SUNY

Upstate Cancer Center

Syracuse, New York

Technical Report 2

Michael Kostick | Structural Option

Evaluation of Alternative Floor Systems

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Table of Contents

Executive Summary

The intent of this report is to investigate three proposed alternative floor systems that could possibly be implemented to the SUNY Upstate Cancer Center in Syracuse, New York. Originally the building was designed using a composite system, made up of composite steel deck atop composite beams and girders. Three alternative systems, pre-cast hollow core concrete planks, two-way flat slab with drop panels, and one-way pan joists and beams were designed to the constraints of a typical interior bay between column lines K'-L' and 3'-4', located in the Central Tower. Each of the four systems, including the existing composite system, was compared against each other on the basis of system cost, weight, and overall depth. In addition, each system's effect on construction, the existing building's architecture and structural system, and serviceability was also accounted for. After summarizing advantages and disadvantages of each system, one assembly was chosen as the most feasible alternative to the existing floor system.

It was decided, after careful consideration, that the two-way flat slab with drop panels was the most feasible alternative to the existing composite steel system. In general comparison, the two-way flab slab is less expensive than the composite steel system and provides an overall system depth of nearly half the composite system. However, the weight of the flat slab system is more than twice that of the composite steel system. This characteristic raises issues concerning the existing structural system of the Upstate Cancer Center. As it stands now, the superstructure of the Upstate Cancer Center is composed of steel. In order to accommodate a two-way flat slab with drop panels, the superstructure will need to be converted to an all concrete system. Bay sizes will be able to remain constant, but the existing lateral, gravity, and foundation systems will need to be completely redesigned.

Because the building is going to be completely designed out of concrete, the columns and subsequently the foundation will need to be resized / altered to accommodate the new weight of the structure. The conversion to a concrete superstructure also affects the lateral forces applied to the structure due to seismic activity. Existing braced frames will have to be exchanged with poured concrete shear walls in an attempt to add more lateral resistance to the building's structure to counter-act the amplified lateral forces due to the increase in building mass.

Although the other two systems, pre-cast hollow core plank and one-way pan joist and beam, may be theoretically possible for the design of the typical bay inside the Upstate Cancer Center, they were not as practical as the two-way flat slab system. In summary, the pre-cast hollow core plank system was too expensive and raised concerns about its structural efficiency in relation to the lateral system. The one-way pan joist and beam system was similar to the two-way flat slab system; however, it was slightly more expensive, had a deeper overall system depth, and required more labor and construction time to accomplish.

By analyzing each system and summarizing their characteristics, it was determined that the two-way flat slab with drop panels was the most practical and feasible alternative floor system to the existing composite system for the SUNY Upstate Cancer Center.

Introduction

The State University of New York's Upstate Medical University, located in Syracuse, New York will serve as the home to the new Upstate Cancer Center. Taking the place of an existing parking lot to the northwest of the Upstate Medical University Hospital, the new center will not only serve as the region's premiere outpatient adult and pediatric cancer center, but also link the university's Regional Oncology Center (ROC), Gamma Knife Center, and the Upstate Medical University Hospital. (See Figure 1)

Figure 1 Aerial map locating the building site. (Courtesy of Google Maps)

Upon its completion, the five-story building will rise 72 feet to the roof level, 90 feet to the top of the rooftop parapets, and encompass 90,000 square feet. Floor one will house administration services, the radiology department, as well as intra operative suites. The second floor will be reserved for medical oncology while the third floor will be devoted entirely for pediatric oncology. Floors four and five will consist of shell space intended for future outfit and expansion. A two-story central plant containing electrical transformers and a full mechanical space serves as linkage between the cancer center and the existing ROC. (See Figure 1 – highlighted green)

The building is primarily clad in a soothing white insulated metal paneling with cold form metal stud back up. This metal paneling is rather haphazardly disrupted by varying widths and heights of vertical bands of glazing. These bands consist of both vision and spandrel glazing, which is used to transition floor levels, hiding mechanical space and the structural floor. The exterior façade culminates at the three-story, northeast facing entrance atrium. Featuring a custom frit pattern, the northeast facing façade is enclosed by a full height, glazed curtain wall which provides solar shading as well as an aesthetically pleasing view. (See Figure 2)

Figure 2 Exterior rendering of northeast entry façade. (Courtesy of EwingCole)

Upstate is committed to providing a comforting environment for its patients, providing amenities such as a meditation room, a boutique for gifts and apparel, and a four-season roof top healing garden. These gardens not only serve as a refreshing oasis, but also help to reduce the cooling costs for the Upstate Cancer Center, adding to Upstate's goal of achieving USGBC LEED Silver certification. Preliminary Construction on the 74 million dollar center began in March of 2011 and is expected to be completed by September of 2013.

Structural Systems

Structural Key Plan

In an attempt to better understand the building geometries, a key plan overview of the site has been created. Main divisions of the building were divided and designated based on the location of expansion joints. Included in this reference diagram are basic dimensions, story counts, roof elevations, and primary building function or name. These building names will apply to data, calculations, and descriptions later in this report.

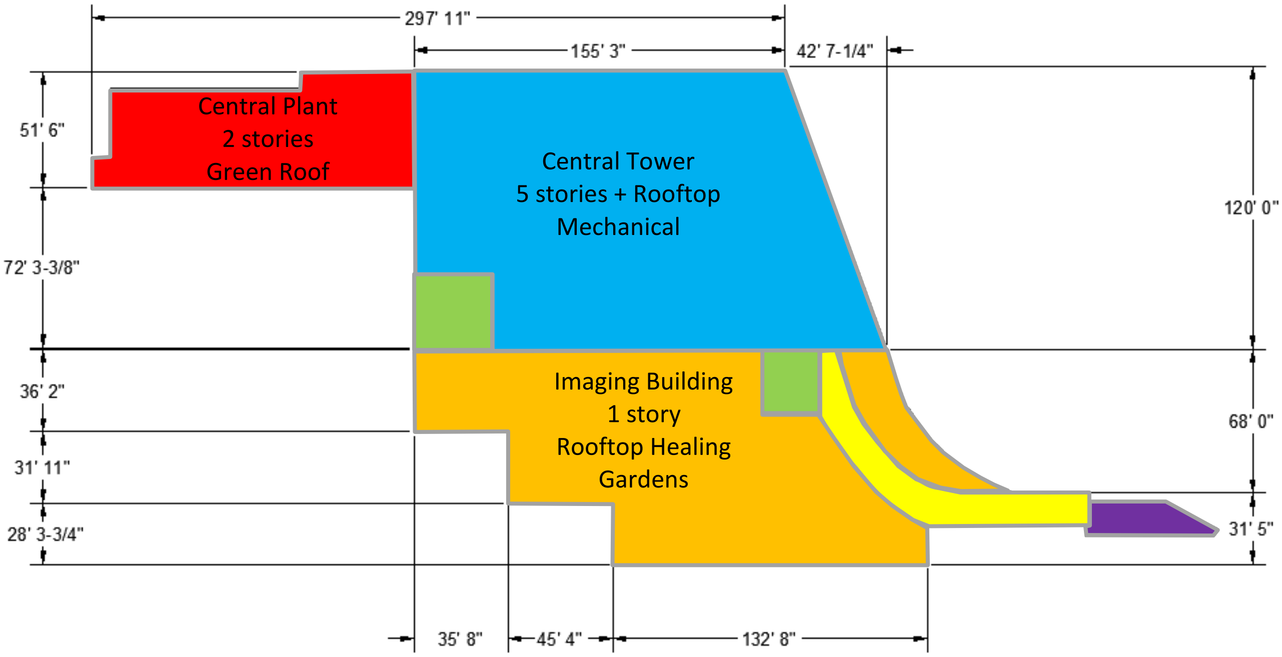


Figure 3 Building key plan showing main building divisions, dimensions, and description. Diagram key given below.

Diagram Key / Roof Elevations

🟦 Central Tower – 72'-0"
🟥 Central Plant – 30'-0"
🟨 Public Access Corridor – 30'-0"
🟧 Imaging Building – 16'-0"
🟩 Elevator Core Shafts – 86' 6"
🟪 Covered Entry Walkway

Foundation

Atlantic Testing Laboratories (ATL), at the request of Upstate Medical University, conducted a subsurface and geotechnical evaluation of the project site. Testing purposes were to determine the subsurface soil and ground water conditions at the site, and assess their engineering significance. Several boring tests, locations specified by architect/engineer EwingCole, were performed by ATL, to a minimum depth of 12 feet throughout the site. Subsurface soil composition beneath the initial layers of top soil and asphalt, mainly consisted of silty, gravelly, sand; silty clay and clayey silt, organic silt; debris (brick and ash); and weathered gypsum. Weathered bedrock was discovered at depths ranging from 12 to 28 feet at different boring locations. Beneath the weathered rock lies bedrock that consists of shale, gypsum, and dolostone deposits.

ATL's discoveries resulted in their recommendation of using a structural slab supported by a deep foundation system consisting of drilled piers (caissons) bearing on the dolostone bedrock. The allowable rock bearing capacity of the specified bedrock was assessed at 40 kips per square foot (40 ksf). ATL recommends a minimum pier diameter of 30 inches drilled a minimum of 24 inches into the bedrock.

Following these recommendations, EwingCole designed a foundation consisting of cast-in-place grade beams (4000 psi minimum compressive strength) resting on drilled caissons (5000 psi minimum compressive strength) with a poured slab on grade (4000 psi minimum compressive strength). All reinforcing was specified as ASTM A615 Grade 60. Grade beams range in depth from 16 to 66 inches and in width from 18 to 116 inches. Typical longitudinal bars are number eights to number tens with use of number three or number four stirrups. The slab on grade is most commonly a depth of six inches with some areas up to twelve inches thick, reinforced with number four to number six longitudinal bars. A typical grade beam section is shown below. (Figure 5)

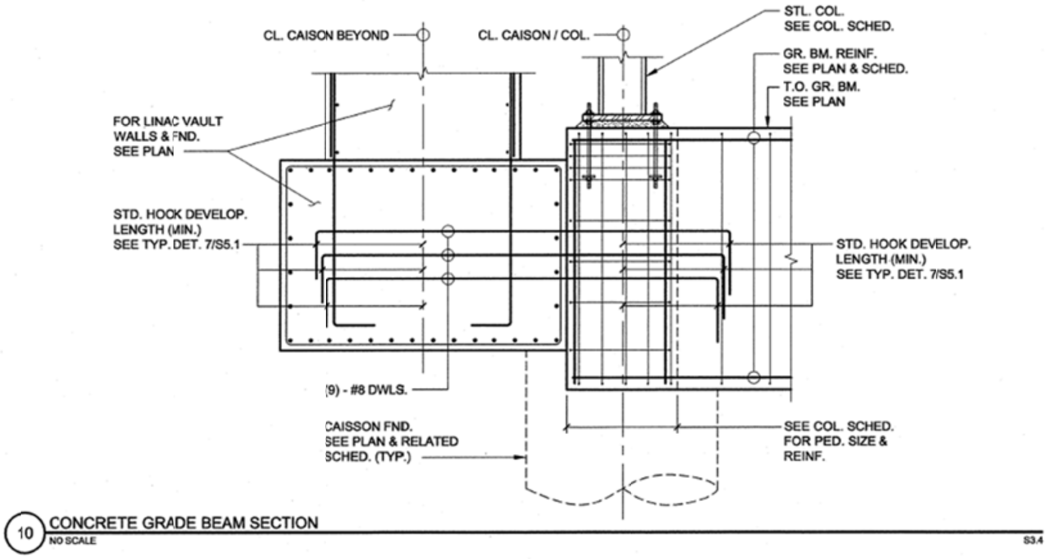


Figure 5 Typical grade beam section from sheet S3.4
(Courtesy of EwingCole)

Framing System

The superstructure of the Upstate Cancer Center is composed of structural ASTM A992 GR 50 wide flange steel shapes. Columns are almost exclusively sized as W12's with a few exceptions, W14's, and spliced at a height of 36 feet, mid-way through floor three. This provides a typical floor to floor height of 14 feet with a ground floor height of 16 feet. Column weights vary from 24 lb/ft to 210 lb/ft.

A typical bay size throughout the building measures 30'-0" by 30'-0" with infill beams spaced evenly at a distance of 10'-0" on center, spanning 30'-0" from girder to girder. Beams and Girders were designed compositely with the floor system through use of ¾" by 5 inch long shear studs welded on the center line of the members. In addition to this, infill beams were generally designed with a ¾" camber to compensate for excessive deflection. On a typical floor, beams range in size from W12x14's to W16x31's with the most common size being a W16x26. Girders range in size from W18x35's to W30x90's with the most common size being a W24x68 on a typical floor. Figure 6 shows a typical floor framing plan for floors two through four.

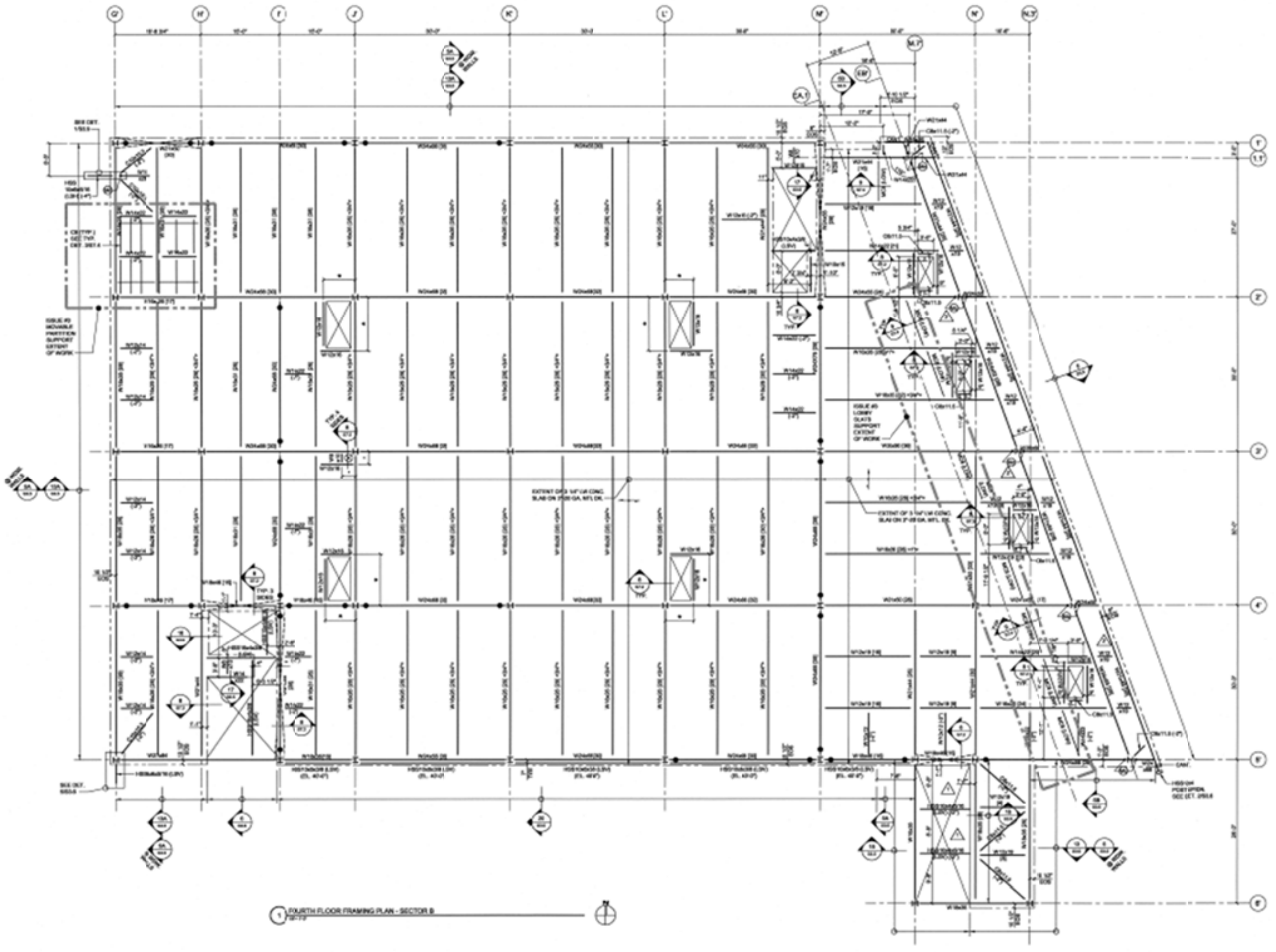


Figure 6 Typical framing layout (Central Tower) Floors two – four (Courtesy of EwingCole)

Floor System

All elevated floors of the cancer center utilize a composite flooring system working integrally with the structural framing members discussed in the previous section. A typical floor assembly is comprised of 3 inch 20 gage galvanized steel deck with 3 ¼ inch lightweight concrete topping (110 pcf, 3000 psi minimum compressive strength), a total thickness of 6 ¼ inches. The deck is reinforced with ASTM A185 6x6 welded wire fabric (WWF). On the fifth floor, a 60’-0” by 30’-0”, two bay, section of floor reserved for a future MRI or PET-CV unit, uses a larger topping thickness of 5 ¼ inches. The floor assembly for this particular area results as 3 inch 20 gage galvanized steel deck with 5 ¼ inch lightweight concrete topping, a total thickness of 8 ¼ inches, and ASTM A185 6x6 welded wire fabric.

All decking is specified as a minimum of two span continuous. The typical span length is approximately 10’-0” spanning perpendicular to the infill beams, typically W16x26’s. In the two story central plant, housing the center’s mechanical equipment, typical deck spans decrease to approximately 6’-0” to 7’-0”. The decrease of span length allows the floor system to support a larger superimposed load, i.e. mechanical and electrical equipment.

