redesigned when using the flat slab with drop panels or one-way joist and beam systems. Finally, deflection and vibration changes may need to be studied further. The composite steel system was checked and satisfies deflection requirements, and the concrete systems were designed to meet ACI minimum slab requirements. This ensures that the slabs will in fact meet deflection requirements, but if a specific comparison is to be made between all four systems, a more complex computer analysis will need to be conducted.

Architecture

Each alternative system was considered on how it would alter the architecture of the existing building. The flat slab system with drop panels would have very little impact. It would actually offer a slightly thinner floor thickness, and it would not be necessary to alter the column spacing in any way. The pre-cast hollow core plank system would have some impact on the buildings current architecture. This design would in fact require a minimal shift of the column spacing to allow for the placement of the 4' planks. It also requires very large girders to carry the load

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of the planks, and therefore the floor thickness is much larger than the existing structure. Perhaps the most impact would be seen from the one-way joist and beam systems, which would require the column spacing to be altered by almost two feet. This may not have an effect on the aesthetic look of the building, but it could very well alter the layout of the floors.

Fireproofing

All of the systems considered in this study have a two hour fire rating as prescribed by code. The current composite steel design has an increased cost due to the need for spray on fire proofing. Each of the concrete systems would have inherent fireproofing due to the thickness of the concrete.

Constructability

Constructability is a major concern when building a structure. The current composite steel system is easy to design and simple to construct, and so a change in the floor system would require a system just as simple. The flat slab system with drop panels would be a possibility under this requirement. Even though this system would call for the use of formwork, this formwork would be simple and easy to construct due to the nature of the flat slab. The pre-cast plank system is also extremely easy to construct. Each of the panels is fabricated off-site, transported to the site, and lifted in to place by a crane. The one-way joist and beam system would not be as easily constructed. This system requires complex formwork, which not only takes time, but costs more.

Cost

The most important consideration when choosing a floor system may be cost. Table 1 above shows the cost per square foot for each of the systems analyzed. It can be seen that the current composite steel system is actually priced the highest. This may be due to the fact that it requires a large amount of concrete, as well as structural steel members and detailed connections. The pre-cast plank system has the second highest cost, and this is somewhat expected due to the fact that they are fabricated off-site and trucked where they are needed. The two remaining concrete systems have similar prices, but for different reasons. Flat slab systems with drop panels need less formwork and labor, but require more cubic yards of concrete. The one-way joist and beam system will require less concrete, but has a need for complex formwork. While these estimates give a good comparison of the different prices, if the decision is to be made on cost alone, a more in-depth cost analysis should be completed to assure that the correct system is selected.

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Conclusion

This second technical report has investigated three alternative floor systems for the Kaleida Health and University at Buffalo Global Heart and Vascular Institute.

Analysis of a flat slab system with drop panels shows that this system is the most viable option for an alternative design. The flat slab system works well with moderate spans and heavy live loads; both of which are seen in GHVI. It would have a minimal impact on the architecture of the building, and would not alter the bay sizes. For this building, a flat slab system would be easy to construct, would contain the necessary two hour fireproofing, and would cost the least.

The pre-cast hollow core plank system would also be a viable option. Although it would increase the floor depth considerably, and slightly alter the column bays, it would be easy to construct and would be fire-rated for two hours. This system would come at a higher cost than the other two alternative systems, but it would be less expensive than the current composite steel design.

A one-way joist and beam design would be the most difficult system to construct, but it is the lightest and second least expensive system. This strategy would be a viable alternative, but it would be necessary to modify the column bays by almost two feet in each direction.

After careful analysis of each of these systems, it can be concluded that all three are viable options, but that the flat slab system may be more efficient and cost effective for this building. Future computer analysis will be conducted to examine lateral and foundation designs, and a more specific cost breakdown may be necessary.

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Appendix

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Appendix A: Typical Floor Plans and Elevations

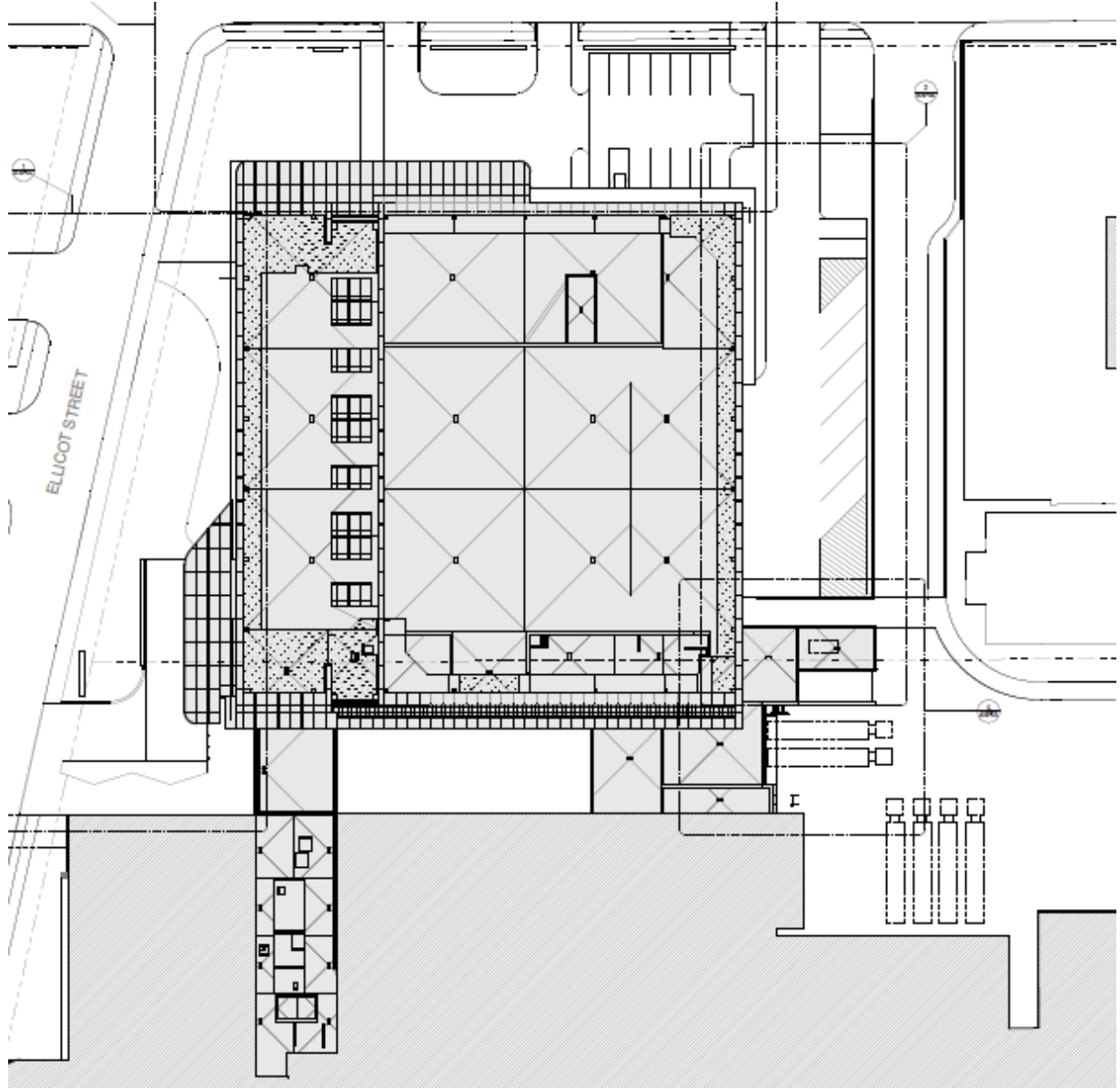


Figure G – Site Plan (Cannon Design)

