ECMC Skilled Nursing Facility

Architectural Engineering Senior Thesis 2011

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Faculty Consultant:	Dr. Ali Memari
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Brian Brunnet | Architectural Engineering | Structural Option

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Executive Summary

The purpose of Technical Report 2 is to explore the strength, serviceability, weight, cost, and aesthetic features between the existing floor system at ECMC Skilled Nursing Facility and three other different types of floor systems. Using hand calculations and current industry standards such as ASCE 7, the AISC Steel Construction Manual, and the ACI Building Code Requirements, each system will be evaluated and tested for viability as a floor system alternative. The existing floor system consists of a 5¼" thick LWC composite slab with composite steel beams and girders. The three systems designed in this report include:

- Non-Composite Steel Framing with Non-Composite Steel Deck
- One-way Post Tensioned Concrete Flat Plate
- Precast Hollow core Plank on Steel Girders

The design of the non-composite steel system results in 4" concrete topping on 2" Vulcraft 2C22 non-composite deck. The framing is W18x35 infill beams spanning 29'-2" with W21x48 girders spanning 26'-0". This is a simple system to design and nearly similar in deck weight, however because of the lack of composite action, the beams and girders must be larger in section to support the full stresses involved. This system does have the ability to be cored without receiving any significant structural strength issues. This system lacks in adequate fireproofing and would need either a spray-on fire protection or fire resistive drop ceiling. This system is relatively uneconomical and with an existing system using composite action, it was deemed as an unacceptable choice.

An 8" thick LWC slab using tendons composed of (12) 0.196" diameter prestressing wires resulted from the post tensioned floor system design. This post tensioned slab system weighed more than the composite system due to an increased slab thickness; however, it had the least system depth due to the absence of infill beams or girders. The cost of the system was also the lowest of the four. The only issues found with the PT system are that the slab cannot be easily cored for any future changes, and it increases the difficulty to construct the post tensioned slab system due to additional details. The benefits mainly outweigh the flaws, making this system a viable alternative.

Using design data sheets from Nitterhouse Concrete Products, a hollow core plank system was designed consisting of a 6"x4'-0" hollow core plank with 2" of concrete topping. These planks utilized (6) ½" diameter low-relaxation steel strands to create an uplifting camber. Steel girders varying in size were used to support the hollow core planks. The column layout required an extra set of columns to reduce the largest span by about 10 feet, which decreased the floor plan layout availability. Large lead times and a high cost are a few drawbacks of hollow core plank systems; however the constructability of this system is very easy and makes it a feasible alternative.

Introduction

The new ECMC Skilled Nursing Facility serves as a long term medical care center for citizens found throughout the region. The building is located on the ECMC campus found at 462 Grider Street in Buffalo, NY. This site was chosen to bring residents closer

to their families living in the heart of Buffalo. As you can see here in Figure 1, the site sits right off the Kensington Expressway, providing ease of access to commuters visiting the ECMC Skilled Nursing Facility. Since the Erie County Medical Center is found within close proximity of the new building, residents can receive fast and effective care in an event of emergency.

The new facility is the largest of four new structures being built on the ECMC

Figure 1: Aerial view of ECMC Skilled Nursing Facility site shown in white. Photo courtesy of Bing Maps.

campus located in central Buffalo, NY. The new campus will also contain a new Renal Dialysis Center, Bone Center, and parking garage. Each of the three new facilities will be connected to the main medical center via an axial corridor, which provides enclosed access to emergency rooms, operation rooms, and other facilities found within the Erie County Medical Center.

Architectural Overview

The new Erie County Medical Center Skilled Nursing Facility is a five-story 296,489 square-foot building offering long-term medical care for citizens in the region. The facility consists of an eight-wing design with a central core. The main entrance to the building is located to the east and is sheltered from the elements by a large porte-cochere. There is a penthouse level that contains the facility's mechanical and HVAC units. Each floor features one garden terrace, providing an outdoor space accessible to both residents and staff. The exterior of the building is clad in brick, stone veneers, composite metal panels, and spandrel glass curtain wall system.

Figure 2: Exterior view of stacked garden terraces, green wall, and the building's vertical and horizontal shading panels. Rendering courtesy of Cannon Design.

The facility also incorporates green building into many of its elegant features. The composite metal panels that run vertically and horizontally across each wing of the building, visible in Figure 2, provide solar shading along with architectural accent. A green wall is featured on each outdoor garden terrace, providing residence with a sense of nature and greenery. The ECMC Skilled Nursing Facility provides an eclectic, modern atmosphere and quality care for long-term care patients found within the Buffalo area.
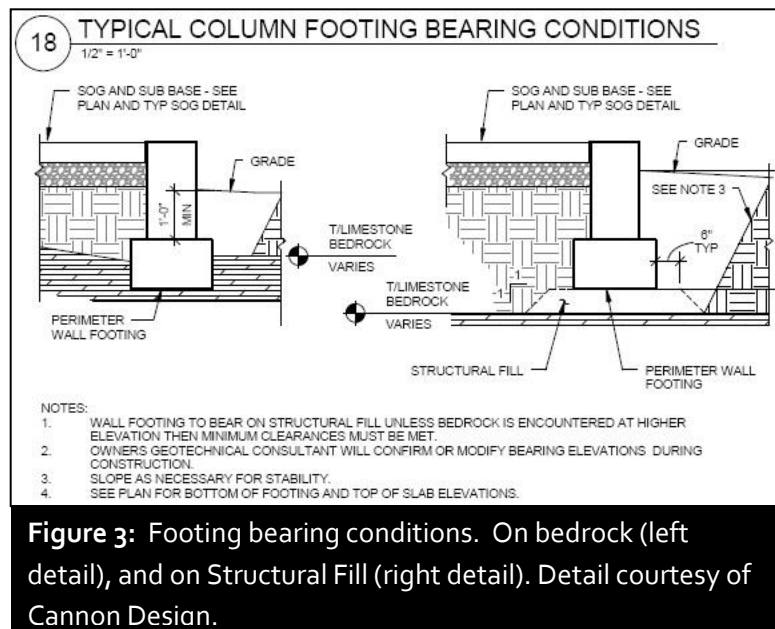
Structural Systems Overview

The ECMC Skilled Nursing Facility consists of 8 wings and a central core, with an overall building footprint of about 50,000 square feet. The building sits at a maximum height of 90' above grade with a common floor to floor height of 13'-4". The ECMC Skilled Nursing Facility mainly consists of steel framing with a 5" concrete slab on grade on the ground floor. The Penthouse level contains 6.5" thick normal weight concrete slab on metal deck. All other floors have a 5.25" thick lightweight concrete on metal deck floor system. All concrete is cast-in-place.

Foundation System

The geotechnical report was conducted by Empire Geo Services, Inc. The study classified the soils using the Unified Soil Classification System, and found that the indigenous soils consisted mainly of reddish brown and brown sandy silt, sandy clayey silt, and silty sand. The ECMC Skilled Nursing Facility foundations sit primarily on limestone bedrock, although in some areas the foundation does sit on structural fill. Depths of

Figure 3: Footing bearing conditions. On bedrock (left detail), and on Structural Fill (right detail). Detail courtesy of Cannon Design.
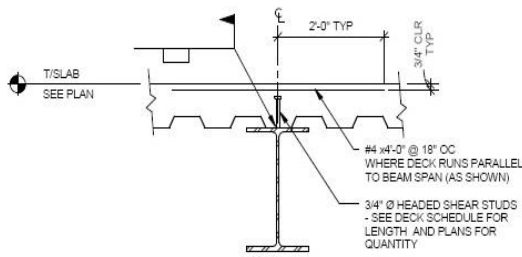
limestone bedrock range from 2ft to 12ft. The building foundations of the ECMC Skilled Nursing Facility are comprised of spread footings and concrete piers with a maximum bearing capacity of 5,000 psf for footings on structural fill and 16,000 psf for footings on limestone bedrock. Concrete piers range in size from 22" to 40" square.

Floor System

The floor system on all floors except at the penthouse level consists of a 5.25" thick lightweight concrete floor slab on 2" - 20 gage metal decking, creating a one-way composite floor slab system. The concrete topping contains 24 pounds per cubic yard of blended fiber reinforcement. Steel decking is placed continuous over three or more spans except where framing does not permit. Shear studs are welded to the steel framing system in accordance to required specification. Refer to Figures 4 and 5 for composite system details.

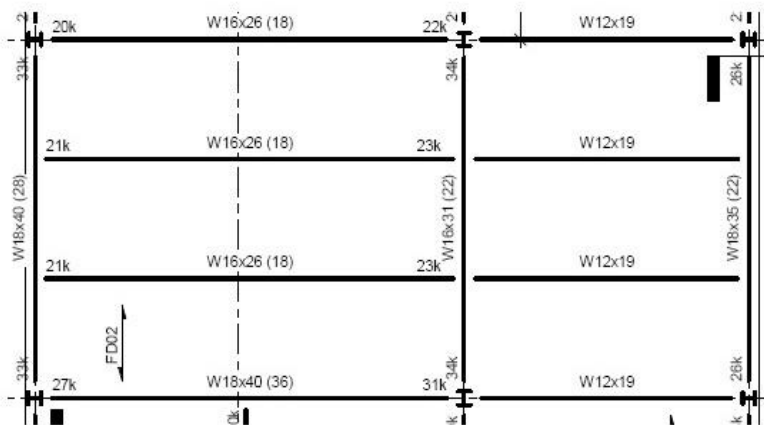**Figure 4:** Composite deck system (parallel edge condition). Detail courtesy of Cannon Design.
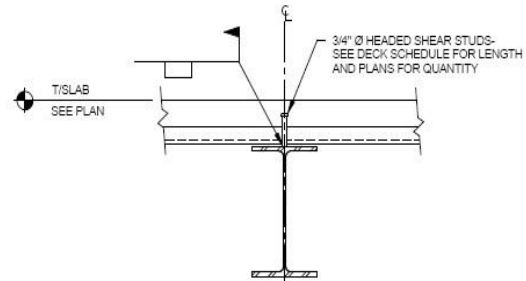
Figure 5: Composite deck system (perpendicular edge condition). Detail courtesy of Cannon Design.

Framing System

**Figure 6:** Typical bay layout for building wing. Detail courtesy of Cannon Design.

The structural framing system is primarily composed of W10 columns and W12 and W16 beams; however the girders vary in sizes ranging from W14 to W24, mainly depending on the size of the span and applied loads on the girder. Typical beam spacing varies from 6'-8"o.c. to 8'-8"o.c. Figure 6 shows a typical grid layout for a building wing. Columns are spliced at 4' above the 2nd and 4th floor levels, and typically span between 26'-8" and 33'-4".