Roof System

The Upstate Cancer Center uses three separate roofing assemblies; metal roof deck; concrete roof deck; and a green roof. The metal roof deck is the most commonly used assembly of the three and consists of a 60 mil EPDM membrane, 5/8 inch cover board, 4 inch minimum rigid insulation, and a gypsum thermal barrier. This composition is used in combination with a 3 inch 18 gage galvanized metal roof deck atop the five story central tower, and with a 1 ½ inch 18 gage galvanized metal roof deck atop the second floor public access corridor spanning from the Upstate Cancer Center to the Upstate Medical University Hospital. In place of the metal deck and gypsum thermal barrier, the concrete roof deck assembly employs a poured concrete deck with a minimum of 2 inches of concrete topping. This assembly is used in one location, the lower level roof supporting auxiliary mechanical equipment.

Green roofing systems have been incorporated into the design of the Upstate Cancer Center for both aesthetic and energy saving purposes. The typical green roof assembly consists of native plants grown in approximately 12 inches of top soil. Beneath the soil surface is a composition of a drainage boards, rigid insulation, a root barrier, as well as roofing membrane. All of this is supported by a composite 3 inch 20 gage galvanized steel deck with 3 ¼ inch lightweight concrete topping, making a total thickness of 6 ¼ inches, reinforced with ASTM A185 6x6 welded wire fabric. The green roof assemblies are located atop the two story central plant as well as the single story imaging building.

Lateral System

Lateral forces acting on the building are mainly opposed by a series of ordinary steel braced frames running in the East-West and North-South directions inside the central tower. These braced frames generally run the full height of the building, from ground level to the roof. Braced frames are located, surrounding the elevator cores, along the exterior wall of the building, and along interior framing lines. (See Figure 7 for braced frame locations, highlighted in blue)

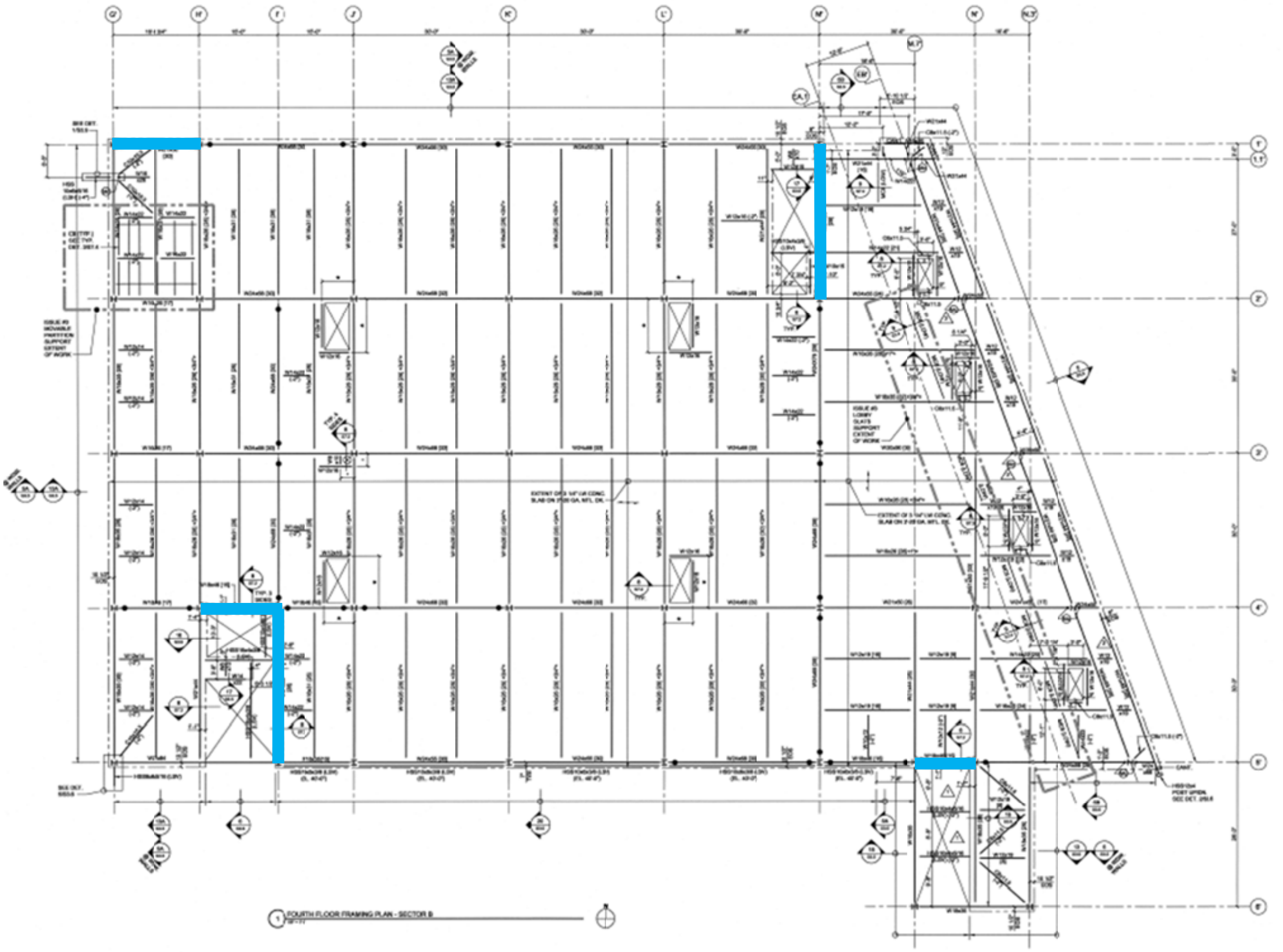


Figure 7 Location of braced frames in the central tower. (Courtesy of EwingCole)

All columns used in the braced frames are W12's ranging in size from a W12x106 to a W12x210. The diagonal members used for the frames are generally W10's with W8's being used at the upper levels. Sizes of these members range from W8x31 to W10x88. The bolted connections for the frames were not detailed for seismic resistance and therefore a response modification factor of 3.0 was used for calculation purposes. Figure 8 below displays an elevation of the braced frame located long grid line I' between lines 4' and 5'.

Braced frames are used in conjunction with moment frames in the central plant. Braced frames run in the East-West direction along the exterior walls of the building, while moment frames run in the North-South direction along interior framing lines. The moment frames allow for more accessible floor space to be utilized for the movement of mechanical equipment. The brace frame composition for the central plant is similar to that described previously. The typical moment frame uses a bolted moment connection with most welding prefabricated in the shop.

Similar braced frames are used as the main lateral resisting system within the imaging building. Figure 9 displays the location of braced (blue) and moment (red) frames in the central plant as well as the imaging building.

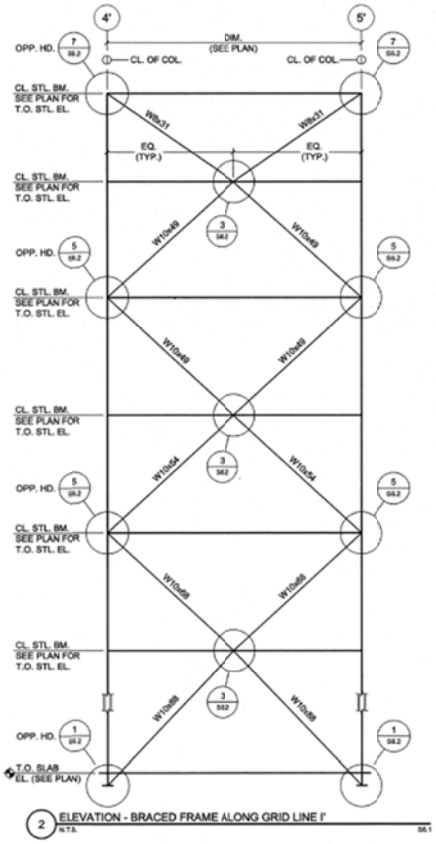
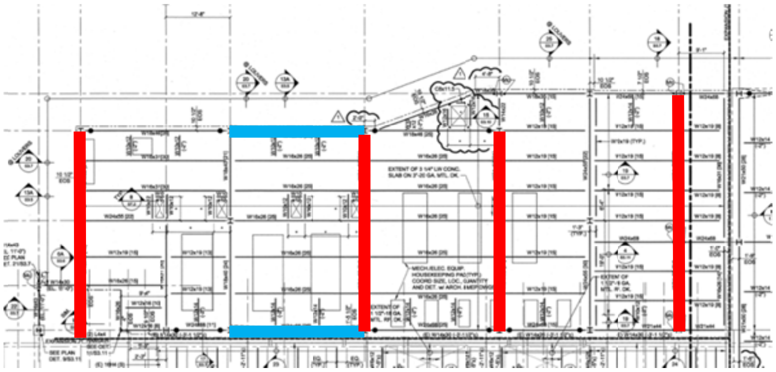


Figure 8 Braced frame elevation along grid line I' between lines 4' & 5' (Courtesy of EwingCole)



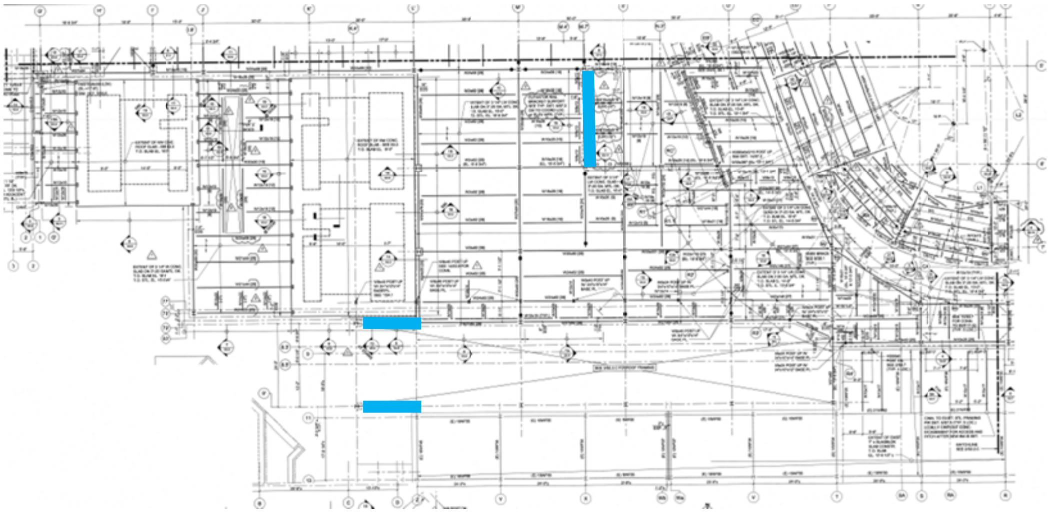


Figure 9 Floor plans showing braced (blue) and moment (red) frames locations in the central plant (above) and imaging building (right).
(Courtesy of EwingCole)

Design Codes and Standards

Referencing sheet G.2.1, the following codes were applicable in the design of the Upstate Cancer Center:

- 2007 Building Code of New York State (Based on IBC 2003)
    - IBC 2003 - International Building Code, 2003 Edition
    - ASCE 7-02 – Minimum Design Loads for Buildings and Other Structures, 2002 Edition
- 1997 Life Safety Code (NFPA 101)
- Sprinkler Code – NFPA 13-02
- National Electrical Code, 2005 Edition
- 2007 Plumbing Code of New York State (Based on the 2003 IPC)
- 2007 Fire Code of New York State (Based on the 2003 IFC)
- 2007 Energy Conservation Construction Code of New York State
- 2007 Mechanical Code of New York State (Based on the 2003 IMC)
- 2007 Fuel Gas Code of New York State (Based on the 2003 IFGC)
- Accessibility – ICC/ANSI A117.1-03
- 1997 AIA Guidelines for Design & Construction of Healthcare Facilities
- Health Care – NFPA 99-1996
- Fire Alarm Code – NFPA 72-02 (Amended)
- AISC Manual of Steel Construction, Load Resistance Factor Design (LRFD)

Calculations and analyses included within this report have been carried out with use of the following codes and standards:

- IBC 2009 – International Building Code, 2009 Edition
- ASCE 7-10 – Minimum Design Loads for Building and Other Structures, 2010 Edition
- AISC Manual of Steel Construction, 14th Edition, Load Resistance Factor Design (LRFD)
- ACI 318-08 – Building Code Requirements for Structural Concrete and Commentary
- Vulcraft Steel Roof and Floor Deck 2008

*NOTE: References made to 2007 Building Code of New York State for special case items.

Materials

Structural Steel		
Item	**Grade**	**Strength, fy (ksi)**
Wide Flange Structural Shapes	A992 GR 50	50
Base Plates / Moment Plates / Spice Plates	ASTM 572 GR 50	50
Hollow Structural Steel	ASTM A 500 GR B	46
Angles / Channels / Other Plates	A36	36
Concrete		
Item	**Weight (pcf)**	**Strength, f'c (psi)**
Piers / Caissons	Normal Weight (145)	5000
Slab on Grade (SOG)	Normal Weight (145)	4000
Walls / Beams / Equipment Pads / Sidewalks	Normal Weight (145)	4000
Lower Mechanical Roof Slab Deck	Normal Weight (145)	3500
Typical Slab Deck	Light Weight (110)	3000
Masonry		
Item	**Grade**	**Strength (psi)**
Concrete Masonry Unit (CMU)	ASTM C 90	1900
Type S Mortar	ASTM C 270	1800
Fine Grout	--	3000
Cold Formed Metal Framing		
Item	**Grade**	**Strength (ksi)**
6" Cold Form Metal Framing	ASTM 653	50

Table 1 Compilation of building materials used in the design and construction of the Upstate Cancer Center.

Building Loads

The following sections convey the various loads that were tabulated for the Upstate Cancer Center and used to spot check selected member sizes and design. Loads considered acting on the structure were dead, live, snow, wind, and seismic. Values were verified against provided data for accuracy where given.

Dead Load

Dead load was calculated for the building accounting for loading that was considered permanent over the life of the building. Items that were included in the dead load determination consisted of framing members (beams and girders); columns; floor assemblies (metal deck, concrete slab, etc.); exterior wall assemblies (façade weights); mechanical, electrical, and plumbing (MEP) equipment; ceiling and floor finishings; and any permanent equipment that was specified. Values for weights of common building materials were either gathered from literature or assumed based on engineering judgment. In cases of uncertainty, values were always calculated conservatively.

Because the building is separated into three separate pieces, loads were tabulated individually for each piece. Discrepancies between listed weights are most likely due to different assumptions of superimposed dead loads. The table below (Table 2) lists typical values for various components of the structural system. It should be noted that MEP equipment, ceiling and floor finishings are considered in one category, superimposed dead load. Also, any weights particular to a specific floor, such as air handling units or medical equipment, are not included.

Dead Loads	
Description	**Load**
Beams / Girders	6.5 psf
Columns	2.25 psf
Floor Systems:	
1-1/2" Metal Roof Deck	13.74 psf
3" Metal Roof Deck	14.56 psf
3" Composite Deck w/ 3-1/4" LW Topping	46 psf
3" Composite Deck w/ 5-1/4" LW Topping	64 psf
Green Roof	154.5 psf
Facades:	
Curtain Wall Glazing	15 psf
Insulated Metal Paneling	21 psf
Brick Veneer	45 psf
Super Imposed Dead Load:	
Central Tower / Imaging Building	25 psf
Central Plant	60 psf

Table 2 Break down of typical dead loads. Note: Central Plant Superimposed Dead Load considers the weight of unaccounted mechanical equipment.

In order to determine the weight of individual floors and subsequently the total weight of the building, individual assembly weights were taken by their appropriate area and summed.

Live Load

Design live loads were specified on sheet SG.1 in accordance with the 2007 New York State Building Code. The loads given were not descriptive of their classification, but simply were listed as "Typical Floor Live Load," etc. To produce accurate and comparable loads, assumptions were made with engineering judgment regarding usage of spaces as well as future changes. Because floors four and five are left unoccupied for future expansion, they will be designed to the highest live load found on the remaining three floors to compensate for the uncertainty of occupancy. Live load values were obtained from the International Building Code, 2009 edition, using Table 1607.1, and cross-referenced with ASCE 7-10 using Table 4-1. Table 3 below summarizes the comparison of live load values chosen for design versus the live load values used for analyses in this report.

Live Loads			
Occupancy Type	Design Live Load (psf) N. Y. State Building Code (2007)	Analysis Live Load (psf) IBC 2009 / ASCE 7-10	Comments
Public Space / Typical Floor	100	100	Use of higher load to account for undesigned core floors four and five
Corridors	100	100	
Mechanical Building Spaces	250	250	Heavy manufacturing areas used for comparison
Typical Roof	45	20	Snow Load may control over roof live load
Rooftop Gardens	100	100	
Rooftop Mechanical Locations	150	125	Light manufacturing areas used for comparison

Table 3 Live load comparison between initial design and loads used in analyses in this report

Floor System Analysis

Structural bays of the Upstate Cancer Center are rather regular with a typical bay size of 30'-0" by 30'-0". A typical bay was picked between column lines K' - L' and 3' - 4' for design and analysis purposes. (See Figure 10) The original floor construction, composite steel deck on composite steel beams and girders, was analyzed and compared among three proposed floor systems for the Upstate Cancer Center. The floor systems that were chosen for analysis are as follows:

- Composite Steel deck on Composite Steel Beams and Girders
- Pre -Cast Concrete Hollow Core Planks on Structural Steel Framing
- Two-Way Flat Slab with Drop Panels
- One-Way Pan Joist and Beam System

Each flooring system is detailed in the following sections noting their advantages and disadvantages. Systems were evaluated based on the assembly weight, overall system depth, assembly cost, constructability, serviceability, as well as their effects on the building's existing structural systems and architectural features. (NOTE: All cost data was obtained from RS Means CostWorks Online data base) All floor systems were designed and assessed solely under gravity loading. A summary comparing the four floor systems can be found in Table 4. Calculations pertaining to the design of each floor system are included in the appropriate Appendices.