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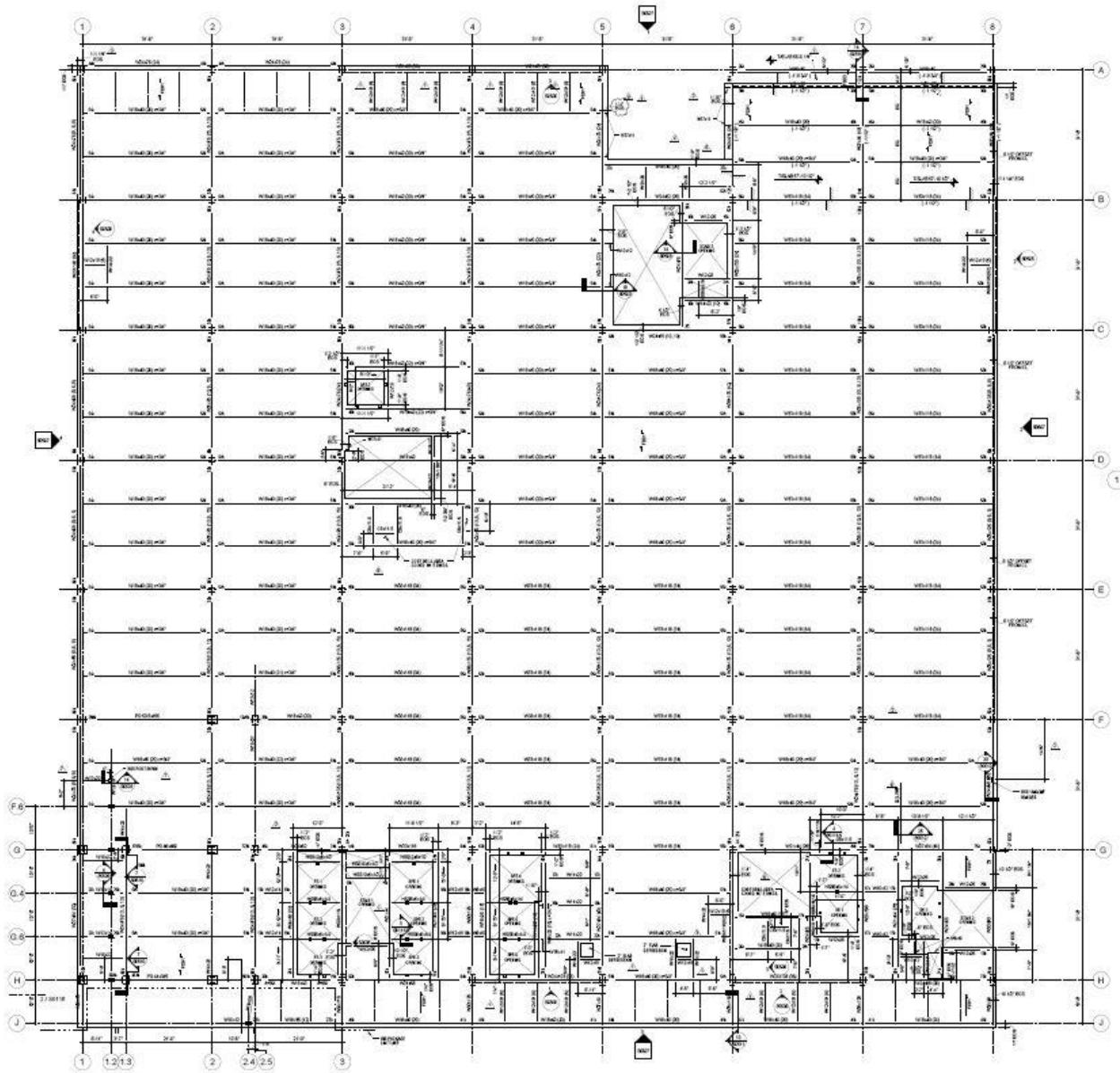


Figure H – Typical floor framing plan (Cannon Design)

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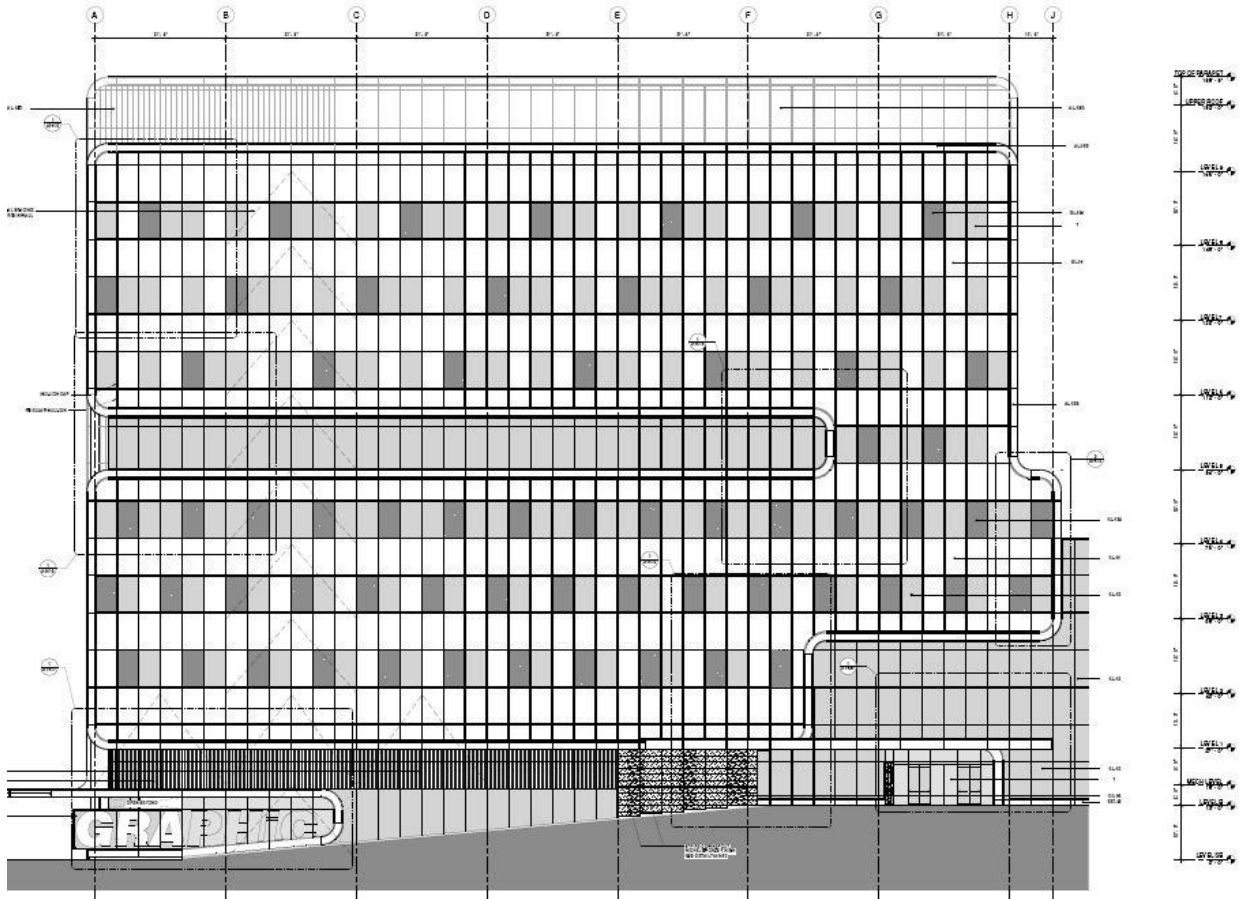


