NORTH SHORE EQUITABLE BUILDING

PITTSBURGH, PA

STEPHAN NORTHROP - STRUCTURAL OPTION

TECHNICAL REPORT #2

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EXECUTIVE SUMMARY

In Technical Report 2, an analysis was performed on four possible structural systems for the design of the North Shore Equitable Building in Pittsburgh Pennsylvania. This report includes a brief overview of the findings of report 1, an analysis of the existing structural system of this building and an investigation of three alternate structural systems that could be used in the design of this building. The objective of this report was to become familiar with the four systems chosen, compare the advantages and disadvantages of each system and ultimately choose one system to investigate further as a potential replacement for the existing system.

The four structural systems investigated in this report are;

- Steel frame structure with composite deck (existing system)
- One way wide module skip joist and beam system (alternate #1)
- Precast, pre-stressed hollow core planks (alternative #2)
- Two way post-tensioned slab (alternative #3)

The main design features that are compared between each system are floor weight, floor depth, cost, susceptibility to vibration, architectural impact and fire protection. There are a handful of other design features that are evaluated as well.

From this investigation, it was determined that the existing composite steel system is an excellent system for this design and offers several advantages including long spans and low building weight. The two way post-tensioned slab system and the hollow core plank system were ruled out as alternatives due to their excessive weight and inflexibility with regards to the existing column grid. The one way skip joist and beam system proved to be a solid structural alternative due to its smaller floor depth, long span ability and reasonable weight. Upon conclusion of this report, the one way skip joist and beam system was selected as the potential alternative structural system warranting further investigation.

1. INTRODUCTION

The North Shore Equitable Building is a 6 story, 180,000 square foot low rise commercial office building located on Pittsburgh's North Shore. Completed in 2004, this building is part of the North Shore development project between Heinz Field and PNC Park. Of the building's 180,000 square foot area, 150,000 square feet consists of office space on floors 2 to 5 and the remaining 30,000 square feet is retail space on the ground level. In addition to the 6 above grade levels, one sublevel of parking is also provided, which accommodates 80 vehicles. The North Shore Equitable Building offers its tenants amenities such as an employee fitness center, a test kitchen for product development and the North Shore Riverfront Park which offers access to riverside trails and beautiful views of the Pittsburgh skyline across the Allegheny River.

Figure 1-1: View of the North Shore Equitable building from Mazeroski Way

Among the Equitable building's notable architectural features are what is referred to as a turret, located at the southwest corner of the building and two towers located at the northwest and southeast corners of the building respectively. The majority of the building's façade consists of cast stone masonry units up to the third level and a combination of composite metal paneling and face brick from the third level up to the roof level. Two skylights can be found on the roof as well with the architectural designs including a location for a proposed third skylight which was never built.

2. STRUCTURAL SYSTEMS OVERVIEW

The structural system of the North Shore Equitable Building consists of composite steel beams and girders to resist gravity loads and a combination of braced frames and moment frames to resist lateral loads. These components of the building's structural design, along with all other structural design components, will be described in further detail below.

Foundation

The foundation consists of a 5 ½" slab on grade supported by concrete grade beams and a combination of 18" auger cast piles and steel H-piles. Reinforced concrete retaining walls in the parking garage extend from the top of the grade beams to the first floor framing. These walls are restrained at the top by the first floor framing.

The piles for the Equitable Building pose a unique set of design requirements. The Allegheny Port Authority is currently extending their light rail transit system under the Allegheny River to Pittsburgh's North Shore. This extension consists of two parallel tunnels which are designed to pass directly below the Equitable Building as seen in Figure 2-1. As a result, the foundation is designed as a combination of two types of foundations; driven Steel H-piles (Figure 2-2 on the right) to withstand pressures and settlement resulting from tunneling under the building and 18" auger cast piles (Figure 2-2 on the left) for the remainder of the foundation.

