360 State Street

New Haven, Connecticut

PRO-CON STUDY OF ALTERNATE FLOOR SYSTEMS

STRUCTURAL TECHNICAL REPORT II

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28 OCTOBER 2009

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Senior Thesis: www.engr.psu.edu/ae/thesis/portfolios/2010/szd125/index.html

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EXECUTIVE SUMMARY

The intent of this report is to research, analyze, and compare various floor systems to the existing conditions of 360 State Street located in New Haven, Connecticut. The building is a new landmark for the city consisting of street level retail, parking, and residential living space. The design couples sustainable resources and tactics with location and architectural allure. Overall, the building reaches 32 stories and is a mixture of public and private spaces.

The objective of this report is to recommend an alternative solution to the existing floor system. Research has been gathered for the following:

- ◊ Hollow Core Planks on Staggered Steel Trusses (existing)
- ◊ Composite Cellular Beams
- ◊ Hambro® Composite Floor Systems
- ◊ Girder-Slab
- ◊ Two-Way Flat Plate

A variety of floor systems have been designed throughout 360 State Street. The base of the building consists of cast-in-place and post-tensioned slabs that were specifically designed to handle the heavy loads of an open air parking garage. The remaining portion of the building is composed of hollow core planks. To simplify this report, only the floor system of the residential tower will be considered.

The existing floor system consists of 8" hollow core plank on staggered steel trusses which were incorporated into the architectural design. Often seen in buildings with doubly loaded center corridors, the trusses are able to maintain design flexibility in the interior spaces while fulfilling several structural functions. Alternative floor systems could capitalize on the characteristics where the existing system fell short; such as, improving the floor-to-floor height or decreasing the overall building weight, without sacrificing rigidity.

To become a viable candidate, each alternative floor system had to have similar characteristics to the existing system. In addition, each system had to improve a quality that did not diminish the integrity of the structure. Cellular beams were considered because they can increase rigidity without additional weight. The Hambro® Composite System was chosen based on its unique design to achieve full composite action. The Girder-Slab system was selected on its ability to minimize the floor depth while maintaining strength. And lastly, a flat plate system was considered for its low material costs. Overall, a typical bay was designed for each system to evaluate.

Comparing the systems' non-structural advantages to their structural capabilities, the Girder-Slab system is recommended for further investigation as an alternative floor system. It optimizes the floor-to-floor height and slightly decreases the overall building weight. Although it requires additional columns, it does not negatively impact the current interior floor plan. Girder-Slabs are comparable to the existing floor system and have the ability to enhance the structure furthermore.

ARCHITECTURAL INTRODUCTION

360 State Street is an innovative building project by the firm Becker + Becker Associates. Located in downtown New Haven, Connecticut, it is situated on the corner of Chapel and State Street just two blocks east of the historic town green. It is also located across the street from an Amtrak train station which services lines to New York and Boston. 360 State Street is a thirty-two story residential tower with four levels of parking and street-level retail. The designer of the building is also the owner who plans to rent the apartment units to students attending Yale University and locals attracted by an urban lifestyle. Becker + Becker also hopes to attract a grocery store to the retail space.

Figure 1: Corner of Chapel & State Street View of 360

Previously, the corner of Chapel and State Street consisted of an abandoned building with an adjacent parking lot that occupied an acre and half of land. With its redevelopment, 360 State Street now covers the site with the exception of a small plaza in the northwest corner. The building begins one level below grade; this area functions as the loading dock for the retail space. The primary entrances are located at grade. A parking garage extends from the second to the fifth floor with a ramp that circles the elevator core. On the sixth level, the residential tower begins. Its area is roughly a third of the building's footprint and is centered on the site. The sixth level contains all the amenities which include a fitness center, library, and lounge. The lower roof also doubles as a terrace for 360's residents. It consists of a landscaped garden, an outdoor pool, and a patio. The residential tower extends from the seventh to the thirty-first floor. The units include studio, one, two, and three bedroom apartments. At the roof of the building is a mechanical room which houses 360's cooling towers.

Overall, 360 State Street tops off at 338'-7", the second tallest building in New Haven. It is clad with architectural pre-cast concrete panels, masonry, and glazing. Ornamentation also decorates the façade on the lower levels. Sustainable features include recycled building materials, rooftop gardens, and geothermal walls. The design goal is to achieve LEED® Silver certification and encourage an urban lifestyle. 360 State Street is a milestone to the city's redevelopment and environmental efforts.

Figure 2: Interior View of Apartment Unit

DESIGN CRITERIA

The following data is provided to illustrate the general design criteria used for 360 State Street.

Codes & Design Standards

Applied to Original Design
2005 Connecticut State Building Code consisting of the 2003 International Building Code as modified by the 2005 Connecticut Supplement∵
American Institute of Steel Construction *Specification for Structural Steel Buildings – Allowable Stress Design and Plastic Design* 01 June 1989 (AISC)∵
American Concrete Institute *Building Code Requirements for Structural Concrete* ACI 318-02 (ACI)∵
American Concrete Institute *Building Code Requirements for Masonry Structures* ACI 530-99 (ACI 530)
American Iron and Steel Institute *Specification for the Design of Cold-Formed Steel Structural Members* 1996 (AISI)

∵Substituted for Analysis
American Society for Civil Engineers *Minimum Design Loads for Buildings and Other Structures* ASCE-7-05
American Institute of Steel Construction *Steel Construction Manual, Thirteenth Edition* April 2007 (AISC)
American Concrete Institute *Building Code Requirements for Structural Concrete and Commentary* ACI 318-08 (ACI)

Table 1: Codes & Standards used for Original & Analyzed Design
Note: Thesis Design Analysis was conducted using Load and Resistance Factor Design (LRFD).

Material Strength Requirements

Material	Strength Requirement
Structural Steel:	
All Rolled Shapes	ASTM A572 (A992), Grade 50
Connection Materials	ASTM A36
Metal Deck	ASTM A611 or A653 w/ ASTM A653 G60 Galv.
Cast-In-Place Concrete:	
Foundations	4 ksi NWC
Slabs-On-Grade	4 ksi NWC
Formed Slabs	5 ksi NWC
Columns and Walls	8 ksi NWC (Foundation to 6th Floor)
Reinforcement	ASTM A615, Grade 60 Except all #11 Bars are Grade 75
Light Gage Framing	ASTM A653, Grade 50

Table 2: Material Strength Requirements as per drawing S001

Deflection Criteria

Construction[1]	Live	Snow or Wind[f]	D + L[g]
Roof Members[e]:			
Supporting Plaster Ceiling	$\ell/360$	$\ell/360$	$\ell/240$
Supporting Non-Plaster Ceiling	$\ell/240$	$\ell/240$	$\ell/180$
Not Supporting Ceiling	$\ell/180$	$\ell/180$	$\ell/120$
Floor Members	$\ell/360$	-	$\ell/240$
Exterior Walls and Interior Partitions:			
With Brittle Finishes	-	$\ell/240$	-
With Flexible Finishes	-	$\ell/120$	-

Table 3: Deflection Limitations outlined by IBC 2003

e. The above deflections do not ensure against ponding.
f. The wind is permitted to be taken as 0.7 times the "component and cladding" loads for the purpose of determining deflection limits herein.
g. For steel structural members, the dead load shall be taken as zero.

Dead & Live Loads

Level	Load Type	Design Dead Load (psf)	Design Super-Imposed Dead Load (psf)	Design Live Load (psf)	Live Load per ASCE 7 - 05(psf)
Foundation	Loading Dock	Varies on Mat Slab Thickness	40	100	-
Grade	Public	150	40	100	100
2nd to 5th	Parking	125	22	40	40
6th Terrace	Amenities	150	25	100	100
	Terrace Typ.	200	160	100	100
	Terrace Planters	200	400	100	100
	Large Tree Planters	250	620	100	-
7th	Residential	61	20	40	40
	Public	61	20	100	100
8th to 31st	Residential	61	20	40	40
	Public	61	20	100	100
Mechanical/Roof	Mechanical	61	20	40	-

Table 4: Dead & Live Load Schedule

Note: According to Section 1606.1 in the International Building Code 2003, dead loads considered for design shall be the actual weight of materials and construction.

Occupancy/Function	psf	Occupancy/Function	psf
Corridor	100	Public Space	100
Storage (Light)	125	Lobby	100
Office	50	Terrace (Private, Public)	60, 100
Residential	40	Parking (Passenger Cars)	40

Table 5: Additional Uniformly Distributed Live Loads from ASCE 7 Table 4-1

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[1] Table 1604.3 Deflection Limits, 2003 International Building Code Portion of the 2005 Connecticut State Building Code

Foundations

The foundation of 360 State Street is a reinforced concrete mat slab located 17' – 3" below grade. The slab varies between 36" to 68" in thickness depending on the programmed area's function. A mat slab was chosen as the primary support because it can evenly distribute heavy column loads across the entire building's area and sufficiently resist hydraulic uplift. It was also chosen based on New Haven's geology and the building's proximity to water. Supporting the slab is a series of pressure injected footings and mini-piles that have a capacity of 75 to 100 tons. Additionally, a foundation wall runs along the perimeter of the residential tower's footprint and 40" x 40" concrete piers provide extra support to the retail space. Overall, the foundation is underpinned to the adjacent Pitkin Tunnel.

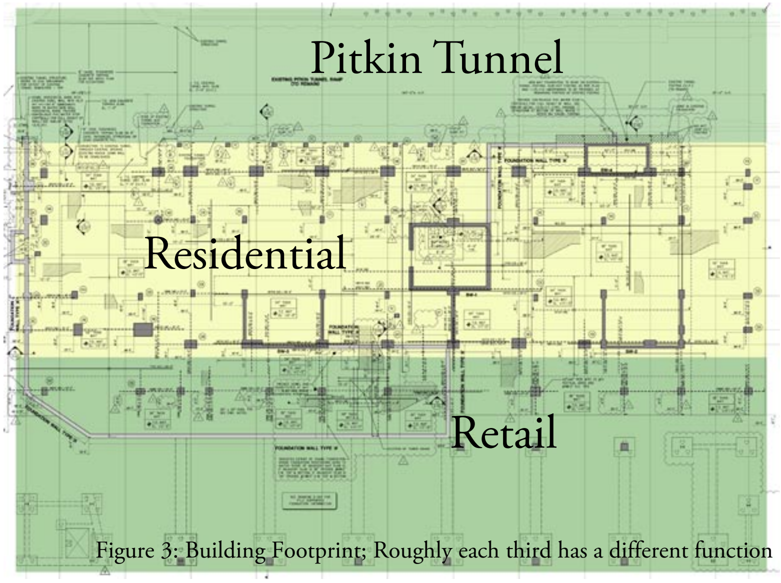


Figure 3: Building Footprint; Roughly each third has a different function

Floor Systems

360 has a variety of concrete floor systems distributed throughout the building. At ground level, there is a 12" slab-on-grade which covers two-thirds of the building's footprint. Between the second and fifth floor, three different slabs are used for each third of the building. The center portion consists of a 10" cast-in-place slab that supports the elevator lobby and unit storage rooms. Above the Pitkin Tunnel, a 7" post- tensioned slab supports the tenant parking. The last third of the footprint is composed of an 8" two-way flat plate slab that is supported by a series of post-tensioned beams and columns.

The intermediate floor between the concrete base and the residential tower has a 12" cast-in-place slab. The lower roof or terrace is composed of a 2" 18 gage galvanized composite floor deck with 3 ¼" concrete. The remainder of the building consists of an 8" hollow core pre-cast plank that is supported by staggered steel trusses. This particular system will be discussed further in detail and will be compared to alternative floor systems viable for 360 State Street.

Gravity Systems

Reinforced concrete is the primary material used in the first six stories of the building. Supporting the floor systems are post-tensioned beams and columns which are spaced at 24' east to west. Within the center portion of the building, the spacing is 14' north to south however; the columns along the exterior are spaced at 50' to provide room for maneuvering and parking cars.

The second half of the building is the slender residential tower which is made up of steel shapes. The beams and columns are primarily found along the exterior perimeter with the exception of those that support the elevator core. Unlike most buildings, 360 uses a system of staggered trusses for its interior framing. There are eleven overall which span 62' across the short length of the building.

Lateral Systems

Although the beams and columns create 360 State Street's skeleton, the floor slabs, shear walls, and cross-bracings give the structure stability. The lateral systems help distribute wind and seismic forces across the entire frame as well as increase its rigidity. Four main shear walls are located in the concrete base, one of which encases the elevator core. None of these walls continue past the fifth floor however; steel cross-bracings continue through the residential tower. The braces consist of hollow structural sections that zigzag along the North/South face of the building. The staggered trusses previously mentioned also helps support in the East/West direction.

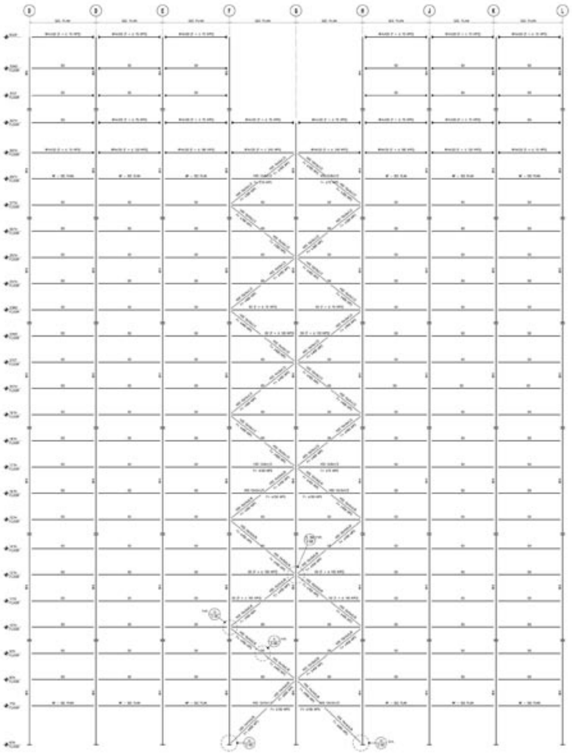


Figure 4: Elevation of North/South Steel Cross-Bracing

Roof Systems

The main roof is composed of the same 8" hollow core planks that are present on the lower levels. Additionally, a waterproof membrane, 12" R40 rigid insulation, ½" DensDeck prime cover board, and EPDM roofing membrane are layered on top. A pre-cast parapet wall runs along the perimeter of the roof at a height of 3' – 6". Flashing and another waterproof membrane tie the construction together. The lower roof has a completely different structure. It is supported by a 2" 18 gage galvanized composite floor deck with a 3 ¼" concrete slab. This level is used as a terrace and includes a landscaped garden which requires the addition of a drainage mat, filter fabric, and a waterproofing membrane to the construction.

Hollow Core Plank with Staggered Steel Truss

The existing floor system for the residential tower of 360 State Street consists of 8" hollow core concrete planks that bear on a series of staggered steel trusses. The planks come in sections of 24' x 8' and eleven trusses span 62' in the short direction. Figure 5 represents the typical framing conditions throughout the tower. (See Appendix A for typical floor plans.) The alternative floor systems will be designed and compared to the highlighted bay. Additionally, the intent of the highlighted bay is to maintain the column locations that correspond within the base of the building. This will minimize the necessity of transfer girders.

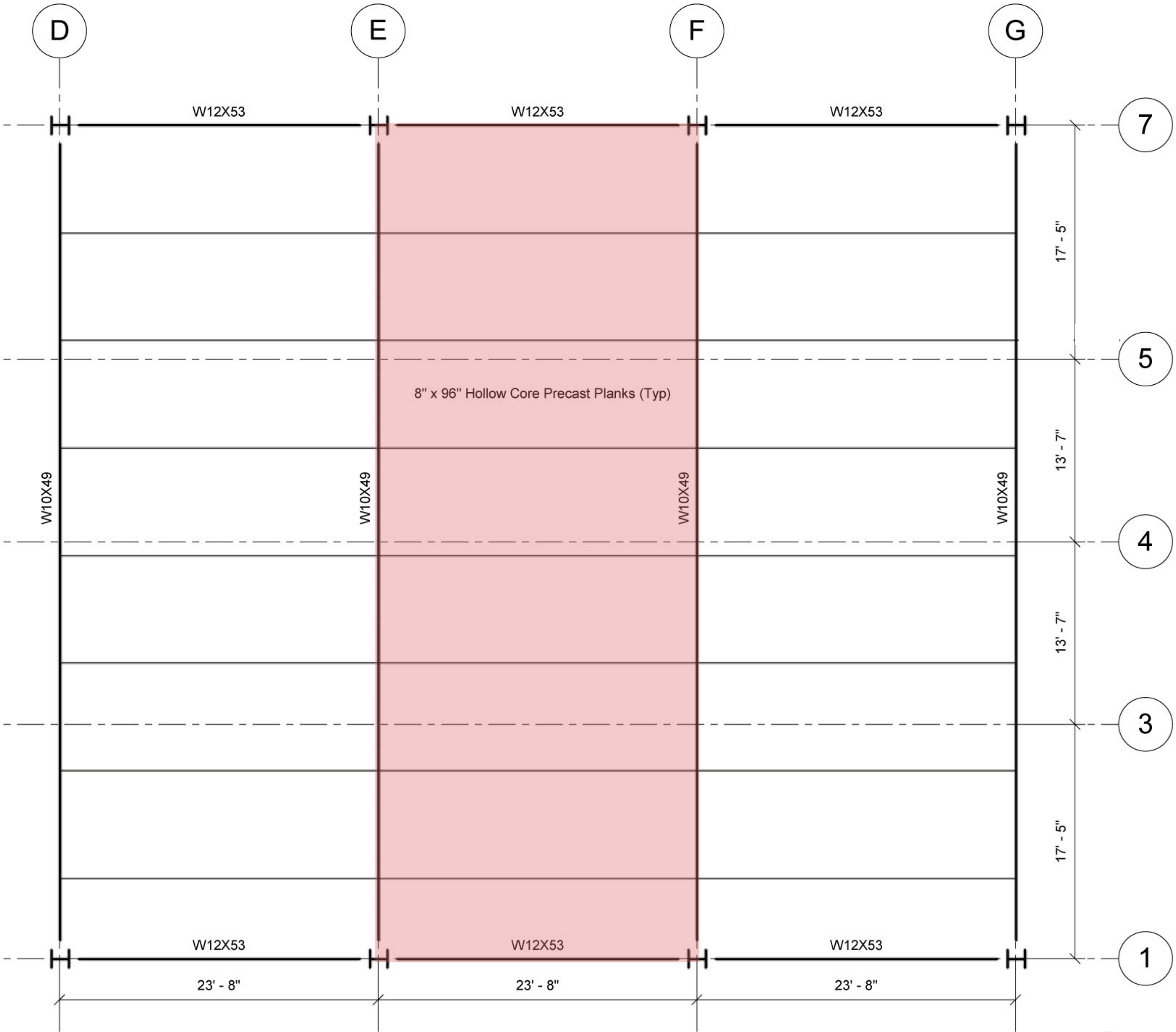


Figure 5: Typical Framing Plan, Highlighted Bay will be Analyzed for Alternative Floor Systems

Hollow core planks first came into production during the 1960's and were popular in Eastern Europe and Scandinavia. They were typically used in multi-story apartment buildings that were geared toward affordable housing. The planks are now produced with various thicknesses in long lengths and are later cut according to project specifications. Each plank is cast with tubular voids at a diameter close to its thickness. These voids are simply filled with air. In between the openings, steel strand are placed and tensioned to counter-balance undesirable stresses under heavy loads. The planks are additionally fire rated at 2 hours and can span up to 30'.

Staggered steel trusses were also developed in the 1960's. Beginning as a study for US Steel, William LeMessurier of the Massachusetts Institute of Technology designed a steel frame that could achieve similar floor-to-floor heights as a flat plate concrete system (roughly 8'-8"). LeMessurier's research led him to analyze buildings as a cantilever beam in which the various structural members perform as a single unit. A system of staggered trusses was found to maximize the strength of a building as well as its rigidity. The location of the trusses alternate with the column lines such that the long axis is always situated intermediately between levels. Each truss is additionally composed of W-shapes and hollow structural sections. Sometimes a camber is incorporated into the top and bottom chords to account for dead loads. Staggered trusses are now typically seen in buildings that have a doubly-loaded center corridor or repetitive floor plans such as high-rise apartment buildings, hotels, and hospitals.