Figure I – West elevation (Cannon Design)

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Appendix B: Existing Floor System – Composite Steel

WILLIAM McDEVITT	TECH REPORT #2	COMPOSITE STEEL	①
<p><u>METAL DECKING</u></p> <p>CHECK MOST COMMON DECK TYPE MARK FDO1, ON LEVEL 4 TOTAL SLAB THICKNESS = 7 1/2" DECK TYPE = 3" GAGE = 18 $f'_c = 3000 \text{ psi}$, 145 pcf PLACE OVER MINIMUM OF THREE SPANS</p> <div style="display: flex; align-items: center;"> <div style="flex: 1;"> </div> <div style="flex: 1; padding-left: 20px;"> <p>LEVEL 4 IS PROCEDURE FLOOR LL = 125 psf SDL = 25 psf TOTAL LOAD = 150 psf</p> </div> </div> <p>USING VULCRAFT DECK CATALOG:</p> <p>CHECK MAXIMUM UNSHORED SPAN 7.5" SLAB 3 OR MORE SPANS } 3VLI18 → 13'-3" > 10'-6" ✓ ∴ OK</p> <p>CHECK SUPERIMPOSED LL 7.5" SLAB 10'-6" CLEAR SPAN → 275 psf >> 150 psf ✓ ∴ OK</p> <p style="text-align: right; margin-top: 20px;">THIS SEEMS EXCESSIVE BUT IN ORDER TO OBTAIN THE REQUIRED FIRE RATING A THICKER DECK IS NEEDED</p>			

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BEAM

CHECK BEAM ON LEVEL 4

W21x44 (22)
3/4" DIA. SHEAR STUDS

$A_t = 10.5 (31.5) = 330.75 \text{ ft}^2$

FLOOR DL = 75 psf + 25 psf = 100 psf = 0.100 ksf
 ↑ ↑
 DECK/SLAB SUPER

LL = 125 psf (SHALL NOT BE REDUCED > 100 psf, SUPPORTING ONE FLOOR - 4.7.3)
 = 0.125 ksf

TOTAL LOADS: BEAM SW
↓

TOTAL DL = 0.100 ksf (10.5') + 0.044 klf = 1.094 klf
 TOTAL LL = 0.125 ksf (10.5') = 1.313 klf

CONSTRUCTION LOADS:

CDL (UNSHORED) = (75 psf)(10.5') + 44 plf = 0.832 klf
 ↑ ↑
 DECK/SLAB BEAM SW

CLL (UNSHORED) = (20 psf)(10.5') = 0.210 klf

$w_u = 1.2 \text{ DL} + 1.6 \text{ LL} = 1.2(1.094 \text{ klf}) + 1.6(1.313 \text{ klf}) = 3.41 \text{ klf}$

$V_u = \frac{w_u l}{2} = \frac{3.41 \text{ klf} (31.5')}{2} = 53.7 \text{ k}$

$M_u = \frac{w_u l^2}{8} = \frac{3.41 \text{ klf} (31.5')^2}{8} = 422.9 \text{ 'k}$

$b_{\text{eff}} = \begin{cases} 2 \times \frac{\text{SPAN}}{8} = 2 \times \frac{(31.5' \times 12 \text{ in/ft})}{8} = 94.5 \text{ in} \\ 2 \times \frac{1}{2} \text{ SPACING} = 2 \left[\frac{1}{2} (10.5' \times 12 \text{ in/ft}) \right] = 126 \text{ in} \end{cases}$

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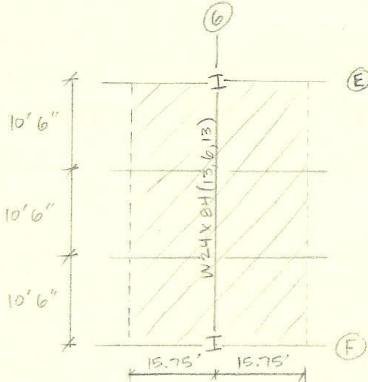
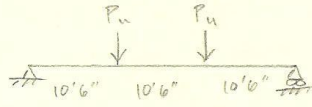
	WILLIAM McDEVITT	TECH REPORT #2	COMPOSITE STEEL 3
	<p>ASSUME $a \approx 1$</p> $y_2 = t_{SUB} - \frac{a}{2} = 7.5 - \frac{1}{2} = 7"$ <p>$G_n = 17.2$ FOR 4 KSI NWC W/ DECK PERPENDICULAR</p> <p>TRY W21x44 - $\phi M_n = 541'k$, $\phi_b M_p = 358'k$, PNA = 7 $\Sigma Q_n = 162$ $a = \frac{\Sigma Q_n}{0.85 F'_c b_{eff}} = \frac{162}{0.85(4)(94.5)} = 0.504 < 1" \checkmark$ $y_2 = 7.5 - \frac{0.504}{2} = 7.25" > 7" \leftarrow$ CONSERVATIVE</p> <p>SHEAR STUDS: $\frac{162}{17.2} = 9.42 \rightarrow 10$ STUDS/HALF $\rightarrow 20$ STUDS</p> <p>CHECK UNSTORED STRENGTH</p> $w_u = 0.210 \text{ klf}$ $w_{DL} = (75 \text{ psf})(10.5') + 44 \text{ plf} = 0.832 \text{ klf}$ $w_u = 1.2(0.832) + 1.6(0.210) = 1.334 \text{ klf}$ $M_u = \frac{w_u l^2}{8} = \frac{1.334 \text{ klf} (31.5')^2}{8} = 165.5'k < 358'k = \phi_b M_p \checkmark \therefore \text{OK}$ <p>CHECK MEMBER STRENGTH</p> $\phi M_n = 541'k > 422.9'k = M_u \checkmark \therefore \text{OK}$ $\phi V_n = 217 \text{ k} > 53.7 \text{ k} = V_u \checkmark \therefore \text{OK}$ <p>CHECK LL DEFLECTION:</p> $w_{LL} = 125 \text{ psf} (10.5') = 1.313 \text{ klf}$ $I_{LB} = 1620 \text{ in}^4$ $\Delta_{LL} = \frac{5 w_{LL} l^4}{384 EI} = \frac{5(1.313)(31.5')^4 (1728)}{384(29000)(1620)} = 0.619 \text{ in}$ $\frac{l}{360} = \frac{31.5' (12 \text{ in/ft})}{360} = 1.05 \text{ in} > 0.619 \text{ in} \checkmark \therefore \text{OK}$ <p>CHECK WET CONCRETE DEFLECTION</p> $w_{WC} = 75 \text{ psf} (10.5') + 44 \text{ plf} = 0.832 \text{ klf} \quad I_x = 843 \text{ in}^4$ $A_{WC} = \frac{5 w_{WC} l^4}{384 EI} = \frac{5(0.832)(31.5')^4 (1728)}{384(29000)(843)} = 0.754 \text{ in}$ $A_{WCmax} = \frac{l}{240} = \frac{31.5' (12 \text{ in/ft})}{240} = 1.575" > 0.754" \checkmark \therefore \text{OK}$ <p>USE W21x44 [20]</p>		