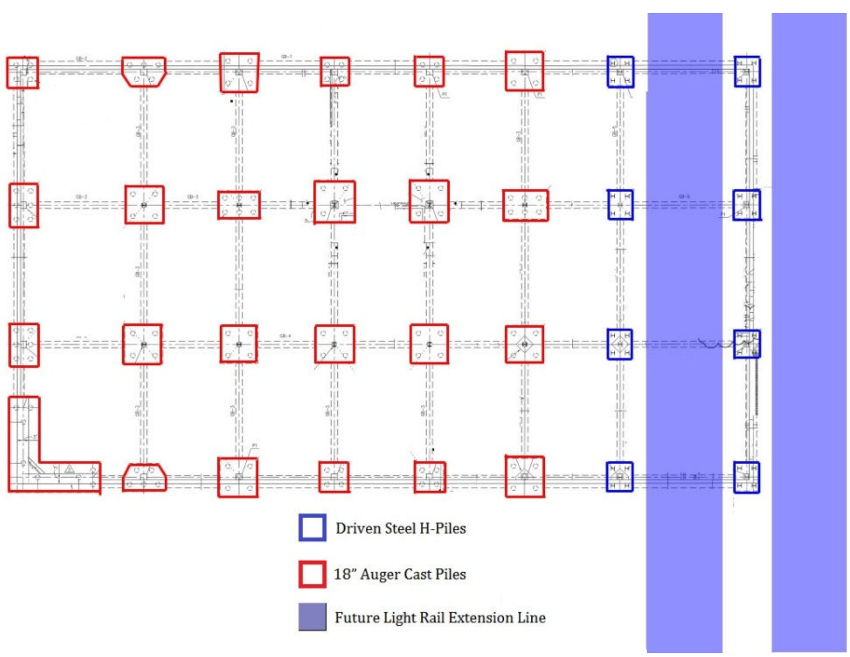


Figure 2-1: Foundation plan with future transit line extension

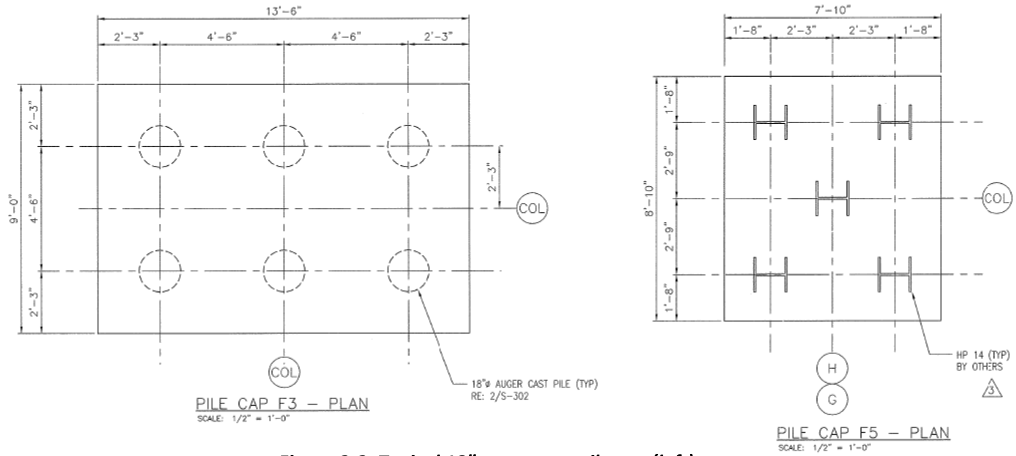


Figure 2-2: Typical 18" auger cast pile cap (left) and typical steel H pile cap (right)

General Floor Framing

Due to the equitable building's rectangular shape, the framing follows a simple grid pattern (128' wide by 228' long). Framing consists of a lightweight concrete slab supported by steel beams girders and columns. The slab has a total depth of 5 ½" consisting of 3 ½" lightweight concrete over a 2" 18 gage composite galvanized metal floor deck. The floor is supported by steel beams, typically W18x40's in exterior bays and W21x44's in interior bays, framing into girders ranging in size from W24x62 to W30x116. There are 7 bays on each level (approximately 30' x 42' or 40' x 42' for exterior bays and 30' x 44' or 40' x 44' for interior bays). The beams span 44' in the interior bays and 42' in the exterior bays and are spaced no more than 10' apart. The girders typically span either 30 or 40 feet. Shear studs (4 ½" length, ¾" diameter) are used to create composite action between the deck and the steel beams. Figure A-1 on page 25 shows the typical floor plan for the existing structural system.