Lateral System

The lateral resisting system consists of a concentrically brace frame system composed of shear connections with HSS cross bracing. Lateral HSS bracing is predominantly located at the end of each wing, and also found surrounding the central building core. Because of the radial shape of the building and symmetrical layout of the structure, the brace framing can oppose seismic and wind forces from any angle. The HSS bracing size is mainly HSS 6x6x3/8, but can increase in size up to HSS 7x7x1/2 in some ground floor areas for additional lateral strength. Figure 7 contains multiple details and an elevation of a typical brace frame for the ECMC Skilled Nursing Facility.
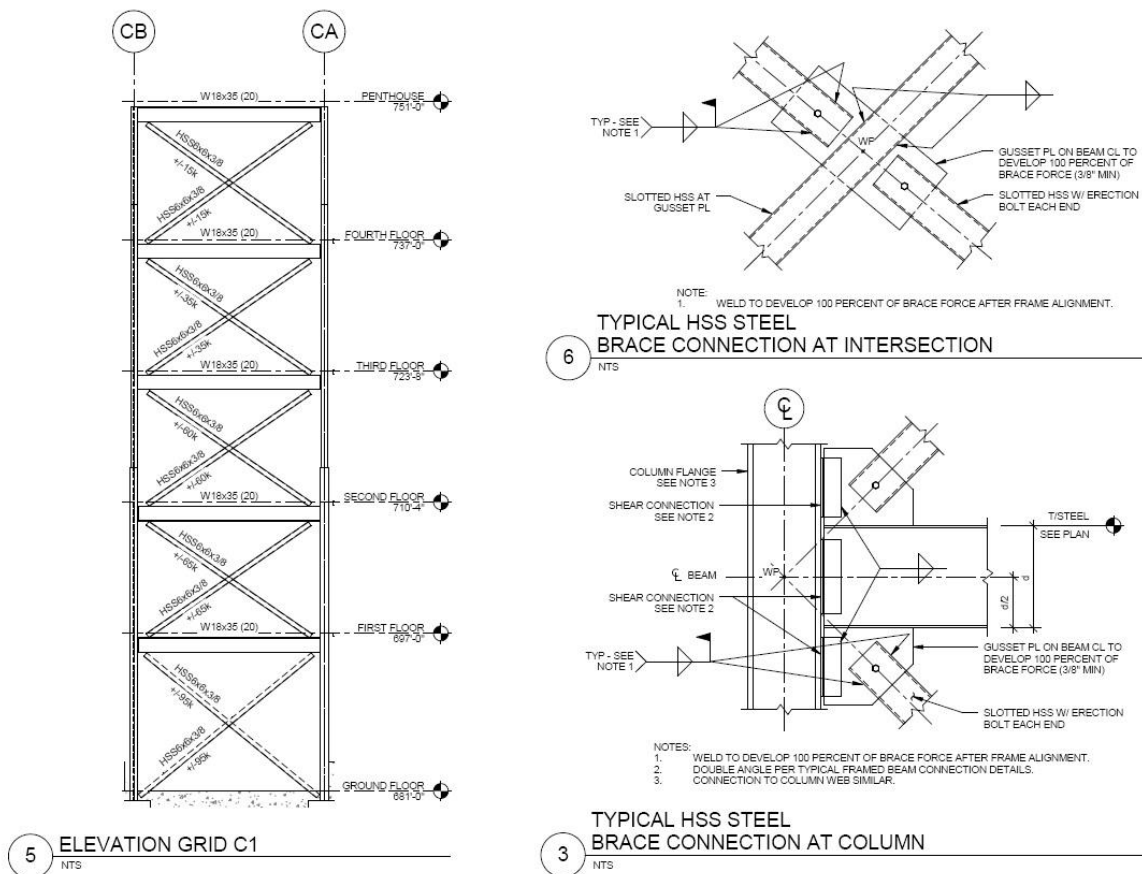
Figure 7: Typical lateral HSS brace frame (left). Typical HSS steel brace connection at intersection (upper right). Typical HSS steel brace connection at column (lower right). Details courtesy of Cannon Design.

Design Codes and Standards

Original Codes:

Design Codes:

- ACI 318-02, *Building Code Requirements for Structural Concrete*
- ACI 530-02, *Building Code Requirements for Masonry Structures*
- AISC LRFD - 3rd Edition, *Manual of Steel Construction: Load and Resistance Factor Design*
- AWS D1.1 - 00, *Structural Welding Code - Steel*

Model Code:

- NYS Building Code - 07, *Building Code of New York State 2007*

Structural Standard:

- ASCE 7-02, *Minimum Design Loads for Buildings and Other Structures*

Thesis Codes:

Design Codes:

- ACI 318-08, *Building Code Requirements for Structural Concrete*
- AISC Steel Construction Manual - 13th Edition (LRFD), *Load and Resistance Factor Design Specification for Structural Steel Buildings*

Model Code:

- IBC - 06, *2006 International Building Code*

Structural Standard:

- ASCE 7-10, *Minimum Design Loads for Buildings and Other Structures*

Material Properties

Structural Steel		
Wide Flange Shapes, WT Sections	ASTM A992	
Channels and Angles	ASTM A36	
Pipe	ASTM A53 Grade B	
Hollow Structural Sections (Rectangular and Round)	ASTM A500 Grade B	
Base Plates	ASTM A36 UNO	
All Other Steel Members	ASTM A36 UNO	
High Strength Bolts, Nuts, and Washers	ASTM A-325 / A-490 (Min. 3/4" Diameter)	
Anchor Rods	ASTM F1554	
Steel Shape Welding Electrode	E70XX	
Concrete	**F'c (psi)**	**Unit Weight (pcf)**
Footings	f'c = 3000psi	145
Foundation Walls	f'c = 4000psi	145
Slabs-on-Grade	f'c = 3000psi	145
Slabs-on-Steel Deck (Floor Deck 1)	f'c = 3000psi	145
Slabs-on-Steel Deck (Floor Deck 2)	f'c = 3000psi	115
All Other Concrete	f'c = 4000psi	145
Reinforcement		
Typical Bars	ASTM A-615 Grade 60	
Welded Bars	ASTM A-706 Grade 60	
Welded Wire Fabric	ASTM A-185	
Steel Fibers	ASTM A-820 Type 1	
Decking		
Floor Deck (both types)	2" Composite Metal Deck, 20 Ga.	
Roof Deck Type 1	1 1/2" Type B Metal Roof Deck, 20 Ga.	
Roof Deck Type 2	1 1/2" Type B Metal Roof Deck, 18 Ga.	
3/4" Shear Studs	ASTM A-108	

Table 1: This table describes material properties found throughout the building.

Design Loads

Dead and Live Loads

The original structure of the ECMC Skilled Nursing Facility was designed using ASCE 7-02 and the 2007 NYC Building Code. These load cases are compared to the newer ASCE 7-10 standard. Their differences can be seen in Table 2 below. Loads used for thesis analysis are from the ASCE 7-10 standards unless unspecified in the code. Refer to Appendix B for Dead Load Calculations/Assumptions.

Superimposed Dead Loads			
Description	**Location**	**NYC-BC 2007**	**ASCE 7-10**
Roof Deck 1	Roof	2 psf	2 psf
Roof Deck 2	Penthouse Roof	3 psf	2 psf
Floor Deck 1	Penthouse Floor	2 psf	2 psf
Floor Deck 2	Floors 1-4	2 psf	2 psf
Floor Finishings	Floors 1-4	2 psf	2 psf
Roofing & Insulation	Roof + Penthouse Roof	8 psf	8 psf
Leveling Concrete	Floors 1-4	5 psf	5 psf
Ceilings	Floors 1-4 + Penthouse	5 psf	5 psf
Typical Suspended MEP	Floors G-4	5 psf	5 psf
Penthouse Suspended MEP	Penthouse	8 psf	8 psf
Partitions	Floors 1-4	18 psf	18 psf
Pavers, Potted Plants	Floors 1-4	80 psf	- -
Green Wall (4"thick)	Floors 1-4	20 psf	- -
Live Loads			
Description		**NYC-BC 2007**	**ASCE 7-10**
Resident Rooms	Floors G-4	40 psf	40 psf
Ground Floor Corridors	Floor G	80 psf	100 psf
Balconies	Floors 1-4	Not Specified	100 psf
Resident Corridors	Floors 1-4	80 psf	80 psf
Penthouse Floor	Penthouse	150 psf	150 psf
Public Spaces/Exit Corridors/ Stairs/Lobbies	Floors G-Penthouse	100 psf	100 psf
*Live load reductions used where applicable ** Snow drift included where applicable			

Table 2: The table above shows a list of dead and live loads used in the various calculations found in this report, along with a comparison of loads between the NYC BC-2007 versus ASCE 7-10.

Floor Systems

Composite Beam & Girder System (Existing)

The existing system for the ECMC Skilled Nursing Facility consisted of composite beams and girders using ¾" diameter headed shear studs to help transfer compressive stresses to the concrete in the slab. This method greatly increases the strength capabilities of beams and girders, allowing the use of smaller shapes and longer spans.

Because of the building's unusual shape, it does not have a simple rectangular bay layout throughout. However, a bay of 29'-2"x26' was selected for analysis and design, since this bay was one of the largest bays on the residential floors. A 2VLI20 composite floor deck with a total slab depth of 5-1/4" was chosen in order to match the typical floor deck, concrete type, and slab thickness as specified in the drawings. Within this bay were 2 intermediate W16x31 wide flange beams, which both required 20 shear studs to transfer compressive loads to the slab. Both beams along with another set of W14x22 beams connected to a W18x35 transfer girder that required 22 shear studs for strength.

Upon checking the existing system, it was found that the 2VLI20 slab contained adequate strength to meet the load requirements. The designer possibly chose this deck for an ease of constructability as well as it has a 2 hour fire rating. The W16x31 beams and the W18x35 transfer girder adequately carried the loads when considering shear and moment strength, and upon comparison to other systems they were both relatively overdesigned for the loads calculated. The reason for this may be due to specific loads considered in the design, but primarily it is ultimately believed that deflection limitations controlled the design.

Advantages

Composite beam and girder systems provide many advantages to a framing system. This system allows beams to span longer distances due to the transferred compressive strength acquired from the floor slab. By causing the slab to undergo compressive stress, the system as a whole gains more moment and shear capacity. Another advantage for this system is that you can save considerably on project cost since you will use smaller steel shapes. Since the concrete takes some of the stresses off the steel, the steel shape can be downsized to make the system more economical. A great advantage of using composite decking is that you can erect the decking without use of shoring, which greatly cuts down on labor costs and installation times.

Disadvantages

The main disadvantage to using a composite beam and girder system is the issue with ceiling heights. An economical composite system usually creates a relatively deep floor system due to deeper beams usually ranging from 1 or more feet in depth. To compensate for this, the architect usually increases the floor to floor height, which can increase construction costs. Increasing floor to floor height may also cause zoning issues depending on the area that the building is being built, where it may have height restrictions. Architecturally, the exposed steel beneath is generally unappealing and is typically hidden using a suspended ceiling, which may also lead to a considerable increase in building cost.

Figure 8: Composite Girders used in a steel bridge (left).
Typical composite construction. (right).
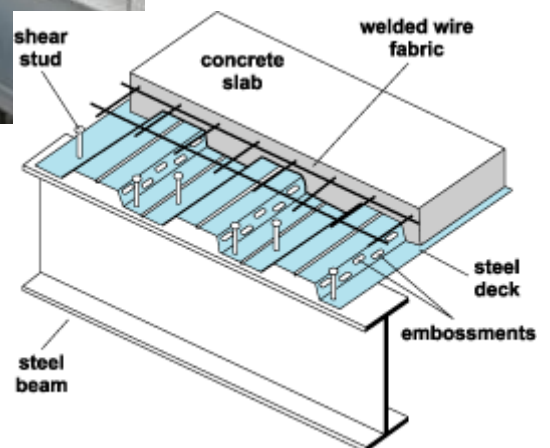
Non-Composite Steel Framing System

Non-composite steel framing systems are generally very easy to design and construct. Known as a common system in the early 20th century, this system allowed buildings at the time the opportunity reach new heights such as in the form of skyscrapers. However advancements in building technology such as composite decking and spray on fireproofing have made this structural system outdated and more costly than typical systems today.

In this report, the general bay size of 29'-2"x26' was used to compare this system with the existing system, as well as followed the same framing layout. This allows the reader to see the structural advantages that come with composite decking and composite framing systems. The report also evaluates non-composite decking when compared to the existing composite floor decking, which also shows the differences in slab thickness and clear span strengths.

Upon evaluation of the non-composite system, it is found that a 2C22 non-composite deck with 6" of total slab thickness was chosen to match fire protection requirements of the original, and calculations conclude that the deck is adequate for the loads. This slab was slightly thicker than the original; however it provided less capacity at the specific clear span of 8'-8". Furthermore, as for the intermediate beams within the bay, a W18x35 beam would be sufficient in carrying the loads. When compared to the existing structure, the W18x35 beams are deeper and heavier than the W16x31 beams and also provided less strength in both shear and moment along with larger deflections. The girder calculation produced a similar situation as was found with the beams. A W21x48 transfer girder would be effective at meeting the design criterion; however it is also deeper and heavier than the existing W18x35 transfer girder.