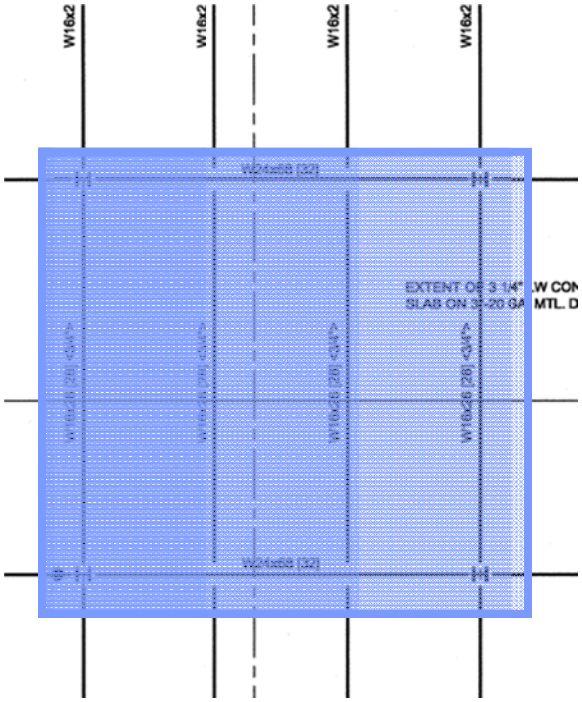


Figure 10 Typical Bay (highlighted in blue) between column lines K'-L' and 3'-4'. (Courtesy of EwingCole)

Composite Steel Deck on Composite Beams and Girders

Description

The existing floor system used for the elevated floors of the Upstate Cancer Center is a composite system consisting of composite metal deck atop composite beams and girders. Hand calculated spot checks were performed for this particular system in the previous technical report, Technical Report 1, and have been referenced for use in this technical report. Decking is specified as 3 inch 20 gage composited deck with 3 ¼ inches of lightweight concrete topping, making a total thickness of 6 ¼ inches. Utilizing the Vulcraft Steel Roof and Floor Deck 2008 catalog, a 3VLI20 deck with 3 ¼ inches of lightweight concrete topping meets the specified requirements, is capable of carrying the applied loading, and is sufficient for unshored construction. In addition, this assembly provides a two-hour fire rating as specified by Underwriter's Laboratory.

Supporting the floor assembly are W16x26 composite infill beams spaced at 10'-0" on center which tie into the larger W24x68 composite girders. These framing elements achieve composite action through the use of ¾" diameter, 5 inch long shear studs welded along the center line of the members. Beams and girders were evaluated for strength and serviceability requirements. The members are adequate to carry more than the required loading, are within acceptable deflection limits, and meet requirements for unshored construction. Detailed calculations for the composited steel deck on composite beams and girders can be found in Appendix A. Figure 11 illustrates a typical bay layout for the system.

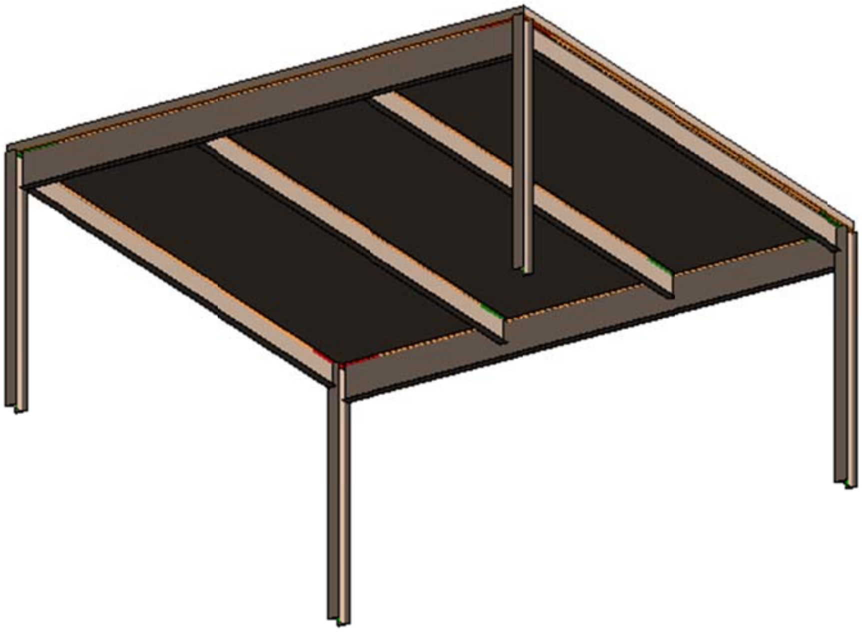


Figure 11 Typical bay layout for Composite Steel Deck on Composite Beams and Girders. (30'-0" by 30'-0" between column lines K'-L' and 3'-4') (Modeled in Revit Structure)

Advantages

A composite steel system provides an effective and efficient design utilizing the strengths of both concrete in compression and steel in tension. This dual action allows the beam depths to be much less than those of a traditional non-composite steel system. Subsequently a composite steel system could provide longer span lengths and carry a larger load. Formwork is minimal for this system simply because the concrete is poured directly onto the metal deck; this allows for a quickened construction pace. This system in particular does not require shoring during construction, once again cutting down on construction time and cost. In addition to having shallower, lighter framing members, normal weight concrete has been substituted for lightweight concrete further reducing the weight of the system.

Disadvantages

In order to achieve the composite action desired by the design of a composite steel system, shear studs need to be installed onto the framing members, requiring more labor and inspections for proper installment. Adding shear studs also drives up the cost of the assembly. It is generally taken that the cost of one shear stud is approximately equal to ten pounds of steel. Although the assembly meets a two-hour fire rating, the underside of the deck as well as framing members must be protected by some sort of fireproofing, adding extra cost to the system. Even though beam depths are shallower in a composite system, girder sizes can sometimes be rather deep causing issues coordinating other disciplines throughout the building.

Pre-Cast Hollow Core Planks on Steel Framing

Description

Pre-cast hollow core planks supported by steel framing was selected as the first alternative floor system for the SUNY Upstate Cancer Center. Common plank widths are normally 4’-0” or 8’-0”. The hollow core planks were selected to span 30’-0” in the East-West direction of the building. Selecting the 4’-0” wide plank, the North-South bays of the Upstate Cancer Center had to be adjusted in order to accommodate a whole number of planks without altering the overall dimension of the building. The solution was to modify their lengths to 20’-0”, allowing five planks to fit side by side. Although the typical bay size changes from 30’-0” by 30’-0” to 30’-0” by 20’-0” (See Figure 12), the overall dimensions of the building remain the same.

Referencing Spancrete’s Pre-Cast Hollow Core Plank Catalog, a 4’-0” wide by 8” deep, Standard Spancrete Hollow Core Plank with 1 ¾” strand cover, would be sufficient to carry the required superimposed load. These planks would contain the strand configuration of 1.75D-8712T, and would be topped with 2 inches of concrete for floor leveling purposes.

Girders spanning the 30’-0” direction were sized for the appropriate loading and checked against live load and total load deflection. The appropriate size was calculated to as a W30x90. In an attempt to minimize the overall system depth, the hollow core planks would be connected to the girders per detail HL-31 (Figure 13) provided by Spancrete. All calculations pertaining to the design of the Pre-cast hollow core plank alternative floor system can be found in Appendix B.

Figure 12 Typical bay layout for Pre-Cast Hollow Core Plank on Steel Framing. (30’-0” by 20’-0” between adjusted column lines K’-L’ and 3’-4’) (Modeled in Revit Structure)

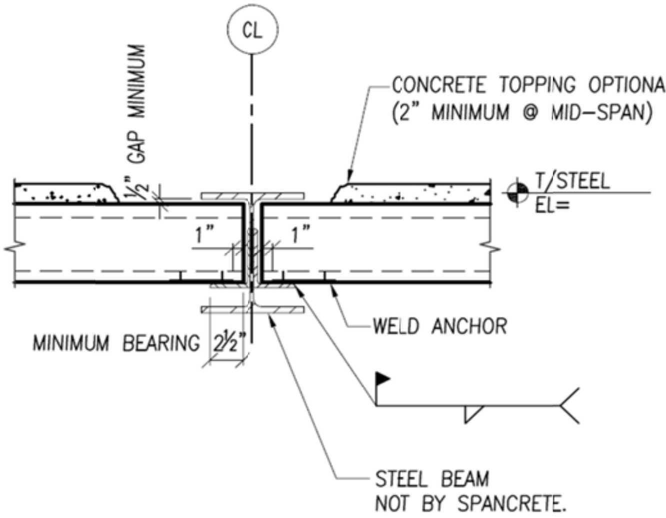


Figure 13 Typical Hollow Core Plank to Girder connection detail (HL-31) specified by Spancrete. (Courtesy of Spancrete)

Advantages

Construction time for this system would be greatly reduced due to the fact that the pre-cast hollow core concrete planks would be manufactured beforehand at a pre-cast concrete plant. After erection the planks would be covered with a topping for finishing purposes. During construction, shoring would not be required, allowing construction to progress above the recently installed levels. The weight of the system is greatly reduced due to the voided cores along the length of the planks. In addition, the pre-cast hollow core plank assembly would also reduce sound and heat transmission between floors. Fireproofing would not be required to protect the underside of the planks; however, the exposed framing members would still need protection, perhaps a cemetitious spray.

Disadvantages

This system is essentially constructed by laying these preformed beams across an open span connecting to steel girders by means of a welded plate. Although they are topped with a two inches of concrete, it is unclear whether the planks together would have enough rigidity to transfer the applied lateral loads to the existing lateral system. Additional resistance, in the form of braced framing, could possibly need to be implemented in order to address this issue. Although deeper girders were needed to carry the load of the planking, the connection detail allowed the overall system depth to remain the same as the composite steel system. Bay sizes in the North-South direction would have to be altered in order to accommodate the preformed plank widths. This would change the existing column layout of the Upstate Cancer Center, ultimately leading to changes in the location of caissons as well as the size and location of grade beams in the foundation. In terms of constructability, the pre-cast hollow core planks generally require a long lead time to produce, and are not very useful for applications including irregular or curved perimeters.

Two-Way Flat Slab with Drop Panels

Description

The second alternative floor system for the SUNY Upstate Cancer Center was designed as a two-way flat slab with drop panels. Drop panels were assumed to be needed due to the large live load and resulting punching shear. In order to employ a flat slab with drop panels, the existing steel columns would need to be exchanged with cast-in-place concrete columns. These columns were estimated as square measuring 24 inches by 24 inches. A minimum slab thickness of 9.33 inches was determined according to ACI 318-08 Section 9.5.3.2, assuming an interior slab with drop panels. For ease of construction, a depth of 9 ½" was used for the remaining calculations.

According to ACI 318-08 Section 13.6.1 the flat slab was able to be design by the Direct Design Method. All reinforcing steel was taken to have a yield strength of 60,000 psi, and all cast-in-place concrete was taken to have a compressive strength of 4,000 psi. Because the typical bay used for analysis is equal in both dimensions, the reinforcement required to resist the moments in the column and middle strips is identical for both directions.

Two-way shear, or punching shear, was checked at the columns without the use of drop panels. The analysis resulted in a failure, supporting the earlier claim about the need for drop panels in this system. An analysis carried out using a 6 inch drop was successful; however, a drop of 6 ¼ inches will be used due to the ease of constructability. The 6 ¼ drop is well beyond the minimum depth of 2.38 inches as prescribed by ACI 318-08. In accordance with code, all drop panels will have a 10'-0" by 10'-0" dimension. The floor system configuration can be seen in Figure 14. All calculations pertaining to the two-way flat slab with drop panels alternative flooring system can be found in Appendix C.

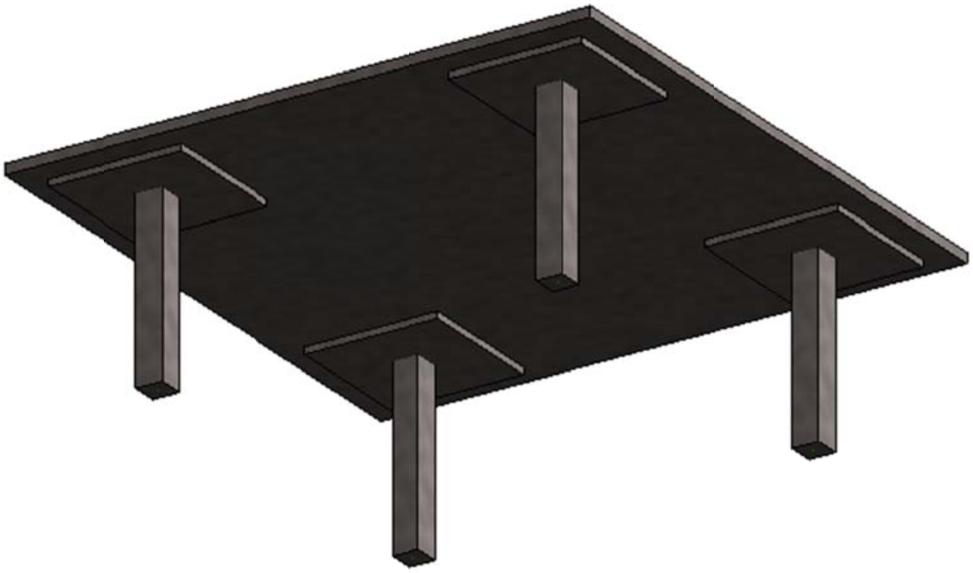


Figure 14 Typical bay layout for Two-Way Flat Slab with Drop Panels. (30'-0" by 30'-0" between column lines K'-L' and 3'-4') (Modeled in Revit Structure)

Advantages

One of the most appealing aspects of the two-way flat slab with drop panels floor system is the ability to create a low story height. Because there is no limitation to the overall height of the Upstate Cancer Center, this system could be used to create a higher story height while keeping the total building height the same. Formwork used to construct this system is relatively simplistic and it could possibly be reused for pouring other floors. The simplicity of the formwork also allows for ease of construction decreasing the erection time of the building. In general, poured concrete flooring systems, such as the flat slab with drop panels, offer more mass and intern provide better vibration control over that of a steel floor system. The drop panels located around the columns not only provide strength to prevent punching shear but also increase the systems stiffness further improving its rigidity and vibratory resistance. Because the entire superstructure is composed of concrete, no additional fire protection needs to be added, so long as proper cover is provided.

Disadvantages

Although the flat slab with drop panels system offers many benefits, it suffers from some key drawbacks. One of the main concerns of using this system is the need for future core drilling. Because the system is designed to carry moment in both directions, penetrating the slab for future renovation or tenant purposes could severely hamper the structural integrity of the system. Similarly drilling through the drop panels could result in a two-way shear failure. Because the cancer center is owned and is part of the Upstate Medical University, its use and occupancy has been accounted for, and will most likely remain the same over the life of the building.

Converting from a steel superstructure to a concrete superstructure, will increase the overall mass of the building significantly. As a result, lateral forces attributed to seismic activity will increase from the original design values. This will result in an entire redesign of the lateral system of the building. Most likely, poured concrete shear walls will replace the braced frames as the lateral resisting elements of the structure. It should also be noted that due to the increase in building weight, alterations to the foundation would also need to be made in order to effectively carry the load of the building.

One-Way Pan Joist and Beam

Description

A one-way pan joist and beam system was chosen as the third and final alternative flooring system for the Upstate Cancer Center. This system was chosen because it is often used in larger bay sizes, and was widely popular for healthcare construction in previous years. The system casts the joists, running in one direction, monolithically with a wide beam, running normal to the joists. Since the system consists of cast-in-place concrete members, the steel superstructure of the building must be converted to concrete. Once again, the concrete column sizes were assumed square with a dimension of 24 inches by 24 inches. A 30 inch pan size with a 5 inch width joist was used for design, making the center-to-center of the joists 35 inches apart. A typical slab depth for a one-way pan joist and beam system is 4 ½ inches. This depth also provides a two hour fire rating. The joists frame into the wide beam girder, measuring 36 inches across.

Using the superimposed loads, reinforcement was design for the slab assuming one-way action, and was specified as (1) #3 bar. Joist depth was required to meet a minimum value specified by ACI 318-08 Table 9.5(a). Since the resulting value was calculated to be 15.4 inches, a joist depth of 16 inches was chosen, making the overall depth of the system 20.5 inches. Joist reinforcing was specified as (2) # 6 bars for negative moment, and (1) #7 bar for positive moment. Reinforcement was found for the beam was calculated similarly to the joist assuming it had the same 16 inch depth. Negative moment reinforcement was specified as (10) # 9 bars, while the positive moment reinforcement was (7) #9 bars. Figure 15 illustrates the layout of a typical bay employing the one-way pan joist and beam alternative floor system. Calculations pertaining to this system can be found in Appendix D.

Figure 15 Typical bay layout for One-Way Pan Joist and Beam (30’-0” by 30’-0” between column lines K’-L’ and 3’-4’) (Modeled in Revit Structure)

Advantages

One-way pan joist and beam systems allow for the use of longer spans and wider spaced columns, which is beneficial for the layout of the Upstate Cancer Center. This system is also are fitted to carry larger live loading, as in the case of the Upstate Cancer Center. Although the voids from the pans reduce the dead load of the system, overall the structure is very massive, which is useful when considering vibration dampening. Voids formed by the pans can also be used to house electrical, mechanical, and plumbing equipment such that there is no need to increase the depth of the ceiling cavity to accommodate such equipment. Because the formwork is repetitive for this system it can be reused to cast additional floors and bays, cutting down on construction cost. However, the formwork is a bit more complicated than that of the previous flat slab with drop panels, leading to a slightly higher labor cost and slightly longer erection time. Once again, because the superstructure is composed of concrete, the need for additive fireproofing is unnecessary, further reducing the cost of the building as compared to other systems. The one-way pan joist and beam system does allow for easier future renovations as compared to the flat slab system.

Disadvantages

Some of the drawbacks of the two-way flat slab with drop panels floor system also plague the one-way pan joist and beam floor system. As stated earlier, a switch from a steel superstructure to a concrete superstructure will significantly change the mass of the building. Ultimately, considerable changes or a complete redesign of both the lateral resisting system and foundation system of the Upstate Cancer Center will be necessary. Also stated previously, the cost of formwork for this system is slight more expensive than that of the flat slab system, and requires a bit more skill and labor to form and remove.