Columns for the Equitable Building are all W14 wide flange columns ranging in weight from W14x311 on the first level to W14x48 extending up to the roof level. Columns are spliced at two locations along the vertical length of each column line at 4' above the floor level indicated. A typical column splice detail is shown to the right in Figure 2-3.

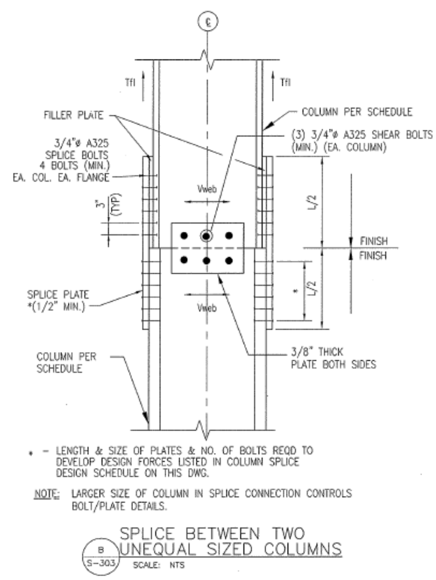


Figure 2-3: Typical column splice detail

Turret Framing Plan

For the turret at the southwest corner of the building, members of varying sizes are used as seen to the right in Figure 2-4. The columns for the turret are HSS columns ranging in size from HSS 6x6x 1/2 (on the first level) to HSS 6x6x 3/16 extending up to the roof level. These HSS columns are spliced at three locations along the column line.

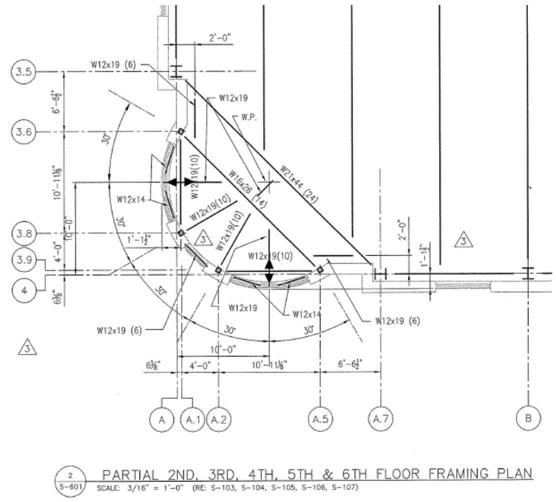


Figure 2-4: Turret framing plan

Roof Framing Plan

The roof framing system, like the floor framing system, is laid out in a simple rectangular grid. It consists of a 1 ½" 20 gage type B galvanized roof deck supported by open-web K-series joists (Figure 2-5) which frame into wide flange girders. The roof deck spans longitudinally which is perpendicular to the joist span direction. The K-series joists are generally either 28" or 30" deep and span either 44' (in interior bays) or 42' (in exterior bays). These joists are spaced no further apart than 5' typically.

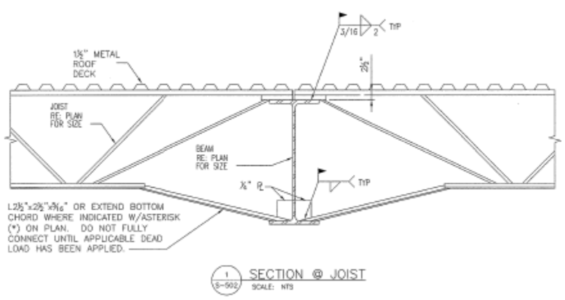


Figure 2-5: Section at joist

The girders in the roof plan vary greatly in both size and span length. Girders carrying the typical roof load vary in size from W18x35's to W30x116's (spanning anywhere from 16' to 44'). The roof girders above the core of the building supporting mechanical equipment are mainly W12x19's and W24's with a few W14's and W18's used as well. 10" and 30" deep KCS-Type open-web K-series joists are also used to help support this equipment.