Since this system doesn't allow the concrete slab to carry any stresses, the steel must carry the full effect from the applied loads. This would cause larger beam sizes when compared to a composite system using the same bay size and framing plan.

Advantages

The advantage to using this system is that it has been used for many years in the past, and the majority of building designers and structural engineers easily understand how to design this system economically and efficiently. Many construction companies and steel erectors are also familiar with constructing this system, which can save time and money when erecting a building using this system.

Disadvantages

The main disadvantage to using this system is the fact that the structural framework receives greater stresses under loading since it carries the full set of loads. In composite construction, the concrete slab contains the compressive stresses involved, allowing the structural steel framework to be smaller in size at the same spans. Another disadvantage to this system is that since the steel is under higher stresses, it leads to considerably deeper and heavier steel shapes, with wide flanges ranging in depth from 2 to 3 feet. This leads to issues with ceiling heights and floor to floor heights, similar to that mentioned with composite construction earlier in the report.
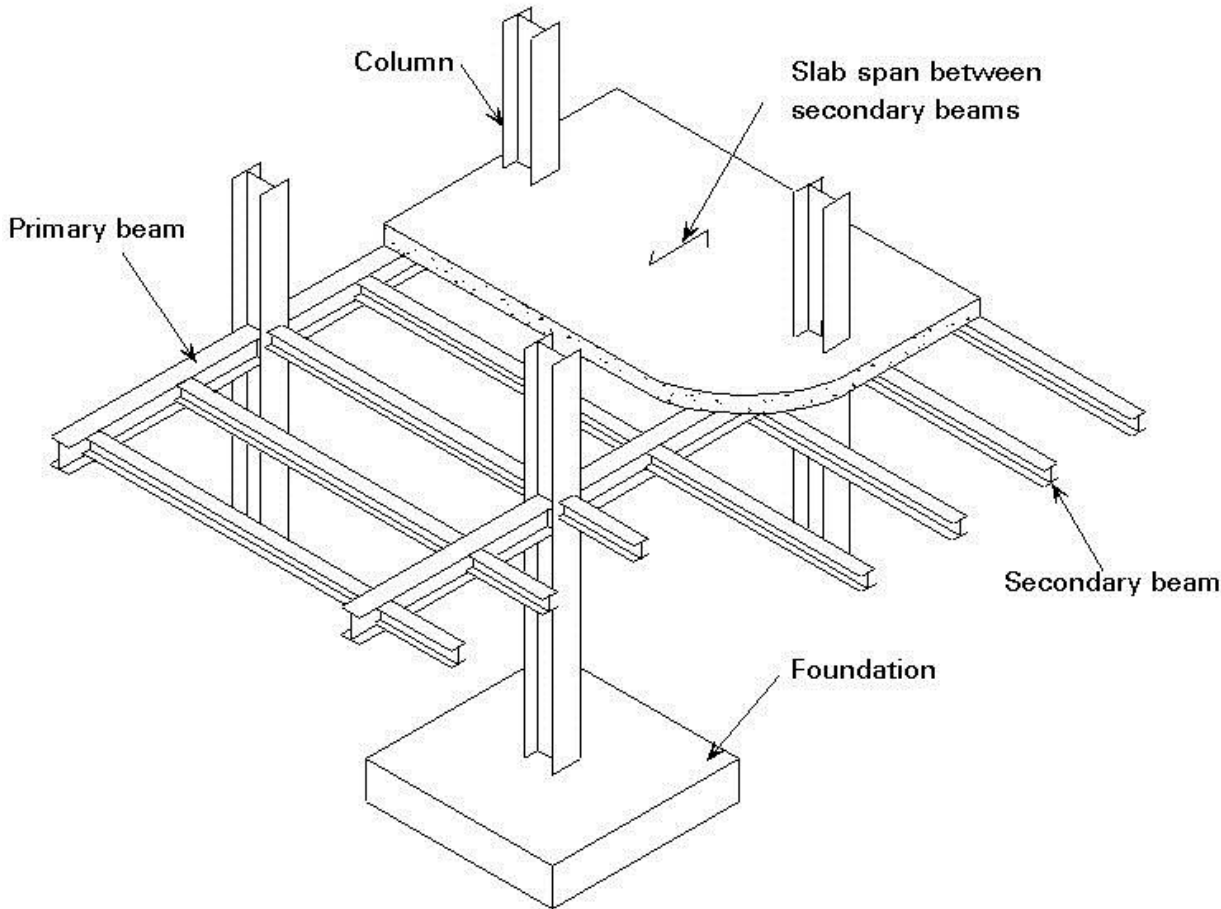
Figure 9: Typical non-composite steel system.

Hollow Core Planks on Steel Framing

Hollow core planks are very common in hotels and residential construction since it is a relatively thin floor system, allowing for high ceilings and lower floor to floor heights. In the hotel industry, this is crucial because if you can create more floors at a lower height, you can reach zoning and height requirements while maximizing occupancy, which leads to larger gains in revenue.

Because of the large spans involved, the column layout has slightly changed from the existing structure, adding another set of columns along the interior wall. This created bays of 19'-4" on the exterior with a central hallway that spans 9'-10". Refer to the framing layout in the appendices to gain further information on span direction and lengths.

Design data sheets from Nitterhouse Concrete Products were used to specify the adequate hollow core plank to support the loads. A 6"x4'-0" hollow core plank with (6) ½" diameter low-relaxation strands was chosen, which provides a maximum design moment of 92.6 ft-k, which is about a third the strength of the existing composite structure. The hollow core planks were not evaluated in deflection due to the complexity of camber calculations, however hollow core planks perform notably well in deflection since they do use camber which in most cases is slightly larger than calculated. The steel framing system supporting the planks is designed similarly to the non-composite framing system since it will receive full loads transferred from the planks to the beams. Upon further review, a W24x62 beam and a W21x44 beam would both be adequate to transfer the loads over the specific spans.

Advantages

Advantages toward using hollow core planks are that you can create higher ceilings and lower floor to floor heights since the system is relatively thin. This can help increase the occupancy in a building. They are also very reliable and often do not usually have constructability issues since they are precast at a plant. This eliminates any weather conditions when forming and casting the concrete, and also allows for very precise measurements. You can also order special shapes and specific lengths to meet your needs.

Disadvantages

Some disadvantages to precast hollow core plank systems involve the costs of transporting the planks from the plant to the jobsite. Since you don't form the planks on site, you are limited to a specific length and width in order to safely transport the planks. In some cases you can tilt the planks on edge to transport wider planks; however this involves additional costs and limits space on the truck. Another disadvantage to this

specific system is that it limits floor plan layouts due to the steel framework. Although the planks are thin, the beams will be deep to carry the loads, so usually they will be hidden within walls and partitions. This restricts room layouts and sizes that can cause aesthetic issues.

Figure 10: Typical precast hollow core plank w/ embedded reinforcement.

One-Way Post-Tensioned Flat Plate System

When designing this system, the similar column layout from the existing plans was used to help keep floor plan opportunities open, so I had the tendons span over the 19'-4" and 29'-2" bays. Upon review, it was found that using an 8" slab with 3000psi concrete, the design calls for a tendon consisting of (12) 0.196" diameter strands to carry the loads. The eccentricity at mid-span on the larger span would be at 2", which was at the maximum due to cover requirements. The eccentricity on the shorter span was at 0.318", which allows the tendon to create a general balanced upward force over the entire slab to resist dead loads. Spacing between each tendon was calculated to be 18" on center. The max moment found was 13.1 ft-k / ft width, or 113.6 ft-k when comparing it to the tributary width of the existing structure, which is significantly weaker than the existing composite system. Calculated shear produced similar results.

Design criteria such as deflection or vibration were not checked in this report due to the inherent complexity of PT systems. However, post-tensioned systems perform notably well against deflection issues, since the balanced moment supplied by the stressed tendons creates a camber effect on the slab, reducing deflections significantly. This is a main reason why it can be used for larger spans.

Advantages

Post-tensioned systems can offer a solution toward long span conditions. Since PT systems apply an upward force from the tendon, they create a camber effect on the slab which when loads are applied to the slab, the slab balances these gravity forces. This allows the concrete to span large bays without negative effects from large deflections. PT flat plate systems also offer adequate fireproofing due to the thick 8" of concrete between each level.

Disadvantages

The main disadvantage of post tensioned systems comes from constructability issues. Placing the tendons is a very time consuming job, since each tendon must have the correct amount of drape in order to function as intended. Safety is also an issue when jacking the tendons. Forces in the hundreds of thousands are being applied to these tendons, creating a very destructive outcome if one was to rupture. If any types of repairs are necessary in an older building using post tensioned slabs, cutting through a post tensioned slab is highly dangerous since you would be releasing some of these internal forces.

Impact on Lateral System

Post Tensioned and Hollow Core Plank Systems

Floor systems mainly utilizing concrete such as the post tensioned and hollow core systems provide a slightly more massive system, depending on the thickness of the slab. This increase in mass would create larger story shear forces in an earthquake. However, a building made of a concrete structural and lateral system tends to be quite rigid, meaning that the structure will have a low period of vibration. Although the site is located in Buffalo, NY, earthquake forces are prevalent and must be compensated for in design. A low period of vibration in this area would be suitable since many of the earthquakes seen here are infrequent and usually quite low in magnitude. Both post tensioned systems and hollow core plank systems usually work well with a concrete frame, which most often incorporates shear walls for their lateral system. If one of these systems were used for this building, it would be wise to reconsider different lateral systems that may be more compatible with concrete construction.

Composite and Non-Composite Systems

Composite and non-composite floor systems tend to mainly utilize steel for its structural system. This use of steel makes it easy to tie the floor system into either a brace frame or moment frame lateral system. Steel generally behaves well in a building and generally flexes with each passing wave. Because of the general height and shape of the building, a mainly steel structure should perform well in this location.

Foundations

The foundation is mainly sitting on limestone bedrock with some structural fill supporting it as well. If you are using lightweight concrete in either post tensioned or hollow core plank systems, you shouldn't have much of a settlement issue or any types of punching shear problems, similar with steel structural systems.

Systems Comparison

Each system is compared based on the following criteria: slab weight, slab depth, system depth, vibration control, fire rating, additional fireproofing, constructability, formwork, floor to floor height, lead time, system cost, and feasibility. Table 3 below illustrates the system comparison by highlighting best choice in green and worst choice in red.

	Composite	Non-Composite	Hollow Core Plank	Post Tensioned
Slab Weight	42psf	48psf	49psf	80psf
Slab Depth	5.25"	6"	6"	8"
System Depth	17.7"	20.6"	23.7"	8"
Vibration Control	Yes	Yes	No	Yes
Fire Rating	2 hr.	2 hr.	2 hr.	2 hr.
Additional Fireproofing	Yes	Yes	No	No
Constructibility	Easy	Easy	Easy	Hard
Formwork	No	No	No	Yes
Floor to Floor Height	Increased	Increased	Decreased	Decreased
Lead Time	Short	Short	Long	Short
System Cost	$24.20	$28.60	$34.20	$22.60
Feasibility	Existing	Outdated	Possible	Most Possible

Table 3: Comparison Data

Here are some strength comparisons between each floor system via Table 4 below.

	moment capacity (ft-k)	shear capacity (k)	deflection live (in.)	deflection dead (in.)
composite:	-	-	-	-
beam	394	344	0.2	0.83
girder	483.3	376.2	0.225	0.933
Non-composite:	-	-	-	-
beam	249	159	0.385	1.34
girder	398	217	0.297	1.23
hollow core	92.6	-	-	-
beam b1	574	306	0.33	1.33
beam b2	358	217	0.252	1.01
post tensioned	13.1 /ft width	5.243 /ft width	-	-

Table 4: Comparison Data for Strength & Deflection

Final Summary

Technical Report 2 is meant to explore the differences in strength, serviceability, cost, weight, and structural depth between the existing structural floor system and three other types of floor systems. These different systems were reviewed in order to discover which system would be fitting for future design considerations.