Technical Report #2

WILLIAM McDEVITT	TECH REPORT #2	COMPOSITE STEEL	4
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GIRDER

CHECK GIRDER ON LEVEL 4

TOTAL LOADS:

TOTAL DL = 1.094 klf (2 · 15.75') = 34.5 K
 TOTAL LL = 1.313 klf (2 · 15.75') = 41.4 K

CONSTRUCTION LOADS

CDL (UNSHORED) = 0.832 klf (2 · 15.75') = 26.2 K + BEAM SW
 CLL (UNSHORED) = 0.210 klf (2 · 15.75') = 6.6 K

$P_u = 1.2DL + 1.6LL = 1.2(34.5) + 1.6(41.4) = 107.64 K$
 $V_u = P_u = 107.64 K$
 $M_u = 10.5' (107.64 K) = 1130.2 'K$

$b_{eff} = \begin{cases} 2 \times \frac{SPAN}{8} = 2 \times \frac{(31.5' \times 12 \text{ in/ft})}{8} = 94.5 \text{ in} \\ 2 \times \frac{1}{2} SPACING = 2 \left[\frac{1}{2} (31.5' \times 12 \text{ in/ft}) \right] = 378 \text{ in} \end{cases}$ (min)

ASSUME $a \approx 1$

$Y_2 = t_{SLAB} - \frac{a}{2} = 7.5 - \frac{1}{2} = 7''$
 $Q_n = 21.2$ FOR 4 ksi NWC W/ DECK PARALLEL

TRY W24x84 - $\phi M_n = 1210 'K$, $\phi_b M_p = 840 'K$, PNA = 7
 $\Sigma Q_n = 309$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{309}{0.85(4)(94.5)} = 0.962 < 1'' \checkmark$

$Y_2 = 7.5 - \frac{0.962}{2} = 7.02 \text{ in} > 7 \text{ in} \leftarrow \text{CONSERVATIVE}$

SHEAR STUDS: $\frac{309}{21.2} = 14.58 \rightarrow 15 \text{ STUDS/HALF} \rightarrow 30 \text{ STUDS}$

Technical Report #2

WILLIAM McDEVITT	TECH REPORT #2	COMPOSITE STEEL	5
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CHECK UNSHORED STRENGTH ← BEAM SW

$$w_u = 1.2DL + 1.6LL = 1.2(0.084 \text{ klf}) = 0.101 \text{ klf}$$

$$P_u = 1.2DL + 1.6LL = 1.2(26.2 \text{ k}) + 1.6(6.6 \text{ k}) = 42.0 \text{ k}$$

$$M_u = \frac{w_u l^2}{8} + \frac{P_u l}{3} = \frac{0.101(31.5)^2}{8} + \frac{42.0(31.5)}{3} = 453.5' \text{ k} < 840' \text{ k} = \phi_b M_p \quad \checkmark \therefore \text{OK}$$

CHECK MEMBER STRENGTH

$$\phi M_n = 1210' \text{ k} > 1130.2' \text{ k} = M_u \quad \checkmark \therefore \text{OK}$$

$$\phi V_n = 340 \text{ k} > 107.6 \text{ k} = V_u \quad \checkmark \therefore \text{OK}$$

CHECK LL DEFLECTION

$$P_{LL} = 41.4 \text{ k} \quad I_{LB} = 2940 \text{ in}^4$$

$$\Delta_{LL} = \frac{P_{LL} l^3}{28EI} = \frac{41.4(31.5)^3(1728)}{28(29000)(2940)} = 0.937 \text{ in}$$

$$\Delta_{LL \text{ MAX}} = \frac{l}{360} = \frac{31.5'(12 \text{ in/ft})}{360} = 1.05 \text{ in} > 0.937 \text{ in} \quad \checkmark \therefore \text{OK}$$

CHECK WET CONCRETE DEFLECTION

$$P_{wc} = [75 \text{ psf}(10.5') + 44 \text{ plf}](2 \cdot 15.75') = 52.4 \text{ k} \quad I_x = 2370 \text{ in}^4$$

← BEAM

$$w_{wc} = 75 \text{ psf}(9.02''/12 \text{ in/ft}) + 84 \text{ plf} = 0.140 \text{ klf}$$

← b_f OF GIRDER

$$\Delta_{wc} = \frac{S_w l^4}{384 EI} + \frac{P l^3}{28 EI} = \frac{5(0.140)(31.5)^4(1728)}{384(29000)(2370)} + \frac{52.4(31.5)^3(1728)}{28(29000)(2370)}$$

$$\Delta_{wc} = 0.045 + 1.47 = 1.515$$

$$\Delta_{wc \text{ MAX}} = \frac{l}{240} = \frac{31.5'(12 \text{ in/ft})}{240} = 1.575 \text{ in} > 1.515 \text{ in} \quad \checkmark \therefore \text{OK}$$

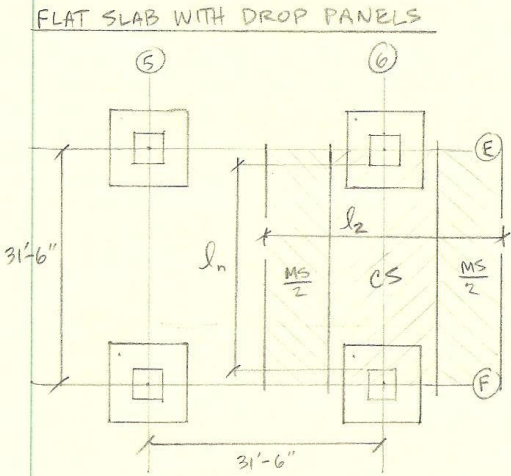
USE W24 X 84 [30]

Technical Report #2

	WILLIAM McDEVITT	TECH REPORT #2	COMPOSITE STEEL (b)
	<p>COST ANALYSIS USE RS MEANS 2011, p 93</p> <p>FOR 30' x 30' BAY, W SHAPE, COMPOSITE DECK AND SLAB</p> $\$28.85/\text{ft}^2 (101.6)/100 = \$29.31/\text{ft}^2$ <p>↖ BUFFALO, NY</p>		