The framing of the tower roofs consists of C10x20's, W10x22's and L2 ½ x 2 ½ x ¼ horizontal bridging, as seen in Figure 2-6. The framing of the turret roof consists of curved C6x13 members and wide flange members of varying lengths as seen in Figure 2-7.

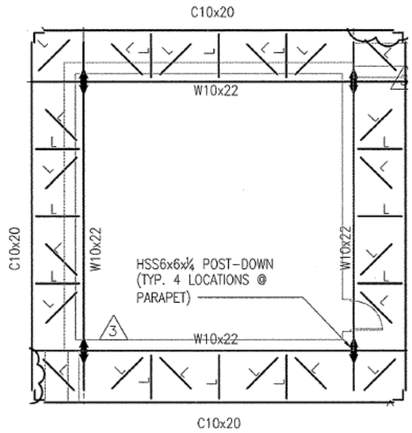


Figure 2-6: Tower roof framing plan

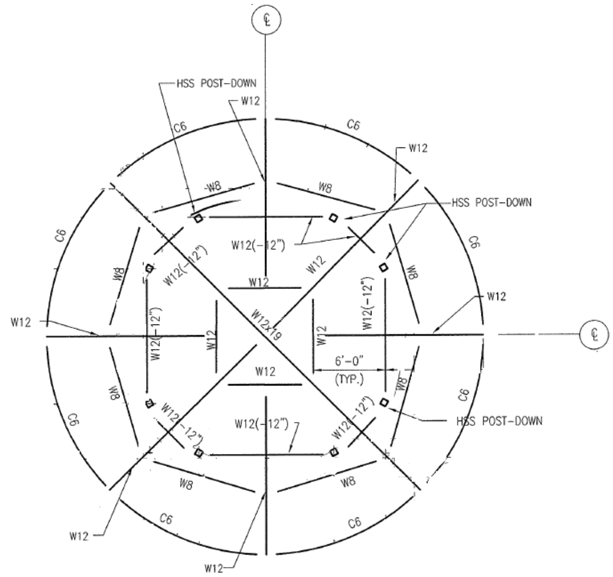


Figure 2-7: Turret roof framing plan

Lateral Resisting System

Lateral stability in the North Shore Equitable Building is achieved through the use of a combination of braced frames and moment frames. Braced frames run in the transverse direction and moment frames run in the longitudinal direction as seen in Figures 2-8 and 2-9 below. The floor and roof decks, which act as horizontal diaphragms, transfer lateral forces to the frames. Elevation views of these frames can be seen in Figures 2-10 and 2-11. The connections in the moment frames are semi rigid connections. Details of a typical braced frame connection and a moment frame connection are shown in Figures 2-12 and 2-13 respectively.