The existing composite beam and girder system seems to be the best solution. They use a minimal amount of steel, it is easy to construct, it is relatively cheap and relatively low weight. However the post tensioned floor system provided some surprising results such as system depth that help qualify it as another close possibility. It almost matched every quality when looking at aesthetic topics such as floor to floor height, yet its main drawback is the difficulty it creates to construct during construction. Otherwise, the benefits of the post tensioned floor system mainly outweighed the flaws, making it my second best choice and a viable alternative.

The hollow core plank floor system seemed to be relatively strong, reduced system depth and floor to floor height, and lightweight, the cost of the floor system was considerably larger than every other system. This system would also have some architectural design issues because of floor plan restrictions as well as possible ceiling finishes. Vibration would be more prevalent in this system as well. The system was therefore rejected, and will no longer be considered as an alternative floor system.

The non-composite floor system design is a relatively older system, however is very well understood in the industry. It is easy to design, construct, and attain materials to build a steel non-composite floor system. It is very similar to the composite system, however the steel framing below the concrete decking mainly carries the entire set of stresses due to the loads above. In composite construction, the slab helps the steel carry these stresses which reduce the amount of structural steel needed to support the loads. Since the existing system uses a composite floor system already, it is virtually and economically unnecessary to use this floor system. Therefore this system was rejected as well and will no longer be considered as an alternative.

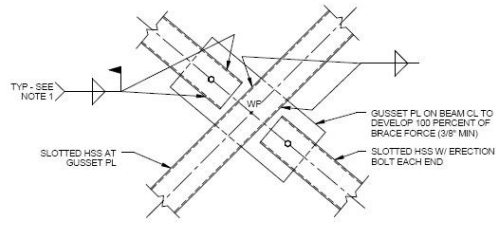# Appendices

Appendix A: Framing Plan & Elevations
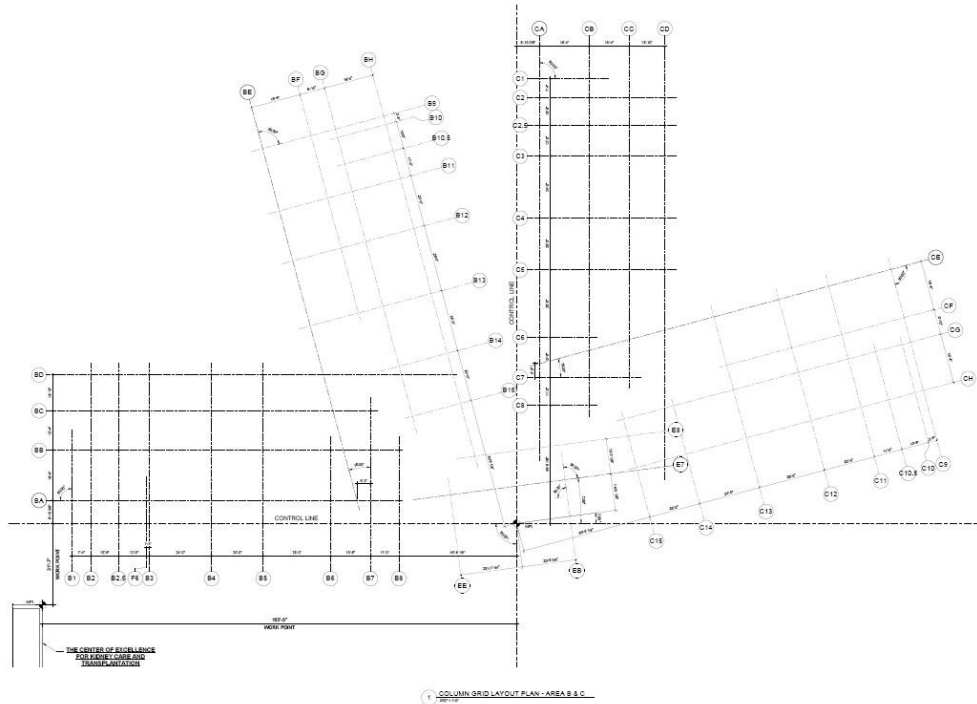
Figure 11: Column Grid Layout Plans (East End on bottom, West End on top) Details courtesy of Cannon Design.
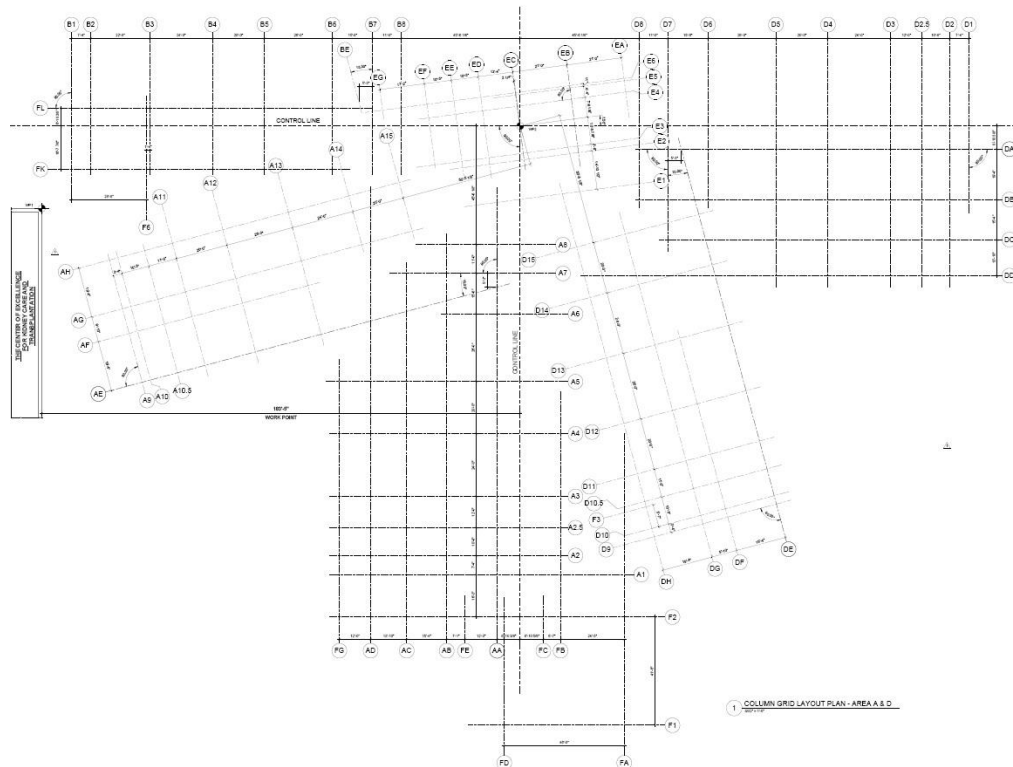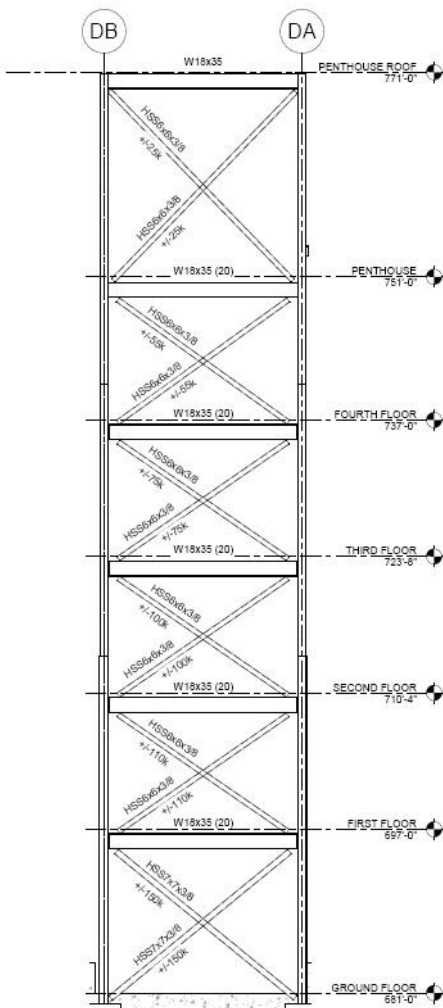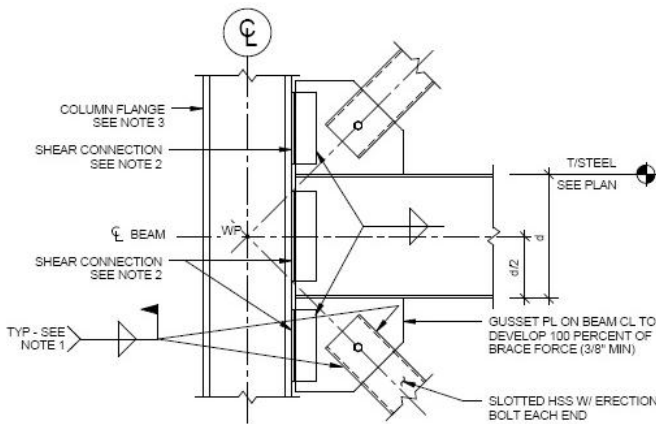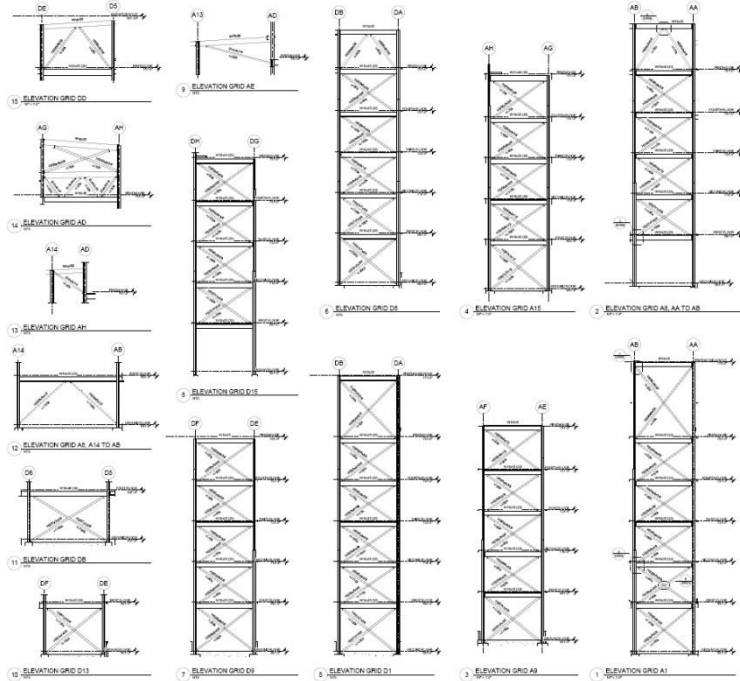
Figure 12: Concentric HSS Brace Frames and Connection Details. Details courtesy of Cannon Design.

Figure 13: Typical Floor Plan of existing structure with bays used in calculations highlighted. Drawings courtesy of Cannon Design.

Appendix B: Composite Steel Analysis (existing structure)

Design Loads	Tech 2 Report	BRIAN BRUNNET	1

<u>Dead Loads:</u>

- Roof DL
  + insulation : — 2psf
  + MEP: ———— 18psf
  + Metal Deck: — 3psf
  + Framing: ———— 10psf

  33psf

- Penthouse Floor DL
  + Metal Deck: — 3psf
  + NWC topping: $145pcf \times \frac{6"}{12"} = 72.5psf$
  + Blended Fiber Reinforcement: $24pcf \times \frac{6"}{12"} = 12psf$
  + MEP: 20psf
  + Framing: 10 psf

  117.5psf

- Floors (1→4) DL
  + Metal Deck: 3psf
  + LWC topping: $115pcf \times \frac{6"}{12"} = 57.5psf$
  + Blended fiber reinforc.: $24pcf \times \frac{6"}{12"} = 12 psf$
  + MEP: 18psf
  + Framing: 10 psf

  100.5psf

<u>Live Loads:</u> (per ASCE 7-10)

  + Corridors: 40psf
  + Lobbies: 100 psf
  + Balconies: 100 psf
  + Resident Rooms: 40psf
  + Stairs/Exits : 100 psf
  + 1st floor corridor: 100psf.