Floor System Summary

The chart provided below is a simple summary recapping key strengths and weaknesses of each of the four systems described prior. The final row is base on personal opinion about whether the system is worthwhile to implement as an alternative floor design for the SUNY Upstate Cancer Center.

	Systems			
	Existing	Alternatives		
Consideration	**Composite Steel Deck on Composite Beams & Girders**	**Precast Hollow Core Planks on Steel Girder**	**Two-Way Flat Slab with Drop Panels**	**One-Way Pan Joist System**
General Information				
Weight	50.9 psf	91 psf	127.7 psf	104.8 psf
Overall Depth	30"	30"	15.75"	20.5"
Slab Depth	6.25"	10"	9.5"	4.5"
Assembly Cost	20.04 $/sf	25.96 $/sf	17.44 $/sf	18.33 $/sf
Architectural				
Bay Size	30'-0" x 30'-0"	30'-0" x 20'-0"	30'-0" x 30'-0"	30'-0" x 30'-0"
Fire Rating	2 HR - UL Assembly	2 HR - Unrestrained	2 HR	2 HR
Other	Requires Additional Fireproofing for Underside of Deck & Framing Members	Fireproofing Needed for Exposed Framing Members Change in Bay Size	Increase in Floor to Floor Height Superstructure Changes to Concrete	Increase in Floor to Floor Height Superstructure Changes to Concrete
Structural				
Gravity System Alterations	No Change	Increase Girder Size - Resize Columns Due to Altered Bay Sizes	No Beams/Girders - Concrete Columns w/ Drop Panels	Joists w/ Wide Beam Girders - Concrete Columns
Lateral System Alterations	No Change	Possible Addition of Braced Frames	Change From Braced Frames to Shear Walls	Change From Braced Frames to Shear Walls
Foundation Alterations	No Change	Alter Size and Location of Caissons & Grade Beams	Increase Foundation Size to Carry Larger Building Weight	Increase Foundation Size to Carry Larger Building Weight
Construction				
Formwork Required	Minimal	None	Yes	Yes
Constructability	Slightly Moderate	Easy	Moderate	Slightly Difficult
Lead Time	Moderate	Long	Moderate	Moderate
Serviceability				
Vibration Control	Moderate	Slightly Moderate	Good	Great
Feasible	**YES**	**NO**	**YES**	**YES**

Table 4 Summary comparing existing floor system and three proposed alternative systems.

(NOTE: Cost data obtained from RS Means CostWorks Online Database)

Additional Consideration

It should be brought to attention that this report has discussed floor system design based on a typical bay located within the Central Tower of the Upstate Cancer Center. Although this design covers the majority of the floor area throughout the cancer center, irregular bays do exist within the structure. In particular, on the fifth floor of the Central Tower lie two adjacent 30’-0” by 30’-0” bays that have been designed and reserved for future MRI space. The floor structure in this location, as mentioned in previous reports, contains a thicker slab on deeper, heavier members. This change in construction is most likely to account for floor vibrations due to the MRI machinery. Although it was out of the scope of this technical assignment, there has been interest in studying these two bays and determining if a different floor system could be implemented to better dampen vibrations from the machine. If so, further research would be made to see if the system would be cost effective and practical to use throughout the remainder of the building. The goal of attempting such a feat would be to increase sound and vibration dampening, improving the quality inside the building from a serviceability stand point. This topic could provide motive for a possible thesis proposal.

Conclusion

In summary, the intention of this report was to investigate three alternative flooring systems that could be implemented for the SUNY Upstate Cancer Center in Syracuse, New York. In addition to the existing floor system, composite steel deck on composite steel beams and girders, a pre-cast hollow core plank system, a two-way flat slab system, and a one-way pan joist system were selected for analysis and comparison. The four systems were compared on the basis of system weight, depth, cost, construction facts and information, alterations to the existing building architecture and structural systems, as well as serviceability issues.

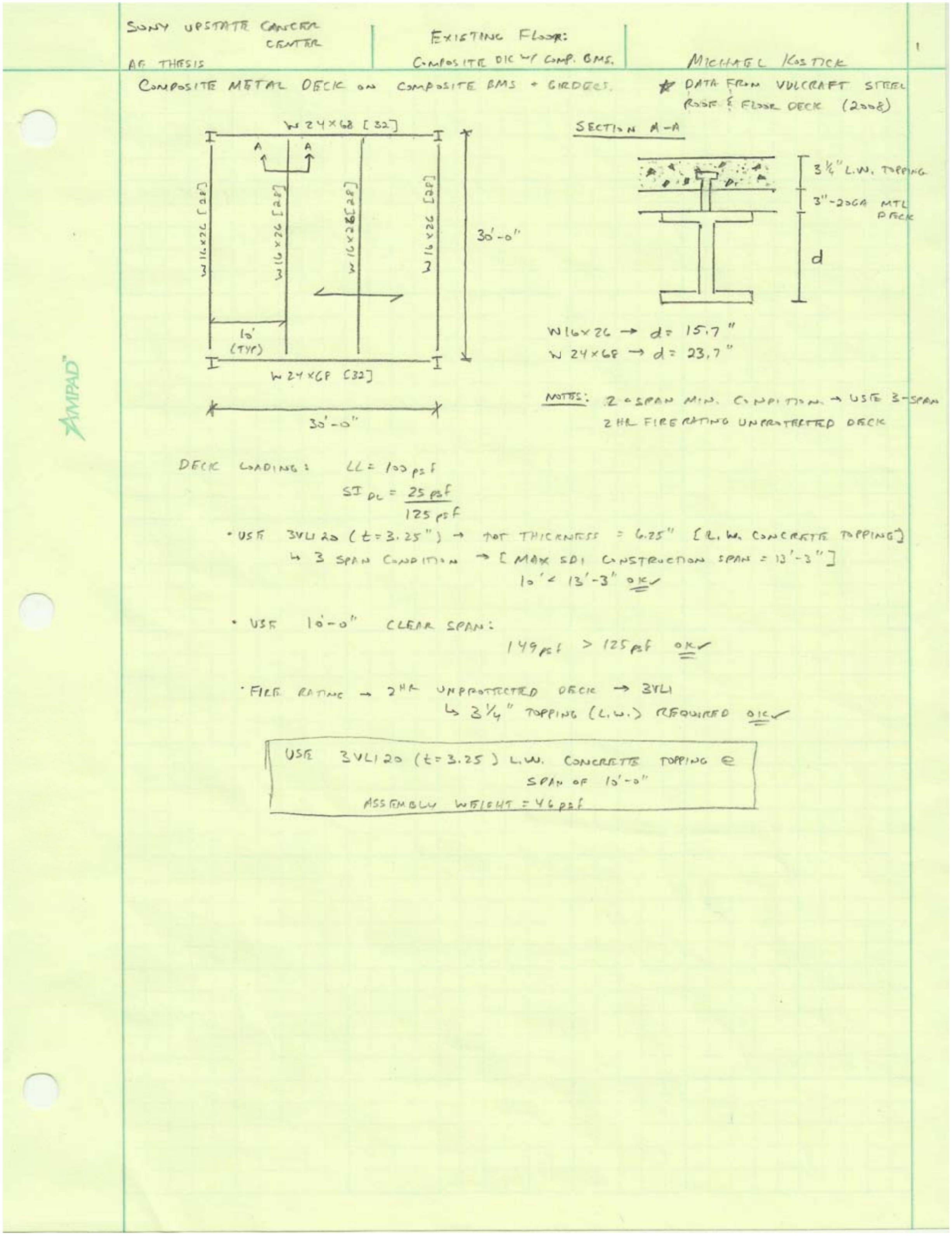
Overall the goal of this assignment was to determine the most feasible alternative floor system from the three systems that were proposed. After carefully considering the strengths and weaknesses of each system, it was decided that if the SUNY Upstate Cancer Center was not to be designed using a composite deck and composite framing system, the next best option would be the two-way flat slab with drop panels. This system has the least cost of all the systems, maintains the original bay sizes, and cuts the original system depth nearly in half. The main issues with selecting the two-way flat slab system are that the gravity and lateral systems will need to be completely redesigned.

The assembly weight of the flat slab with drop panels is more than twice as much as the existing composite steel system. A concrete superstructure would have to take the place of the existing steel superstructure. Foundation sizes would need to be increased to carry the additional load and provide a stable base for the structure. The lateral system would need to account for the concrete superstructure, perhaps using concrete shear walls in place of braced frames. In addition, lateral forces due to seismic activity would need to be reevaluated due to the increase in building mass.

The additional systems did not seem as practical as the two-way flat slab with drop panel. Pre-cast hollow core planks proved to be considerably higher in price and raised concerns about structural issues related to the lateral system. Although the one-way pan joist and beam system is comparable to the two-way flat slab with drop panels, it is slightly more expensive and requires more labor and construction time to accomplish.

From the research gathered from this report, the two-way flat slab with drop panels floor system will be further investigated as an alternative floor system for the SUNY Upstate Cancer Center.

Appendix A: Composite Steel Deck on Composite Beams & Girders



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EXISTING FLOOR:
COMPOSITE DK W/ COMP. BMS.

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1

COMPOSITE METAL DECK ON COMPOSITE BMS + GIRDERS.

★ DATA FROM VULCRAFT STEEL ROOF & FLOOR DECK (2008)

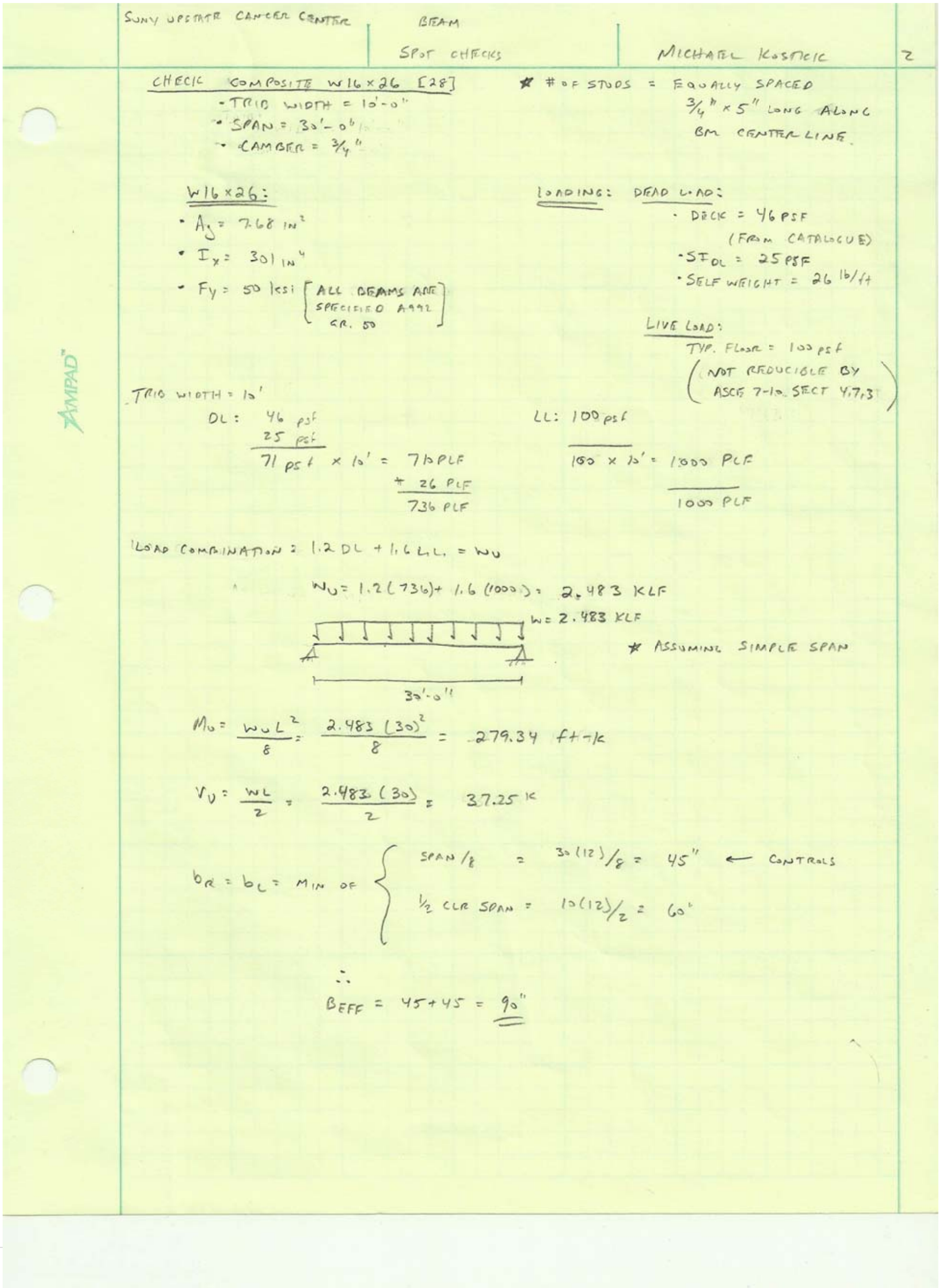
W16×26 → d = 15.7"
W 24×68 → d = 23.7"

NOTES: 2-SPAN MIN. CONDITION → USE 3-SPAN
2 HR FIRE RATING UNPROTECTED DECK

DECK LOADING: LL = 100 psf
SI_{DL} = 25 psf
125 psf

- USE 3VLI 20 (t = 3.25") → TOT THICKNESS = 6.25" [L.W. CONCRETE TOPPING]
  ↳ 3 SPAN CONDITION → [MAX SDI CONSTRUCTION SPAN = 13'-3"]
  10' < 13'-3" OK ✓
- USE 10'-0" CLEAR SPAN:
  149 psf > 125 psf OK ✓
- FIRE RATING → 2 HR UNPROTECTED DECK → 3VLI
  ↳ 3 1/4" TOPPING (L.W.) REQUIRED OK ✓

USE 3VLI 20 (t = 3.25) L.W. CONCRETE TOPPING @ SPAN OF 10'-0"
ASSEMBLY WEIGHT = 46 psf



SUNY UPSTATE CANCER CENTER
BEAM SPOT CHECKS
MICHAEL KOSTICK
2

CHECK COMPOSITE W16×26 [28]

- TRIB WIDTH = 10'-0"
- SPAN = 30'-0"
- CAMBER = 3/4"

★ # OF STUDS = EQUALLY SPACED 3/4" × 5" LONG ALONG BM CENTER LINE.

W16×26:

- $A_g = 7.68 \text{ IN}^2$
- $I_x = 301 \text{ IN}^4$
- $F_y = 50$ ksi [ALL BEAMS ARE SPECIFIED A992 GR. 50]

LOADING: DEAD LOAD:

- DECK = 46 PSF (FROM CATALOGUE)
- SI_{DL} = 25 PSF
- SELF WEIGHT = 26 lb/ft

LIVE LOAD:

TYP. FLOOR = 100 psf (NOT REDUCIBLE BY ASCE 7-10 SECT 4.7.3)

TRIB WIDTH = 10'

DL: 46 psf
25 psf
71 psf × 10' = 710 PLF
+ 26 PLF
736 PLF

LL: 100 psf
100 × 10' = 1000 PLF
1000 PLF

LOAD COMBINATION: 1.2 DL + 1.6 L.L. = w_u

$$w_u = 1.2(736) + 1.6(1000) = 2.483 \text{ KLF}$$

w = 2.483 KLF

★ ASSUMING SIMPLE SPAN

30'-0"

$$M_u = \frac{w_u L^2}{8} = \frac{2.483(30)^2}{8} = 279.34 \text{ ft-k}$$

$$V_u = \frac{wL}{2} = \frac{2.483(30)}{2} = 37.25^k$$

$$b_R = b_L = \text{MIN OF} \begin{cases} \text{SPAN}/8 = 30(12)/8 = 45'' & \leftarrow \text{CONTROLS} \\ 1/2 \text{ CLR SPAN} = 10(12)/2 = 60'' \end{cases}$$

$$\therefore \quad B_{EFF} = 45 + 45 = \underline{\underline{90''}}$$

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AE THESIS

BEAM
SPOT CHECK

MICHAEL KOSTICK

3

★ VALUES OBTAINED FROM AISC STEEL CONSTRUCTION MANUAL 14TH EDITION

ASSUME $a = 1''$

$\therefore$

$Y2 = t - a/2 = 6.25 - 1/2 = 5.75''$

TRY W16X26 → $\phi M_p = 166$ ft-k [FROM AISC 14TH ED TABLE 3-19]

↳ PNA = BFL $\phi M_N = 307.5$ FT-K

↳ $\Sigma Q_n = 194$ K

$\phi M_n = 307.5$ FT-K > 279 FT-K $\underline{\text{OK}}$ ✓

CHECK a, Y2

$f'_c = 3000$ psi (LW CONC.)

$$a = \frac{\Sigma Q_N}{(.85)\, f'c\, b_{eff}}$$

$$= \frac{194^K}{(.85)(3)(90)} \rightarrow a = .85 < 1.0 \therefore \underline{\text{OK}} ✓$$

$Y2 = 6.25 - \frac{.85}{2} = 5.825 > 5.75$ $\underline{\text{OK}}$ ✓

CHECK STUD #

$$\#\ \text{STUD/BM} = \frac{\Sigma Q_N}{Q_N}$$

$Q_n = 17.2^K$ [FROM TABLE 3-21]
↳ ASSUMING DECK IS ⊥ TO BM + 1 WEAK STUD PER RIB.

$$= \frac{194}{17.2} \rightarrow 11.28 \approx 12 \quad \therefore \text{24 STUDS REQ} < \text{28 STUDS PROVIDED}$$

$\underline{\text{OK}}$ ✓

CHECK SHEAR

$\phi V_N = 106$ kips > $V_u = 37.25^K$ $\underline{\text{OK}}$ ✓ [TABLE 3-2]