Figure 6: 360 State Street Under Construction, October 2009

Advantages	Disadvantages
High strength steel and concrete can be used.	SST: Floor plans dependent on placement of trusses.
Optimizes floor-to-floor heights; Increases useable square footage; Minimizes number of required columns; Possible to have long spans.	Placing planks can be difficult as they need to be lowered between trusses to place. This is time consuming and slows down steel erection.
Other trades can begin work immediately at or below a level with planks. No shoring required. Placement not dependent on weather conditions. Once trusses are up, the façade can be installed two levels below.	HCP: Each plank experiences creep and shrinkage independently overtime without a topping slab; can lead to uneven surfaces.
Planks are precast and trusses are prefabricated; ideal for fast-tracked projects. Fewer pieces to assemble and store onsite.	Although HCP are light weight, SST are heavy; no increase or decrease to original foundation size.
HCP are inherently fire rated; SST can be fireproofed or enclosed by fire-rated partitions.	SST: Long lead time for fabrication.
HCP: Voids provide natural thermal insulation and eliminate sound permeations and vibrations.	Optimal floor-to-floor height can be compromised by MEP coordination.
HCP: Ceilings can be applied directly to underside of slab. Floor finish can be applied directly to top of slab.	
HCP: Less material required to form; very economical.	
HCP: There are many manufactures which allow planks to be readily available locally.	
SST: Minimizes moment across frame with cantilever action and all members working together as one unit.	
SST: Foundations are along column lines of trusses; less formwork required.	
SST: Resists lateral loads by distributing forces through rigid diaphragms.	

Evaluation

Hollow core planks carry with them many advantages that benefit the overall construction and sustainability of the building. As precast slabs, less material and labor is required for installation. This decreases the pollution caused by equipment and limits residual waste. Additionally, this significantly decreases the construction cost for the majority of the building. Although the slabs are thicker than usual for residential projects, the added benefit of thermal and sound insulation saves the owner more time and money. By and large, the quality of the residential units improves as well as its marketability.

The staggered trusses also provide a large advantage to 360 State Street's construction. Steel erection is fairly quick since the trusses can be placed directly off of the delivery trucks. Although they require some lead time in terms of fabrication, the trusses provide the designer more flexibility in the interior spaces with long open spans. The added stability provides the building with ample strength required to carry the lateral forces with ease.

The combination of the two elements creates an attractive system that allows the architectural design to flourish. The benefits outweigh the disadvantages however; some improvements can be made. With the depth of the steel, thickness of the slab, and space required for mechanical ductwork, the current floor-to-floor height adds up to 9'-4". Alternative systems that can optimize this height will largely benefit the owner with more rentable space. Additionally, floor systems that can decrease the overall weight of the building and the size of the foundations without sacrificing rigidity should be researched. Any system that can overshadow hollow core planks on staggered trusses without diminishing the interior space would be a capital improvement to the building.

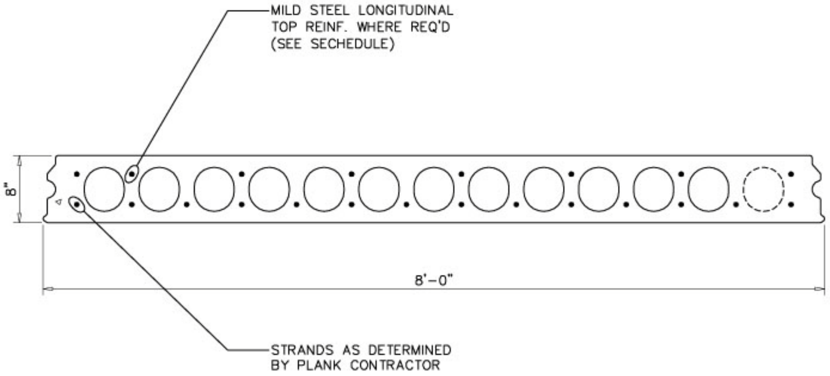


Figure 7: Section of a typical 8" Hollow Core Concrete Plank

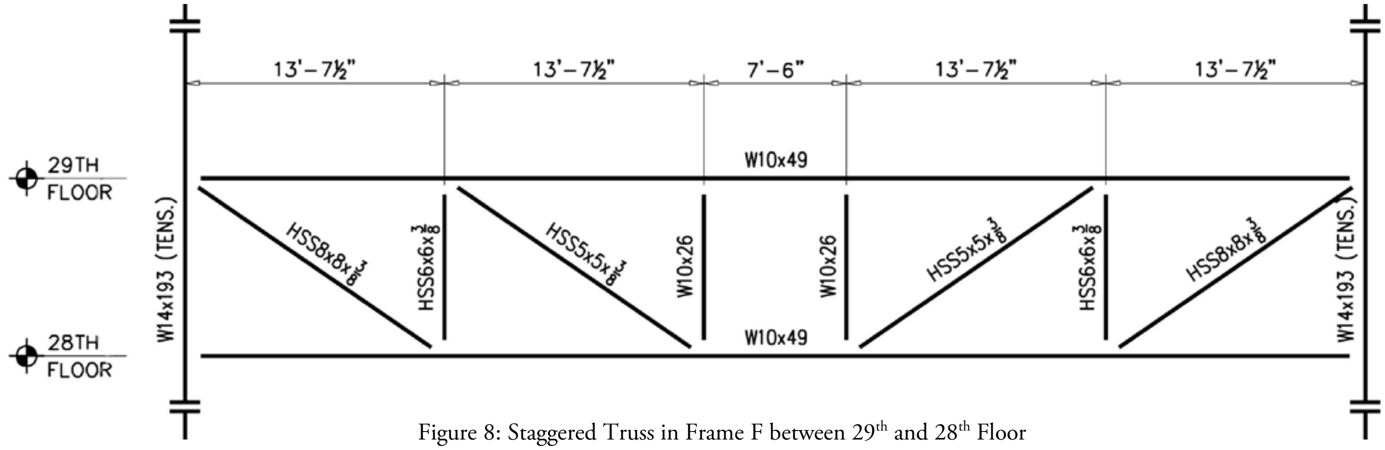


Figure 8: Staggered Truss in Frame F between 29th and 28th Floor

Composite Cellular Beam System

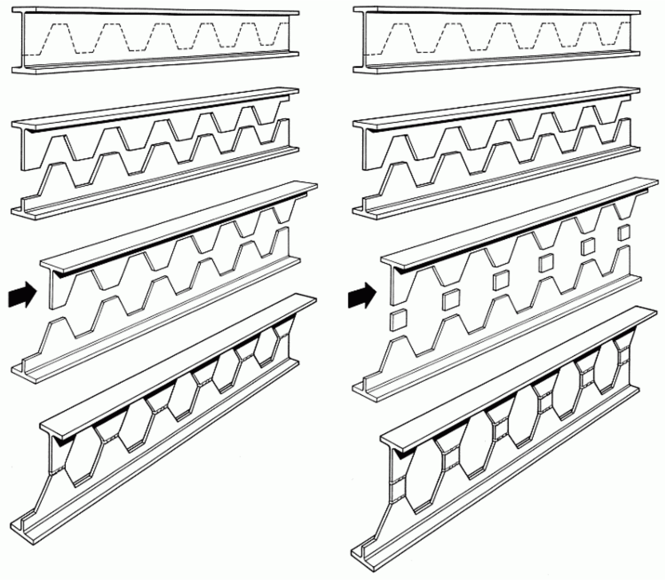


Figure 9: Diagram of Fabrication; photo courtesy of Grünbaver Bv

The design of a cellular beam originates from a traditional WF steel shape. The beam is cut half in the long direction in a specific pattern. The pieces are then staggered and welded back together. There are two types of beams that can be formed in this way—cellular and castellated. Cellular beams have circular openings in the web and castellated beams have hexagonal openings. The idea of the design is to expand a standard steel shape to become 35% to 50% deeper and stronger while maintaining its original weight. Studies regarding material strengths have concluded that the proportion of steel, not the amount of steel, determines the beam's abilities. The configuration also allows the distribution of stresses to be taken around the edge of the openings. This eliminates the presence of stress concentrations that can lead to failure. Overall, this design was developed in response to steel shortages during the world wars and was eventually patented in 1937.

Figure 10: Cellular Beams, photo courtesy of CMC Steel Products

CMC Steel Products now manufactures these shapes under the brand name of SmartBeams®. They are ideally used in composite floor systems and can efficiently span 40' to 60'. Cellular and castellated beams are typically seen in office buildings, parking garages, and suspended floor structures. Each beam is created with up to 90% recycled steel at ASTM Standards, Grade 50. CMC Steel Products also maintains 50 fabrication shops nation-wide to ensure that local materials are used.

Advantages	Disadvantages
Aesthetically pleasing if left exposed.	Does not perform well with high concentrated loads.
Ductwork and conduits can easily pass through openings; floor-to-floor heights can be optimized.	Tedious and potentially time-consuming to coordinate MEP trades.
Performs well in long spans; can minimize the number of interior columns necessary resulting in open floor plans.	If additional strength required, plates can be installed over the openings however; welding materials and labor costs are expensive. Potentially time consuming.
Vibrations are decreased in floors because the increased depth increases stiffness.	Fireproofing needs to be 20% thicker in order to account for openings; increases labor costs and construction time.
Sustainability; LEED® points can be received for the use of recycled and local materials.	Potential lateral instability.
Can be painted, galvanized or fireproofed up to a 3 hour rating for interior use, 1 ½ hour for roof construction.	
Saves time and money in terms of fabrication and erection; fewer members required.	
Overall lighter construction will reduce foundation size.	

Evaluation

Reviewing the pros and cons, cellular beams appear to be a viable alternative to the existing floor system in 360 State Street. Coordinating the MEP trades might be tedious however; it would be beneficial in order to optimize the floor-to-floor heights. Coupled with the beam's ability to perform well in long spans, minimizing the number of interior columns will increase the design flexibility of the interior spaces. The increased depth of the beams can also significantly increase the stiffness required to minimize lateral movement and vibrations. Additionally, the use of local and recycled materials can help the owners achieve a higher LEED® certification.

Figure 11 represents the typical framing plan applicable to 360 State Street with cellular beams. The calculations for this arrangement assumed a 4" thick poured slab on top of a 3" metal deck. Shear studs were also incorporated to achieve full composite action. This design does not alter the placement of the columns and maintains the spacing of girders that clear-span between the columns. This system decreases the overall building weight and the size of the foundations. Although the bay appears to retain the original layout, an investigation of the remaining framing system is required. Additional columns may be necessary to maintain stability in the girder. This would need to be coordinated with the location of the corridor. A cellular beam system poses a suitable solution for a sturdy floor system but it neglects the impact of lateral forces. Significant bracing would be required between grid lines 1 and 7.

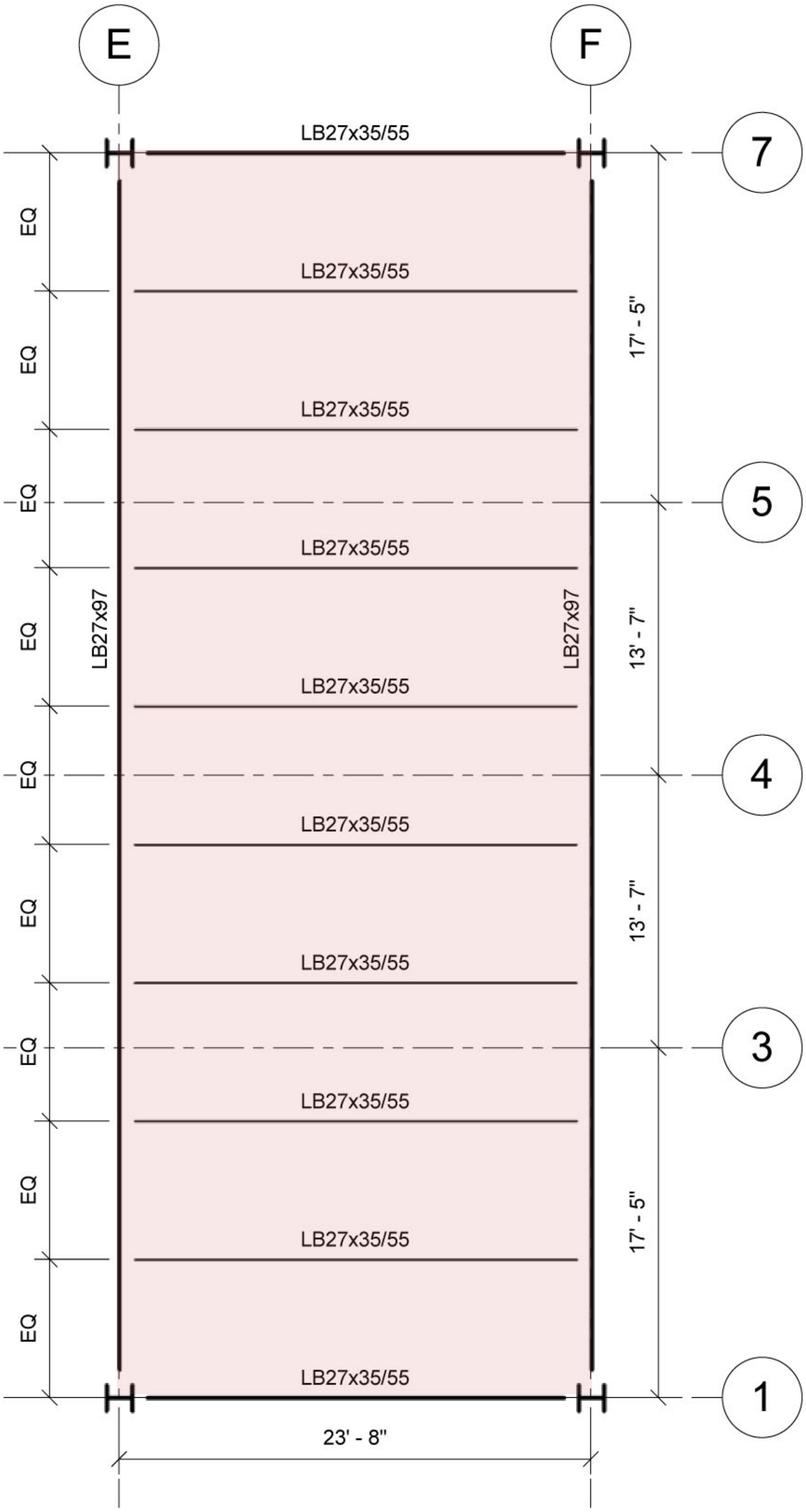


Figure 11: Typical Framing with Cellular Beam System

Hambro Composite Floor

Hambro, a division of Canam Group, is company that designs, fabricates, and markets various construction products. In this case, they have developed the D500 which is a composite floor system targeted towards residential buildings. Their latest product consists of an open web joists topped with a thin concrete slab.

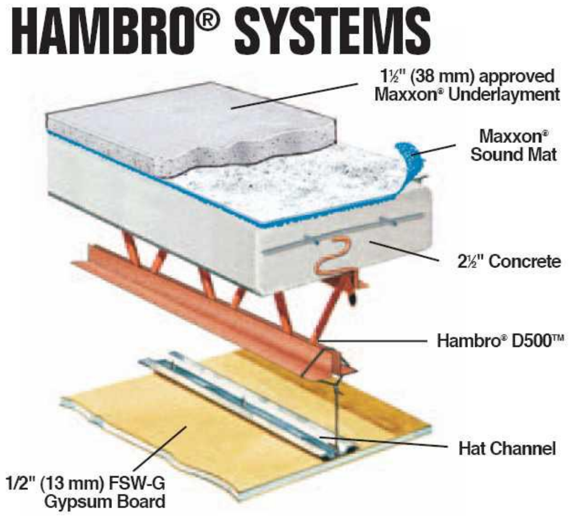


Figure 12: Detail of D500 Floor System courtesy of Hambro®

The bottom chord of the joist is the tension member in the system. It is comprised of double angles. Bent rods acts as the web of the joist and tie the chords together. The top chord is a unique WT beam that carries the compression forces. Welded onto the top is a special bar that protrudes into the slab. This distinctive feature acts is the shear connector between the joists and the slab in order to achieve composite action. Additionally, welded wire fabric is placed within the poured concrete slab as reinforcing. This unique design was developed to simplify one-way slabs. The shear connector is acts as a "high chair" to induce negative moments in the slab. This ensures one-way behavior in the system. Overall, the design proves to be very versatile as it is compatible with any type of framing. The joists can easily be supported by masonry walls, concrete beams or metal studs.

Advantages	Disadvantages
Can achieve high acoustic properties with added treatments.	Fireproofing may be expensive and time consuming to achieve required ratings.
Versatility with framing systems. Able to bear on any framing; angles may be installed to provide additional bearing.	Time consuming. Bearing walls must be up first before installation however; floor must be completed before next level initiated. Holds up other trades.
MEP trades can easily run through joists with some coordination; minimizes floor-to-floor height.	Not readily available; offices only located in Florida, Washington, and Wisconsin.
No negative impact on architectural design.	Connections must be welded; time consuming and expensive.
Joists can support drop-down ceilings.	Limited configuration of joists.
Economical; less concrete and steel required.	Vibration and rigidity issues with thin slab and flimsy joist.

Evaluation

The Hambro® Composite System equally shares positive and negative aspects in its design. It may be compatible with numerous framing types but the overall rigidity of the structure comes into question. The alternative design for 360 State Street includes a 3” slab with a 10” joist as seen in Figure 13. Unfortunately, Hambro®’s design guide does not include detailed information to accurately calculate deflection. Further investigation into this system is required however; the amount of materials necessary to ensure proper stability and framing might discourage the use of the system entirely.

Upon further investigation, this system could potentially govern over the interior floor plans since additional columns were required to effectively carry the girder loads. This system will also dominate the construction schedule as it requires the trades to wait until it is nearly installed. Moreover, it is doubtful that a 10” joist will allow excessive ductwork and conduits to pass through.

The Hambro® Composite System incorporates shear fasteners to the top of its joists and is compatible with numerous framing systems. However, it appears to be best suited for smaller scale residential projects.

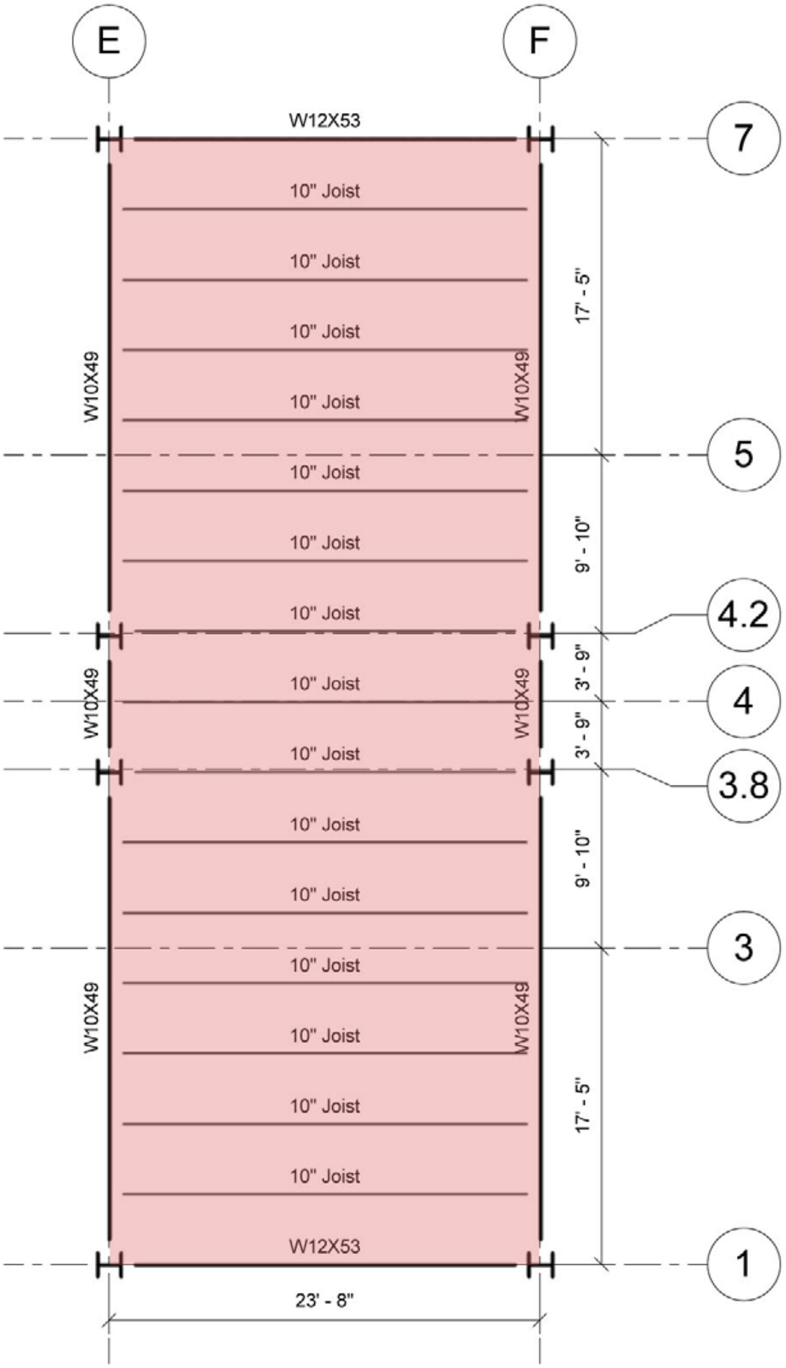


Figure 13: Typical Framing with Hambro® Composite System

Girder-Slab

Girder-Slab® Technologies, LLC is a company that specializes in composite steel and precast systems. Targeting the residential market, the objective in their latest design was to optimize floor-to-floor heights as well as decrease construction time. The design combines the advantages of typical steel frames with innovative engineering. Story heights as low as 8’-0” can be achieved as well as easy on-site assembly. Developed in 1990 by two engineers, the girder-slab system is now seen in hotels, dormitories, and high-rise apartment buildings.