Typical Floor Deck (side margin)

Loads:

- LL = 40 psf

- Floor DL:

  + Deck: 1.97 psf
  + LWC Topping: $115 pcf \times \frac{5.25''}{12''} = 50.3 psf$
  + Blend. Fiber Reinf.: $24 \times \frac{5.25''}{12''} = 10.5 psf$

  + MEP: 18 psf

  + Framing: 10 psf
  + Superimposed DL: 20 psf
  + partitions: 18 psf

  <u>128.8 psf</u>

FD02:
- 2VLI20 Floor Deck w/ total thickness = $5\frac{1}{4}''$

$Span = \frac{26'}{3} = 8.67'$

From Vulcraft Deck Manual:

- 2VLI20 w/ tot. slab depth = 5.25"

  Results:

  + SDI Max Unsh. Clear Span: $10'-11'' > 8.67'$ ☑ ok
    (3 span condition)

  + Superimp. LL (psf): 184 psf > 128.8 psf ☑ ok at 8.67' span
    (interpolated)

  + Deck wt.: 42 psf < 50.3 psf (calculated) ☑ ok (conservative)

  + Fire Rating: w/ unprotect. deck → | 2-hr. Fire Rating |
    ⤳ $t = 3\frac{1}{4}''$ (total $t = 5\frac{1}{4}''$)

(right side diagram)

W18×35 (22 studs)

W16×31 (20 studs)

FD02

29'-2"

26'
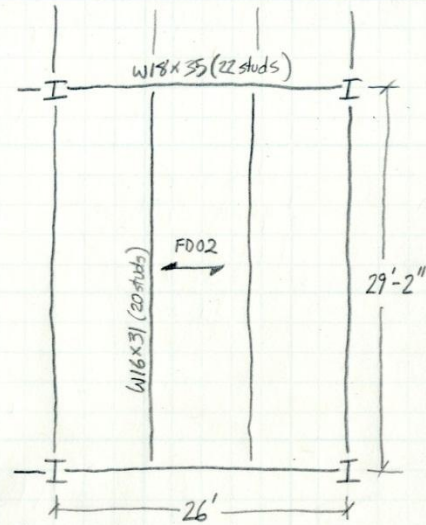
Assumed:
+ unprotected deck

Beam: $\boxed{\text{W16}\times 31 \text{ (20 shear studs)}}$

Deck: 2" 20 Ga. Composite Deck (2VLI20)
w/ $5\frac{1}{4}$" tot. thickness ($t = 3\frac{1}{4}$")

Properties:
- spacing: 8.67'
- span: 29'-2"
- $F_c' = 3^{ksi}$
- $F_y = 50^{ksi}$
- Studs: $\frac{3}{4}$" / 1 per rib
- Deck: perpendicular / LWC

Typical Beam

(beam section drawing: $5\frac{1}{4}$" total, 2" deck)
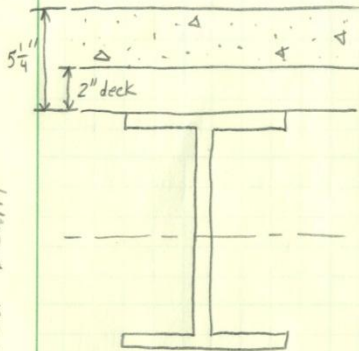
$b_{eff} \begin{cases} \dfrac{29.167}{4} = 7.29' \text{ (governs)} \\ \text{spacing} = 8.67' \end{cases}$

\# studs = $20 = \dfrac{\Sigma Q_n}{Q_n/stud}$

$V_c' = (.85)(3)(87.5")(3.25") = 725.2^k$

$V_s' = (9.13 in^2)(50 ksi) = 456.5^k$

$20 = \dfrac{\Sigma Q_n}{17.2^k}$

Since $V_q' < V_s' + V_c'$: partially composite

$\Sigma Q_n = 344^k = V_q'$

$a = \dfrac{344}{(.85)(3)(87.5)} = 1.542"$ → since $a < 2$" deck depth sufficient conc. is available

$M_u = \dfrac{wL^2}{8} = \dfrac{(1.88)(29.167)^2}{8}$

$M_u = 200^{ik}$

$A_{s-c} = \dfrac{456.5 - 344}{2(50)} = 1.125 in^2$

$x = \dfrac{A_{s-c}}{b_f} = \dfrac{1.125}{5.53} = 0.203" < t_f = 0.440"$ (PNA in flange)

$M_n = 456.5\left(\dfrac{15.9"}{2}\right) + 344\left(5.53 - \dfrac{1.542}{2}\right) - 112.5\left(\dfrac{.203}{2}\right) = 5255$ in-kip

$\phi M_n = (.9)\left(\dfrac{5255}{12}\right) = \boxed{394^{ik}} > 200^{ik}$ ☑

$$\Delta = \frac{5wL^4}{384 EI_{LB}} \quad \underline{\text{Deflection:}}$$

$$\Delta_{DL} = \frac{5(1.1)(29.167)^4(1728)}{384(29000)(982\,in^4)} = \boxed{0.629''}$$

↖ from Table 3-20

$$\Delta_{LL} = \frac{5(0.35)(29.167)^4(1728)}{384(29000)(982)} = \boxed{0.200''} < 0.97'' \text{ ok}$$

$$\Delta_{TL} = \frac{5(1.45)(29.167)^4(1728)}{384(29000)(982)} = \boxed{0.83''} < 1.46'' \text{ ok}$$

$$\phi V_n = V_q'$$

$$\phi V_n = \boxed{344^k} > 27.4 \text{ ok}$$

$$V_u = \frac{wL}{2} = \frac{(1.88)(29.167)}{2} = 27.4^k$$

Typical Beam

Dead Load:

Deck DL: 118.8 psf

Beam DL: 3.6 psf

—————————

122.4 psf

Live Load:

— Floor LL: 40 psf

spacing

$$w_D = (122.4)(8.67') = 1.1 \text{ klf}$$

$$w_L = (40)(8.67) = 0.35 \text{ klf}$$

$$W_{tot} = 1.2(1.1) + 1.6(0.35)$$

$$W_{tot} = 1.88 \text{ klf}$$

$$\Delta_{LLallow} = \frac{L}{360} = \frac{(29.167)(12)}{360} = 0.97''$$

$$\Delta_{TLallow} = \frac{L}{240} = \frac{(29.167)(12)}{240} = 1.46''$$

Typical Girder

40.9^k 40.9^k

$26'$

Girder: W18×35 (22 studs)
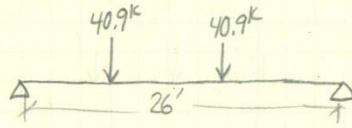
b_{eff} $\begin{cases} \dfrac{26'}{4} = 6.5' \text{ (governs)} \\ \\ \text{spacing} = 14.33' \end{cases}$

$V_c' = (.85)(3)(78)(3.25) = 646.4^k$

$V_s' = (10.3in^2)(50) = 515^k$

since $V_q' < V_s' + V_c'$
↳ partially composite

$a = \dfrac{376.2}{(.85)(3)(78)} = 1.89 < 2''$ ☑ ok

$A_{s-c} = \dfrac{515 - 376.2}{2(50)} = 1.388 in^2$

$x = \dfrac{1.388}{6.00} = 0.231'' < 0.425 \quad (\text{ANA in flange})$

$M_n = 515\left(\dfrac{17.7}{2}\right) + 376.2\left(6.00 - \dfrac{1.89}{2}\right) - 138.8\left(\dfrac{.231}{2}\right) = 6443.4 \text{ in-kip}$

$\phi M_n = (.9)\left(\dfrac{6443.4}{12}\right) = \boxed{483.3^{ik}} > 354.6^{ik}$ ☑ ok

Deflection:

$\Delta = \dfrac{PL^3}{28EI_{LB}}$

$\Delta_{DL} = \dfrac{(23.9)(26)^3(1728)}{28(29000)(1263.7in^4)} = \boxed{0.708''}$

↖ Table 3-20 (interp.)

$\Delta_{LL} = \dfrac{(26)(26)^3(1728)}{28(29000)(1263.7)} = \boxed{0.225''} < 0.87''$ ☑

$\Delta_{TL} = \dfrac{(26)^3(1728)}{28(29000)(1263.7)} = \boxed{0.933''} < 1.3'$ ☑ ok

#studs $= \dfrac{\Sigma Q_n}{Q_n}$

$22 = \dfrac{\Sigma Q_n}{17.1}$

$V_q' = \Sigma Q_n = 376.2^k$

large span:
$W_{tot} = 1.88 \text{ klf}$

$P = (1.88)\left(\dfrac{29.167}{2}\right) = 27.4^k$

small span:
deck DL $= 118.8 \text{ psf}$
beam DL $= 2.54 \text{ psf}$
     121.3 psf

$W_D = (121.3)(8.67) = 1.1 \text{ klf}$
$W_L = 0.35 \text{ klf}$
$W_{tot} = 1.2(1.1) + 1.6(.35) = 1.88 \text{ klf}$
$P = (1.88)\left(\dfrac{14.33}{2}\right) = 13.5^k$

$P_u = 27.4^k + 13.5^k = \underline{40.9^k}$

$V_u = 40.9^k$
$\phi V_n = V_q' = \boxed{376.2^k} > 40.9^k$ ☑ ok

$M_u = (40.9^k)(8.67) = 354.6^{ik}$

$P_D = \left[1.1\left(\dfrac{29.167}{2}\right) + 1.1\left(\dfrac{14.3}{2}\right)\right] = 23.9^k$

$P_L = \left[.35\left(\dfrac{29.167}{2}\right) + .35\left(\dfrac{14.3}{2}\right)\right] = 7.6^k$

$P_{tot} = 40.9^k$

$\Delta_{LLall} = \dfrac{(26)(12)}{360} = 0.87''$

$\Delta_{TLall} = \dfrac{(26)(12)}{240} = 1.3''$

Appendix C: Non-Composite Steel Design

| Design Non-Composite | Tech 2 Report | BRIAN BRUNNET | 1 |

Metal Decking

Loads:

$LL = 40 psf$

$\underline{DL = 100.5 psf}$

$140.5 psf$

$Span = \dfrac{26'}{3} = 8.67'$

Assume:
- sprayed fiber insulation

for 2 hr. fire rating

Try: 2C Deck w/ at least $2\frac{3}{4}"$ topping ($\geq 4\frac{3}{4}"$ total)

	3 span Condit:	Super. TL:	t:
2C22 →	9'-9"	✗	3"
w/ t=3.00" (tot. depth = 5")		← too thin	

| 2C22 → | 9'-1" | 150.67 | 4" |
| w/ t=4" (tot. depth=6") | | | |

Results:

3 span Condit.: $9'-1" > 8'-8"$ ☑   2hr. Fire Rat.: $4" > 2\frac{3}{4}"$ ☑

Sup TL: $150.67 psf > 140.5 psf$ ☑

USE   2C22 w/ t= 4" (6" tot. thickness) LWC
   & 4×4 W2.9 × W2.9 WWF

2-hr. Fire Rating

Typical Beam

Loads:

Span = 29'

$LL = 40 psf$

$DL = 100.5 psf$

$$w_u = \left[1.2(100.5) + 1.6(40) \right](8.67') = 1.6 \text{ klf}$$

$$M_u = \frac{w_u L^2}{8} = \frac{(1.6)(29.167')^2}{8} = \underline{170.1^{'k}}$$

$$M_n = M_p = F_y Z$$

$$M_a \le \phi M_n = \phi F_y Z$$

$$Z_{req} = \frac{M_u}{\phi F_y} = \frac{170.1(12)}{(0.9)(50)} = \underline{45.4 \, in^3}$$