CHECK UNSHORED STRENGTH

$$W_u = 1.2(\underset{\substack{\uparrow \\ \text{DECK WT}}}{46})(10) + 1.2(\underset{\substack{\uparrow \\ \text{S.W.}}}{26}) + 1.6(\underset{\substack{\uparrow \\ \text{CONSTRUCTION WEIGHT}}}{20})(10) = .903 \text{ KLF}$$

$$M_u = \frac{(.903)(30)^2}{8} = 101.6 \text{ FT-K} < 166 \text{ FT-K}$$ $\underline{\text{OK}}$ ✓ FOR NO SHORING

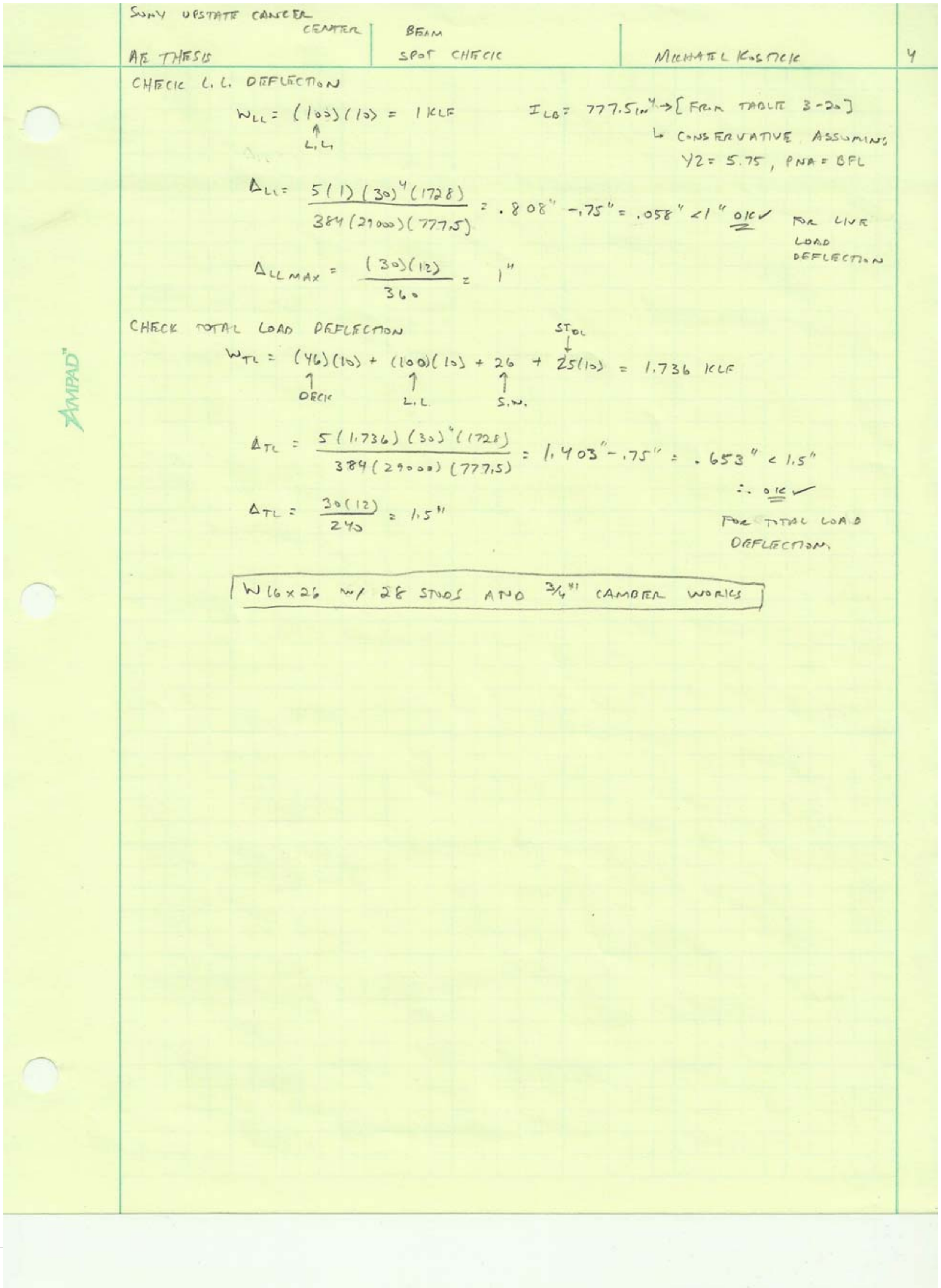
CHECK WET CONCRETE DEFLECTION

$W_{wc} = 46(10) + 26 = .486$ KLF $\quad I = 301 \text{ IN}^4$

$$\Delta_{wc} = \frac{5\, wL^4}{384\, EI} = \frac{5(.486)(30)^4(1728)}{384(29000)(301)} = 1.015'' - .75'' = .265'' < 1.5''$$

$\underline{\text{OK}}$ ✓ FOR W.C. DEFLECTION

$$\Delta_{WCMAX} = \frac{L}{240} = \frac{(30)(12)}{240} = 1.5''$$



CHECK L.L. DEFLECTION

$w_{LL} = (100)(10) = 1\ \text{KLF}$ ← L.L.

$I_{LB} = 777.5\text{in}^4 \rightarrow$ [FROM TABLE 3-20]
↳ CONSERVATIVE ASSUMING Y2 = 5.75, PNA = BFL

$$\Delta_{LL} = \frac{5(1)(30)^4(1728)}{384(29000)(777.5)} = .808'' - .75'' = .058'' < 1''\ \underline{\underline{\text{OK}}}\ ✓$$

FOR LIVE LOAD DEFLECTION

$$\Delta_{LL\,MAX} = \frac{(30)(12)}{360} = 1''$$

CHECK TOTAL LOAD DEFLECTION

$$w_{TL} = \underset{\text{DECK}}{(46)(10)} + \underset{\text{L.L.}}{(100)(10)} + \underset{\text{S.W.}}{26} + \overset{ST_{DL}}{25(10)} = 1.736\ \text{KLF}$$

$$\Delta_{TL} = \frac{5(1.736)(30)^4(1728)}{384(29000)(777.5)} = 1.403'' - .75'' = .653'' < 1.5''$$

$\therefore\ \underline{\underline{\text{OK}}}$ ✓

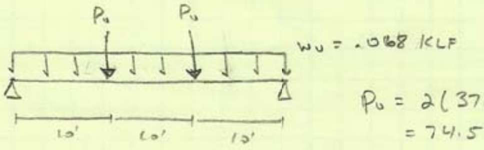
FOR TOTAL LOAD DEFLECTION

$$\Delta_{TL} = \frac{30(12)}{240} = 1.5''$$

$\boxed{\text{W16×26 w/ 28 STUDS AND 3/4'' CAMBER WORKS}}$

CHECK COMPOSITE GIRDER: W 24 x 68 [32] ★ 3/4", 5" LONG STUDS EQUALLY SPACED

- TRIB WIDTH = 30'-0"
- SPAN = 30'-0"
- NO CAMBER



W 24 x 68:

- $A_g = 20.1 \text{ IN}^3$
- F_y = 50 ksi
- $I_y = 1830 \text{ IN}^4$

$P_u = 2(37.25) = 74.5$ K

$$V_U = P_U + \frac{w_u L}{2} = 74.5 + \left[\frac{(.068)(30)}{2}\right]1.2 = 75.72^K$$

$$M_U = P_a + \frac{wL^2}{8} = (74.5)(10) + \left[\frac{(.068)(30)^2}{8}\right]1.2 = 754.18 \text{ FT-K}$$

$$b_R = b_L = \text{MIN OF} \begin{cases} \text{SPAN}/8 = (30)(12)/8 = 45'' \leftarrow \text{CONTROLS} \\ \tfrac{1}{2} \text{ CLR SPAN} = 30(12)/2 = 180'' \end{cases}$$

$\therefore$

$b_{eff} = 45 + 45 = \underline{\underline{90''}}$

ASSUME $a = 1.0 \rightarrow Y2 = 6.25 - 1.0/2 = 5.75''$

TRY W 24 x 68 → ϕM_p = 664 FT-K [FROM TABLE 3-19]
PNA = 7 ϕM_n = 937.5 FT-K [INTERPOLATION]

ϕM_N = 937.5 FT-K > 754.18 FT-K <u>OK</u> ✓
ΣQ_N = 251 K

CHECK a, Y2

$$a = \frac{251}{(.85)(3)(90)} = 1.094 > 1.0 \rightarrow \text{RECALCULATE } \phi M_N.$$

USING a = 1.094

$$Y_2 = 6.25 - \frac{1.094}{2} = 5.703 \rightarrow \text{TRY USING } 5.5 \text{ (CONSERVATIVE)}$$

W24x68 → ϕM_p = 664 FT-K [FROM TABLE 3-19]
PNA = 7 ϕM_n = 933 FT-K → USING Y2 = 5.5
ΣQ_N = 251

ϕM_N = 933 FT-K > 754.18 FT-K <u>OK</u> ✓

CHECK STUD #

$$\#\text{STUDS}/\text{DM} = \frac{\Sigma Q_N}{Q_N} = \frac{251}{17.2} = 14.6 \simeq 15$$

$\therefore$

30 STUDS/DM REQ. < 32 PROVIDED
OK ✓

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AE THESIS

GIRDER SPOT CHECK

MICHAEL KOSTICK

6

CHECK SHEAR

$$\phi V_N = 295\,k > 75.72^k \quad \underline{\underline{OK}}\checkmark \quad (\text{TABLE } 3\text{-}2]$$

CHECK UNSHORED STRENGTH

$$P_U = [1.2(46](10) + 1.2(68) + 1.6(20)(10)] \times 30 = 28.6^k \quad (\text{PT LOAD})$$

$$M_U = P_U(a)$$

$$= 28.6(10^k) = 286\ k\text{-}ft < 664\ k\text{-}ft \quad \underline{\underline{OK}}\checkmark \text{ FOR NO SHORING}$$

CHECK WET CONCRETE DEFLECTION

$$P_{WC} = [(46)(10)(30)] = 13.8^k \quad (\text{PT-LOAD}) \qquad I = 1830\ in^4$$

$$w_{WC} = .068\ KLF$$

$$\Delta_{WC} = \frac{PL^3}{28EI} + \frac{5wL^4}{384EI} = \frac{(13.8)(30)^3(1728)}{28(29000)(1830)} + \frac{5(.068)(30)^4(1728)}{384(29000)(1830)}$$

$$\Delta_{WC} = .457'' < 1.5'' \quad \underline{\underline{OK}}\checkmark \text{ FOR WET CONC DEFLECTION.}$$

$$\Delta_{WC\,MAX} = \frac{(30)(12)}{240} = 1.5''$$

CHECK LIVE LOAD DEFLECTION

$$P_{LL} = [100(10)(30)] = 30\,k \quad (\text{PT-LOAD})$$

$$\Delta_{LL} = \frac{(.30)(30)^3(1728)}{28(29000)(3040)} \qquad I_{LB} = 3040\ in^4 \begin{bmatrix} \text{TABLE } 3\text{-}20 \\ \text{PNA} = 7 \\ Y2 = 5.5 \end{bmatrix}$$

$$= .567'' < 1'' \quad \underline{\underline{OK}}\checkmark \text{ FOR LIVE LOAD DEFLECTION.}$$

$$\Delta_{LL_{MAX}} = \frac{30(12)}{360} = 1''$$

CHECK TOTAL LOAD DEFLECTION

$$P_{TL} = [(46)(10)(30) + (25)(10)(30) + (26)(30) + 100(10)(30)] = 52.1^K$$

$$w_{TL} = .068\ KLF$$

$$\Delta_{TL} = \frac{52.1(30)^3(1728)}{28(29000)(3040)} + \frac{(5)(.068)(30)^4(1728)}{384(29000)(3040)}$$

$$\Delta_{TL} = 1.13'' < 1.5'' \quad \underline{\underline{OK}}\checkmark \text{ FOR TOTAL LOAD DEFLECTION.}$$

$$\Delta_{TL\,MAX} = \frac{(30)(12)}{240} = 1.5''$$

$$\boxed{\text{W } 24\times68 \text{ w/ } 32 \text{ STUDS WORKS}}$$

Appendix B: Pre-Cast Hollow Core Plank on Structural Steel Framing

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ALT. 1
HOLLOW CORE PLANKS

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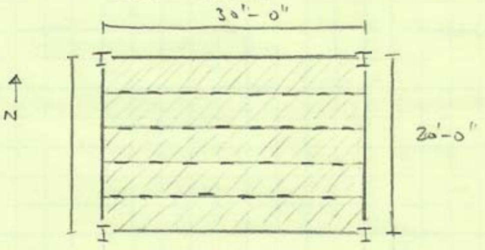
1

HOLLOW CORE PLANKS



(TYPICAL BAY)

RESIZED TYPICAL 30'-0" x 30'-0" BAYS TO ACCOMODATE 4'-0" WIDE HOLLOWCORE CONCRETE PRECAST PLANKS

E-W DIRECT → ORIGINAL BAY DIMENSIONS
N-S DIRECTION → ORIGINALLY [4]-30'-0" BAYS
= 120'-0"
↳ NOW - [6]-20'-0" BAYS
= 120'-0"

★ NOTE: NEED TO ALTER. SIZE OF MRI IMAGING BAYS ★

LOAD: $LL = 100\ psf$
$SI_{DL} = 25\ psf$
$125\ psf$

★ USE A 2" TOPPING FOR FLOOR FINISH.

USING SPANCRETE HOLLOW CORE PLANKS
↳ SPAN = 30'-0"
↳ 2 HR UNRESTRAINED FIRE RATING
↳ 2" CONCRETE TOPPING (FINISH)
↳ 125 psf S.I. LOAD.

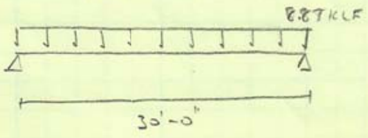
- USE 8" STANDARD SPANCRETE, 1.75" STRAND COVER w/ 2" MIN STRUCTURAL TOPPING @ MIDSPAN.
- STRAND SERIES → 1.75 0- 8712T
  (SEE CUT SHEET)
  ↳ $129_{psf} > 125_{psf}$ OK✓
- ASSEMBLY WT = 88 psf

DESIGN GIRDERS

LOADS: DL: 88 psf - PLANKS
25 psf - SI_{DL}
113 psf
x 30'
3.39 KLF

LL: 100 psf - FLOORS
100 psf
x 30'
3.00 KLF

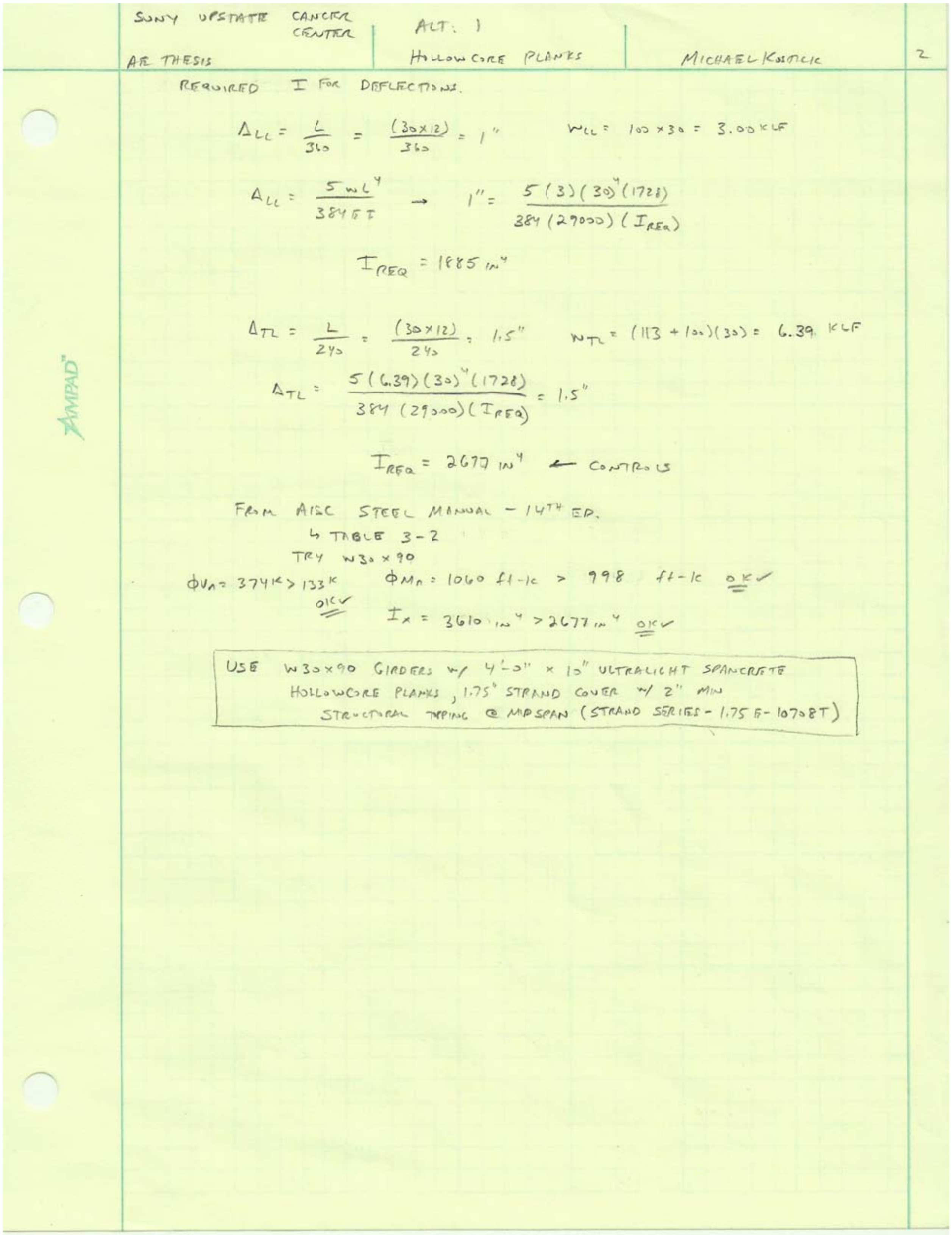
$W_u: 1.2(3.39) + 1.6(3.00) = 8.87\ KLF$



$$M_u = \frac{wL^2}{8} = \frac{(8.87)(30)^2}{8} = 1998\ ft\text{-}k$$

$$V_u = \frac{wL}{2} = \frac{(8.87)(30)}{2} = 133^k$$

★ USE SPAN CONNECTION DETAIL → H-31 ★
(ASSUME PINNED)