The assembly is composed of hollow core precast planks that rest on the bottom flange of a D-beam girder. Originally a rolled WF section, the girder is inverted and a flat plate is welded to the top. The bottom flange is the widest portion of the shape and thus gets its name as the dissymmetric beam. Trapezoidal openings are additionally cut into the web so that full composite action can be achieved when the joints are grouted. For shear reinforcing, rebar can also be threaded through the openings. The overall girder-slab system is designed to eliminate masonry bearing walls and flat plate concrete systems.

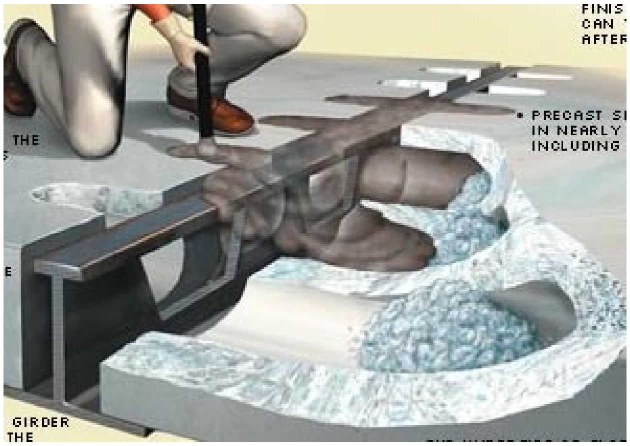


Figure 14: Girder-Slab Detail courtesy of Girder-Slab® Technologies

Advantages	Disadvantages
No formwork or shoring required.	Limited D-beam sizes.
Optimizes floor-to-floor height; shallow floor depth.	Spans limited by load and deflection requirements.
MEP trades can easily run ductwork and conduits below slab; coring into slab possible for utilities.	Availability of D-beam; offices located only in New Jersey. Certification for patent use must be received before local steel mill can fabricate.
Slab placement independent of weather conditions.	Columns still required; increases column grid.
Other trades can begin immediately after placement.	
Economical.	
Fairly light-weight though sufficiently rigid.	
Underside & top of slab ready-finished for ceiling & floor.	
Ease of construction; prefabricated and assembled-in-place; short erection time.	
Fire rated assembly though fireproofing required for remaining steel framing.	

Evaluation

The Girder-Slab system combines the advantages of typical steel framing with the benefits of hollow core precast planks and applies them towards residential projects. This system successfully optimizes floor-to-floor heights without sacrificing strength or rigidity in the structure. Although a long lead time may be required for the D-beams, the system is quickly assembled in place and does not hold up the other trades. Next to placing the members with a crane, the only time sensitive activity is reinforcing and grouting the joints between the planks. Afterwards, the floor is ready for its finish.

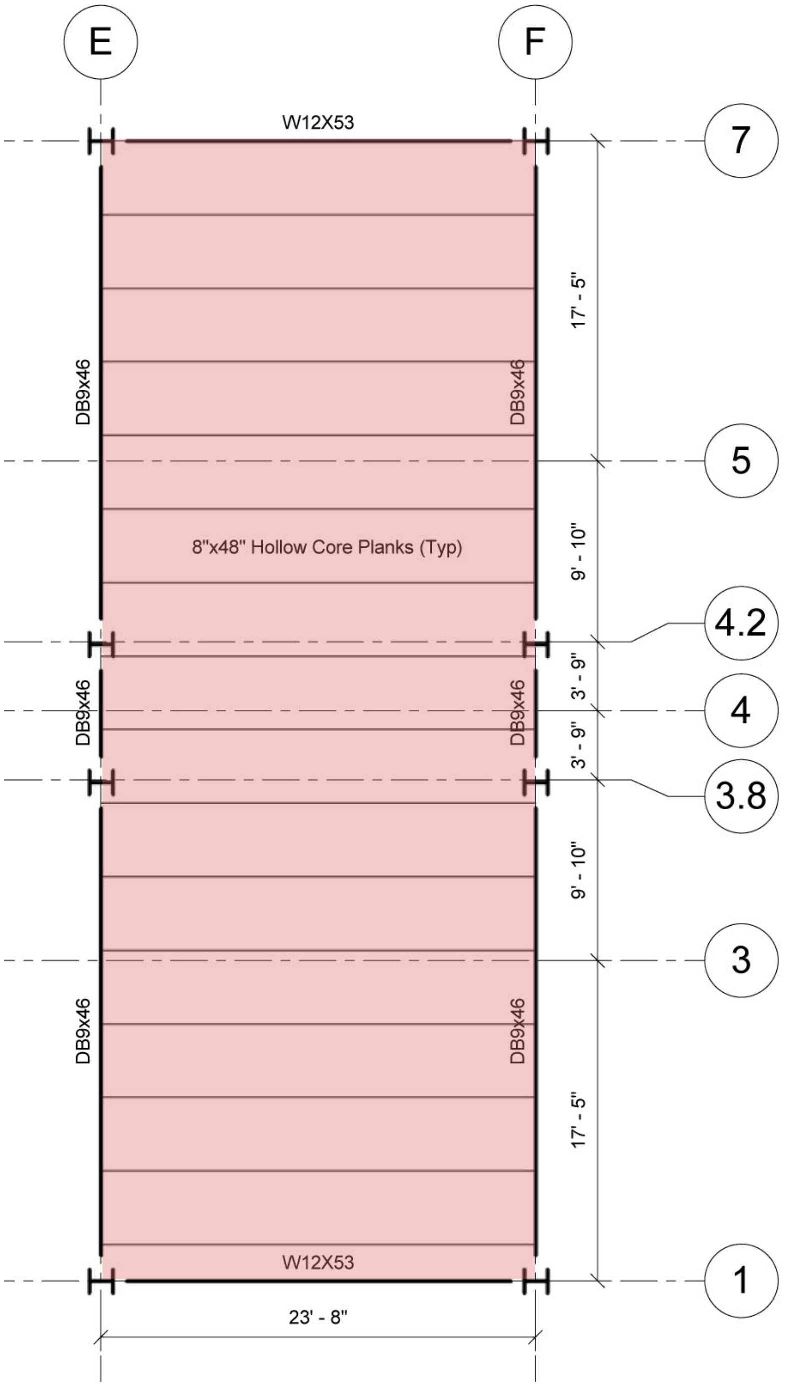


Figure 15: Typical Framing Plan with Girder-Slab System

The alternate design for 360 State Street maintains the original layout of each bay however; additional columns are required. If the original interior floor plan is maintained, the columns could easily hide in the walls. This is certainly a positive tradeoff when the floor depth has been minimized to 10” and rigidity is not sacrificed. The Girder-Slab system appears to be a good alternate for 360 State Street however; the remaining framing must be analyzed to ensure compatibility.

Two-Way Flat Plate

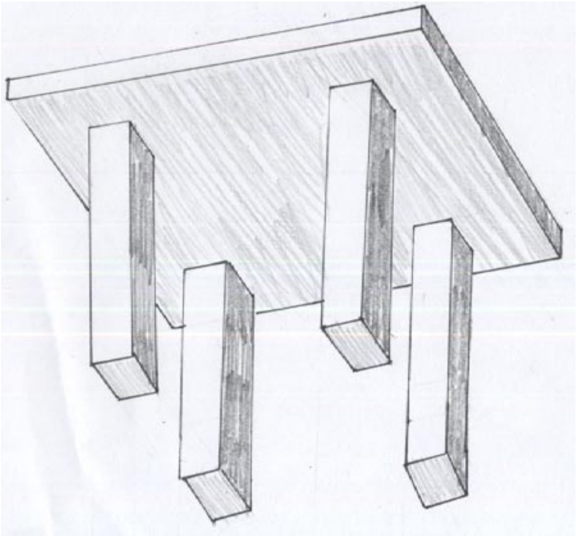


Figure 16: Sketch of Typical Flat Plate System

Flat plate systems are cast-in-place concrete slabs supported by concrete columns. They can exist with or without drop panels, column capitals, and beams. These elements typically aid in shear resistance and load distribution. Formwork and shoring is required however; installation can be simple depending upon what elements need to be formed and if floor plans are repeated. Flat plate systems are ideal for moderate span lengths and building types with light weight loads. Architecturally, columns dominate the floor plan but the advantage of closely spaced columns can optimize the floor-to-floor height.

Two-way slabs are a common floor system found in both residential and commercial building types. Their prominent feature is the ability to carry loads in two directions. This helps to minimize the amount of force distributed to each adjacent member. Lacking lateral resistance, flat plate slabs are often used in conjunction with shear walls. The thickness of a slab is dependent upon the design criteria of a project. Reinforcing bars and chairs are included within the slab to induce negative moments across a span. The steel provides the tension capabilities that concrete lacks; it is also a safety measure if a crack occurs.

Advantages	Disadvantages
Easy to construct: cast-in-place.	Formwork and shoring required; limits other trades.
Formwork is simple to erect; reusable if floor plans allow.	Time required for curing; fast-tracking schedule not possible.
Very short lead time for materials.	Weather and temperature dependent for pouring & curing.
Economical.	Might require drop-ceiling to hide MEP.
Low maintenance costs.	Reinforcing required; could increase cost.
Mass of floors limits vibrations and has acoustical advantages.	Foundations increase in size due to weight of slabs and additional columns; cost increases as well.
Inherent fire-rating of 2 hours.	Lacks lateral resistance.

Evaluation

A two-way flat plate system appears to be well-suited for residential construction however; this system may not be the best alternative for a tall building. The advantages are found in the economical cost of the materials and the straight forward installation process. The inherent fire-rating and acoustic properties would allow resources to be directed towards other design aspects. The disadvantages begin with the mass of 360 State Street. Twenty-six stories would have to be formed, poured, and left to cure. This would limit the progress of other trades and essentially increase the entire construction schedule without mentioning the impact of inclement weather.

Figure 17 illustrates a typical framing plan associated with a flat plate floor system. The design includes a 10” slab reinforced with #5 bars and three new interior columns along each North-South gridline. Together, the slab and additional columns would significantly increase the overall building weight. Consequently, this would also increase the size of the foundations. The 10” slab would not optimize the floor-to-floor height and the interior columns along gridline 4 would require architectural coordination with the corridor. Another disadvantage of this system is the lack of lateral resistance. Without the original staggered truss design, shear walls would have to be designed to provide rigidity to the structure. Overall, a flat plate floor system would not be considered a viable alternative. The system does not provide significant advantages and would require a complete redesign of the building.

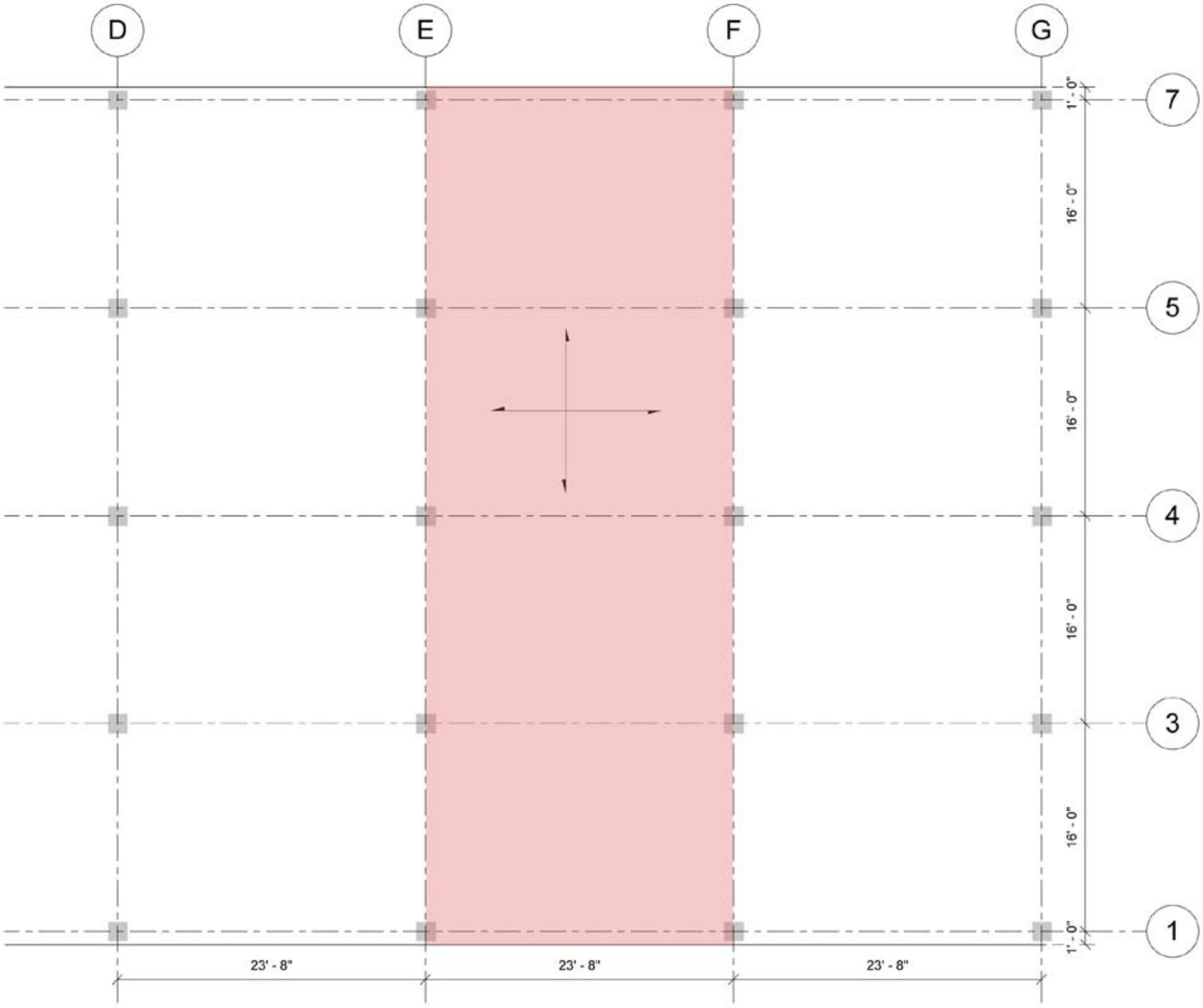


Figure 17: Typical Framing Plan with Flat Plate System

Criteria	Hollow Core Plank w/ Staggered Truss	Composite Cellular Beam	Hambro® Composite	Girder—Slab	Flat Plate
Relative Cost	$21/psf	$13.56/lf	$19.10/lf	$29/psf	$14..60
Structural Depth	18” + MEP	34”	13” + MEP	10” + MEP	10” + MEP
Structural Weight	99 plf	85 plf	~77 plf	96 plf	125 plf
Additional Fireproofing	Required	Required	Required	Not Required	Not Required
Vibration Issues	Not Likely	Not likely	Most Likely	Not Likely	Not Likely
Rigidity	Ample	Sufficient	Insufficient	Ample	Ample
Lead Time	Medium	Short	Long	Medium	Short
Construction Difficulty	Easy	Medium	Medium	Easy	Easy
Formwork	No	Yes	Yes	No	Yes
Schedule	(Baseline)	Maintains	Increases	Decreases	Increases
Architectural Impact	Positive	Positive	Negative	Potentially Negative	Potentially Negative
Coordination w/ Trades	Flexible	Significant	Significant	Flexible	Significant
Column Grid	(Baseline)	Maintains	Slightly Increases	Slightly Increases	Decreases
Building Weight (Impact on Foundations)	(Baseline)	Decreases	Decreases	Maintains	Increases
Weather’s Impact	Insignificant	Insignificant	Insignificant	Insignificant	Significant
Overall Viability	Yes	Yes	No	Yes Investigate Further	No

Table 6 Comparison of Floor Systems

CONCLUSIONS

360 State Street is comprised of a variety of different functions—retail, parking, and residential. Each space requires a specific floor system that can handle its particular loads. The base of the building is composed of three different cast-in-place slabs. Concrete was chosen in order to minimize maintenance in the open air parking garage. The remainder of the building is supported by a singular floor system—hollow core planks on staggered steel trusses. The intent of this report was to research, compare, and recommend an alternate system for 360 State Street. In order to maintain simplicity, this document focused on the individual floor system that supports the residential tower.

The majority of the building consists of twenty-six stories of residential space. The effect of optimizing a single system found in such a large volume creates a significant impact on the overall cost of the building, structural integrity, and construction schedule. The existing structure was analyzed in addition to three alternatives. In order to be considered, each system had to demonstrate similar advantages to the existing system. Furthermore, each had to magnify a particular structural quality that may have been overlooked or sacrificed.

The existing system is a combination of hollow core precast planks and a fully developed steel frame. Eleven frames composed of story-height trusses span across the short length of the building. At any given time, five or six trusses are located on each level. The advantage of staggering the trusses allows for the even distribution of loads from the slab while providing an open floor plan. The interaction between the trusses and the remaining steel members allow the structure to resist forces as a single unit. Furthermore, high concentrations of stresses are minimized with the distribution of loads through the truss chords and diagonal members.

The hollow core planks are 8” thick and come in sections of 24’x8’. Although the planks are thicker than typical residential floor constructions, the 8” provide more rigidity to the structure. Pre-stressed steel strands are additionally incorporated into the slabs; this allows the sections to span between the girders with minimal deflections. Analyzing the system, the staggered trusses and hollow core planks were found to have sufficient strength to withstand 360’s loads. The disadvantages of this system include a floor-to-floor height of 9’-4” and a heavy burden on the foundations.

The first alternative considered was composite cellular beams. They were chosen for their ability to increase the rigidity of the structure without significantly increasing the building weight. The system is composed of standard beams that have been cut, staggered, and welded back together with circular openings. Mechanical ductwork and electrical conduits can easily pass through the openings however; tedious coordination is required. A composite cellular beam system has the ability to capitalize on the weaknesses of the existing design however; the results of the analysis have concluded otherwise. Further investigation is required to ensure lateral stability and the overall frame will need to be redesigned to become compatible with the cellular beams. Although the benefits of the system include optimal floor heights and maintained weight, the design analysis has additionally concluded the system increases both properties. Coupled with the qualities expressed in Table 6, cellular beams pose a viable solution to the existing floor system however; other solutions are worth considering.

The second alternative explored was the Hambro® Composite System developed specifically for residential construction. It enhances a typical composite assembly with a unique shear bar welded to the top flange of an open web joist. The design's objective is to achieve full composite action and distribute negative moments across the slab. This feature allows the joists to span further with more ease. The immediate downside to this system is the weak construction compared to the existing system. The analysis concluded a 3" slab can be carried by 10" joists across a 24' span at a 4' spacing. Although it was desired to see a decrease in the building's weight, Hambro® sacrifices rigidity. Further investigation is required to determine if the available strength is sufficient for 360 State Street's loads. Additionally, the technical manual provided by Hambro® does not provide adequate information to make such a complete judgment. This system appears to be geared towards small scaled residential projects and does not provide enough viability to be recommended as an alternate.

The third system researched and analyzed for 360 State Street was the Girder-Slab. This system combines the strength of typical steel framing with the numerous benefits of hollow core planks. To minimize the floor depth, planks are supported by an inverted WF section. The wide bottom flange provides enough bearing to support the slabs. Additionally, the shape is castellated so that rebar and grout can tie the slabs together. The system is also prefabricated and can be quickly assembled onsite. Girder-Slabs provide a unique design that capitalizes where the existing staggered truss system does not; it also has the ability to enhance the integrity of the structure. The analysis has concluded that the system successfully decreases the floor depth and slightly decreases the overall building weight. Although additional columns are required for support, this system does not negatively impact the interior floor layout. The Girder-Slab system proves advantageous all around and is highly recommended for further investigation.