Technical Report #2

Appendix C: Alternative Floor System 1 – Flat Slab with Drop Panels

	WILLIAM McDEVITT	TECH REPORT #2	FLAT SLAB WITH DROP PANELS ①
	<p>UTILIZE 31'-6" x 31'-6" BAYS</p> <p>ASSUME: 24" x 24" COLUMNS</p> <p>BECAUSE THIS IS A SQUARE BAY, BOTH DIRECTIONS WILL BE THE SAME</p>		
<p>DETERMINE SLAB THICKNESS</p> <p>- SLAB WITHOUT INTERIOR BEAMS, WITH DROP PANELS</p> <p>ACI 9.5.3.2:</p> $t = \max \left[\begin{array}{l} 4 \text{ in} \\ \frac{l_n}{36} = \frac{29.5' (12 \text{ in/ft})}{36} = 9.8 \text{ in} \end{array} \right] \rightarrow \text{USE } 10" \text{ SLAB}$ <p style="text-align: right;">$l_n = 31.5' - 2(\frac{3}{2}) = 29.5'$</p>			
<p>METHOD FOR COMPUTING DESIGN MOMENTS</p> <p>DIRECT DESIGN METHOD CAN BE USED:</p> <ul style="list-style-type: none"> - MORE THAN 3 CONTINUOUS SPANS IN EACH DIRECTION ✓ - $\frac{\text{LONG SPAN}}{\text{SHORT SPAN}} = 1 < 2$ ✓ - SUCCESSIVE SPAN LENGTHS SHALL NOT DIFFER MORE THAN $\frac{1}{3}$ LONGER SPAN ✓ - COLUMN OFFSETS ✓ - GRAVITY ONLY LOADS + UNIFORMLY DISTRIBUTED ✓ - UNFACTORED LL $\leq 2 \cdot$ UNFACTORED DL <p style="margin-left: 40px;"> $LL = 125 \text{ psf}$ $SDL = 25 \text{ psf}$ $SLAB = \frac{10"}{12 \text{ in/ft}} (150 \text{ pcf}) = 125 \text{ psf}$ </p> <p style="text-align: right;">$125 \leq 2(25 + 125)$ ✓</p>			
<p>COMPUTE POSITIVE + NEGATIVE MOMENTS IN SLAB</p> $M_o = \frac{q_u l_x l_n^2}{8}$ $q_u = 1.2 \left[\frac{10"}{12 \text{ in/ft}} (150 \text{ pcf}) + 25 \text{ psf} \right] + 1.6 (125 \text{ psf}) = 0.380 \text{ ksf}$			

Technical Report #2

WILLIAM McDEVITT	TECH REPORT #2	FLAT SLAB WITH DROP PANELS ②
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$$l_n = 31.5' - 2\left(\frac{2}{2}\right) = 29.5'$$

$$l_2 = 2\left(\frac{31.5'}{2}\right) = 31.5' \quad l_1 = 31.5'$$

$$M_o = \frac{0.380(31.5)(29.5)^2}{8} = 1302' \text{ k}$$

NEGATIVE MOMENT = $-0.65 M_o = -0.65(1302) = -846' \text{ k}$

POSITIVE MOMENT = $0.35 M_o = 0.35(1302) = 456' \text{ k}$

DISTRIBUTE TO COLUMN STRIP + MIDDLE STRIP

$\alpha_{F1} = 0$ (NO BEAMS)

$$\frac{l_2}{l_1} = 1$$

NEGATIVE MOMENTS:

COLUMN STRIP NEG = $0.75(-846) = -634.5' \text{ k} / 15.76' = -40.3' \text{ k/ft}$
↑ ACI 13.6.4.1

MIDDLE STRIP NEG = $(1 - 0.75)(-846) = -211.5' \text{ k} / 15.74' = -13.4' \text{ k/ft}$

POSITIVE MOMENTS:

COLUMN STRIP POS = $0.60(456) = 273.6' \text{ k} / 15.76' = 17.4' \text{ k/ft}$
↑ ACI 13.6.4.4

MIDDLE STRIP POS = $(1 - 0.60)(456) = 182.4' \text{ k} / 15.74' = 11.6' \text{ k/ft}$

COLUMN STRIP SIZE:

$$l_{min}/4 = \begin{matrix} \frac{l_1}{4} = \frac{31.5}{4} = 7.88' \\ \frac{l_2}{4} = \frac{31.5}{4} = 7.88' \end{matrix}$$

CS
MS
CS
15.76'
15.74'
15.76'

NEG

-40.3' k/ft -13.4' k/ft -40.3' k/ft

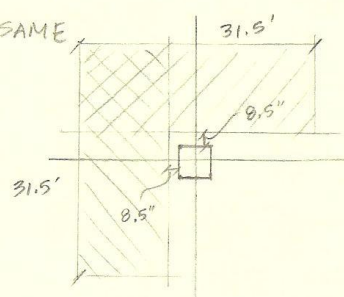
POS

17.4' k/ft 11.6' k/ft 17.4' k/ft

Technical Report #2

	WILLIAM McDEVITT	TECH REPORT #2	FLAT SLAB WITH DROP PANELS (3)
	<p>DESIGN SLAB REINFORCING BECAUSE OF THE SQUARE BAY, THE REINFORCEMENT IN BOTH DIRECTIONS WILL BE THE SAME</p> <p>$d_{LONG\ SPAN} = 10'' - 1.15 = 8.85''$</p> <p>$d_{SHORT\ SPAN} = 10'' - 1.9 = 8.1'' \leftarrow$ THIS IS CONSERVATIVE FOR BOTH DIRECTIONS</p> <p>ASSUME $jd = 0.95d = 0.95(8.1) = 7.70$ (USE $jd = 0.85d = 6.87$ WHEN NEEDED)</p> <p>- COLUMN STRIP POSITIVE MOMENT = 17.4'K/FT</p> $A_{sreq} = \frac{M_u}{\phi f_y j d} = \frac{17.4(12^2/4)}{0.9(60)(7.7)} = 0.502 \text{ in}^2/\text{FT}$ $a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.502(60)}{0.85(4)(12)} = 0.74 \text{ in}$ $c = \frac{a}{\beta_1} = \frac{0.74}{0.85} = 0.87 \text{ in} \leq 0.375 d_t = 0.375(8.1) = 3.04 \text{ in} \checkmark \therefore \phi = 0.9$ $jd = d - \frac{a}{2} = 8.1 - \frac{0.74}{2} = 7.73 > 7.70 \therefore \text{CONSERVATIVE}$ $A_{smin} = 0.0018bh = 0.0018(12'')(10'') = 0.216 \text{ in}^2 < 0.502 \text{ in}^2 \checkmark \therefore \text{OK}$ $S_{MAX} = \begin{matrix} Zh = 2(10) = 20'' \\ \min(18'') \end{matrix} \rightarrow \text{USE } 12'' \text{ SPACING SINCE DESIGNING A } 12'' \text{ STRIP}$ <p>USE #7 @ 12" WITH $A_s = 0.60 \text{ in}^2 > 0.502 \text{ in}^2$</p> <p>- COLUMN STRIP NEGATIVE MOMENT = 40.3'K/FT</p> $A_{sreq} = \frac{40.3(12)}{0.9(60)(6.87)} = 1.30 \text{ in}^2/\text{FT} > 0.216 \text{ in}^2 \checkmark \therefore \text{OK}$ <p style="text-align: center;">\uparrow A_{smin}</p> $a = \frac{1.30(60)}{0.85(4)(12)} = 1.91 \text{ in}$ $c = \frac{a}{\beta_1} = \frac{1.91}{0.85} = 2.25 \text{ in} \leq 0.375 d_t = 0.375(8.1) = 3.04 \text{ in} \checkmark \therefore \phi = 0.9$ $jd = 8.1 - \frac{1.91}{2} = 7.15 > 6.87 \therefore \text{CONSERVATIVE}$ $A_{req-4} = 1.30 \text{ in}^2/\text{ft} \left(\frac{1}{3} \right) = 0.43 \text{ in}^2/4 \text{ in} \rightarrow \text{USE } \#6 @ 4''$ <p>USE #6 @ 4" WITH $A_s = 0.44 \text{ in}^2 > 0.43 \text{ in}^2$</p>		