SUNY UPSTATE CANCER CENTER
AE THESIS

ALT: 1
HOLLOW CORE PLANKS

MICHAEL KOSTICK

2

REQUIRED I FOR DEFLECTIONS.

$$\Delta_{LL} = \frac{L}{360} = \frac{(30\times 12)}{360} = 1'' \qquad w_{LL} = 100 \times 30 = 3.00\,\text{KLF}$$

$$\Delta_{LL} = \frac{5wL^4}{384EI} \rightarrow 1'' = \frac{5(3)(30)^4(1728)}{384(29000)(I_{REQ})}$$

$$I_{REQ} = 1885\,\text{in}^4$$

$$\Delta_{TL} = \frac{L}{240} = \frac{(30\times 12)}{240} = 1.5'' \qquad w_{TL} = (113+100)(30) = 6.39\,\text{KLF}$$

$$\Delta_{TL} = \frac{5(6.39)(30)^4(1728)}{384(29000)(I_{REQ})} = 1.5''$$

$$I_{REQ} = 2677\,\text{in}^4 \leftarrow \text{CONTROLS}$$

FROM AISC STEEL MANUAL – 14TH ED.
↳ TABLE 3-2
TRY W30×90

$\phi V_n = 374^K > 133^K$ OK✓

$\phi M_n = 1060$ ft-k > 998 ft-k OK✓

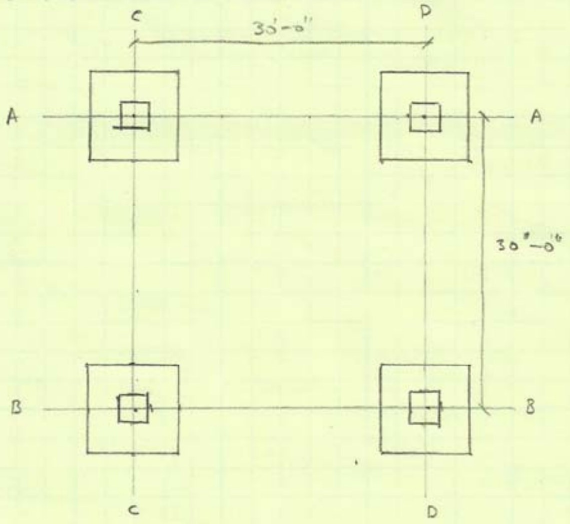
$I_x = 3610\,\text{in}^4 > 2677\,\text{in}^4$ OK✓

$$\boxed{\text{USE W30×90 GIRDERS w/ 4'-0" × 10" ULTRALIGHT SPANCRETE HOLLOWCORE PLANKS, 1.75" STRAND COVER w/ 2" MIN STRUCTURAL TOPPING @ MIDSPAN (STRAND SERIES - 1.75 E-10708T)}}$$

Appendix C: Two-Way Flat Slab with Drop Panels

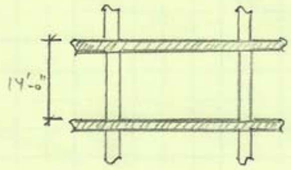
SUNY UPSTATE CANCER CENTER
ALT FLOOR #2
FLAT SLAB W/ DROP PANELS
MICHAEL KOSTICK
1

USE FLAT SLAB W/ DROP PANELS



☆ DATA FROM ACI 318-08

ASSUME COLUMN SIZE OF 24"×24" (CONCRETE)
↳ FROM "PRELIMINARY SITECAST CONCRETE SIZE GUIDLINE"



FLR - FLR = 14'-0"

LL: 100 psf DL: 25 psf SDL
SLAB WT

$f_y = 60000 \text{ psi}$ $f'_c = 4000 \text{ psi}$

SQUARE BAY → $L_1 = L_2$ → BOTH DIRECTIONS ARE THE SAME

DETERMINE SLAB THICKNESS:
↳ SLAB - NO INTERIOR BM, W/ DROP PANELS

$L_n = S_n \therefore L_n/S_n = 1.0 < 2.0$ → USE TABLE 9.5(c)

ACI 318: 9.5.3.2

$$t = \text{MAX OF} \begin{cases} 4'' \rightarrow \text{SLAB W/ DROP PANEL} \\ \dfrac{l_n}{36} \rightarrow \text{SLAB W/ DROP PANEL} \end{cases}$$

↳ INT PANEL W/ $f_y = 60000$ psi

$$l_n = 30 - 2\left(\frac{2'}{2}\right) = 28'$$

$$t = \text{MAX OF} \begin{cases} 4'' \\ \dfrac{28 \times 12}{36} = 9.33'' \end{cases} \rightarrow \text{USE } 9.5'' \text{ SLAB}$$

DETERMINE DESIGN PROCEDURE ACI 318-08: 13.6.1

DIRECT DESIGN METHOD:

- MIN 3 CONTINUOUS SPANS IN EACH DIRECTION ✓
- PANEL SPAN RATIO ≤ 2 ✓
- SUCCESIVE SPANS DON'T DIFFER BY MORE THAN 1/3 LENGTH ✓
- OFFSET COLUMNS NO GREATER THAN 10% ✓
- ONLY GRAVITY LOADING - UNIFORM ✓
- UNFACTOR LL ≤ 2 UNFACTORED D.L ✓

LL = 100 psf

DL = 25 + 15 psf

$$\text{SLAB WT} = \frac{9.5}{12} \times (150) = 119 \text{ psf}$$

$$100 \le 2(119 + 25)$$ OK ✓

SUNY UPSTATE CANCER CENTER
AE THESIS

ALT FLOOR #2
FLAT SLAB w/DROP PANELS

MICHAEL KOSTICK

2

FIND MOMENTS IN SLAB $L_N = 28'$ $L_2 = 30'$ $L_1 = 30'$

$w_L = 100$ psf
$w_D = 119 + 25 = 144$ psf

$w_u = 1.2DL + 1.6LL$
$= 1.2(144) + 1.6(100)$
$= 333$ psf

ALL FRAMES: $M_o = \frac{1}{8} w_u L_2 L_N^2 = \frac{1}{8}(.333)(30)(28)^2 = 979$ ft-k

ACI 318-08: 13.6.3.2 → INTERIOR SPAN - STATIC MOMENT
DISTRIBUTED → .65 - NEGATIVE
.35 - POSITIVE

FRAME A-A, B-B, C-C, D-D (ft-k)

.65	.35	.65
-636	343	-636

FIND COLUMN + MIDDLE STRIP WIDTHS → ALL FRAME EQUAL.

TRIB WIDTH = 30'-0" COLUMN STRIP = 15' MIDDLE STRIP = 15'

$$\frac{30(12)}{4} = 90'' \rightarrow 7.5'$$

$$\frac{L_1}{L_2} = \frac{30}{30} = 1.0 \qquad \alpha = 0 \quad B/c \quad I_b = 0 \text{ [NO BEAMS]}$$

$$\therefore \frac{L_1}{L_2}\alpha = 0 \qquad \beta_t = \frac{C}{2 I_s} = 0$$

NEGATIVE MOMENT @ INTERIOR SPAN

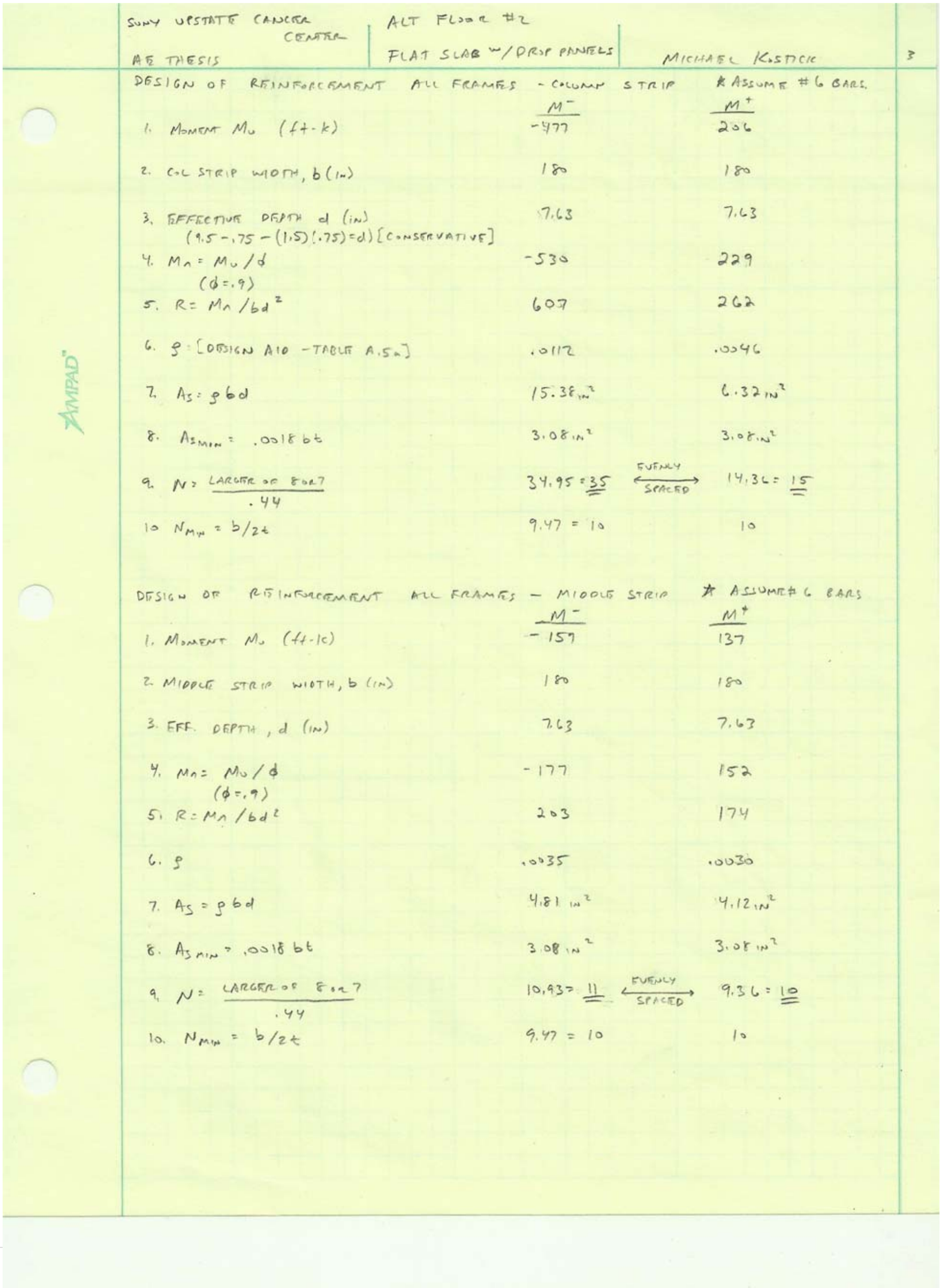
ACI 318: 13.6.4.1 { ↳ 75% TO COL. STRIP → (B/c $L_1/L_2 \alpha = 0$ → 100% TO SLAB)
↳ REMAINS 25% TO MID STRIP SLAB.

POSITIVE MOMENT FOR INT. SPAN

ACI 318: 13.6.4.4 { ↳ 60% TO COL. STRIP → (100% TO SLAB)
↳ 40% TO MID STRIP.

DISTRIBUTION → ALL FRAMES (ft-k)

TOTAL MOMENT	-636	343	-636
MOM. IN COL STRIP	-477	206	-477
MOM IN MID STRIP	-159	137	-159



SUNY UPSTATE CANCER CENTER
AE THESIS

ALT FLOOR #2
FLAT SLAB w/ DROP PANELS

MICHAEL KOSTICK

3

DESIGN OF REINFORCEMENT ALL FRAMES - COLUMN STRIP ★ASSUME #6 BARS.

	M^-		M^+
1. MOMENT M_u (ft-k)	-477		206
2. COL STRIP WIDTH, b (in)	180		180
3. EFFECTIVE DEPTH d (in) (9.5 - .75 - (1.5)(.75) = d) [CONSERVATIVE]	7.63		7.63
4. $M_n = M_u / \phi$ (ϕ = .9)	-530		229
5. $R = M_n / bd^2$	607		262
6. ρ : [DESIGN A10 - TABLE A.5a]	.0112		.0046
7. $A_s = \rho b d$	15.38 in^2		6.32 in^2
8. $A_{s_{MIN}}$ = .0018 bt	3.08 in^2		3.08 in^2
9. $N = \frac{\text{LARGER OF 8 OR 7}}{.44}$	34.95 = $\underline{\underline{35}}$	$\xleftrightarrow[\text{SPACED}]{\text{EVENLY}}$	14.36 = $\underline{\underline{15}}$
10 $N_{MIN} = b/2t$	9.47 = 10		10

DESIGN OF REINFORCEMENT ALL FRAMES - MIDDLE STRIP ★ ASSUME #6 BARS

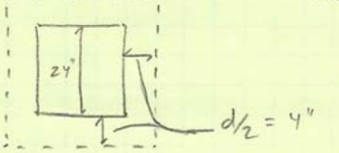
	M^-		M^+
1. MOMENT M_u (ft-k)	-157		137
2. MIDDLE STRIP WIDTH, b (in)	180		180
3. EFF. DEPTH, d (in)	7.63		7.63
4. $M_n = M_u / \phi$ (ϕ = .9)	-177		152
5. $R = M_n / bd^2$	203		174
6. ρ	.0035		.0030
7. $A_s = \rho b d$	4.81 in^2		4.12 in^2
8. $A_{s_{MIN}}$ = .0018 bt	3.08 in^2		3.08 in^2
9. $N = \frac{\text{LARGER OF 8 OR 7}}{.44}$	10.93 = $\underline{\underline{11}}$	$\xleftrightarrow[\text{SPACED}]{\text{EVENLY}}$	9.36 = $\underline{\underline{10}}$
10. $N_{MIN} = b/2t$	9.47 = 10		10

SUNY UPSTATE CANCER CENTER
AE THESIS

ALT FLOOR #2
FLAT SLAB W/ DROP PANELS

MICHAEL KOSTICK

CHECK SHEAR – TWO WAY PUNCHING SHEAR



$d_{avg} = d = 9.5 - .75 - .75 = 8"$

$b_o = 2(32 + 32) = 128"$

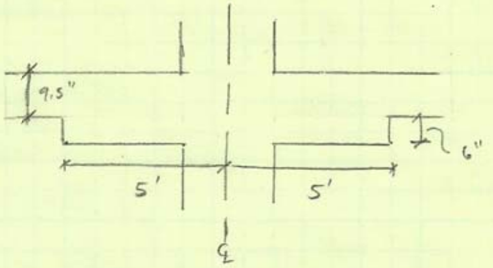
$A_T = (30 \times 30) - \left(\frac{32}{12} \times \frac{32}{12}\right) = 892.8 \text{ ft}^2$

$V_u = .333(892.8) = 297.3 \text{ kips}$

$$V_c = \text{SMALLEST OF} \begin{cases} 4\lambda\sqrt{f'c}\,b_o d = (4)(1)\sqrt{4000}(128)(8) = 259^k \\ \left(2 + \frac{4}{\beta}\right)\lambda\sqrt{f'c}\,b_o d = \left(2 + \frac{4}{1}\right)(1)\sqrt{4000}(128)(8) = 387^k \\ \quad \beta = \frac{24}{24} = 1.0 \\ \left(\frac{\alpha_s d}{b_o} + 2\right)\lambda\sqrt{f'c}\,b_o d = \left(\frac{40(8)}{128} + 2\right)(1)\sqrt{4000}(128)(8) = 291^k \\ \quad \alpha_s = 40 \text{ [INT. COLUMN]} \end{cases}$$

$V_c = 259^k$

$V_u = 297.3 K \geq \phi V_c = (.75)(259) = 194^k$ NO GOOD.
ADD DROP PANELS



$l/6 = \frac{30}{6} = 5'-0"$

$h_d = \frac{9.5}{4} = 2.38"$ MIN
↓
USE 6 INCHES.

$9.5 + 6 = 15.5"$

$d = 15.5 - .75 - .75 = 14"$

$b_o = 2(38 + 38) = 152"$

$A_T = (30 \times 30) - \left(\frac{38}{12} \times \frac{38}{12}\right) = 890 \text{ ft}^2$

$V_u = .333(890) = 296.4 \text{ kips}$

$$V_c = \text{SMALLEST OF} \begin{cases} 4(1)(\sqrt{4000})(152)(14) = 538^k \\ \left(2 + \frac{4}{1}\right)(1)\sqrt{4000}(152)(14) = 808^k \\ \left(\frac{(40)(14)}{152} + 2\right)(1)\sqrt{4000}(152)(14) = 765^k \end{cases}$$

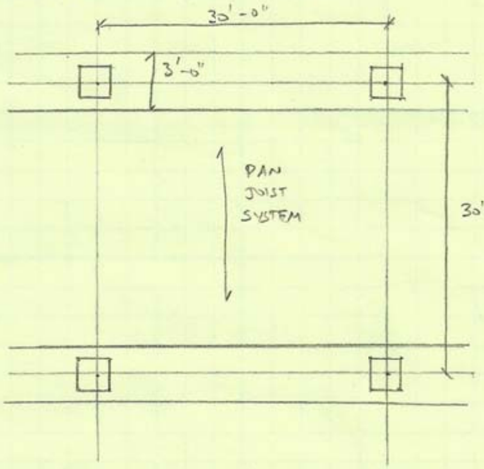
$V_c = 538^k$

$V_u = 296.4^k \leq \phi V_c = (.75)(538) = 403^k$ OK ✓

$\boxed{\text{USE 6.25' PANELS FOR EASE OF CONSTRUCTION}}$

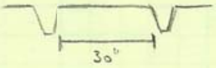
Appendix D: One-Way Pan Joist and Beam

ONE-WAY PAN JOIST SYSTEM.