Lastly, a two-way flat plate slab was configured to 360's floor plan. This concrete system is ideal for residential construction with moderate spans and light weight loads. The analysis concluded an addition of three interior columns as well as a reinforced 10" slab. The system is fairly economical with a short lead time on materials however; the rate of construction is slow. Architecturally, the floor plans would need to be redesigned to coordinate with the new column locations. Overall, the flat plate system significantly increases the building weight and the foundations accordingly. The closely spaced columns also did not decrease the slab thickness or optimize story height. This system appears to best fit shorter building types. Without any noteworthy advantages, two-way flat plate slabs are not a viable option.

The alternative floor systems share a target market—residential construction. Upon reviewing the existing structure, the alternatives were rated against their ability to capitalize on the disadvantages of the existing system. In addition to optimizing the floor-to-floor height and decreasing the overall building weight, each system had to provide benefits that did not diminish the quality of the construction or the integrity of the structure. The systems were compared in Table 6 for their overall qualities. A typical bay was also designed for each system to compare structurally. Although the staggered steel truss system appears to be the best fit for 360 State Street, Girder-Slabs are a viable consideration for an alternative design.

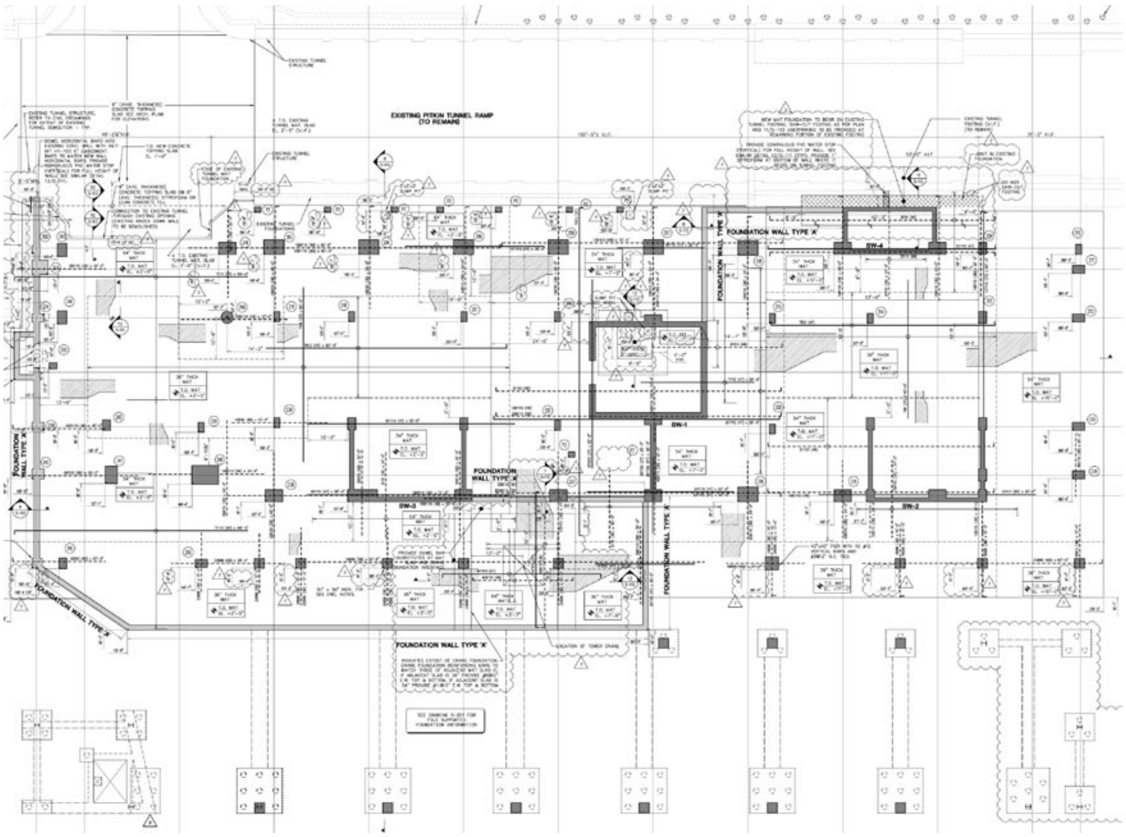


Figure A.1 Foundation Plan, Shear Walls are Shaded

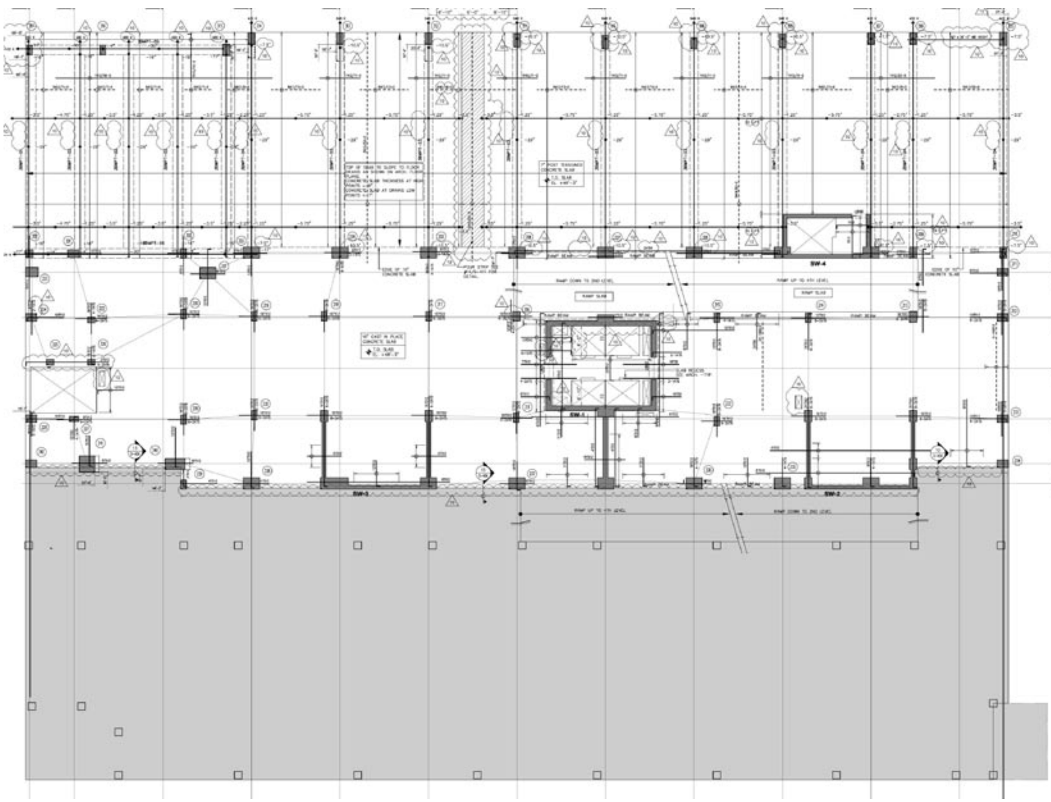


Figure A.2 Second - Fifth Typical Floor Plan

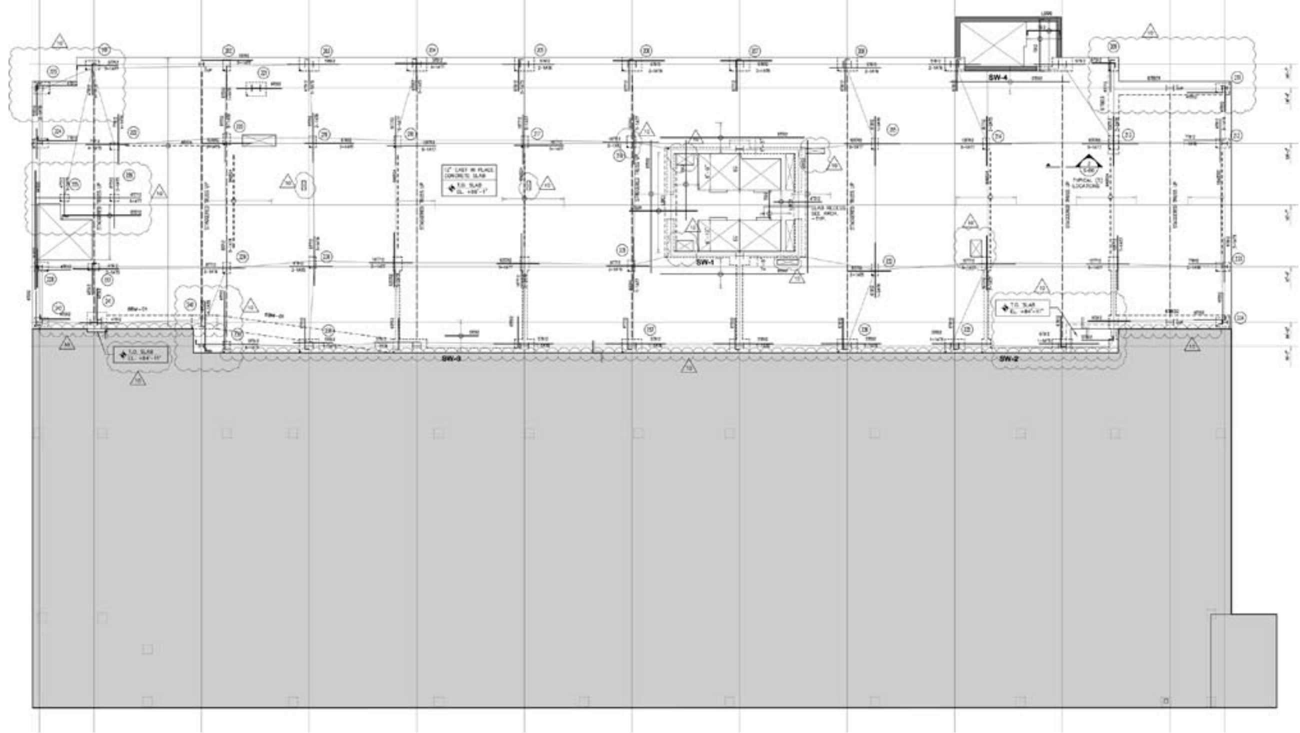


Figure A.3 Terrace & Sixth Floor Plan

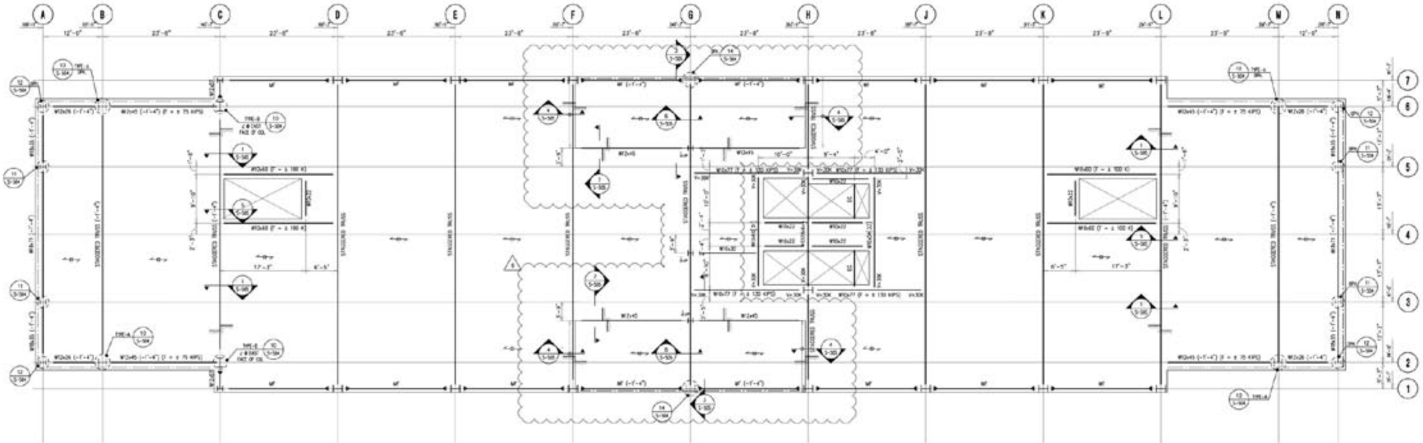


Figure A.4 Typical Floor Plan for Residential Tower

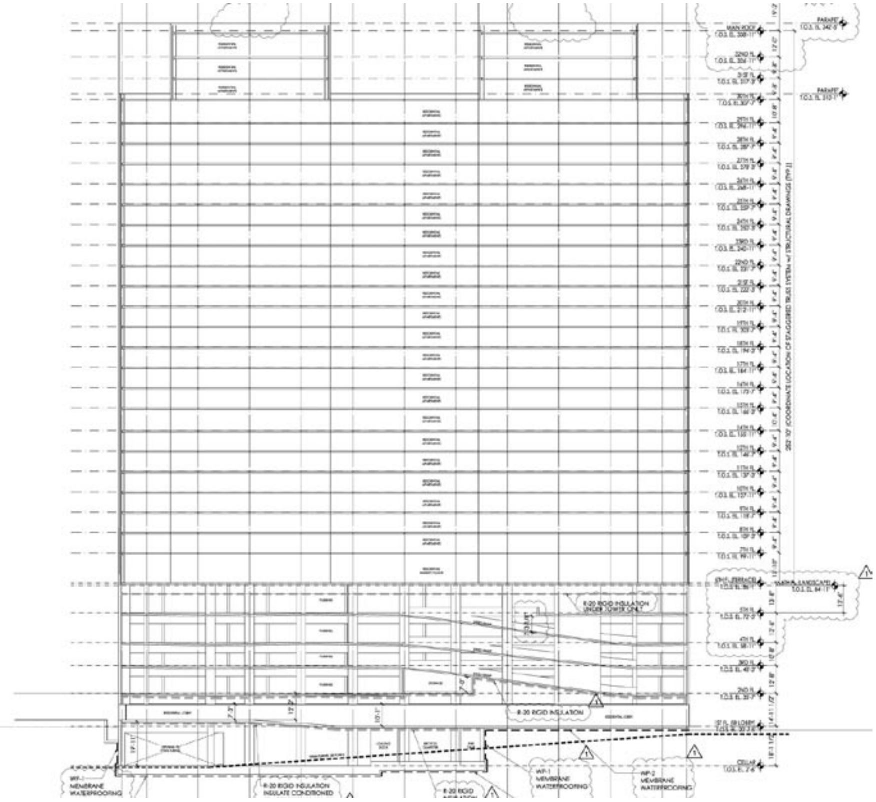


Figure A.5 North/South Building Elevation

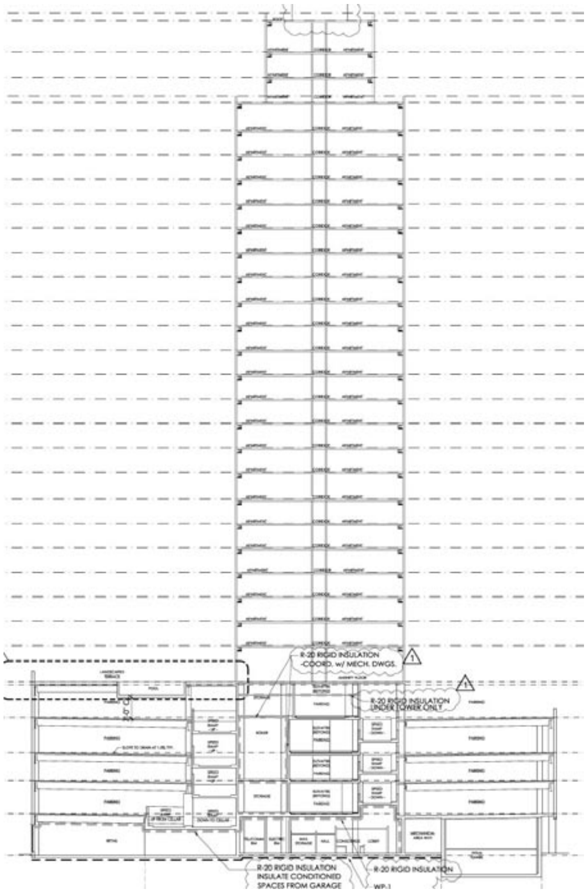
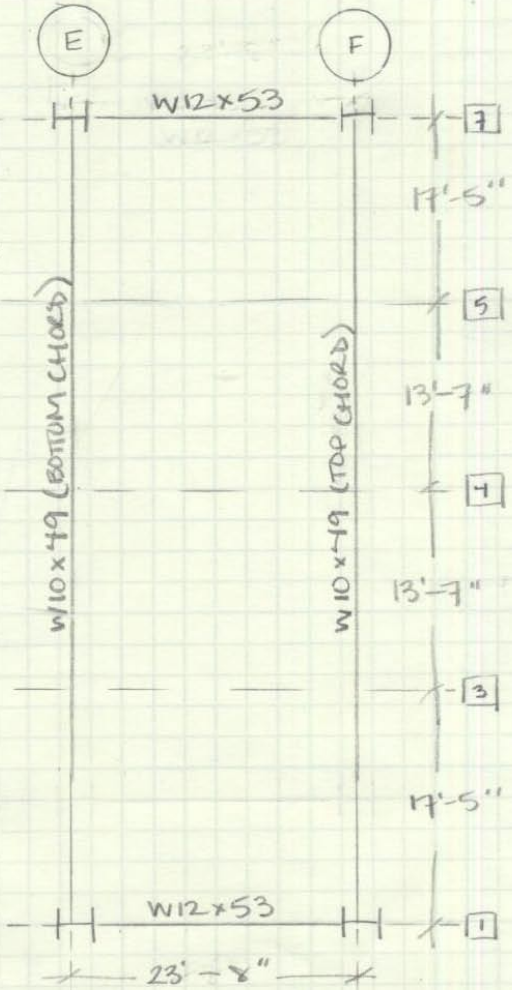


Figure A.6 East/West Building Elevation



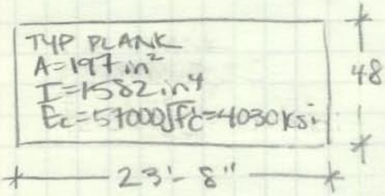
EXISTING FLOOR SYSTEM HOLLOW CORE PLANKS ON STAGGERED TRUSS

EVALUATED AT 29TH FLOOR
LOADS TAKEN FROM DRAWING S001
$W_{LL} = 100\text{psf}$
$W_{DL} = 80\text{psf}$ ← INCLUDES SUPER IMPOSED

TYPICAL 8"×96" PLANKS HOWEVER EVALUATING AT 8"×48" FOR LACK OF SPECIFIC LOAD TABLE

2" TOPPING INCLUDED
$f'_c = 5000\text{psi}$ (MIN)
7 ½" ϕ PRE-STRESSING STRANDS
$f_y = 160{,}000\text{psi}$

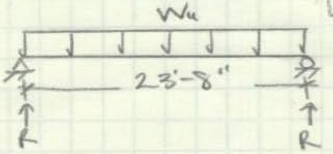
DESIGN BASED ONLY ON GRAVITY LOADS; TRUSS NEEDS TO BE DESIGNED FOR LATERAL LOADS


TYP PLANK
$A = 197\,\text{in}^2$
$I = 1582\,\text{in}^4$
$E_c = 57000\sqrt{f'_c} = 4030\,\text{ksi}$

$W_u = 1.2W_D + 1.6W_L = 256\text{psf}$ $\quad W_u = 256\text{psf}(4') - 1024\text{plf}$

$W_{SERVICE} = W_D + W_L = 180\text{psf}$ $\quad W_{SER} = 180\text{psf}(4') = 720\text{plf}$

$\boxed{W_u \text{ EXCEEDS LOAD TABLE BY 4 psf}}$



$$M_{max} = \frac{Wl^2}{8} = \frac{1024\text{plf}(23.7')^2}{8 \times 1000} = 71.9\,\text{kft}$$

$$R = \frac{Wl}{2} = \frac{1024\text{plf}(23.7')}{2 \times 1000} = 12.1^k$$

$$\Delta_{wservice} = \frac{5wl^4}{384EI} = \frac{5(720\text{plf})(23.7')^4(1728\,\text{in}^3)}{384(4030 \times 10^3\text{psi})(1582\,\text{in}^4)} = 0.801"$$

$$\Delta_{LIVE} = \frac{5wl^4}{384EI} = \frac{5(100\text{psf} \times 4')(23.7')^4(1728\,\text{in}^3)}{384(4030 \times 10^3\text{psi})(1582\,\text{in}^4)} = 0.445"$$