Technical Report #2

	WILLIAM McDEVITT	TECH REPORT #2	FLAT SLAB WITH DROP PANELS (4)
	<p>- MIDDLE STRIP POSITIVE MOMENT = 11.6' k/ft</p> $A_{s, req} = \frac{11.6(12)}{0.9(60)(7.7)} = 0.33 \text{ in}^2 > 0.216 \text{ in}^2 \checkmark \therefore \text{OK}$ <p style="text-align: center;">$\uparrow A_{s, min}$</p> $a = \frac{0.33(60)}{0.85(4)(12)} = 0.49 \text{ in}$ $c = \frac{a}{\beta_1} = \frac{0.49}{0.85} = 0.58 \text{ in} < 3.04 \text{ in} \checkmark \therefore \phi = 0.9$ $jd = 8.1 - \frac{0.49}{2} = 7.86 > 7.70 \therefore \text{CONSERVATIVE}$ <p>USE #6 @ 12" WITH $A_s = 0.44 \text{ in}^2 > 0.33 \text{ in}^2$</p> <p>- MIDDLE STRIP NEGATIVE MOMENT = 13.4' k/ft</p> $A_{s, req} = \frac{13.4(12)}{0.9(60)(7.7)} = 0.39 \text{ in}^2 > 0.216 \text{ in}^2 \checkmark \therefore \text{OK}$ <p style="text-align: center;">$\uparrow A_{s, min}$</p> $a = \frac{0.39(60)}{0.85(4)(12)} = 0.57 \text{ in}$ $c = \frac{a}{\beta_1} = \frac{0.57}{0.85} = 0.67 \text{ in} < 3.04 \text{ in} \checkmark \therefore \phi = 0.9$ $jd = 8.1 - \frac{0.57}{2} = 7.82 > 7.70 \therefore \text{CONSERVATIVE}$ <p>USE #6 @ 12" WITH $A_s = 0.44 \text{ in}^2 > 0.39 \text{ in}^2$</p> <p>DESIGN FOR SHEAR - BOTH DIRECTIONS THE SAME</p> <p>ONE-WAY SHEAR:</p> $d = 10 - \frac{3}{4} - 0.75 = 8.5''$ <p style="text-align: center; margin-left: 20px;">COVER d_b</p> $V_u = 0.380 \text{ ksf} \left(\frac{31.5'}{2} - \frac{24''}{12 \text{ in/ft}} + 8.5'' \right) (31.5')$ $V_u = 168.1 \text{ k}$ $\phi V_n = \phi V_c$ $\phi V_c = 0.75(2\lambda\sqrt{f'_c}bd)$ $= 0.75(2)(1.0)\sqrt{4000} (31.5')(12 \text{ in/ft})(8.5) = 305 \text{ k} > 168.1 \text{ k} \checkmark \therefore \text{OK}$ <div style="text-align: right;">  </div>		

Technical Report #2

WILLIAM McDEVITT	TECH REPORT #2	FLAT SLAB WITH DROP PANELS (5)
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TWO-WAY PUNCHING SHEAR:

$$V_u = 0.380 \text{ ksf} \left[(31.5') \times (31.5') - \left(\frac{24 + 2(8.5/2)}{12 \text{ in/ft}} \right) \left(\frac{24 + 2(8.5/2)}{12 \text{ in/ft}} \right) \right] = 374 \text{ k}$$

$$b_o = 4(24 + 2(8.5/2)) = 130' \text{ in}$$

$$V_c = \begin{cases} 4 \lambda \sqrt{f'_c} b_o d = 4(1.0) \sqrt{4000} (130)(8.5) = 279.5 \text{ k} \\ \left(2 + \frac{4}{B} \right) \lambda \sqrt{f'_c} b_o d = \left(2 + \frac{4}{1} \right) (1.0) \sqrt{4000} (130)(8.5) = 419.3 \text{ k} \\ \min \left[\frac{X_s d}{b_o} + 2 \right] \lambda \sqrt{f'_c} b_o d = \left[\frac{40(8.5)}{130} + 2 \right] (1.0) \sqrt{4000} (130)(8.5) = 322.6 \text{ k} \end{cases}$$

$\beta = \frac{24}{24} = 1$, $\alpha_s = 40$ FOR INTERIOR COLUMN

$\phi V_c = 0.75(279.5) = 209.6 \text{ k} < 374 \text{ k} = V_u$ X NO GOOD - ADD DROP PANELS

$h_d = \frac{10}{4} = 2.5 \rightarrow$ USE 6.25" FOR EASE OF CONSTRUCTION

$\frac{l_1}{6} = \frac{l_2}{6} = \frac{31.5'}{6} = 5.25' \rightarrow$ USE 6'

$10" + 6.25" = 16.25"$

$d = 16.25 - 3/4 - 0.75 = 14.75"$


$b_o = 4(24 + 2(\frac{14.75}{2})) = 155"$

NEW $V_u = 373 \text{ k}$

$$V_c = \begin{cases} 4(1.0) \sqrt{4000} (155)(14.75) = 578.4 \text{ k} \\ \left(2 + \frac{4}{1} \right) (1.0) \sqrt{4000} (155)(14.75) \\ \min \left[\frac{40(14.75)}{147} + 2 \right] (1.0) \sqrt{4000} (155)(14.75) \end{cases}$$

$\phi V_c = 0.75(578.4) = 433.8 \text{ k} > 373 \text{ k} = V_u$ ✓ \therefore OK - USE DROP PANELS

Technical Report #2

	WILLIAM McDEVITT	TECH REPORT #2	FLATSLAB WITH DROP PANELS	⑥
	<p>COST ANALYSIS USE RS MEANS 2011, pg 63</p> <p>FOR 30'x30' BAY, CAST IN PLACE FLAT SLAB WITH DROP PANELS</p> $\$17.75/\text{ft}^2 (101.6)/100 = \$18.03/\text{ft}^2$ <p>↑ BUFFALO, NY</p>			

Technical Report #2

Appendix D: Alternative Floor System 2 – Pre-Cast Hollow Core Plank

WILLIAM McDEVITT	TECH REPORT #2	PRE-CAST HOLLOWCORE	①
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PRE-CAST HOLLOW CORE PLANK

HOLLOW CORE PLANK COMES IN 4' SECTIONS, SO BAY SIZE OF 31'-6" x 31'-6" WOULD HAVE TO BE ADJUSTED TO 32' x 32' TO ACCOMMODATE PLANKS

LEVEL 4 IS PROCEDURE FLOOR
LL = 125 psf
SDL = 25 psf

SUPERIMPOSED SERVICE LOAD:
125 + 25 = 150 psf

2 HR. FIRE RATING

FROM PCI DESIGN HANDBOOK:
USE 4 HC 10 + 2 → 4'-0" x 10" WITH 2" TUPPING + 32' SPAN
6 - 9/16" φ STRANDS (STRAIGHT)

SAFE SUPERIMPOSED SERVICE LOAD = 153 psf > 150 psf ✓ ∴ OK

DESIGN BIRDER:
LL = 125 psf
SDL = 25 psf
DL OF 4'-0" x 10" PLANK = 93 psf

TOTAL LOADS ← TRIB AREA

TOTAL DL = (25 + 93)(32') = 3.8 klf
TOTAL LL = 125(32') = 4 klf

$W_u = 1.2(3.8) + 1.6(4) = 10.96 \text{ klf}$

$V_u = \frac{10.96(32)}{2} = 175.4 \text{ k}$

$M_u = \frac{10.96(32)^2}{8} = 1402.9 \text{ 'k}$

LL DEFLECTION

$\Delta_{LL \text{ MAX}} = \frac{l}{360} = \frac{32'(12 \text{ in/ft})}{360} = 1.07''$