BAYS REMAINED UNCHANGED
ASSUMING 24"×24" COLUMNS.
MAX SPAN LENGTH → 30'0"
↳ MIN PAN DEPTH = 16"
★(FANELLA , PCA) - CONC. FLR SYS. ★
↓ ↓
ASSUME 36" (3'-0") GIRDER

$f'_c = 4000$ psi;
$f_y = 60000$ psi



SLAB THICKNESS : → t= 4.5" → 2 HR FIRE RATING
↳ USE 30" PANS

LOADS:

DL: SI. DL = 25 psf LL: 100 psf

SLAB WT = $\frac{4.5}{12}(150) = 56.25$ psf

$W_u = 1.2(25+56.25) + 1.6(100) = 258$ psf

DESIGN A 1' STRIP

$258\text{psf}\,(1') = 258\,\text{PLF} = w_u$

ACI 318 : 8.3.3

$$\begin{cases} M_u^- = \dfrac{w_u L_n^2}{11} = \dfrac{(.258)(30/12)^2}{11} = .147 \text{ ft-k / ft of slab} \\ M_u^+ = \dfrac{w_u L_n^2}{16} = \dfrac{(.258)(30/12)^2}{16} = .101 \text{ ft-k / ft of slab} \end{cases}$$

MIN REINFORCEMENT → $.0018\,A_g = \rho_{min}$ (S+T) (ACI 318 : 10.5.4)

$= .0018(4.5)(12") = .0972 \text{ in}^2$

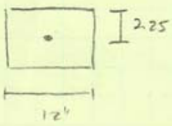
#3 BAR = $.11 \text{ in}^2$

MAX SPACING :

$3t = 3(4.5) = 13.5'$ ← CONTROLS.

$5t$ (S+T) $= 5(4.5) = 22.5"$

$d = \frac{4.5}{2} = 2.25"$



TRY 1 #3 @ 12"

★ REINFORCEMENT BAR @ MID HEIGHT OF SLAB★
(FANELLA)

SUNY UPSTATE CANCER CENTER | ALT FLOOR #3 |
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$a = \frac{A_s f_y}{.85 f'_c b} = \frac{(.11)(60)}{.85(4)(12)} = .162''$

$c = \frac{.162}{.85} = .191'' < .375d = .375(2.25) = .844''$

$\therefore \varepsilon_t > .005 \rightarrow \phi = .9$

$\phi M_n = \phi A_s f_y (d - a/2)$

$= .9(.1)(60)(2.25 - \frac{.162}{2}) = 12.88$ in-k $= 1.07$ ft-k

.147 ft-k / ft of slab > 1.07 ft-k / ft of slab

SIZE JOIST

Min depth $= \frac{l_n}{21}$ [ACI 318: Table 9.5(a) → Both ends continuous ribbed one-way slab]

$l_n = 30 - 2(\overset{36'' \text{ BM}}{3/2}) = \underline{27'}$

Min depth $= \frac{27 \times 12}{21} = 15.4''$ → use 16" pan joist (specified earlier)

↳ try 5" thickness → (Fanella chart)

LOADING:

℄ trib width = 30" + 5" web thickness

DL:

$SI_{DL} = (25)(35/12) = 72.92$ PLF

Slab wt $= (56.25)(35/12) = 164$ PLF

Joist wt $= \frac{(16)(5)(150)}{144} = 83.33$ PLF

L.L.:

$= 100(35/12) = 292$ PLF

DL = 320 PLF   LL: 292 PLF

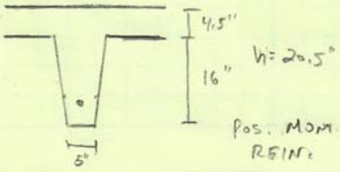
$w_u = 1.2(320) + 1.6(292) = .851$ KLF

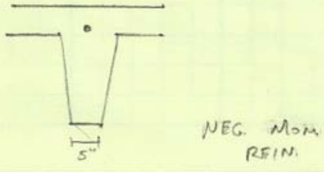
Using ACI 318: 8.3.3 → moment coefficients for interior span

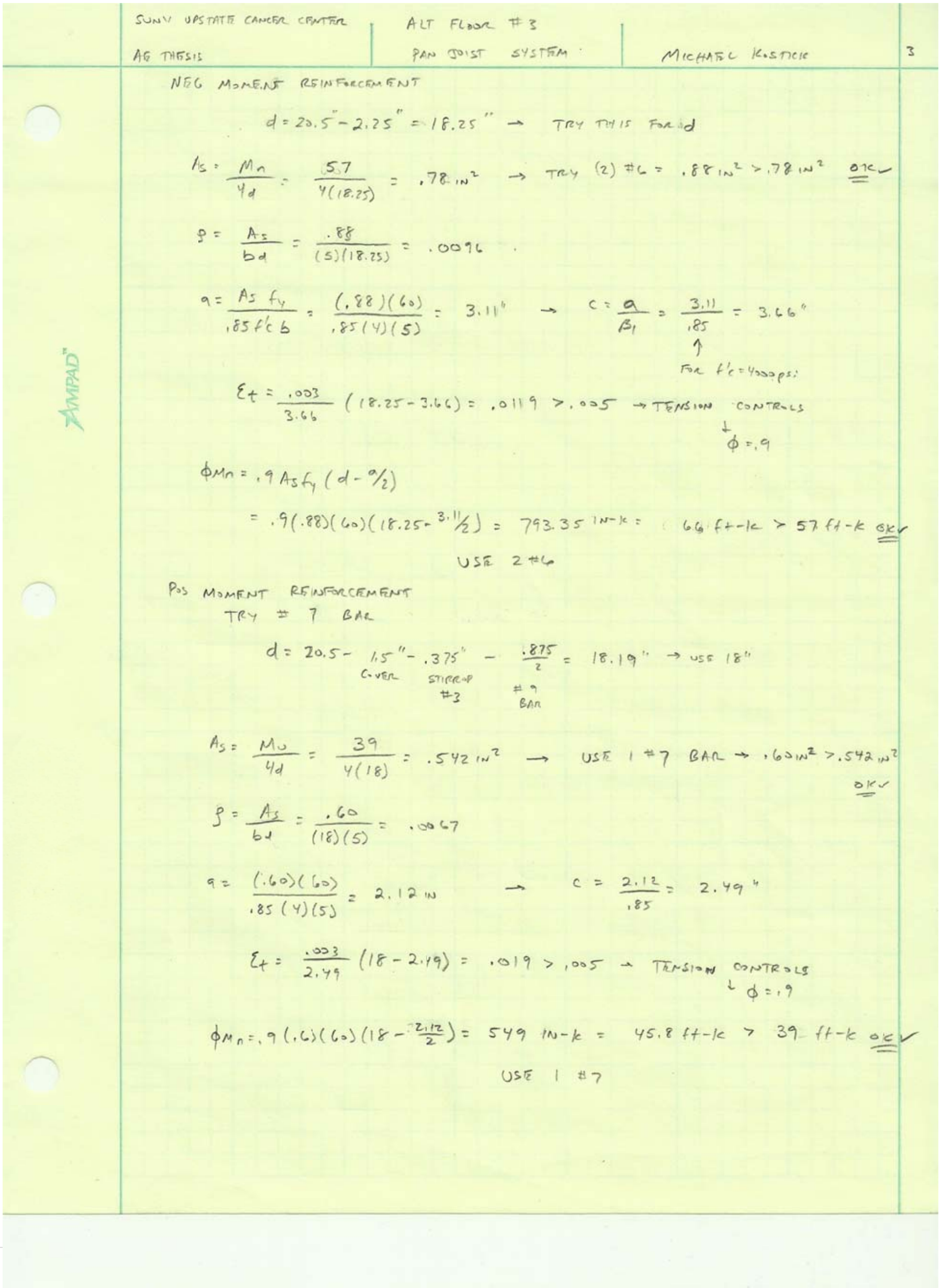
$M^-_{n\,max} = \frac{w_u L_n^2}{11} = \frac{(.851)(27)^2}{11} = 57$ ft-k

$M^+_{n\,max} = \frac{w_u L_n^2}{16} = \frac{(.851)(27)^2}{16} = 39$ ft-k

B/C moments were approximated → no redistribution (ACI 318: 8.4.1)







SUNY UPSTATE CANCER CENTER
AE THESIS
ALT FLOOR #3
PAN JOIST SYSTEM
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3

NEG MOMENT REINFORCEMENT

$d = 20.5'' - 2.25'' = 18.25''$ → TRY THIS FOR d

$A_s = \frac{M_n}{4d} = \frac{57}{4(18.25)} = .78 \text{ in}^2$ → TRY (2) #6 = $.88 \text{ in}^2 > .78 \text{ in}^2$ OK ✓

$\rho = \frac{A_s}{bd} = \frac{.88}{(5)(18.25)} = .0096$

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{(.88)(60)}{.85(4)(5)} = 3.11''$ → $c = \frac{a}{\beta_1} = \frac{3.11}{.85} = 3.66''$

↑ FOR $f'_c = 4000$ psi

$\varepsilon_t = \frac{.003}{3.66}(18.25 - 3.66) = .0119 > .005$ → TENSION CONTROLS

↓ $\phi = .9$

$\phi M_n = .9 A_s f_y (d - a/2)$

$= .9(.88)(60)(18.25 - 3.11/2) = 793.35 \text{ in-k} = 66 \text{ ft-k} > 57 \text{ ft-k}$ OK ✓

USE 2 #6

POS MOMENT REINFORCEMENT

TRY # 7 BAR

$d = \underset{\text{COVER}}{20.5 - 1.5''} - \underset{\text{STIRRUP \#3}}{.375''} - \underset{\text{\#9 BAR}}{\frac{.875}{2}} = 18.19''$ → USE 18"

$A_s = \frac{M_u}{4d} = \frac{39}{4(18)} = .542 \text{ in}^2$ → USE 1 #7 BAR → $.60 \text{ in}^2 > .542 \text{ in}^2$ OK ✓

$\rho = \frac{A_s}{bd} = \frac{.60}{(18)(5)} = .0067$

$a = \frac{(.60)(60)}{.85(4)(5)} = 2.12 \text{ in}$ → $c = \frac{2.12}{.85} = 2.49''$

$\varepsilon_t = \frac{.003}{2.49}(18 - 2.49) = .019 > .005$ → TENSION CONTROLS

↓ $\phi = .9$

$\phi M_n = .9(.6)(60)(18 - \frac{2.12}{2}) = 549 \text{ in-k} = 45.8 \text{ ft-k} > 39 \text{ ft-k}$ OK ✓

USE 1 #7

SUNY UPSTATE CANCER CENTER
AE THESIS

ALT FLOOR #3
PAN JOIST SYSTEM

MICHAEL KOSTICK

4

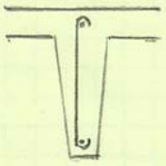
SHEAR CHECK

$$d = 20.5 - \overset{\text{COVER}}{1.5} - \underset{\#3\ \text{STIRRUP}}{.375} - \underset{\#7\ \text{BAR}}{\frac{1.875}{2}} = \underset{\text{USE } 18''}{18.19''}$$

$$V_u = \frac{w\,l_n}{2} = \frac{(.851)(27)}{2} = 11.5^k$$

$$\phi V_c = \phi\, 2\sqrt{f'c}\; b_w d$$

$$= (.75)(2)\sqrt{4000}\,(5)(18) = 8.54^k$$



$$\phi V_s = V_n - \phi V_c = 11.5 - 8.54$$

$$\phi V_s = 2.96^k$$

$$S_{MAX} = d/2 = \frac{18}{2} = 9'' \rightarrow \text{USE } 9''$$

$$\phi V_s = \frac{\phi A_s f_y d}{S_{MAX}} \rightarrow 2.96 = \frac{(.75)(A_{sv})(60)(18)}{9''} \rightarrow A_{sv} = .033\,in^2 < \underset{\#3\ \text{STIRRUP}}{.11\,in^2}$$

O.K.✓

USE ONE LEG #3 STIRRUP.

DESIGN GIRDER

$$w_{u\,FLOOR} = \frac{851}{\left(\frac{35}{12}\right)}(27') = 7.88 \text{ KLF}$$

$$w_{u\,GIRDER\,LL} = 1.6(100)(3) = .48 \text{ KLF}$$

$$w_{u\,GIRDER} = 1.2(25)(3) = .09 \text{ KLF}$$

$$S.W. = 1.2\left[\frac{36(20.5)(150)}{144}\right] = .923 \text{ KLF}$$

$$w_u = 7.88 + .09 + .923 + .48 = 9.37 \text{ KLF}$$

$$l_n = 30' - 2(2/2) = 28'$$

$$M_{n\,MAX}^{-} = \frac{(9.37)(28)^2}{11} = 668 \text{ ft-k}$$

$$M_{n\,MAX}^{+} = \frac{(9.37)(28)^2}{16} = 459 \text{ ft-k}$$

SIZE REINFORCEMENT NEG. MOMENT

$$\text{ASSUME } d = 20.5 - \underset{\text{COVER}}{1.5} - \underset{\#4\ \text{STIRRUP}}{.5} - \underset{\#8\ \text{BAR}}{\frac{1.128}{2}} = 17.9$$

$$A_s = \frac{M_u}{4d} \rightarrow \frac{668}{4(17.9)} = 9.33\,in^2 \rightarrow \text{TRY USING 10 \#9 BARS}$$

$$9.33\,in^2 < 10.0\,in^2$$

O.K.✓

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{(10.0)(60)}{.85(4)(36)} = 4.90'' \rightarrow c = \frac{4.90}{.85} = 5.76''$

$\varepsilon_t = \frac{.003}{5.76}(17.9 - 5.76) = .0063 > .005 \rightarrow$ TENSION CONTROLLED SECTION $\downarrow$ $\phi = .9$

$\phi M_n = (.9)(10.0)(60)(17.9 - \frac{4.90}{2}) = 8343$ in-k $= 695$ ft-k > 668 ft-k OK✓

USE 10 #9 BARS

POSITIVE MOMENT REINFORCEMENT

$A_s = \frac{(459)}{4(17.9)} = 6.41$ in² → TRY USING 7 #9 → 7.00 in² > 6.41 in² OK✓

$a = \frac{(7.00)(60)}{.85(4)(36)} = 3.43'' \rightarrow c = \frac{3.43}{.85} = 4.03''$

$\varepsilon_t = \frac{.003}{4.03}(17.9 - 4.03) = .0103 > .005 \rightarrow$ T.C.S. $\rightarrow \phi = .9$

$\phi M_n = (.9)(7.0)(60)(17.9 - \frac{3.43}{2}) = 6118$ in-k $= 509$ ft-k > 459 ft-k OK✓

USE 7 #9 BARS

CHECK SHEAR

$V_n = \frac{(9.37)(28)}{2} = 131^k$

$\phi V_c = .75(2)\sqrt{4000}(36)(17.9) = 61^k$

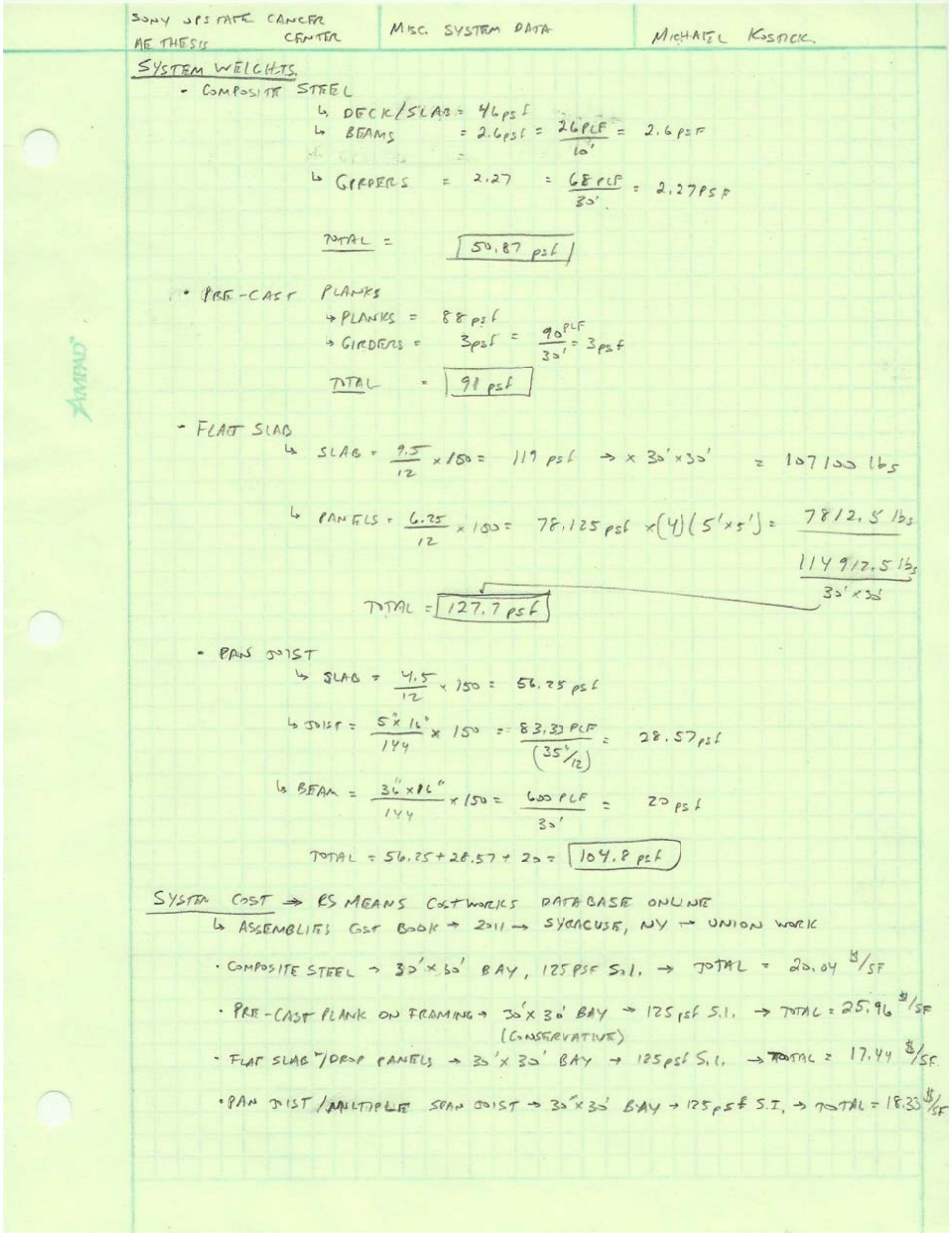
$\phi V_s = 131 - 61 = 70^k$ $S_{max} = \frac{17.9}{2} = 8.95'' \rightarrow$ USE 8.75"

$\phi V_s = \frac{\phi A_s f_y d}{S_{max}} \rightarrow 70 = \frac{(.75)(A_s)(60)(17.9)}{8.75} \rightarrow A_s = .760$ in²

TRY 2 LEGS → #4 → 2(.5) = 1.00 in² > .760 in² OK✓

USE #4 STIRRUP (2 LEGS) @ 8.75"

Appendix E: Miscellaneous Data



SUNY UPSTATE CANCER CENTER
AE THESIS

MISC. SYSTEM DATA

MICHAEL KOSTICK.