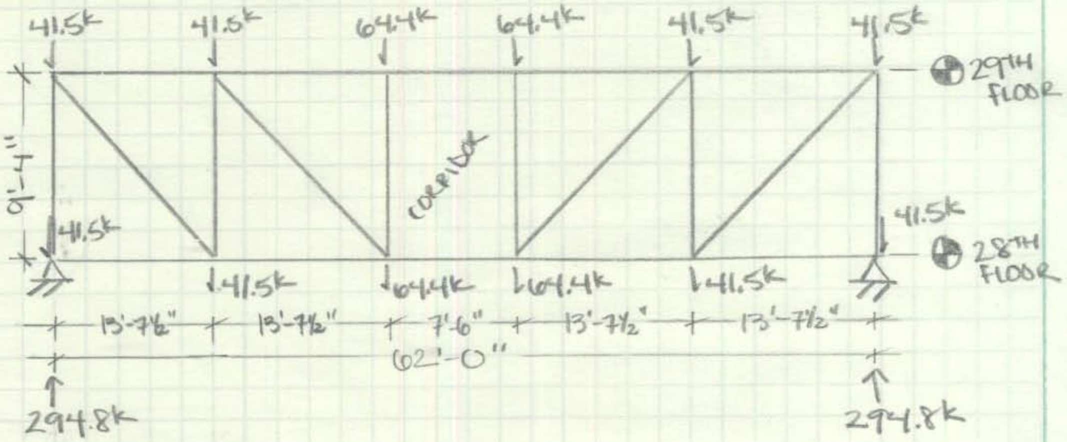
$l/360$ LIVE = 0.789" $\quad l/240 = 1.183"$ $\quad \boxed{\text{HCP OKAY IN } \Delta}$ 1/4

EXISTING FLOOR SYSTEM
HOLLOW CORE PLANKS ON
STAGGERED TRUSS

$w_u = 256$ psf

TRIB AREA = 23'-7" ~ 23.7'

FRAME (F) AT 29TH FLOOR



TOP & BOTTOM CHORDS SUPPORT FLOOR AT 29TH & 28TH LEVEL

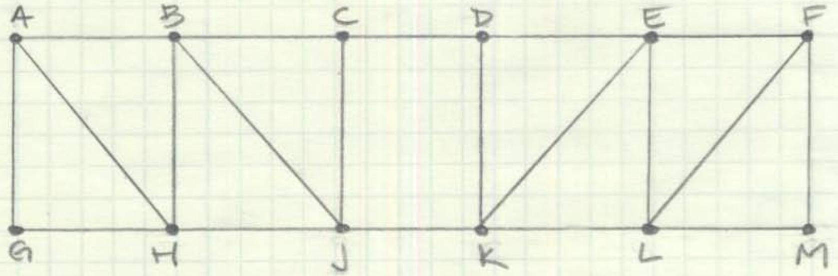
$$\frac{(256\,\text{psf})(23.7')}{1000} \approx 6.1\ \text{k/ft}$$

EXTERIOR $(6.1\ \text{k/ft})(1/2)(13.6') = 41.5^k$

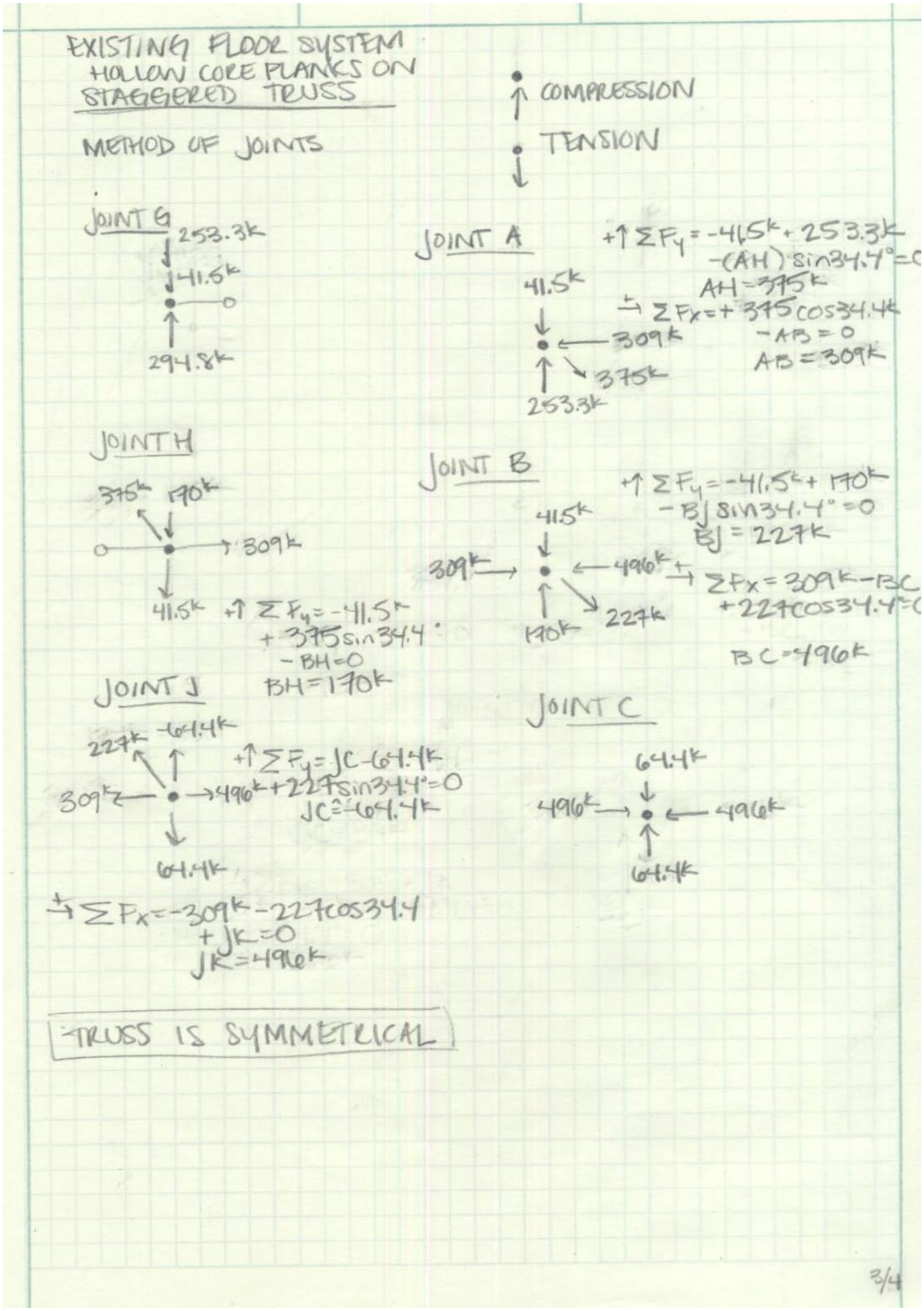
INTERIOR $(6.1\ \text{k/ft})(1/2)(13.6') = 41.5^k$

CORRIDOR $(6.1\ \text{k/ft})(1/2)(7.5' + 13.6') = 64.4^k$

JOINTS



2/4



EXISTING FLOOR SYSTEM
HOLLOW CORE PLANKS ON
STAGGERED TRUSS

METHOD OF JOINTS

↑ COMPRESSION
↓ TENSION

JOINT G

253.3^k
41.5^k
294.8^k

JOINT A

41.5^k
309^k
375^k
253.3^k

$+\uparrow \Sigma F_y = -41.5^k + 253.3^k - (AH)\sin 34.4° = 0$
$AH = 375^k$
$\overset{+}{\rightarrow} \Sigma F_x = +375\cos 34.4^k - AB = 0$
$AB = 309^k$

JOINT H

375^k 170^k
309^k
41.5^k

$+\uparrow \Sigma F_y = -41.5^k + 375\sin 34.4° - BH = 0$
$BH = 170^k$

JOINT B

41.5^k
309^k 496^k
170^k 227^k

$+\uparrow \Sigma F_y = -41.5^k + 170^k - BJ\sin 34.4° = 0$
$BJ = 227^k$
$\overset{+}{\rightarrow} \Sigma F_x = 309^k - BC + 227\cos 34.4° = 0$
$BC = 496^k$

JOINT J

227^k -64.4^k
309^k 496^k
64.4^k

$+\uparrow \Sigma F_y = JC - 64.4^k + 227\sin 34.4° = 0$
$JC \cong -64.4^k$

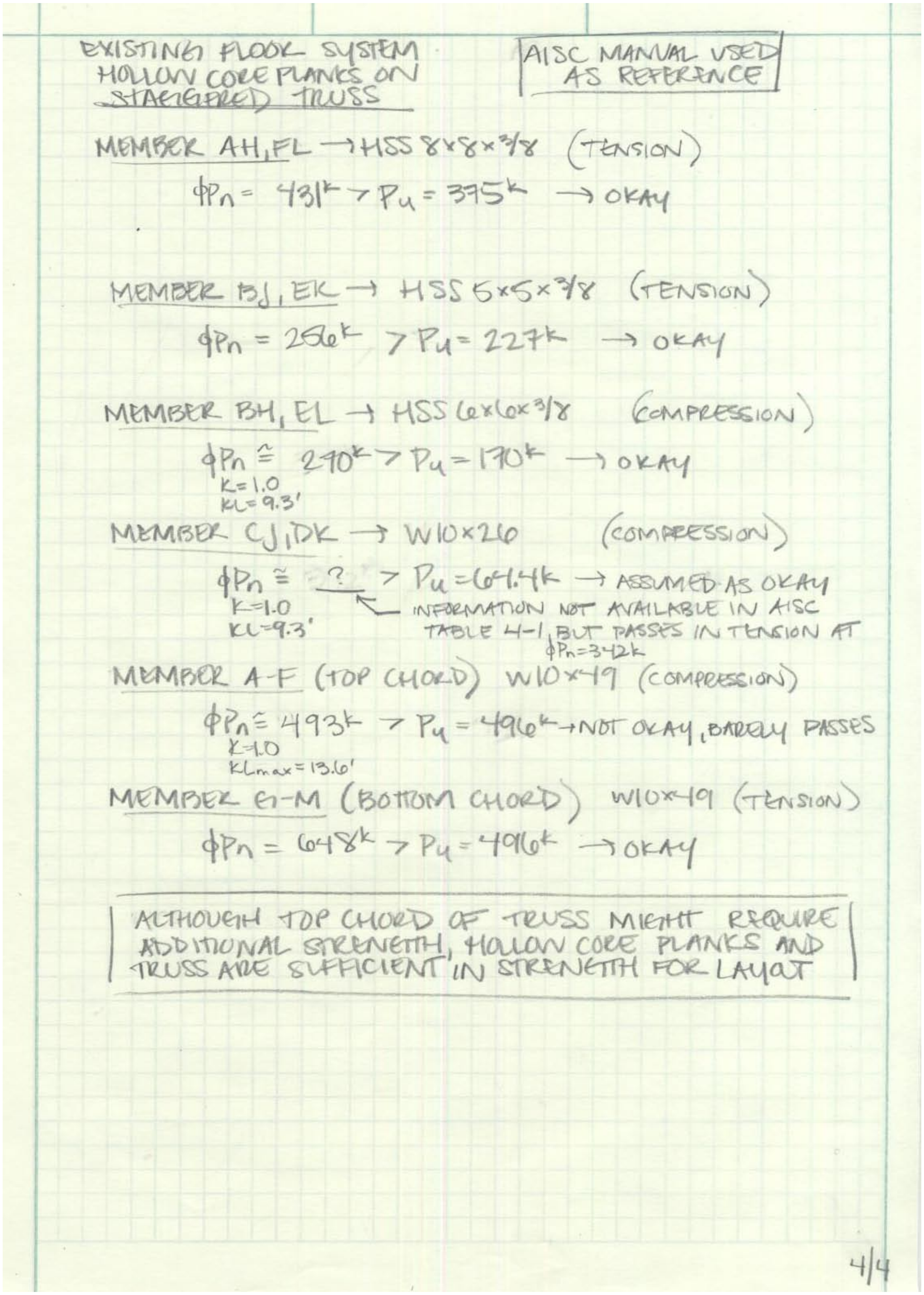
$\overset{+}{\rightarrow} \Sigma F_x = -309^k - 227\cos 34.4 + JK = 0$
$JK = 496^k$

JOINT C

64.4^k
496^k 496^k
64.4^k

TRUSS IS SYMMETRICAL

3/4



EXISTING FLOOR SYSTEM
HOLLOW CORE PLANKS ON
STAGGERED TRUSS

AISC MANUAL USED
AS REFERENCE

MEMBER AH, FL → HSS 8×8×3/8 (TENSION)

$\phi P_n = 431^k > P_u = 375^k$ → OKAY

MEMBER BJ, EK → HSS 5×5×3/8 (TENSION)

$\phi P_n = 256^k > P_u = 227^k$ → OKAY

MEMBER BH, EL → HSS 6×6×3/8 (COMPRESSION)

$\phi P_n \cong 290^k > P_u = 170^k$ → OKAY
$K = 1.0$
$KL = 9.3'$

MEMBER CJ, DK → W10×26 (COMPRESSION)

$\phi P_n \cong$? $> P_u = 64.4k$ → ASSUMED AS OKAY
$K = 1.0$
$KL = 9.3'$
INFORMATION NOT AVAILABLE IN AISC TABLE 4-1, BUT PASSES IN TENSION AT $\phi P_n = 342k$

MEMBER A-F (TOP CHORD) W10×49 (COMPRESSION)

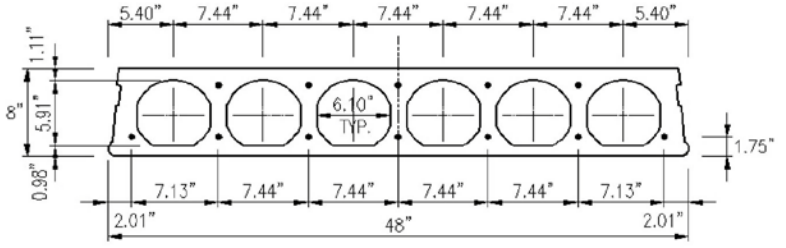
$\phi P_n \cong 493^k > P_u = 496^k$ → NOT OKAY, BARELY PASSES
$K = 1.0$
$KL_{max} = 13.6'$

MEMBER G-M (BOTTOM CHORD) W10×49 (TENSION)

$\phi P_n = 648^k > P_u = 496^k$ → OKAY

ALTHOUGH TOP CHORD OF TRUSS MIGHT REQUIRE ADDITIONAL STRENGTH, HOLLOW CORE PLANKS AND TRUSS ARE SUFFICIENT IN STRENGTH FOR LAYOUT

4/4



Section Properties

		Untopped	Topped (2"
A	=	197 in2	
I	=	1582 in4	2875 in4
Yb	=	4.15 in	5.48 in
Yt	=	3.85 in	4.52 in
Sb	=	381 in3	524 in3
St	=	411 in3	636 in3
Bw	=	11.40 in	11.40 in
wt	=	57 psf	82 psf

Key

180 - Safe superimposed live load, psf
0.2 - Estmated Camber at Erection, in
0.3 - Estmated Long-Time Camber, in

f'c = 5000 psi
f'ci = 3500 psi
f'ct = 3000 psi
fpu = 270 ksi

8" HC (UNTOPPED)

SPAN (Ft)

Strand Pattern	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36
5-3/8	180	155	133	114	98	84	72	62	54	45	38										
	0.2	0.2	0.3	0.4	0.3	0.2	0.2	0.1	0.1	0.1	0.1										
	0.3	0.3	0.3	0.4	0.4	0.4	0.3	0.2	0.1	0	-0.1										
4-1/2	274	236	204	177	154	134	117	102	89	77	67	57	49	42	35						
	0.2	0.2	0.2	0.3	0.3	0.4	0.4	0.3	0.3	0.2	0.1	0	-0.1	-0.2	-0.3						
	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0	-0.1	-0.2	-0.3	-0.4	-0.5						
5-1/2	317	296	275	255	225	200	179	160	143	128	115	104	93	84	76	68	61	55	49	44	39
	0.6	0.6	0.7	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.7	0.7	0.7	0.6	0.5	0.4	0.3	0.2	0.1
	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.4	0.3	0.2	0	-0.2	-0.4	-0.6	-0.9
6-1/2	326	302	284	266	250	236	218	196	176	159	143	130	117	107	97	88	80	72	65	59	54
	0.7	0.8	0.8	0.9	0.9	1.1	1.2	1.3	1.4	1.5	1.6	1.5	1.4	1.3	1.2	1.1	1	0.9	0.8	0.7	0.6
	0.6	0.7	0.8	0.8	0.9	0.9	0.9	1	1	1	1	1	0.9	0.9	0.8	0.7	0.6	0.4	0.2	0	-0.2
7-1/2	335	311	290	272	256	242	229	215	205	188	170	154	141	128	117	106	97	89	81	74	67
	0.6	0.6	0.7	0.8	0.9	1	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.6	1.5	1.3	1.1	0.9	0.7	0.5	0.3
	38	0.8	0.9	1	1	1.1	1.2	1.2	1.3	1.3	1.3	1.3	1.3	1.3	1.2	1.2	1.1	1	0.9	0.7	0.5

8" HC (TOPPED)

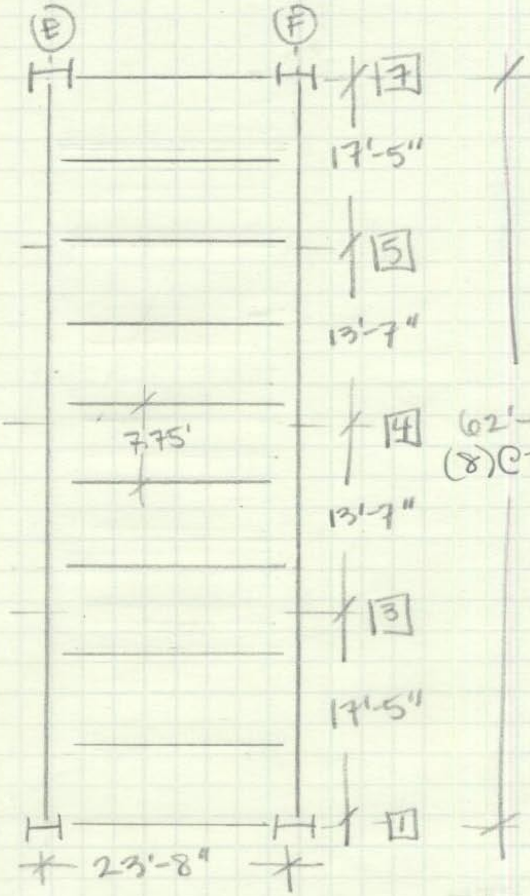
SPAN (Ft)

Strand Pattern	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38
5-3/8	180	155	133	114	98	84	72	62	54	45	38										
	0.2	0.2	0.3	0.4	0.3	0.2	0.2	0.1	0.1	0.1	0.1										
	0.3	0.3	0.3	0.4	0.4	0.4	0.3	0.2	0.1	0	-0.1										
4-1/2	256	222	192	167	145	126	109	94	81	69	59	49	41	33							
	0.2	0.3	0.3	0.4	0.4	0.3	0.3	0.2	0.2	0.2	0.1	0.1	0	-0.1							
	0.2	0.2	0.2	0.1	0.1	0	-0.1	-0.2	-0.3	-0.4	-0.5	-0.6	-0.7	-0.8							
5-1/2	352	317	279	248	220	196	174	156	139	124	111	98	84	71	60	50	40				
	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	1	1	0.9	0.8	0.7	0.6	0.5	0.4	0.3				
	0.5	0.5	0.5	0.5	0.5	0.4	0.4	0.3	0.3	0.2	0.1	-0.1	-0.2	-0.3	-0.4	-0.5	-0.6				
6-1/2		337	316	297	268	239	215	193	173	156	141	127	114	100	87	75	64	54	45	36	
		0.6	0.7	0.8	0.9	1	1.1	1.2	1.3	1.4	1.3	1.2	1.1	1	0.9	0.8	0.7	0.6	0.5	0.4	
		0.6	0.6	0.7	0.7	0.7	0.6	0.6	0.6	0.5	0.4	0.3	0.2	0	-0.2	-0.4	-0.6	-0.9	-1.2	-1.6	
7-1/2		346	325	306	286	271	252	227	205	186	168	152	138	124	111	98	86	76	66	56	47
		0.7	0.8	0.9	1	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.6	1.5	1.4	1.3	1.2	1.1	1	0.9	0.8
		0.8	0.8	0.8	0.9	0.9	0.9	0.9	0.9	0.8	0.8	0.7	0.6	0.5	0.3	0.1	-0.1	-0.3	-0.6	-0.9	-1.3

Figure B.1 Load Table for Hollow Core Planks courtesy of Hoosier Prestress, Inc.