$W_{LL} = 125(32) = 4 \text{ klf}$

$\Delta_{LL} = \frac{5 W_{LL} l^4}{384 EI} = \frac{5(4)(32)^4(1728)}{384(29000) I_{min}} = 1.07''$

$I_{min} = 3041 \text{ in}^4$

Technical Report #2

	WILLIAM McDEVITT	TECH REPORT #2	PRE-CAST HOLLOWCORE	②
		<p>TOTAL LOAD DEFLECTION</p> $\Delta_{TLMAX} = \frac{\delta}{240} = \frac{32'(12 \text{ in/ft})}{240} = 1.6''$ $W_D = (125 + 25 + 93)(32') = 7.8 \text{ klf}$ $\Delta_{TL} = \frac{5 W_D L^4}{384 EI} = \frac{5(7.8)(32')^4(1728)}{384(29000) I_{min}} = 1.6$ $I_{min} = 3966 \text{ in}^4 \leftarrow \text{CONTROLS}$ <p>TRY W30x116 (TABLE 3-3)</p> $\phi M_n = 1420 \text{ k} > 1402.9 \text{ k} = M_u \checkmark \therefore \text{OK}$ $I = 4930 \text{ in}^4 > 3966 \text{ in}^4 = I_{min} \checkmark \therefore \text{OK}$ <p>USE W30x116 GIRDERS WITH 4'-0" x 10" PRE-CAST HOLLOW CORE PLANKS (2" TOPPING, 68-S)</p> <p>COST ANALYSIS USE RS MEANS 2011, pg 70</p> <p>FOR 30' SPAN, PRECAST PLANK WITH 2" CONCRETE TOPPING \$13.48/ft²</p> <p>FOR 30' x 30' BAY, BEAM + GIRDER p 80 \$23.70/ft² ← ASSUME GIRDER IS HALF</p> <p>TOTAL: $(\\$13.48 + \frac{23.70}{2})(101.6)/100 = \\$25.74/\text{ft}^2$ ↑ BUFFALO, NY</p>		

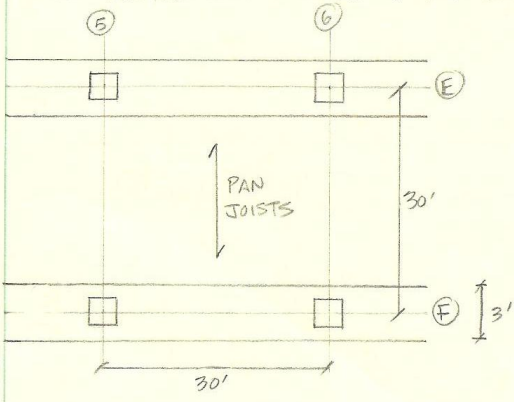
Technical Report #2

Appendix E: Alternative Floor System 3 – One-Way Joist and Beam

WILLIAM McDEVITT	TECH REPORT #2	ONE-WAY JOIST + BEAM	①
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ONE-WAY JOIST AND BEAM

THIS DESIGN WILL UTILIZE 6' PAN JOIST MODULES AND 30' x 30' BAYS



$f'_c = 4000 \text{ psi}$
 $f_y = 60 \text{ ksi}$

ASSUME: 3' GIRDER
24" x 24" COLUMNS

SLAB

USE 4.5" SLAB FOR 2 HR FIRE RATING
 SDL = 25 psf
 LL = 125 psf
 $SW = \left(\frac{4.5}{12} \times 150\right) \text{ pcf} = 56.3 \text{ psf}$

$W_u = 1.2(25 + 56.3) + 1.6(125) = 297.6 \text{ psf}$
 $W_u = 298 \text{ psf (1')} = 298 \text{ plf} \leftarrow \text{DESIGN 1' WIDE STRIP}$

$M_u^- = \frac{W_u L_n^2}{11} = \frac{0.298 \left(\frac{66}{12} \text{ ft}\right)^2}{11} = 0.820 \text{ k/ft OF SLAB WIDTH (ACI 8.3.3)}$
 $M_u^+ = \frac{W_u L_n^2}{16} = \frac{0.298 \left(\frac{66}{12} \text{ ft}\right)^2}{16} = 0.563 \text{ k/ft OF SLAB WIDTH}$

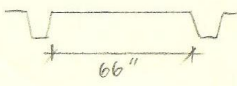
MIN. REINFORCING REQUIRED = $0.0018 A_g$ (ACI 10.5.4)
 $A_{s, req} = 0.0018 (4.5)(12) = 0.0972 \text{ in}^2$

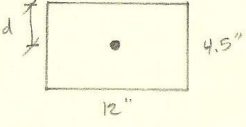
MAX SPACING = $\frac{3t}{5t (S+T)} \rightarrow 3(4.5) = 13.5''$

TRY 1-#3 $A_s = 0.11 \text{ in}^2 > 0.0972 \text{ in}^2 \checkmark : \text{OK}$
 USE 1-#3 @ 12"

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

$\phi M_u = \phi A_s f_y (d - \frac{\alpha}{2}) = 0.9(0.11)(60)(2.25 - \frac{0.162}{2}) = 12.9 \text{ in-k} = 1.07 \text{ k/ft OF SLAB WIDTH}$
 $\phi M_u = 1.07 \text{ k/ft OF SLAB WIDTH} > 0.820 \text{ k/ft OF SLAB WIDTH} \checkmark : \text{OK}$





ASSUME $d = 2.25''$

Technical Report #2

WILLIAM McDEVITT	TECH REPORT #2	ONE-WAY JOIST & BEAM ②
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JOIST

SDL = 25 psf (6') = 150 plf

SLAB = $\frac{4.5''}{12''} (150 \text{ psf}) = 56.3 \text{ psf (6')} = 337.8 \text{ plf}$

SW = $\frac{16''(6'')(150 \text{ psf})}{144} = 100 \text{ plf}$

LL = 125 psf (6') = 750 plf

$W_u = 1.2(150 + 337.8 + 100) + 1.6(750) = 1.9 \text{ klf}$

FOR INTERIOR SPAN:

$M_{u \text{ max}}^- = \frac{W_u L_n^2}{11} = \frac{1.9 (27')^2}{11} = 125.9 \text{ 'k}$

$M_{u \text{ max}}^+ = \frac{W_u L_n^2}{16} = \frac{1.9 (27')^2}{16} = 86.6 \text{ 'k}$

TOP REINFORCEMENT

$A_s = \frac{M_u}{\phi d} = \frac{125.9}{4(18.25)} = 1.72 \text{ in}^2$ $d = 20.5 - 2.25$
 $d = 18.25''$

TRY 3-#7 $A_s = 3(0.60) = 1.80 \text{ in}^2 > 1.72 \text{ in}^2 \checkmark$

$\rho = \frac{A_s}{bd} = \frac{1.80}{(6)(16)} = 0.019 \rightarrow \text{CHECK IF TENSION CONTROLS}$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.80(60)}{0.85(4)(6)} = 5.29 \text{ in}$

$c = \frac{a}{\beta_1} = \frac{5.29}{0.85} = 6.2 \checkmark \leq 0.375 d_t = 0.375(18.25) = 6.8 \text{ in} \checkmark \therefore \phi = 0.9$