SYSTEM WEIGHTS.

- COMPOSITE STEEL
  - ↳ DECK/SLAB = 46 psf
  - ↳ BEAMS = 2.6 psf = $\frac{26 \text{ PLF}}{10'}$ = 2.6 psf
  - ↳ GIRDERS = 2.27 = $\frac{68 \text{ PLF}}{30'}$ = 2.27 psf

  TOTAL = $\boxed{50.87 \text{ psf}}$

- PRE-CAST PLANKS
  - ↳ PLANKS = 88 psf
  - ↳ GIRDERS = 3 psf = $\frac{90^{\text{PLF}}}{30'}$ = 3 psf

  TOTAL = $\boxed{91 \text{ psf}}$

- FLAT SLAB
  - ↳ SLAB = $\frac{9.5}{12} \times 150$ = 119 psf → × 30' × 30' = 107100 lbs
  - ↳ PANELS = $\frac{6.25}{12} \times 150$ = 78.125 psf × (4)(5' × 5') = 7812.5 lbs

  $\frac{114912.5 \text{ lbs}}{30' \times 30'}$

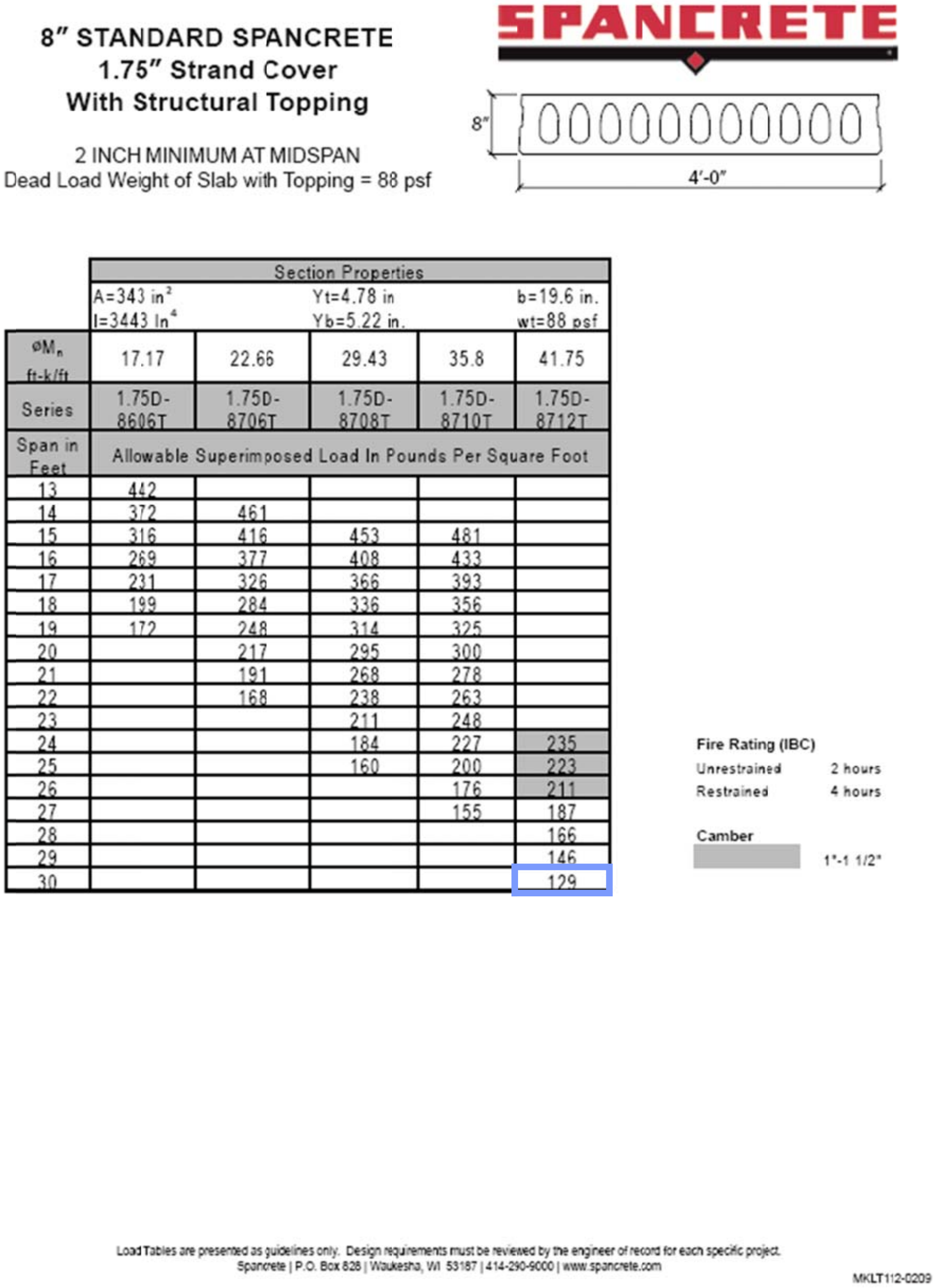
  TOTAL = $\boxed{127.7 \text{ psf}}$

- PAN JOIST
  - ↳ SLAB = $\frac{4.5}{12} \times 150$ = 56.25 psf
  - ↳ JOIST = $\frac{5'' \times 16''}{144} \times 150$ = 83.33 PLF / $(35''/12)$ = 28.57 psf
  - ↳ BEAM = $\frac{36'' \times 16''}{144} \times 150$ = $\frac{600 \text{ PLF}}{30'}$ = 20 psf

  TOTAL = 56.25 + 28.57 + 20 = $\boxed{104.8 \text{ psf}}$

SYSTEM COST → RS MEANS COSTWORKS DATABASE ONLINE
↳ ASSEMBLIES COST BOOK → 2011 → SYRACUSE, NY → UNION WORK

- COMPOSITE STEEL → 30' × 30' BAY, 125 PSF S.I. → TOTAL = 20.04 $/SF
- PRE-CAST PLANK ON FRAMING → 30' × 30' BAY → 125 psf S.I. → TOTAL = 25.96 $/SF
  (CONSERVATIVE)
- FLAT SLAB W/DROP PANELS → 30' × 30' BAY → 125 psf S.I. → TOTAL = 17.44 $/SF
- PAN JOIST/MULTIPLE SPAN JOIST → 30' × 30' BAY → 125 psf S.I. → TOTAL = 18.33 $/SF



SPANCRETE

8" STANDARD SPANCRETE
1.75" Strand Cover
With Structural Topping

2 INCH MINIMUM AT MIDSPAN
Dead Load Weight of Slab with Topping = 88 psf

	Section Properties				
	A=343 in^2 I=3443 In^4		Yt=4.78 in Yb=5.22 in.		b=19.6 in. wt=88 psf
ϕM_n ft-k/ft	17.17	22.66	29.43	35.8	41.75
Series	1.75D-8606T	1.75D-8706T	1.75D-8708T	1.75D-8710T	1.75D-8712T
Span in Feet	Allowable Superimposed Load In Pounds Per Square Foot				
13	442				
14	372	461			
15	316	416	453	481	
16	269	377	408	433	
17	231	326	366	393	
18	199	284	336	356	
19	172	248	314	325	
20		217	295	300	
21		191	268	278	
22		168	238	263	
23			211	248	
24			184	227	235
25			160	200	223
26				176	211
27				155	187
28					166
29					146
30					129

Fire Rating (IBC)

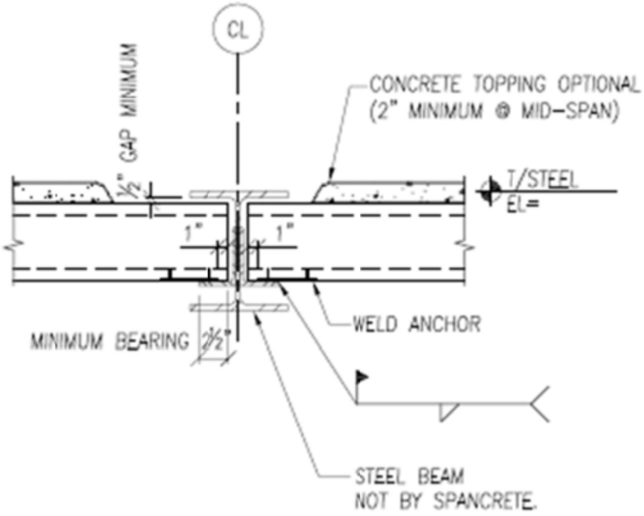
Unrestrained	2 hours
Restrained	4 hours

Camber

▨ 1"-1 1/2"

Load Tables are presented as guidelines only. Design requirements must be reviewed by the engineer of record for each specific project.
Spancrete | P.O. Box 828 | Waukesha, WI 53187 | 414-290-9000 | www.spancrete.com

MKLT112-0209



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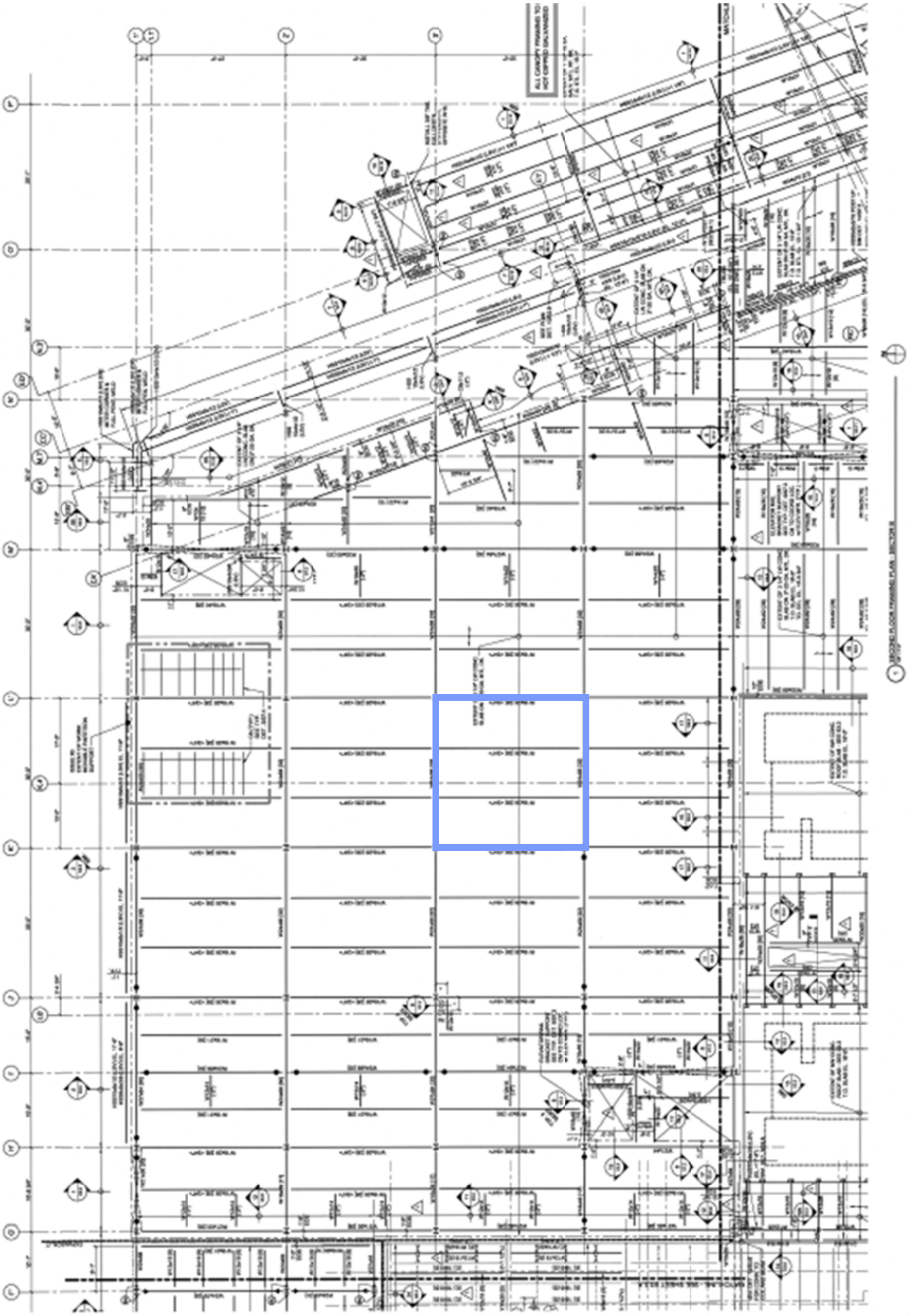

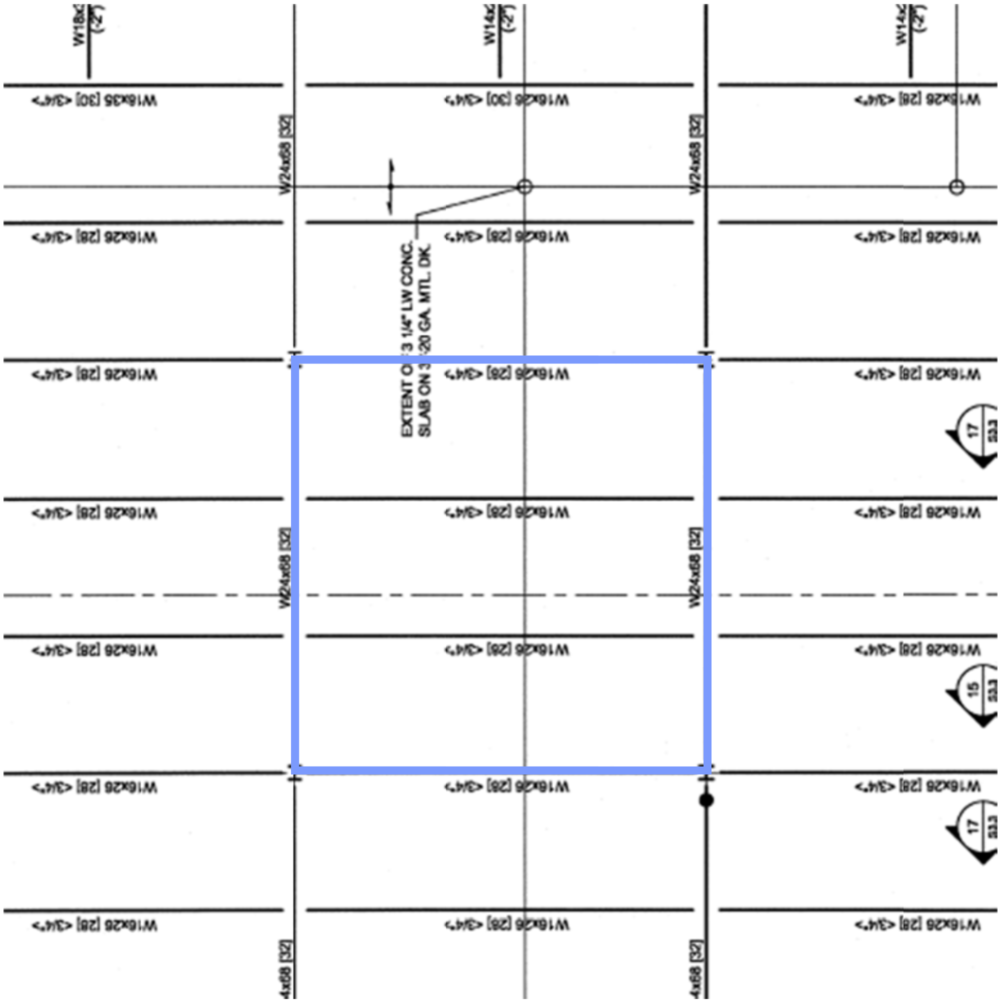
W18x
(-2")
W14x
(-2")
W14x
(-2")
W16x35 [30] <3/4>
W16x26 [30] <3/4>
W16x26 [28] <3/4>
W24x68 [32]
EXTENT O 3 1/4" LW CONC.
SLAB ON 3 20 GA. MTL. DK.
17
S3.3
15
S3.3
17
S3.3