ALTERNATIVE FLOOR SYSTEM
COMPOSITE CELLULAR BEAMS

INPUT FOR SOFTWARE
COURTESY OF CMC STEEL PRODUCTS



(8)@7'-9" ASSUMING SLAB t=4"

BEAM (8) AT 7'-9"
SPAN ~23.7'

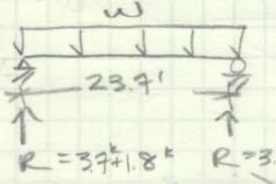
LL = 40psf(7.75') = 310plf

DL = 20 (7.75') = 155plf
↳SUPER IMPOSED ONLY
↳SOFTWARE INCLUDES BEAM SELF WEIGHT

↳SOFTWARE FACTORS LOADS

GIRDER SPACING 23.7'
SPAN = 62'-0"

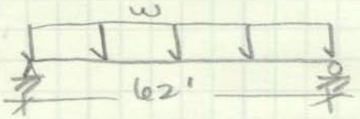
LOADING ON BEAM



$R_L = \frac{310plf(23.7')}{2\times1000} = 3.7k$

$R_D = \frac{155plf(23.7')}{2\times1000} = 1.8^k$

LOADING ON GIRDER



$w_L = 3.7^k/7.75' = 0.477$ klf

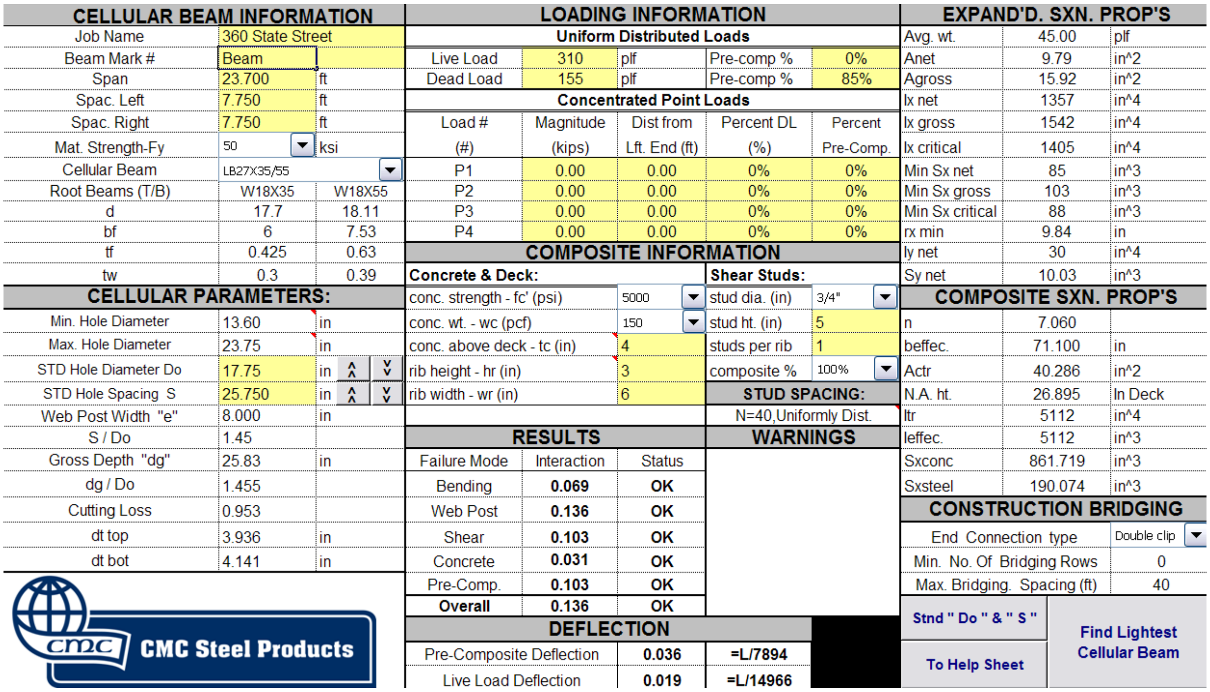
$w_D = 1.8^k/7.75' = 0.232$ klf

$\% DL = \frac{\frac{(150plf)(23.7')}{2}}{\frac{(465plf)(23.7')}{2}}(100) \cong 32\%$

LB27×35/55 SUFFICIENT AS BEAM

LB27×97 SUFFICIENT AS GIRDER
THOUGH ADDITIONAL COLUMNS ARE SUGGESTED TO MINIMIZE DEFLECTION

1/1



CELLULAR BEAM INFORMATION

Job Name	360 State Street	
Beam Mark #	Beam	
Span	23.700	ft
Spac. Left	7.750	ft
Spac. Right	7.750	ft
Mat. Strength-Fy	50	ksi
Cellular Beam	LB27X35/55	
Root Beams (T/B)	W18X35	W18X55
d	17.7	18.11
bf	6	7.53
tf	0.425	0.63
tw	0.3	0.39

CELLULAR PARAMETERS:

Min. Hole Diameter	13.60	in
Max. Hole Diameter	23.75	in
STD Hole Diameter Do	17.75	in
STD Hole Spacing S	25.750	in
Web Post Width "e"	8.000	in
S / Do	1.45	
Gross Depth "dg"	25.83	in
dg / Do	1.455	
Cutting Loss	0.953	
dt top	3.936	in
dt bot	4.141	in

LOADING INFORMATION

Uniform Distributed Loads

Live Load	310	plf	Pre-comp %	0%
Dead Load	155	plf	Pre-comp %	85%

Concentrated Point Loads

Load # (#)	Magnitude (kips)	Dist from Lft. End (ft)	Percent DL (%)	Percent Pre-Comp.
P1	0.00	0.00	0%	0%
P2	0.00	0.00	0%	0%
P3	0.00	0.00	0%	0%
P4	0.00	0.00	0%	0%

COMPOSITE INFORMATION

Concrete & Deck:		Shear Studs:	
conc. strength - fc' (psi)	5000	stud dia. (in)	3/4"
conc. wt. - wc (pcf)	150	stud ht. (in)	5
conc. above deck - tc (in)	4	studs per rib	1
rib height - hr (in)	3	composite %	100%
rib width - wr (in)	6	STUD SPACING:	
		N=40,Uniformly Dist.	

RESULTS

Failure Mode	Interaction	Status
Bending	0.069	OK
Web Post	0.136	OK
Shear	0.103	OK
Concrete	0.031	OK
Pre-Comp.	0.103	OK
Overall	0.136	OK

WARNINGS

DEFLECTION

Pre-Composite Deflection	0.036	=L/7894
Live Load Deflection	0.019	=L/14966

EXPAND'D. SXN. PROP'S

Avg. wt.	45.00	plf
Anet	9.79	in^2
Agross	15.92	in^2
Ix net	1357	in^4
Ix gross	1542	in^4
Ix critical	1405	in^4
Min Sx net	85	in^3
Min Sx gross	103	in^3
Min Sx critical	88	in^3
rx min	9.84	in
Iy net	30	in^4
Sy net	10.03	in^3

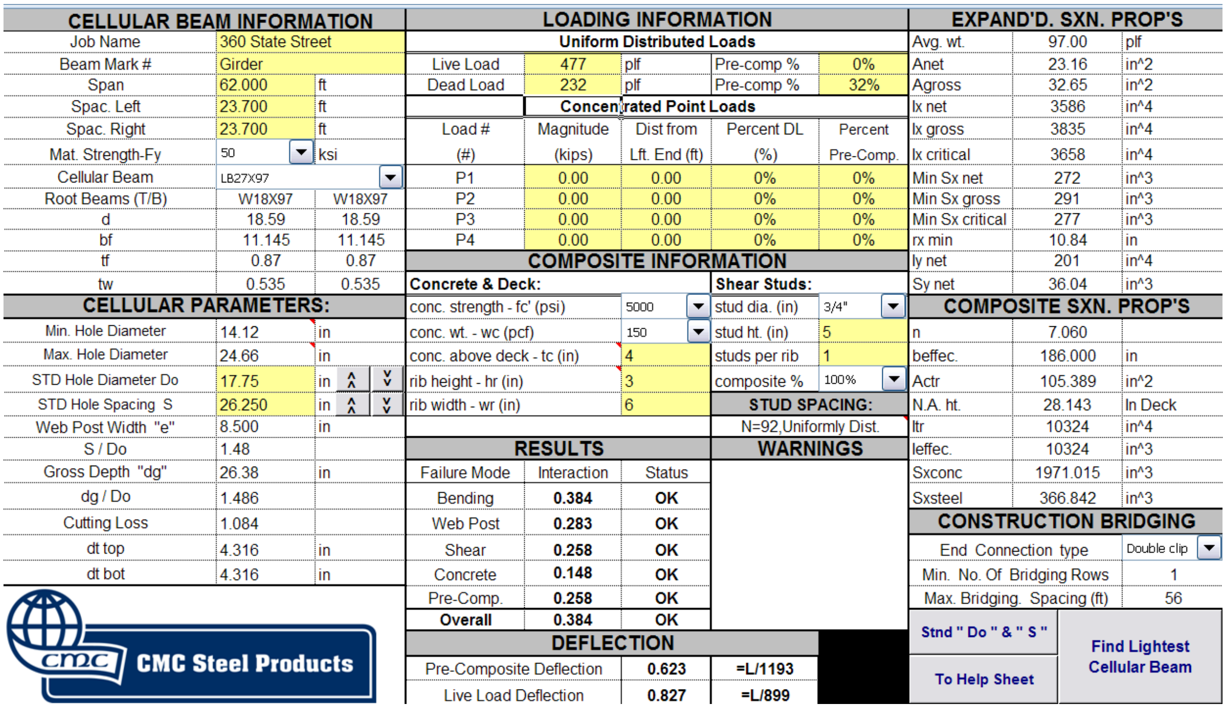
COMPOSITE SXN. PROP'S

n	7.060	
beffec.	71.100	in
Actr	40.286	in^2
N.A. ht.	26.895	In Deck
Itr	5112	in^4
Ieffec.	5112	in^3
Sxconc	861.719	in^3
Sxsteel	190.074	in^3

CONSTRUCTION BRIDGING

End Connection type	Double clip
Min. No. Of Bridging Rows	0
Max. Bridging. Spacing (ft)	40

Figure C.1 Cellular Beam Calculation courtesy of CMC Steel Products



CELLULAR BEAM INFORMATION

Job Name	360 State Street	
Beam Mark #	Girder	
Span	62.000	ft
Spac. Left	23.700	ft
Spac. Right	23.700	ft
Mat. Strength-Fy	50	ksi
Cellular Beam	LB27X97	
Root Beams (T/B)	W18X97	W18X97
d	18.59	18.59
bf	11.145	11.145
tf	0.87	0.87
tw	0.535	0.535

CELLULAR PARAMETERS:

Min. Hole Diameter	14.12	in
Max. Hole Diameter	24.66	in
STD Hole Diameter Do	17.75	in
STD Hole Spacing S	26.250	in
Web Post Width "e"	8.500	in
S / Do	1.48	
Gross Depth "dg"	26.38	in
dg / Do	1.486	
Cutting Loss	1.084	
dt top	4.316	in
dt bot	4.316	in

LOADING INFORMATION

Uniform Distributed Loads

Live Load	477	plf	Pre-comp %	0%
Dead Load	232	plf	Pre-comp %	32%

Concentrated Point Loads

Load # (#)	Magnitude (kips)	Dist from Lft. End (ft)	Percent DL (%)	Percent Pre-Comp.
P1	0.00	0.00	0%	0%
P2	0.00	0.00	0%	0%
P3	0.00	0.00	0%	0%
P4	0.00	0.00	0%	0%

COMPOSITE INFORMATION

Concrete & Deck:		Shear Studs:	
conc. strength - fc' (psi)	5000	stud dia. (in)	3/4"
conc. wt. - wc (pcf)	150	stud ht. (in)	5
conc. above deck - tc (in)	4	studs per rib	1
rib height - hr (in)	3	composite %	100%
rib width - wr (in)	6	STUD SPACING:	
		N=92,Uniformly Dist.	

RESULTS

Failure Mode	Interaction	Status
Bending	0.384	OK
Web Post	0.283	OK
Shear	0.258	OK
Concrete	0.148	OK
Pre-Comp.	0.258	OK
Overall	0.384	OK

WARNINGS

DEFLECTION

Pre-Composite Deflection	0.623	=L/1193
Live Load Deflection	0.827	=L/899

EXPAND'D. SXN. PROP'S

Avg. wt.	97.00	plf
Anet	23.16	in^2
Agross	32.65	in^2
Ix net	3586	in^4
Ix gross	3835	in^4
Ix critical	3658	in^4
Min Sx net	272	in^3
Min Sx gross	291	in^3
Min Sx critical	277	in^3
rx min	10.84	in
Iy net	201	in^4
Sy net	36.04	in^3

COMPOSITE SXN. PROP'S

n	7.060	
beffec.	186.000	in
Actr	105.389	in^2
N.A. ht.	28.143	In Deck
Itr	10324	in^4
Ieffec.	10324	in^3
Sxconc	1971.015	in^3
Sxsteel	366.842	in^3

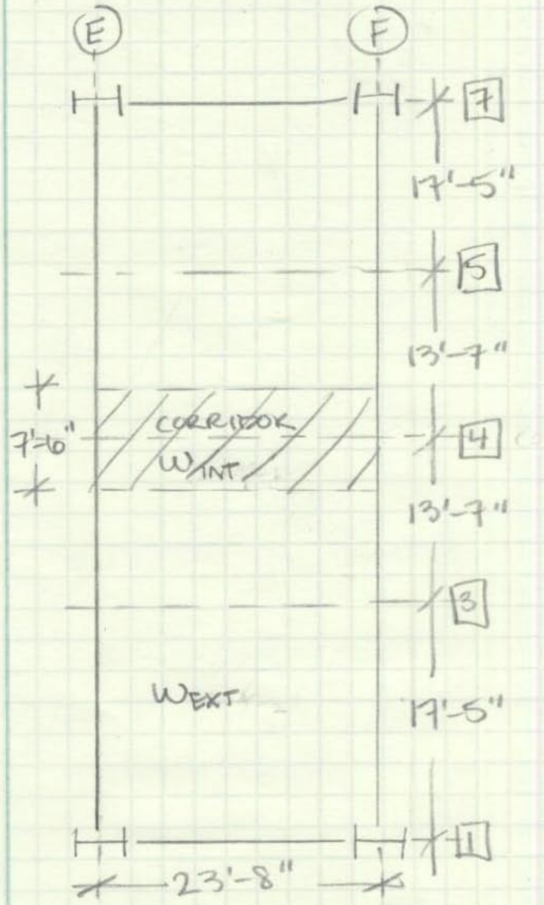
CONSTRUCTION BRIDGING

End Connection type	Double clip
Min. No. Of Bridging Rows	1
Max. Bridging. Spacing (ft)	56

Figure C.2 Cellular Girder Calculation courtesy of CMC Steel Products

ALTERNATIVE FLOOR SYSTEM HAMBRO COMPOSITE

DESIGN BASED STRICTLY ON GRAVITY LOADS, FLOOR SYSTEM WILL NEED TO BE DESIGNED FOR LATERAL LOADS



$w_{LL\,CORR} = 100\,psf$

$w_{LL\,TYP} = 40\,psf$

ASSUME SLAB THICKNESS = 3"

$w_{DL} = 3"/12"(150pcf) + 20psf$ SUPER IMPOSED

$= 57.5 \sim 58psf$

SLAB DESIGN USING TABLE 1 IN HAMBRO MANUAL

$t = 3"$ FOR 2HR FIRERATING

$w_{TOTAL} = 1.7(w_{LL} + w_{DL})$
WHERE 1.7 FACTOR IS RECOMMENDED

$w_{INT} = 1.7(100psf + 58psf) \cong 269 ps$

$w_{EXT} = 1.7(40psf + 58psf) \cong 167 psf$

↳ USE 1/2" ROD WITH 6×6W4×W4 EXTERIOR & INTERIOR JOIST SPACING AT 4'-1 1/4"

$W_{max\,EXT} = 279 psf > w_{EXT} = 167psf$ → OKAY

$W_{max\,INT} = 306 psf > w_{INT} = 269 psf$ → OKAY

JOIST DESIGN USING TABLE 6 IN HAMBRO MANUAL

$W_{SERVICE}$: LL = 40psf DL = 58psf
$t = 3"$
SPANNING ~24'
} CHART RECOMMENDS 10" JOIST

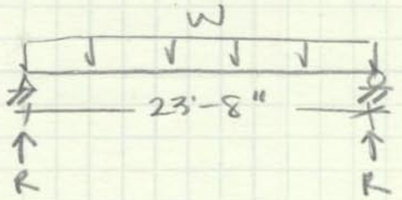
*CHART CONSIDERS $\Delta = \ell/360$

OVERALL DESIGN: 3" SLAB w/ 10" JOISTS AT 4' O.C.
w/ 1/2" ROD AND 6×6W4×W4 WWF

1/2

ALTERNATIVE FLOOR SYSTEM
HAMBRO COMPOSITE

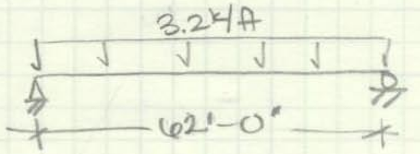
$$w_u = 1.2\underset{58psf}{D} + 1.6\underset{40psf}{L} = 134psf$$



SINGLE JOIST TRIB AREA = 4'

$$W = w_u(A_T) = 134psf(4') = 536plf$$

$$R = \frac{wl}{2} = \frac{536plf(23.7')}{2 \times 1000} \cong 6.4^k$$



SINGLE GIRDER B/T 1-7

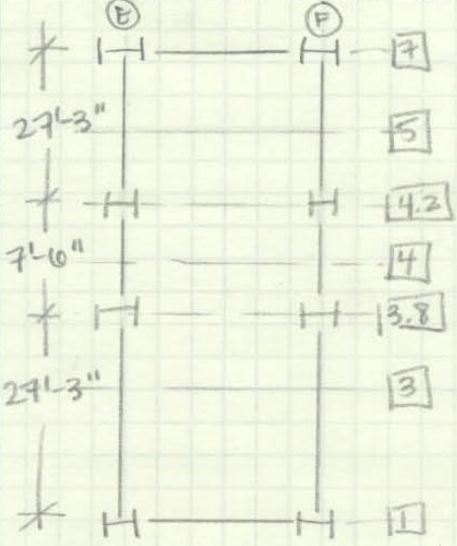
$6.4^k \times 2 = 12.8^k/4'$ FOR BOTH SIDES OF GIRDER

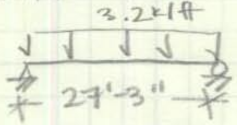
$12.8^k/4' = 3.2^k/ft = w$

$$M = \frac{wl^2}{8} = \frac{(3.2^k/ft)(62')^2}{8} = 1537.6\,kft$$

ANALYZING EXISTING W10x49 AS GIRDER
THIS MOMENT IS BEYOND CAPACITY

→ IF ADDITIONAL COLUMNS WERE PLACED ON E & F

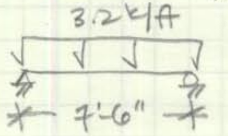




$$M = \frac{(3.2^k/ft)(27.25')^2}{8}$$

$= 297kft$

W10x49 ϕM_n = N/A AT KL=27'
TABLE 3-6 AISC



$$M = \frac{(3.2^k/ft)(4.5')^2}{8}$$

$M = 22.5kft$

W10x49 $\phi M_n = 204kft$ AT KL=8'
TABLE 3-6 AISC

W10x49 IS NOT SUFFICIENT AS A GIRDER
↳ FRAMING SYSTEM MUST BE DESIGNED TO CARRY FLOOR SYSTEM LOADS + ADDITIONAL COLUMNS NECESSARY

2/2

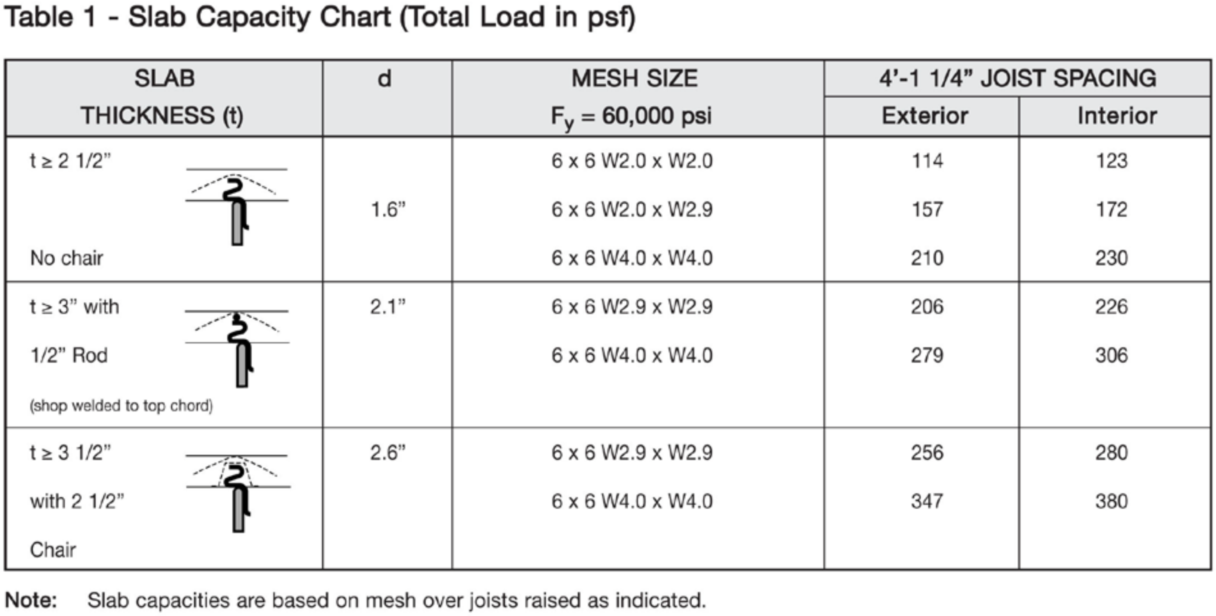


Table 1 - Slab Capacity Chart (Total Load in psf)

SLAB THICKNESS (t)	d	MESH SIZE F_y = 60,000 psi	4’-1 1/4” JOIST SPACING Exterior	Interior
t ≥ 2 1/2” No chair	1.6”	6 x 6 W2.0 x W2.0	114	123
		6 x 6 W2.0 x W2.9	157	172
		6 x 6 W4.0 x W4.0	210	230
t ≥ 3” with 1/2” Rod (shop welded to top chord)	2.1”	6 x 6 W2.9 x W2.9	206	226
		6 x 6 W4.0 x W4.0	279	306
t ≥ 3 1/2” with 2 1/2” Chair	2.6”	6 x 6 W2.9 x W2.9	256	280
		6 x 6 W4.0 x W4.0	347	380

Note: Slab capacities are based on mesh over joists raised as indicated.