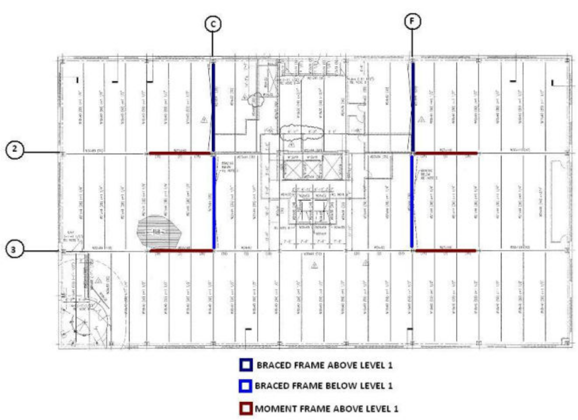


Figure 2-8: Lateral Resisting elements at level 1

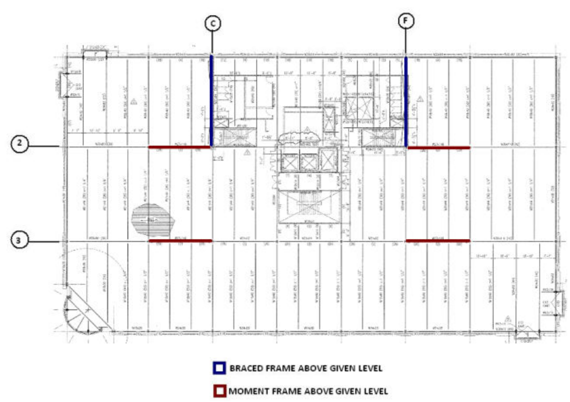


Figure 2-9: Lateral Resisting elements at levels 2-6

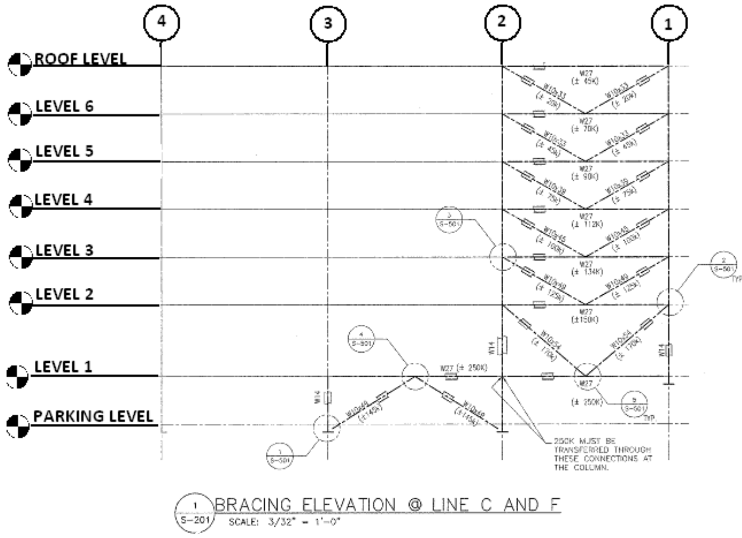


Figure 2-10: Braced frame elevation

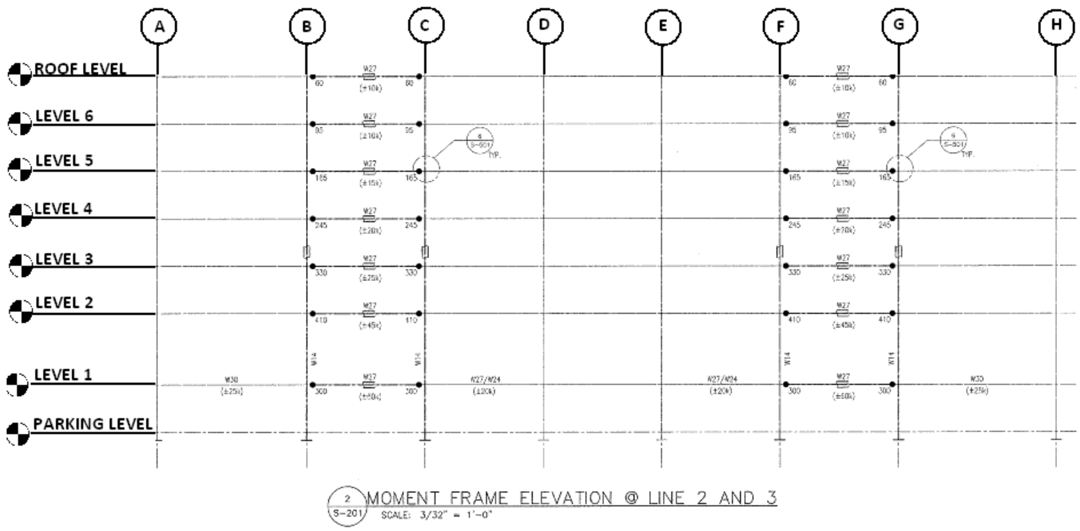


Figure 2-11: Moment frame

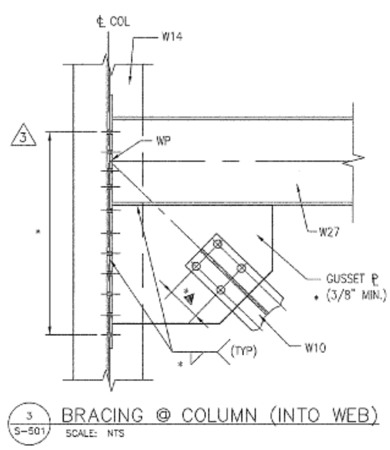


Figure 2-12: Braced frame connection

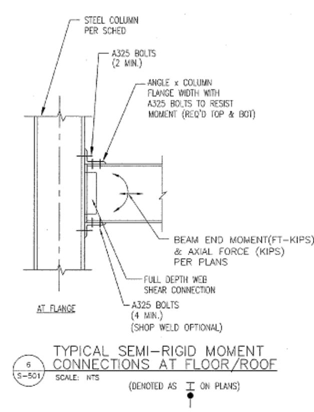


Figure 2-13: Moment frame connection

3. MATERIALS USED

Several different structural material types are used in the design of the North Shore Equitable building. Generally, standard material strengths are used throughout the building. Slabs, footings and grade beams all consist of normal weight concrete (with the exception of the elevated floor slabs). Steel is used for all framing and lateral members, with A992 steel being used for beams, girders and columns and A36 steel being used for all connecting elements (as is customary)

TABLE 3.1 - Concrete Materials Schedule

Structural Element	Weight (pcf)	Strength (f'c)
Footings	150	4000
Drilled Piers	150	4000
Grade Beams	150	4000
Slab On Grade	150	4000
Elevated Floor Slabs	110	4000
Auger Cast Piles	150	4000
All Other Concrete	150	4000

TABLE 3.2 - Masonry Materials Schedule

Structural Element	Compressive Strength
Concrete Masonry	1500 PSI

TABLE 3.3 - Steel Materials Schedule

Structural Element	Yield Strength (ksi)	ASTM Designation
Steel Roof Deck	33 (minimum)	A446
Beams And Columns	50	A992
Rectangular Tube Steel	46	A500 Grade B
Bracing	36	A36
Connections, Plates And All Others	36	A36
Anchor Rods	36	A36
Pipes	35	A53 Grade B
Round Tube Steel	42	A500 Grade B
Light Gage Metal Studs	50	A653
Structural Steel Bolts	92	A325
Column Splice Design Schedule		