$\phi M_{u \text{ in}} = 0.9 A_s f_y (d - \frac{a}{2})$
 $= 0.9(1.80)(60)(18.25 - \frac{5.29}{2}) = 1517 \text{ in-k} = 126.4 \text{ 'k} > 125.9 \text{ 'k} \checkmark \therefore \text{OK}$

USE 3-#7

BOTTOM REINFORCEMENT

TRY 1-#10

$d = 20.5'' - 1.5'' - 0.375'' - \frac{1.27''}{2} = 18.0 \text{ in}$
CLEAR COVER #3 STIRRUP #10 BAR

$A_s = \frac{M_u}{\phi d} = \frac{86.6}{4(18.0)} = 1.20 \text{ in}^2 \rightarrow 1\text{-}\#10 \quad A_s = 1.27 \text{ in}^2$

$\rho = \frac{A_s}{bd} = \frac{1.27 \text{ in}^2}{6''(18.0'')} = 0.012 \rightarrow \text{CHECK IF TENSION CONTROLS}$

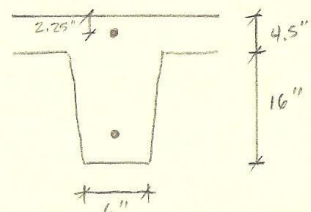
MIN. JOIST THICKNESS
(ACI TABLE 9.5a)

$\frac{l_n}{21} = \frac{27'(12 \text{ in/ft})}{21} = 15.4''$

$l_n = 30' - 2(\frac{3'}{2}) = 27'$

$\therefore \text{USE } 16'' \text{ DEEP JOISTS}$

- INTERIOR SPAN
- APPROXIMATE MOMENTS
 \therefore NO REDISTRIBUTION
(ACI 8.4.1)



Technical Report #2

	WILLIAM McDEVITT	TECH REPORT #2	ONE-WAY JOIST + BEAM	③
		$a = \frac{A_s f_y}{0.85 f_c b} = \frac{1.27(60)}{0.85(4)(6)} = 3.74 \text{ in}$ $c = \frac{a}{\beta_1} = \frac{3.74}{0.85} = 4.4 \text{ in} \leq 0.375 d_t = 0.375(18.0) = 6.75 \text{ in} \checkmark \therefore \phi = 0.9$ $\phi M_n = \phi A_s f_y (d - a/2)$ $= 0.9(1.27)(60) \left(18.0 - \frac{3.74}{2} \right) = 1106 \text{ in-k} = 92.2 \text{ k} > 86.6 \text{ k} \checkmark \text{ OK}$ <p style="text-align: right;">USE 1-#10</p>		
		<p>SHEAR DESIGN</p> $V_u = \frac{w_u L_n}{2} = \frac{1.9(27')}{2} = 25.7 \text{ k}$ <p style="text-align: right;">- INTERIOR SPAN</p> $\phi V_c = \phi 2 \sqrt{f_c} b w d$ $= 0.75(2)(\sqrt{4000})(6)(17.9)$ $= 10.2 \text{ k}$ $\phi V_s = V_u - \phi V_c = 25.7 - 10.2 = 15.5 \text{ k}$ $\phi V_s = \frac{\phi A_s v f_y d}{S_{max}} = 15.5$ $S_{max} = \frac{d}{2} = \frac{17.9}{2} = 9.0$ $A_s v = \frac{\phi V_s S_{max}}{\phi f_y d} = \frac{15.5(9.0)}{0.75(60)(17.9)} = 0.173 \text{ in}^2 < 0.20 \text{ in}^2 \checkmark \#4 \text{ OK}$ $\phi V_c + \phi V_s = 10.2 + \frac{0.75(0.20)(60)(17.9)}{9} = 28.1 \text{ k} > 25.7 \text{ k} = V_u \checkmark \therefore \text{OK}$ <p style="text-align: right;">USE 1-#4 @ 9" SPACING</p>		
		<p>BIRDER DESIGN</p> $W_{u \text{ FLOOR}} = \frac{1.9 \text{ klf}}{6'} (27') = 8.55 \text{ klf}$ $W_{u \text{ AT GIRDER}} = 1.2 \left[\frac{25 \text{ psf}(3')}{\text{SDL}} \right] + 1.6 \left[\frac{125 \text{ psf}(3')}{\text{LL}} \right] = 0.69 \text{ klf}$ $W_{u \text{ SW}} = 1.2 \left[\frac{36" (20.5')(150 \text{ psf})}{144} \right] = 0.92 \text{ klf}$ $W_u = 8.55 + 0.69 + 0.92 = 10.16 \text{ klf}$ <p style="text-align: right;">$l_n = 30' - 2 \left(\frac{2'}{2} \right) = 28'$</p>		
		<p>FOR INTERIOR SPAN</p> $M_{u \text{ MAX}}^- = \frac{W_u L_n^2}{11} = \frac{10.16(28)^2}{11} = 724 \text{ k}$ $M_{u \text{ MAX}}^+ = \frac{W_u L_n^2}{16} = \frac{10.16(28)^2}{16} = 498 \text{ k}$		

Technical Report #2

WILLIAM McDEVITT	TECH REPORT #2	ONE-WAY JOIST + BEAM	④
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GIRDER TOP REINFORCEMENT

$$A_s = \frac{M_u}{\phi d} = \frac{724}{4(18.0)} = 10.1 \text{ in}^2$$

$$d = 20.5 - 1.5 - 0.5 - \frac{1.00}{2} = 18.0''$$

COVER #4 STIRRUP #9 BAR

TRY 11-#9 BARS $A_s = 11.0 \text{ in}^2 > 10.1 \text{ in}^2 \checkmark \text{ OK}$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{11.0(60)}{0.85(4)(36'')} = 5.39 \text{ in}$$

$$c = \frac{a}{\beta_1} = \frac{5.39}{0.85} = 6.34 \checkmark \leq 0.375 d_t = 0.375(18.0) = 6.8 \checkmark \therefore \phi = 0.9$$

$$\phi M_n = \phi A_s f_y (d - \frac{a}{2})$$

$$= 0.9(11.0)(60)(18.0 - \frac{5.39}{2}) = 9091 \text{ in-k} = 758 \text{ k} > 724 \text{ k} \checkmark \therefore \text{OK}$$

USE 11-#9

GIRDER BOTTOM REINFORCEMENT

$$A_s = \frac{498}{4(18.0)} = 6.92 \text{ in}^2$$

TRY 7-#9 BARS $A_s = 7.0 \text{ in}^2 > 6.92 \text{ in}^2 \checkmark \text{ OK}$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{7.0(60)}{0.85(4)(36'')} = 3.43 \text{ in}$$

$$c = \frac{a}{\beta_1} = \frac{3.43}{0.85} = 4.04 \checkmark \leq 0.375 d_t = 0.375(18.0) = 6.8'' \checkmark \therefore \phi = 0.9$$

$$\phi M_n = \phi A_s f_y (d - \frac{a}{2})$$

$$= 0.9(7.0)(60)(18.0 - \frac{3.43}{2}) = 6156 \text{ in-k} = 513 \text{ k} > 498 \text{ k} \checkmark \therefore \text{OK}$$

USE 7-#9

COST ANALYSIS
USE RS MEANS 2011, p66

FOR 30' X 30' BAY, CAST IN PLACE MULTISPAN JOIST SLAB

$$\$19.65/\text{ft}^2 (101.6)/100 = \$19.96/\text{ft}^2$$

↑ BUFFALO, NY