Figure D.1 Slab Capacity Chart courtesy of Hambro®

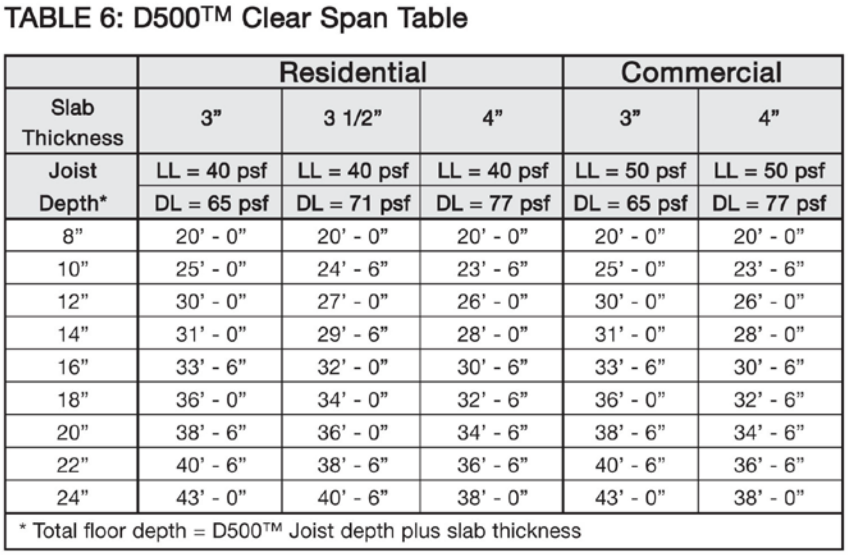


TABLE 6: D500™ Clear Span Table

	Residential			Commercial	
Slab Thickness	3”	3 1/2”	4”	3”	4”
Joist Depth*	LL = 40 psf DL = 65 psf	LL = 40 psf DL = 71 psf	LL = 40 psf DL = 77 psf	LL = 50 psf DL = 65 psf	LL = 50 psf DL = 77 psf
8”	20’ - 0”	20’ - 0”	20’ - 0”	20’ - 0”	20’ - 0”
10”	25’ - 0”	24’ - 6”	23’ - 6”	25’ - 0”	23’ - 6”
12”	30’ - 0”	27’ - 0”	26’ - 0”	30’ - 0”	26’ - 0”
14”	31’ - 0”	29’ - 6”	28’ - 0”	31’ - 0”	28’ - 0”
16”	33’ - 6”	32’ - 0”	30’ - 6”	33’ - 6”	30’ - 6”
18”	36’ - 0”	34’ - 0”	32’ - 6”	36’ - 0”	32’ - 6”
20”	38’ - 6”	36’ - 0”	34’ - 6”	38’ - 6”	34’ - 6”
22”	40’ - 6”	38’ - 6”	36’ - 6”	40’ - 6”	36’ - 6”
24”	43’ - 0”	40’ - 6”	38’ - 0”	43’ - 0”	38’ - 0”

* Total floor depth = D500™ Joist depth plus slab thickness

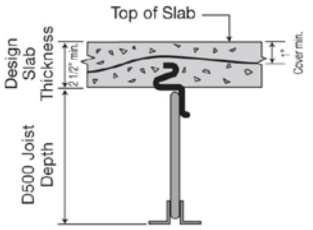
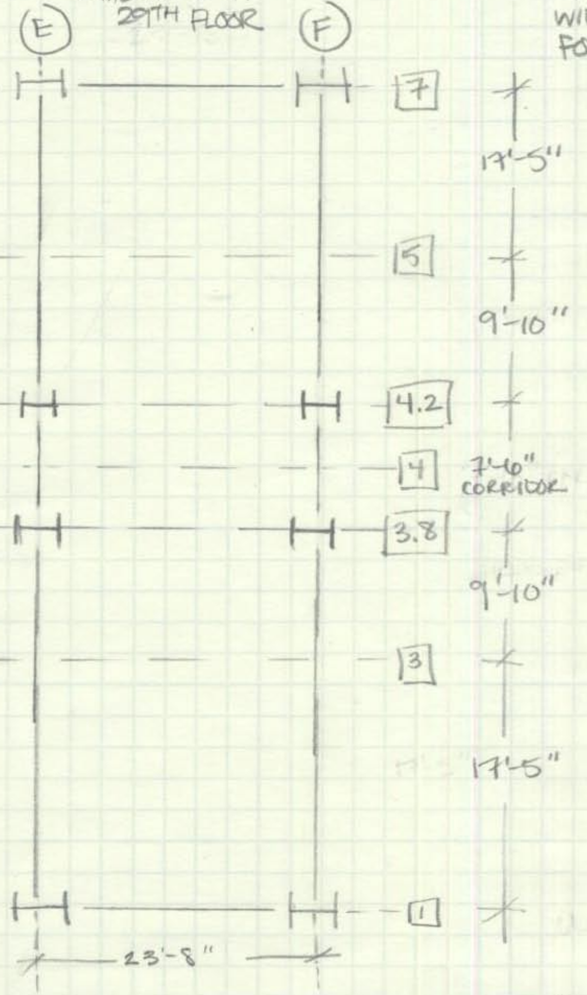


Figure D.2 Joist Depth Chart courtesy of Hambro®

ALTERNATIVE FLOOR SYSTEM
GIRDER-SLAB
EVALUATED AT 29TH FLOOR

DESIGN BASED STRICTLY ON GRAVITY LOADS, FLOOR SYSTEM WILL NEED TO BE DESIGNED FOR LATERAL LOADS



4 COLUMNS ADDED TO ORIGINAL BAY AT CORRIDOR TO COMPENSATE FOR D-BEAMS SPANNING NO MORE THAN 30'

↳ THIS DOES NOT NEGATIVELY IMPACT ARCHITECTURE HOWEVER 2 GRID LINES ARE ADDED

HOLLOW CORE PLANKS USED
8"x48" w/ 2" TOPPING
$f'_c = 5000 psi$
7 1/2" ϕ PRE-STRESSING STRANDS
$f_y = 50{,}000 psi$
$A = 197 in^2$
$I = 1582 in^4$
$E_c = 57000\sqrt{f'_c} = 4030 ksi$

$w_{LL CORR} = 100 psf$
$w_{LL TYP} = 40 psf$
$w_{DL} = 80 psf$ ← INCLUDES SUPERIMPOSED

SPAN 1-3.8 $w_u = 1.2D + 1.6L = 1.2(80psf) + 1.6(40psf) = 160psf$

AT 23'-8" ~ 24' LOAD TABLE: $w_{max} = 252psf > 160psf$ → OKAY

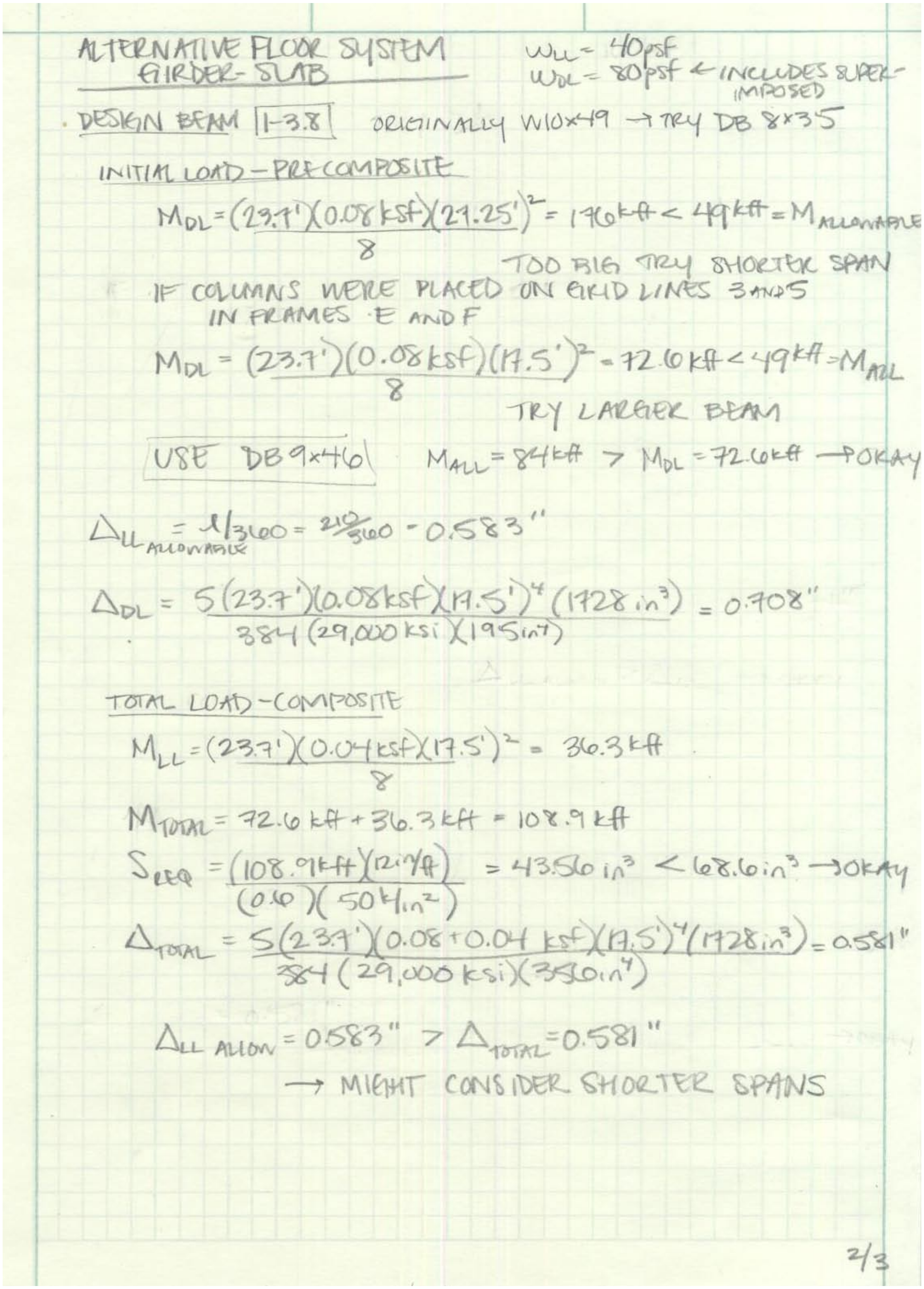
SPAN 3.8-4.2 $w_u = 1.2D + 1.6L = 1.2(80psf) + 1.6(100psf) = 256psf$

AT 23'-8" ~ 24' LOAD TABLE: $w_{max} = 252psf < 256psf$ → NOT OKAY
→ w_u EXCEEDS BY 4psf, ADDITIONAL STRENGTH REQUIRED → POTENTIALLY MORE REINFORCING

PLANK CAPACITY
$w_u = 160psf$ $M_u = \frac{(160psf \times 4')(23.7')^2}{8 \times 1000} = 44.9 k\text{-}ft < \phi M_n = 147.7 k\text{-}ft$
$w_u = 256psf$ $M_u = \frac{(256psf \times 4')(23.7')^2}{8 \times 1000} = 71.9 k\text{-}ft < \phi M_n = 147.7 k\text{-}ft$

$\boxed{\text{8"x48" PLANK w/ 2" TOPPING \& 7 1/2" ϕ STRANDS SUFFICIENT}}$

1/3



ALTERNATIVE FLOOR SYSTEM
GIRDER-SLAB

$w_{LL} = 40\text{psf}$

$w_{DL} = 80\text{psf}$ ← INCLUDES SUPER-IMPOSED

· DESIGN BEAM $\boxed{1\text{-}3.8}$ ORIGINALLY W10×49 → TRY DB 8×35

INITIAL LOAD – PRECOMPOSITE

$$M_{DL} = \frac{(23.7')(0.08\,\text{ksf})(27.25')^2}{8} = 176^{kft} < 49^{kft} = M_{ALLOWABLE}$$

TOO BIG TRY SHORTER SPAN

IF COLUMNS WERE PLACED ON GRID LINES 3 AND 5
IN FRAMES E AND F

$$M_{DL} = \frac{(23.7')(0.08\,\text{ksf})(17.5')^2}{8} = 72.6\,\text{kft} < 49^{kft} = M_{ALL}$$

TRY LARGER BEAM

$\boxed{\text{USE DB9×46}}$ $M_{ALL} = 84^{kft} > M_{DL} = 72.6^{kft}$ → OKAY

$$\Delta_{LL_{ALLOWABLE}} = l/360 = 210/360 = 0.583''$$

$$\Delta_{DL} = \frac{5(23.7')(0.08\,\text{ksf})(17.5')^4(1728\,\text{in}^3)}{384(29{,}000\,\text{ksi})(195\,\text{in}^4)} = 0.708''$$

TOTAL LOAD – COMPOSITE

$$M_{LL} = \frac{(23.7')(0.04\,\text{ksf})(17.5')^2}{8} = 36.3\,\text{kft}$$

$$M_{TOTAL} = 72.6\,\text{kft} + 36.3\,\text{kft} = 108.9\,\text{kft}$$

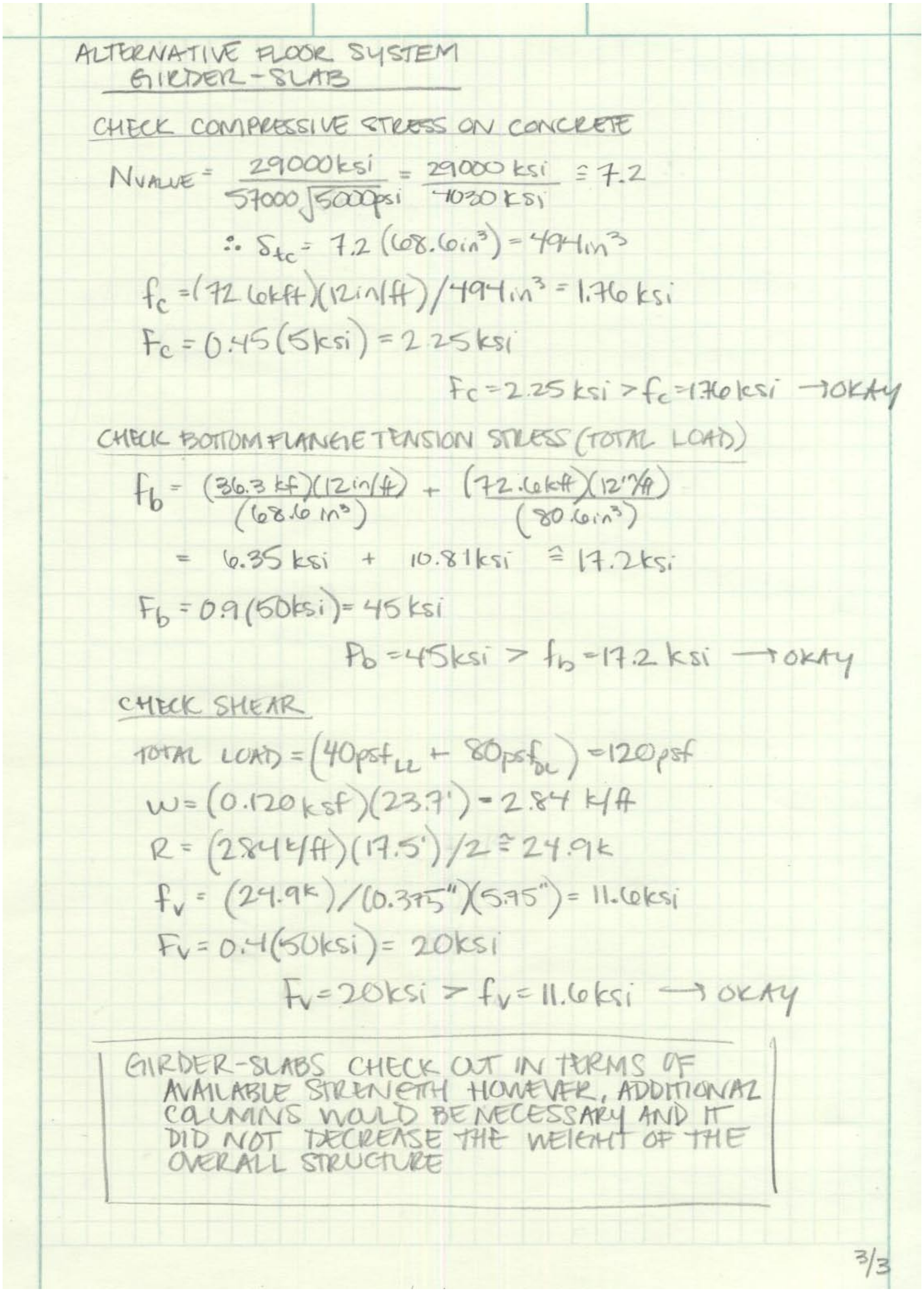
$$S_{REQ} = \frac{(108.9\,\text{kft})(12\,\text{in}/\text{ft})}{(0.6)(50\,\text{k}/\text{in}^2)} = 43.56\,\text{in}^3 < 68.6\,\text{in}^3 \rightarrow \text{OKAY}$$

$$\Delta_{TOTAL} = \frac{5(23.7')(0.08 + 0.04\,\text{ksf})(17.5')^4(1728\,\text{in}^3)}{384(29{,}000\,\text{ksi})(350\,\text{in}^4)} = 0.581''$$

$$\Delta_{LL\ ALLOW} = 0.583'' > \Delta_{TOTAL} = 0.581''$$

→ MIGHT CONSIDER SHORTER SPANS

2/3



ALTERNATIVE FLOOR SYSTEM
GIRDER-SLAB

CHECK COMPRESSIVE STRESS ON CONCRETE

$$N_{VALUE} = \frac{29000\,ksi}{57000\sqrt{5000\,psi}} = \frac{29000\,ksi}{4030\,ksi} \cong 7.2$$

$$\therefore S_{tc} = 7.2\,(68.6\,in^3) = 494\,in^3$$

$$f_c = (72.6\,kft)(12\,in/ft)/494\,in^3 = 1.76\,ksi$$

$$F_c = 0.45\,(5\,ksi) = 2.25\,ksi$$

$F_c = 2.25\,ksi > f_c = 1.76\,ksi \rightarrow$ OKAY

CHECK BOTTOM FLANGE TENSION STRESS (TOTAL LOAD)

$$f_b = \frac{(36.3\,kf)(12\,in/ft)}{(68.6\,in^3)} + \frac{(72.6\,kft)(12''/ft)}{(80.6\,in^3)}$$

$$= 6.35\,ksi + 10.81\,ksi \cong 17.2\,ksi$$

$$F_b = 0.9\,(50\,ksi) = 45\,ksi$$

$F_b = 45\,ksi > f_b = 17.2\,ksi \rightarrow$ OKAY

CHECK SHEAR

TOTAL LOAD $= (40\,psf_{LL} + 80\,psf_{DL}) = 120\,psf$

$$w = (0.120\,ksf)(23.7') = 2.84\,k/ft$$

$$R = (2.84\,k/ft)(17.5')/2 \cong 24.9\,k$$

$$f_v = (24.9\,k)/(0.375'')(5.75'') = 11.6\,ksi$$

$$F_v = 0.4\,(50\,ksi) = 20\,ksi$$

$F_v = 20\,ksi > f_v = 11.6\,ksi \rightarrow$ OKAY

GIRDER-SLABS CHECK OUT IN TERMS OF AVAILABLE STRENGTH HOWEVER, ADDITIONAL COLUMNS WOULD BE NECESSARY AND IT DID NOT DECREASE THE WEIGHT OF THE OVERALL STRUCTURE