From Table 3-2:

   W14×30 $(Z = 47.3 in^3)$

     ↳ $\boxed{\phi M_{px} = 177^{'k}}$ > $M_u = 170.1^{'k}$ ☑ ok $\boxed{\phi V_n = 112^k}$ > $V_u = \frac{(1.6)(29.167)}{2} = 23.3^k$

Deflections:

$$\Delta_{LL} = \frac{5wL^4}{384EI} = \frac{5(0.35)(29.167)^4(1728)}{384(29000)(291)} = \boxed{0.675''} < \frac{L}{360} = \frac{29.167(12)}{360} = 0.973'' \text{ ☑ ok}$$

$$\Delta_{TL} = \frac{5(1.22)(29.167)^4(1728)}{384(29000)(291)} = 2.35'' > \frac{L}{240} = \frac{29.167(12)}{240} = 1.46'' \text{ ☒ N.G.}$$

$$1.46'' = \frac{5(1.22)(29.167)^4(1728)}{384(29000)(I_{req})}$$

$$I_{req} = \underline{469 in^4} \longrightarrow Try: \, W18×35 \quad \begin{bmatrix} \phi M_n = 249^{'k} \\ \phi V_n = 159^k \end{bmatrix}$$

<u>W18×35:</u>

$\phi M_n = 249^{'k} > 170.1^{'k}$ ☑ ok

$\phi V_n = 159^{'k} > 23.3^{k}$ ☑ ok

$\Delta_{LL} = \dfrac{5(.35)(29.167)^4(1728)}{384(29000)(510)} = \boxed{0.385''} < 0.973''$ ☑ ok

$\Delta_{TL} = \dfrac{5(1.22)(29.167)^4(1728)}{384(29000)(510)} = \boxed{1.34''} < 1.46''$ ☑ ok

$$\boxed{\text{USE} \quad W18 \times 35 \quad \text{for beam}}$$

Typical Beam

Typical Girder

Loads:

$w_D = 1.1$ klf

$w_L = 0.35$ klf

$P_3 = \left[1.2(1.1) + 1.6(.35)\right]\left(\frac{14.3'}{2}\right) = 13.5^k$

$P_L = \left[1.2(1.1) + 1.6(.35)\right]\left(\frac{29.167'}{2}\right) = 27.4^k$

$P_u = 13.5 + 27.4 = 40.9^k$

40.9   40.9
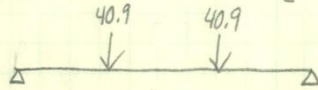
Deflection:

$\Delta_{TL} = \frac{PL^3}{28 EI_{req}} = \frac{(31.5)(26')^3(1728)}{28(29000)I_{req}} = \frac{L}{240} = \frac{26'(12)}{240} = 1.3''$

$I_{req} = 907.4 \, in^4 \longrightarrow$ Try W21×48 $\begin{bmatrix} \phi M_n = 398^k \\ \phi V_n = 217^k \\ I_x = 959 \, in^4 \end{bmatrix}$

W21×48:

$M_u = P_a = 40.9^k (8.67') = 354.6^{1k} < \boxed{398^{1k}}$ ☑ ok

$V_u = 40.9^k < \boxed{217^k}$ ☑ ok

$\Delta_{LL} = \frac{(7.6)(26)^3(1728)}{28(29000)(959)} = \boxed{0.297''} < \frac{L}{360} = \frac{26(12)}{360} = 0.867''$ ☑ ok

$\Delta_{TL} = \frac{(31.5)(26)^3(1728)}{28(29000)(959)} = \boxed{1.23''} < \frac{L}{240} = 1.3''$ ☑ ok

$\boxed{\text{USE \quad W21×48 \quad for girder}}$

3 @ 8.67' = 26'

H   H

19.33'

GIRDER

Beam

Non-composite floor slab

29.167'
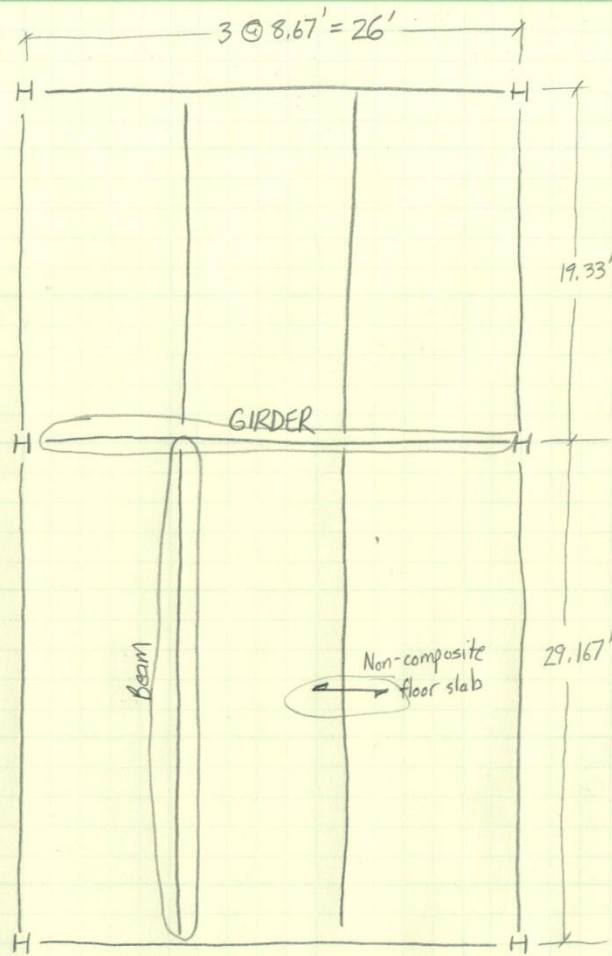
Non-composite floor slab: 2C22 w/ t=4" (6" tot. thickness) LWC
→ 4x4 W2.9 x W2.9 WWF

Beam: W18x35

Girder: W21x48

Appendix D: Post-Tensioned Design

| Design 1 way PT flat plate | Tech 2 Report | BRIAN BRUNNET | 1 |

Loads:

$LL = 40 psf$

PT properties:

$f'_c = 3000 psi$

$f_{pu} = 250,000 psi$

$f_{pi} = 0.7 f_{pu} = 175,000 psi$

minimum cover: $2''$

Losses: 15%

permissible stresses:

$f_{allow} = 1350 psi$ (compress)

$0 psi$ (tension)

Slab Thickness:

- design on basis of $12''$ wide strip:

$$\frac{l}{h} = 45 \qquad h = \frac{29.167' \times 12}{45} = 7.78 \rightarrow 8'' \qquad \therefore Try\ 8''\ slab\ thickness$$

Prestressing Force:

$$h = 2'' + 1'' = 3''$$

Larger Span:

$$W_o = \frac{8Ph}{L^2}$$

$$80 = \frac{8(Ph)}{(29.167)^2(12)}$$

$$Ph = 102,086\ ''^{lb}/_{ft\,width}$$

$$P_e = \frac{102,086}{h = 3''} = 34,029\ {}^{lb}/_{ft\,width}$$

$$P_i = \frac{P_e}{.85} = \frac{34,029}{.85} = 40,034\ {}^{lb}/_{ft\,width}$$

$$W_o = \frac{8''}{12''}(120) = 80 psf \quad \leftarrow LWC$$

Smaller Span:

$$W_o = \frac{8Ph}{L^2}$$

$$80 = \frac{8(Ph)}{(19.3)^2(12)}$$

$$Ph = 44,853\ ''^{lb}/_{ft\,width}$$

$$P_e = 34,029 = \frac{44,853}{h}$$

$$h = 1.318''$$
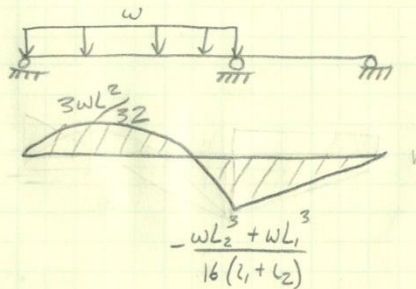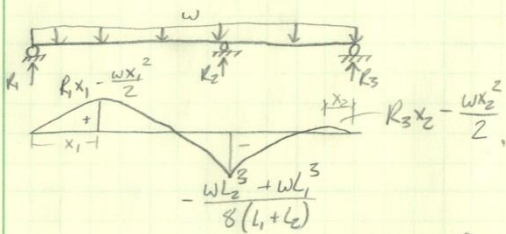
upward uniform load by tendon @ transfer = $\frac{80 psf}{0.85}$ = 94.1 psf

Check Stresses :

$A = 12'' \times 8'' = 96 in^2$

$S = \frac{(12)(8)^2}{6} = 128 in^3$

→ Consider uniform loading once on both spans + once only on larger span to achieve max. effects:

$-\frac{wL_2^3 + wL_1^3}{8(L_1 + L_2)}$

$-\frac{wL_2^3 + wL_1^3}{16(L_1 + L_2)}$

Stresses at transfer: $(P_i + W_o)$ $f_{allow} = 1350 psi$

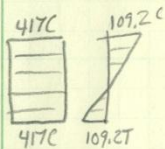w = net uniform load: 94.1 - 80 = 14.1 psf upward

loading on both spans:

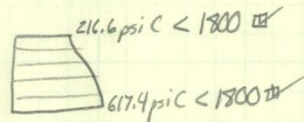$M_{midspan} = -R_1 x_1 + \frac{wx_1^2}{2} = -(2453)(17.4) + \frac{(14.1)(17.4)^2}{2} = -2138.(12) = -25,657$ "lb/ft width

$M_{support} = +\frac{wL_2^3 + wL_1^3}{8(L_1 + L_2)} = \frac{(14.1)(17.33)^3 + (14.1)(29.167)^3}{8(48.5)} = +1164.3(12) = +13,972$ "lb/ft width

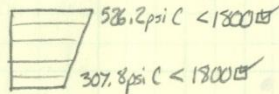stresses: $f = \frac{P_i}{A} \pm \frac{M}{S}$

417 C 200.4 T
417 C 200.4 C
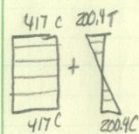
Midspan: $f = \frac{40,034}{96} \pm \frac{25,657}{128}$
(M⁻)
$= 417 \pm 200.4 = \begin{cases} 216.6 \\ 617.4 \end{cases}$

216.6 psi C < 1800 ☑

617.4 psi C < 1800 ☑

417 C 109.2 C
417 C 109.2 T

Center Supp.: $f = 417 \pm 109.2 = \begin{cases} 526.2 \\ 307.8 \end{cases}$
(M⁺)

526.2 psi C < 1800 ☑

307.8 psi C < 1800 ☑

End Supp.: $f = 417/si < 1800$ ☑
M = 0

417 C

417 C

Stress after Losses: (P_e, W_o, W_L on both spans)

$W_{net} = 80^\uparrow - 80^\downarrow - 40^\downarrow = 40$ psf downward

prestress, DL, LL

Loading on both spans:

$M_{midspan} = +R_1 x_1 - \frac{WX_1^2}{2} = (245.5)(17.4) - \frac{(40)(17.4)^2}{2} = -1779(12) = -21,350$ "lb/ft width

$M_{support} = -\frac{WL_2^3 + WL_1^3}{8(L_1 + L_2)} = -\frac{40(17.3)^3 + 40(29.167)^3}{8(48.5)} = -3303(12) = -39,636$ "lb/ft width
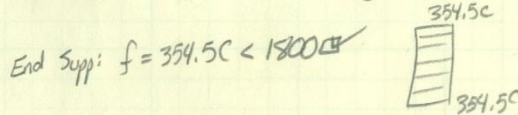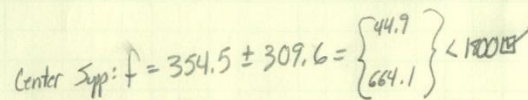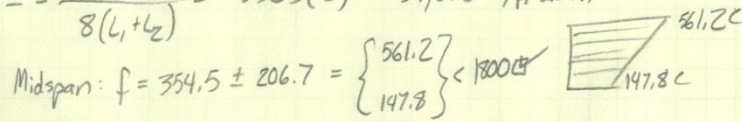
Stresses: $f = \frac{P_e}{A} \pm \frac{M}{S}$