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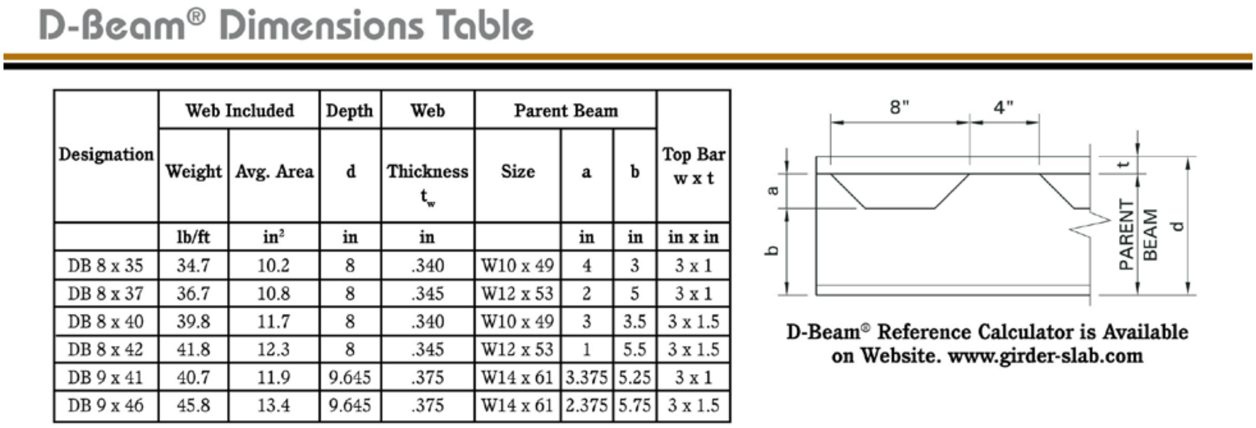
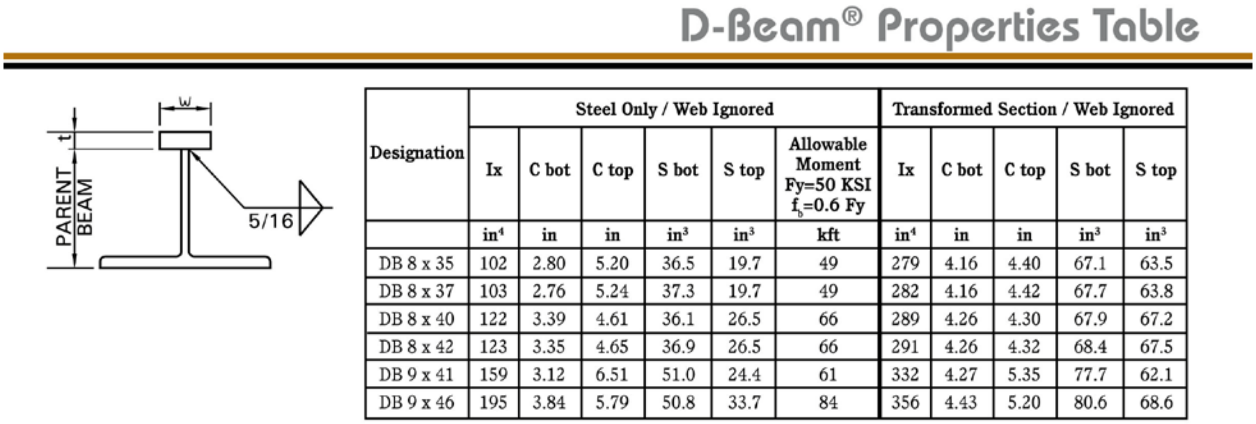


D-Beam® Dimensions Table

Designation	Web Included		Depth	Web	Parent Beam			Top Bar
	Weight	Avg. Area	d	Thickness t_w	Size	a	b	w x t
	lb/ft	in²	in	in		in	in	in x in
DB 8 x 35	34.7	10.2	8	.340	W10 x 49	4	3	3 x 1
DB 8 x 37	36.7	10.8	8	.345	W12 x 53	2	5	3 x 1
DB 8 x 40	39.8	11.7	8	.340	W10 x 49	3	3.5	3 x 1.5
DB 8 x 42	41.8	12.3	8	.345	W12 x 53	1	5.5	3 x 1.5
DB 9 x 41	40.7	11.9	9.645	.375	W14 x 61	3.375	5.25	3 x 1
DB 9 x 46	45.8	13.4	9.645	.375	W14 x 61	2.375	5.75	3 x 1.5

D-Beam® Reference Calculator is Available on Website. www.girder-slab.com

Figure E.1 Beam Dimension Table courtesy of Girder-Slab Technologies



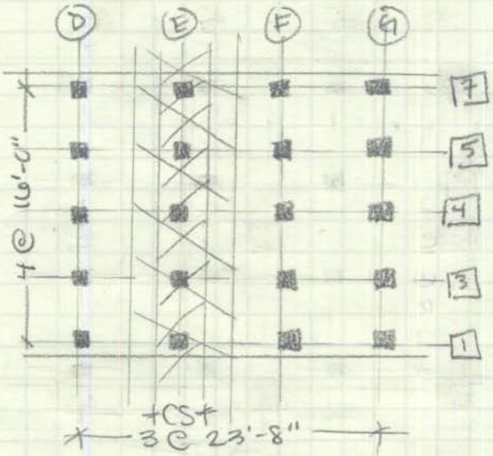
D-Beam® Properties Table

Designation	Steel Only / Web Ignored						Transformed Section / Web Ignored				
	Ix	C bot	C top	S bot	S top	Allowable Moment Fy=50 KSI f_b=0.6 Fy	Ix	C bot	C top	S bot	S top
	in⁴	in	in	in³	in³	kft	in⁴	in	in	in³	in³
DB 8 x 35	102	2.80	5.20	36.5	19.7	49	279	4.16	4.40	67.1	63.5
DB 8 x 37	103	2.76	5.24	37.3	19.7	49	282	4.16	4.42	67.7	63.8
DB 8 x 40	122	3.39	4.61	36.1	26.5	66	289	4.26	4.30	67.9	67.2
DB 8 x 42	123	3.35	4.65	36.9	26.5	66	291	4.26	4.32	68.4	67.5
DB 9 x 41	159	3.12	6.51	51.0	24.4	61	332	4.27	5.35	77.7	62.1
DB 9 x 46	195	3.84	5.79	50.8	33.7	84	356	4.43	5.20	80.6	68.6

Figure E.2 Beam Properties courtesy of Girder-Slab Technologies

ALTERNATIVE FLOOR SYSTEM
FLAT PLATE
EVALUATED AT 29TH FLR

DESIGN BASED STRICTLY ON GRAVITY LOADS, FLOOR SYSTEM WILL NEED TO BE DESIGNED FOR LATERAL LOADS



COLUMNS ASSUMED TO BE 18"x18"
↳ WILL NEED TO BE DESIGNED

$f'_c = 5000$ psi
$f_y = 60{,}000$ psi

$w_{LL} = 100$ psf
$w_{DL_1} = 20$ psf SUPERIMPOSED

DIRECT DESIGN METHOD USED FOR ANALYSIS

CS = COLUMN STRIP
CS = 11.9' WIDTH

CHECK $\ell_2/\ell_1 = 1.49 \leq 2 \rightarrow$ OKAY

$\ell_2 - \ell_1 = 7.8 < \ell_2/3 = 7.93 \rightarrow$ OKAY

→ NO BEAMS OR DROP PANELS

ACI 318-08 TABLE 9-5(c)

$t_{min} = \ell_n/30 = 23.8' \times 12''/30 = 9.52''$

USE SLAB THICKNESS $t = 10''$

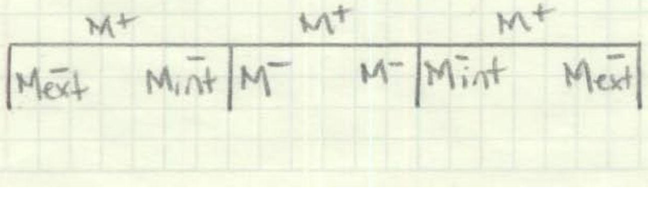
$w_{DL\text{-}SLAB} = (10''/12'')(150\text{pcf}) = 125\text{psf}$

$w_u = 1.2D + 1.6L = 1.2(125 + 20\text{psf}) + 1.6(100\text{psf})$

$w_u = 334\text{psf} \sim 0.334\text{ksf}$

$M_{oE} = \frac{1}{8} w_u \ell_1 \ell_n^2 = \frac{(0.334\text{ksf})(23.8')(16')^2}{8}$

$M_{oE} = 254$ kft



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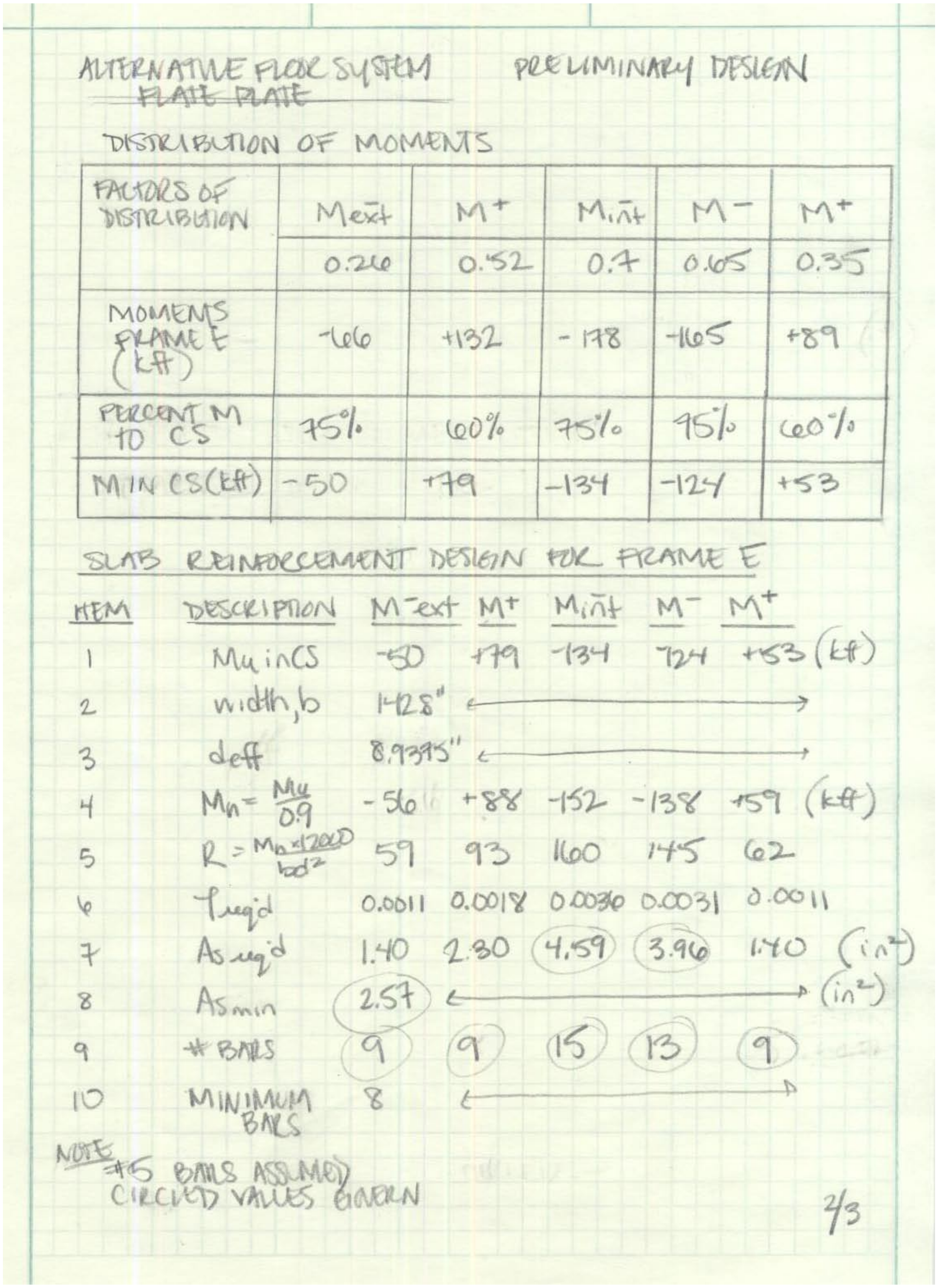

ALTERNATIVE FLOOR SYSTEM PRELIMINARY DESIGN

~~FLATE PLATE~~

DISTRIBUTION OF MOMENTS

FACTORS OF DISTRIBUTION	Mext	M+	Mint	M−	M+
	0.26	0.52	0.7	0.65	0.35
MOMENTS FRAME E (kft)	-66	+132	-178	-165	+89
PERCENT M TO CS	75%	60%	75%	95%	60%
M IN CS (kft)	-50	+79	-134	-124	+53

SLAB REINFORCEMENT DESIGN FOR FRAME E

ITEM	DESCRIPTION	M−ext	M+	Mint	M−	M+	
1	Mu in CS	-50	+79	-134	-124	+53	(kft)
2	width, b	128" ⟷					
3	deff	8.9395" ⟷					
4	$M_n = \frac{M_u}{0.9}$	-56	+88	-152	-138	+59	(kft)
5	$R = \frac{M_n \times 12000}{bd^2}$	59	93	160	145	62	
6	ρ req'd	0.0011	0.0018	0.0036	0.0031	0.0011	
7	As req'd	1.40	2.30	4.59	3.96	1.40	(in²)
8	As min	2.57 ⟷					(in²)
9	# BARS	9	9	15	13	9	
10	MINIMUM BARS	8 ⟷					

NOTE
#5 BARS ASSUMED
CIRCLED VALUES GOVERN

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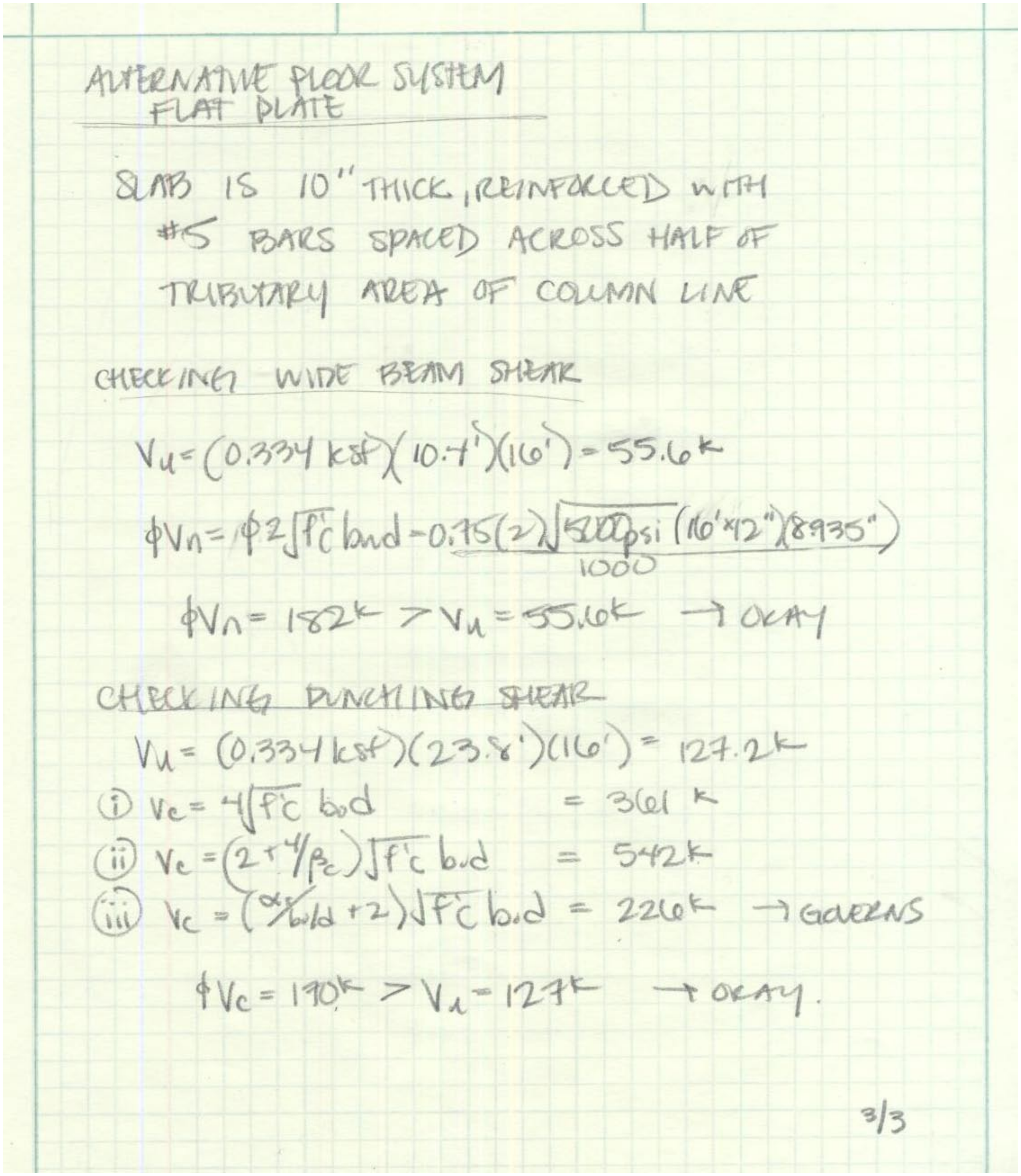

ALTERNATIVE FLOOR SYSTEM
FLAT PLATE

SLAB IS 10'' THICK, REINFORCED WITH #5 BARS SPACED ACROSS HALF OF TRIBUTARY AREA OF COLUMN LINE

CHECKING WIDE BEAM SHEAR

$$V_u = (0.334\text{ ksf})(10.4')(16') = 55.6^k$$

$$\phi V_n = \phi 2\sqrt{f'_c}\, b_w d - \frac{0.75(2)\sqrt{5000\text{psi}}\,(16'\times 12'')(8.935'')}{1000}$$

$$\phi V_n = 182^k > V_u = 55.6^k \rightarrow \text{OKAY}$$

CHECKING PUNCHING SHEAR

$$V_u = (0.334\text{ ksf})(23.8')(16') = 127.2^k$$

ⓘ $V_c = 4\sqrt{f'_c}\, b_o d = 361^k$

ⓘⓘ $V_c = (2 + 4/\beta_c)\sqrt{f'_c}\, b_o d = 542^k$

ⓘⓘⓘ $V_c = (\alpha_s d/b_o + 2)\sqrt{f'_c}\, b_o d = 226^k$ → GOVERNS

$$\phi V_c = 190^k > V_u = 127^k \rightarrow \text{OKAY.}$$

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