Midspan: $f = \frac{34,029}{96} \pm \frac{21,350}{128} = 354.5 \pm 166.8 = \begin{cases} 521.3_{psi} C < 1800 \text{☞} \\ 187.7_{psi} C < 1800 \text{☞} \end{cases}$

521.3 C
187.7 C

Center Supp: $f = 354.5 \pm 309.6 = \begin{cases} 44.9 < 1800 \text{☞} \\ 664.1 < 1800 \text{☞} \end{cases}$

44.9 C
664.1 C

End Supp: $f = 354.5$ psi C < 1800 ☞

354.5 C
354.5 C
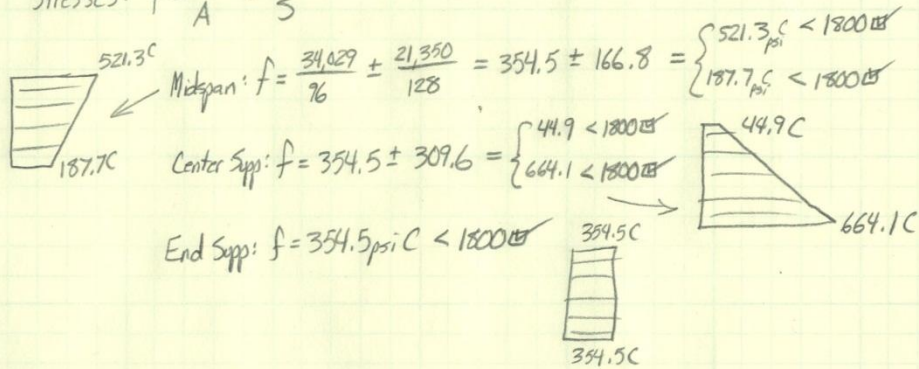
Stress after Losses: (W_L on larger span)

$W = 40$ psf downward

$M_{midspan} = \frac{3}{32} WL_{avg}^2 = \frac{3}{32}(40)\left(\frac{19.3 + 29.167}{2}\right)^2 = 2205(12) = +26,463$ "lb/ft width

$M_{support} = -\frac{WL_2^3 + WL_1^3}{8(L_1 + L_2)} = -3303(12) = -39,636$ "lb/ft width

Midspan: $f = 354.5 \pm 206.7 = \begin{cases} 561.2 \\ 147.8 \end{cases} < 1800 \text{☞}$

561.2 C
147.8 C

Center Supp: $f = 354.5 \pm 309.6 = \begin{cases} 44.9 \\ 664.1 \end{cases} < 1800 \text{☞}$

44.9 C
664.1 C

End Supp: $f = 354.5 C < 1800 \text{☞}$

354.5 C
354.5 C

Prestressing Tendons:

$$f_{pi} = 0.7 f_{pu} = 0.7(250,000) = 175,000 \text{ psi}$$

$$P_i = 40,034 \text{ lb/ft width}$$

$$A_p = \frac{40,034}{175,000} = 0.229 \text{ in}^2/\text{ft width}$$

try (12) 0.196" ⌀ wire w/ $A_p = \frac{\pi(.196)^2}{4}(12) = 0.362 \text{ in}^2 > 0.229$ ✓

$$\text{spacing} = \frac{.362}{.229} \times 12" = 18.97"$$

Ultimate load in flexure:

At midspan + middle supp.: $\rho_p = \frac{0.229}{12(6")} = 0.0032$
(larger)

At smaller midspan: $\rho_p = \frac{0.229}{12(5.318)} = 0.0036$

For unbonded tendons w/ $\frac{\text{span}}{\text{depth}} > 35$

$$f_{ps} = f_{pe} + 10,000 + \frac{f_c'}{300\rho_p} \qquad f_{pe} = \frac{P_e}{A_p} = \frac{34,029}{.229} = 148,598 \text{ psi}$$

$$f_{ps} = 148,598 + 10,000 + \frac{3000}{300(.0032)} = 161,723 \text{ psi}$$

$$f_{ps} < f_{pe} + 30,000 = 148,598 + 30000 = 178,598 \text{ psi}$$

$w_u = 1.2(80) + 1.6(40) = 160 \text{ psf}$

Max Reinforcement: $w_p \le 0.36\beta_1 = 0.36(0.85) = 0.31$

$$w_p = \rho_p \frac{f_{ps}}{f_c'} = 0.0032 \frac{161,723}{3000} = 0.173 < 0.31 \text{ ✓ ok}$$

$$\phi V_c = \phi \, 2\sqrt{f_c'} \, bd_p$$
$$= .75(2)\sqrt{3000}(12)(5.318)$$
$$= 5243 \text{ lb/ft width}$$

Conc. Stress block Depth: $T = C$

$$A_p f_{ps} = 0.85 f_c' \beta a$$

$$a = \frac{(.229)(161,723)}{(.85)(3000)(12)} = 1.21" \quad \text{(conservative)}$$

$$V_u = \frac{5}{8} wL = \frac{5}{8}(160)(21.167) = 2917 \text{ lb/ft w.}$$

$$\boxed{\phi V_c = 5243 \text{ lb/ft width} > V_u \text{ ✓ ok}}$$

$$M_n = A_p f_{ps} \left(d_p - \frac{a}{2}\right) = (.229)(161,723)\left(5.318" - \frac{1.21"}{2}\right) = 174,544 \text{ "lb/ft width}$$

$$\phi M_n = 0.9(174,544) = 157,090 \text{ "lb/ft} = \boxed{13.1 \text{ 'k/ft width}}$$
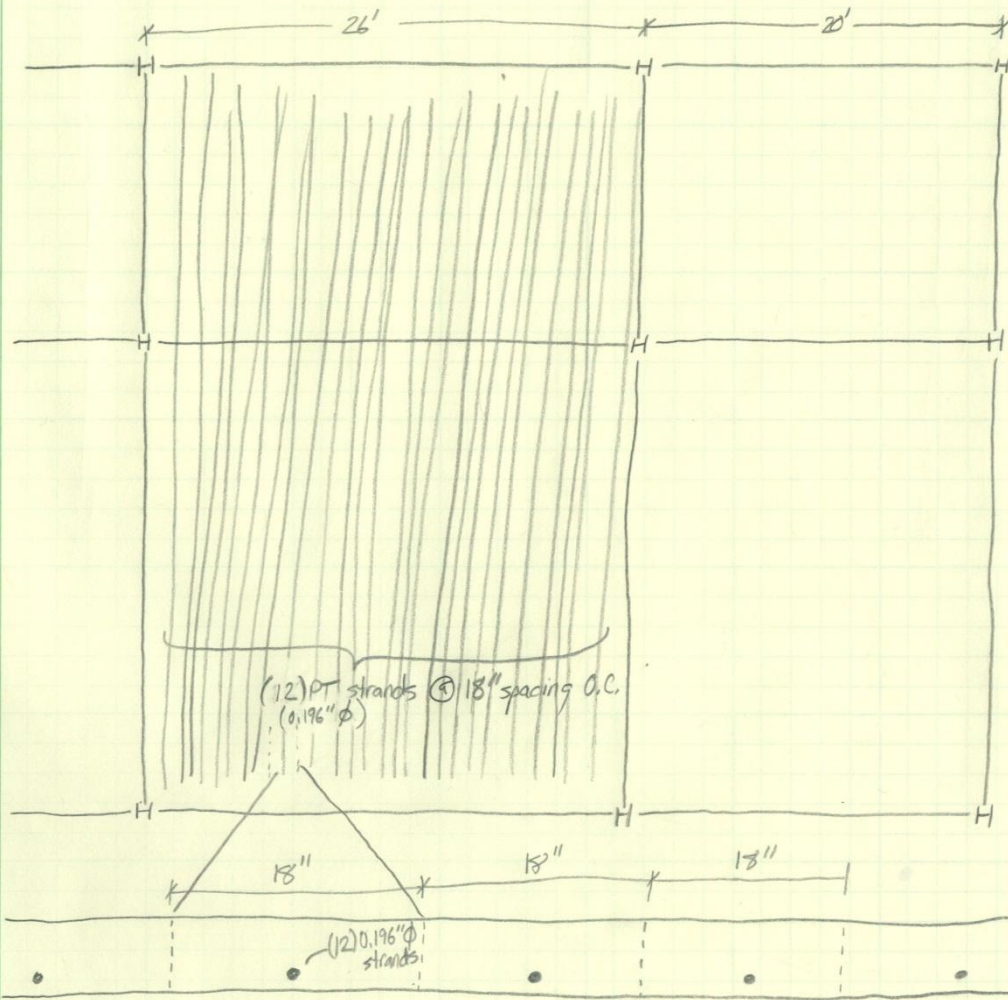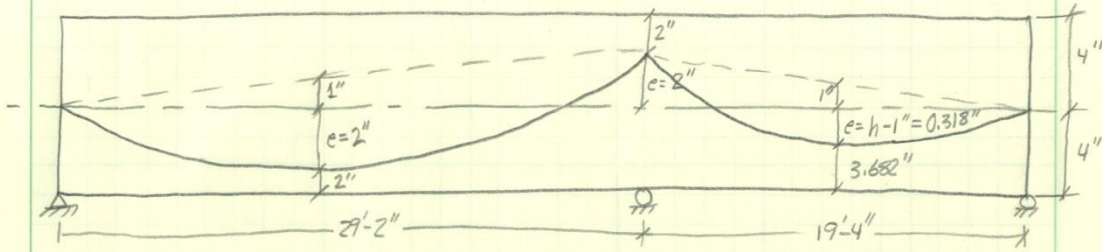
⇒ Refer to design layout for sizes, spacings, + dimensions →

Elevation of post-tensioned tendon profile

2"
e = 2"
1"
e = 2"
2"
e = 2"
1"
e = h-1" = 0.318"
3.682"
4"
4"

29'-2"
19'-4"

Plan view of PT strand layout

26'
20'

(12) PT strands @ 18" spacing O.C.
(0.196" Ø)

18"
18"
18"

(12) 0.196" Ø strands

Isometric View: (Cut)

4"

6"

2"

4.318"

4" 8" thick

(12) strands

(12) 0.196" ⌀ post tentioned strands
@ 18" O.C.

18" spac.
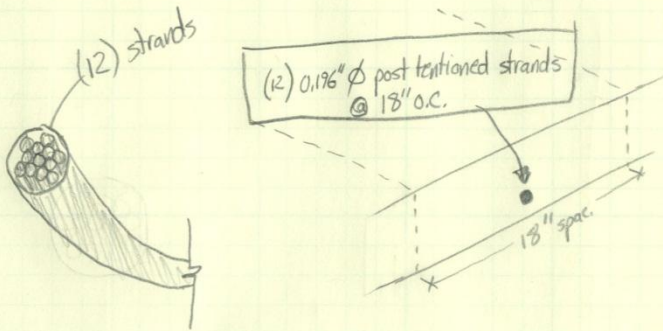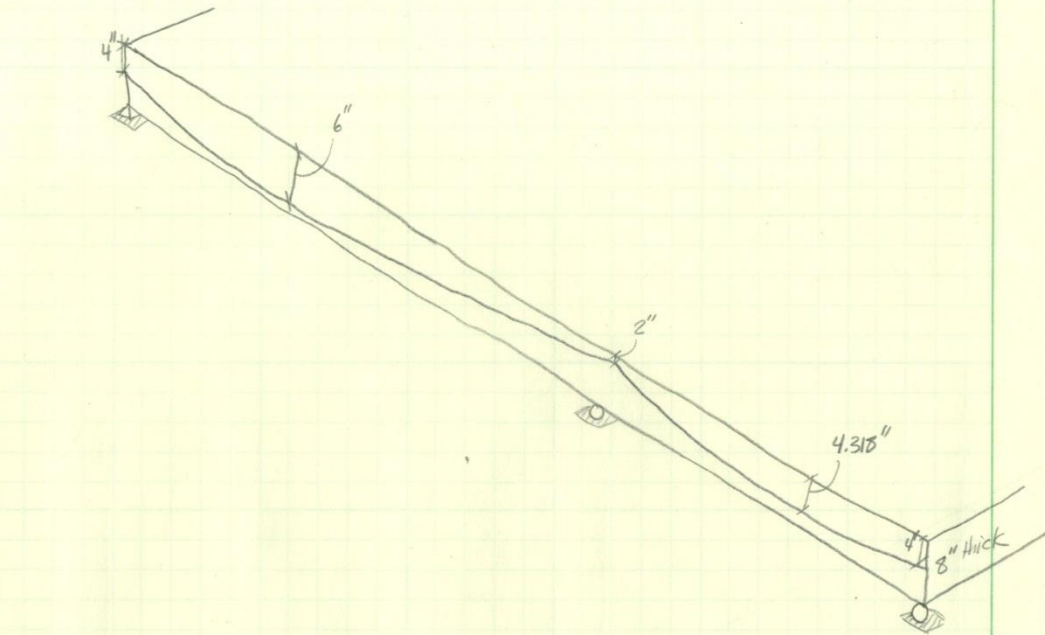
Appendix E: Hollow Core Plank Design

| Design Hollow Core Plank | Tech 2 Report | BRIAN BRUNNET | 1 |

Loads: excludes

$LL = 40 psf$

DL (w/o Hollow Core wt.):
- partitions = 18 psf
- MEP = 18 psf
- Framing = 10 psf

$\overline{\hspace{1.5cm}}$

86 psf

Using Nitterhouse Concrete Products:
- I chose to try $6'' \times 4'-0''$ Hollow Core Plank

Loads w/ Hollow Core Wt.:

$DL: \quad 46 + 48.75 psf + 25 psf = 120 psf$

$\underbrace{\hspace{2cm}}_{\text{precast wt.}} \quad \underbrace{\hspace{1cm}}_{\text{topping}}$

$LL: 40 psf$

$W_u = 1.2(120) + 1.6(40) = \underline{208 psf}$

According to Nitterhouse:

@ 20' Span: Safe Load Capacity (psf):

$6-\frac{1}{2}''\phi$ strands $\longrightarrow$ 237 psf $\boxed{\phi M_n = 92.6^{\,1k}}$ @ 60% jacking force

- Note: I chose to use $6-\frac{1}{2}''\phi$ strands such that because of layout restrictions to follow similar architectural layout, 1 plank would need to be 2' wide w/ $3-\frac{1}{2}''\phi$ strands in it to mimick floor system strength capacity of 4' wide plank w/ $6-\frac{1}{2}''\phi$ strands.

- The hollow core plank w/ $6-\frac{1}{2}''\phi$ strands @ 20' is slightly overdesigned, however this additional capacity is used to meet deflection reqmts.

$\boxed{\text{USE } 6'' \times 4'-0'' \text{ Hollow Core Plank w/ } 6-\frac{1}{2}''\phi \text{ low-relaxation} \atop \text{strands}}$

$\hookrightarrow$ refer to Nitterhouse Design Data Sheet

Beam Sizing: B1 (typical)

$w_u = 208$ psf trib width

$V_u = \dfrac{(.208)(23')(29.167)}{2} = 69.8^k$

$M_u = \dfrac{(.208)(23')(29.167)^2}{8} = 508.7^{1k}$

Using Z_x tables from AISC steel Manual:

$M_u \le \phi M_p$

$Z_{req} = \dfrac{M_u}{\phi F_y} = \dfrac{508.7(12)}{(.9)(50)} = 135.7 \, in^3$

Try W21×62 $(Z = 144 in^3)$ $I = 1330 in^4$ unfactored

$\phi M_p = 540^{1k} > 508^{1k}$ ✓ ok $\Delta_{TL} = \dfrac{5wL^4}{384EI} = \dfrac{5(3.68)(29.167)^4(1728)}{384(29000)(1330)} = \boxed{1.55"} > \frac{\ell}{240}$ ✗

$\phi V_n = 252^k > 69.8^k$ ✓ ok

$\Delta_{TL} = \dfrac{5wL^4}{384EI_{req}} = \dfrac{5(3.68 klf)(29.167)^4(1728)}{384(29000)(I_{req})}$ $I_{req} = 1416.9 in^4 \rightarrow$ Try W24×62 $(I = 1550)$

$\phi M_p = \boxed{574} > 508$ ✓ ok $\Delta_{TL} = \dfrac{5(3.68)(29.167)^4(1728)}{384(29000)(1550)} = \boxed{1.33"} < \frac{\ell}{240} = \dfrac{29.167(12)}{240} = 1.46"$ ✓ ok

$\phi V_n = \boxed{306} > 69.8$ ✓ ok $\Delta_{LL} = \dfrac{5(.920)(29.167)^4(1728)}{384(29000)(1550)} = 0.33" < 0.97"$ ✓ ok

$\boxed{\text{USE W24×62 for beam B1}}$

Beam Sizing: B2 (typical)   Using I_x tables:

$w_u = 208$ psf trib $\Delta_{TL} = \frac{L}{240} = \dfrac{26(12)}{240} = 1.3" = \dfrac{5(2.4 klf)(26)^4(1728)}{384(29000)(I_{req})}$ $I_{req} = 654.6 in^4$

$V_u = \dfrac{(.208)(15')(26')}{2} = 40.6^k$

W21×44:   Try W21×44 $(I = 843)$

$\phi M_p = 358 > 263.6$ ✓ ok $\Delta_{LL} = \dfrac{5(.6)(26)^4(1728)}{384(29000)(843)}$

$M_u = \dfrac{(.208)(15)(26)^2}{8} = 263.6^{1k}$

$\phi V_n = 217 > 40.6$ ✓ ok $\Delta_{LL} = 0.252" < 0.87"$ ✓ ok

$\Delta_{LL \, allow} = \frac{L}{360} = \dfrac{26(12)}{360} = 0.87"$ $\Delta_{TL} = \dfrac{5(2.4)(26)^4(1728)}{384(29000)(843)} = 1.01" < 1.3"$ ✓ ok $\boxed{\text{USE W21×44 for beam B2}}$

True ~~Project~~ North
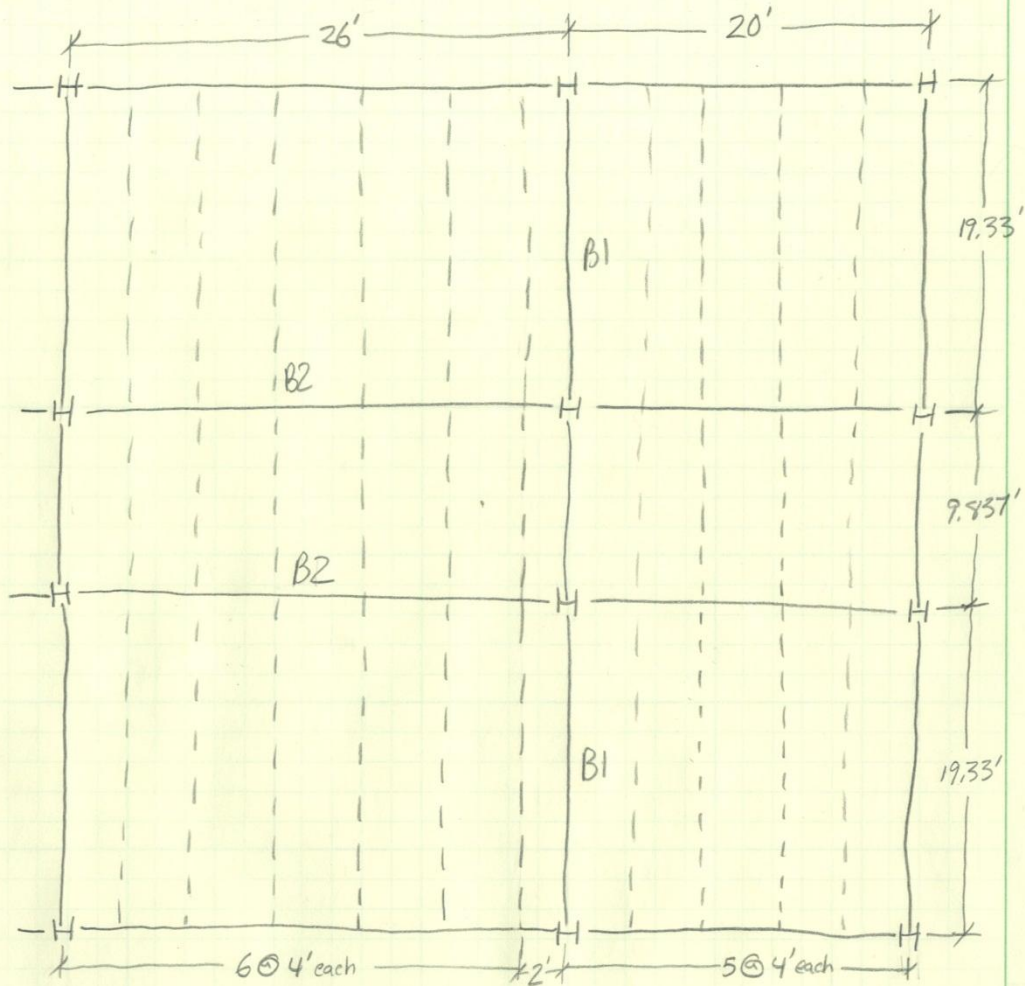
Prestressed Concrete
6"x4'-0" Hollow Core Plank

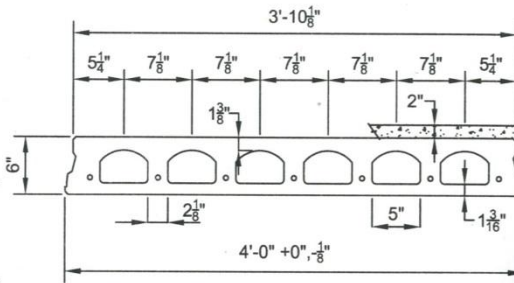### 2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 253 in.^{2}	Precast b_w = 16.13 in.
I_c = 1519 in.^{4}	Precast S_{bcp} = 370 in.^{3}
Y_{bcp} = 4.10 in.	Topping S_{tct} = 551 in.^{3}
Y_{tcp} = 1.90 in.	Precast S_{tcp} = 799 in.^{3}
Y_{tct} = 3.90 in.	Precast Wt. = 195 PLF
	Precast Wt. = 48.75 PSF

DESIGN DATA

1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   - 4-1/2"Ø, 270K = 67.4 k-ft at 60% jacking force
   - 6-1/2"Ø, 270K = 92.6 k-ft at 60% jacking force
   - 7-1/2"Ø, 270K = 95.3 k-ft at 60% jacking force
7. Maximum bottom tensile stress is $10\sqrt{f'c}$ = 775 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS		IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)																		
Strand Pattern		**SPAN (FEET)**																		
		12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
4 - 1/2"ø	LOAD (PSF)	349	317	290	258	227	197	174	149	127	108	92	78	66	55					
6- 1/2"ø	LOAD (PSF)	524	478	437	377	334	292	269	237	215	188	165	142	122	104	88	73	61	49	39
7 - 1/2"ø	LOAD (PSF)	541	492	451	416	364	331	293	274	242	214	190	167	144	124	107	91	77	64	53

NITTERHOUSE
CONCRETE N PRODUCTS

2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17202-9203
717-267-4505   Fax 717-267-4518

11/03/08

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

6F2.0T