Splice Mark	Flange Tension (K)	Web Shear (K)
CS1	60	20
CS2	85	20

4. APPLICABLE CODES

Since the North Shore Equitable building was designed and built between 2003 and 2004, the codes used by the designers are a couple editions older than the codes used for this report. The codes used by the designers and in this report are given below.

Codes Used In the Original Design

- The BOCA National Building Code, 1999
- City of Pittsburgh Amendments to The Boca National Building Code
- ASCE 7-95, Minimum Design Loads for Buildings
- ACI 301, Specifications for Structural Concrete for Buildings
- ACI 318-95, Building Code Requirements for Reinforced Concrete
- ACI 530, Building Code Requirements for Masonry Structures
- AISC/ASD-89, Manual of Steel Construction, 9^{th} Edition
- AISC/LRFD-2001, Manual of Steel Construction, 3^{rd} Edition
- SJI-41^{st} Edition, Standard Specifications and Load Tables for Steel Joists and Joist Girders

Codes Used In Tech 2 Analysis

- ASCE 7-05, Minimum Design Loads for Buildings
- AISC Manual of Steel Construction, 13^{th} Edition
- ACI 318-08, Building Code Requirements for Reinforced Concrete

5. DESIGN LOADS

For the design of this building, the structural engineers at Michael Baker chose to conservatively take the live load as 100 psf rather than the 50 psf recommended by ASCE 7-05. Having worked at Michael Baker as an intern this past summer, it is my understanding that the structural engineers use 100 psf live loads as a general rule of thumb when designing composite steel buildings. For the alternate system analyses in this report, an 80 psf live load is used rather than the ASCE prescribed 50 psf. This was done in an attempt to be conservative but also to try to avoid overdesigning the alternate systems.

TABLE 5.1 - Live Loads

Load Type	As Designed (psf)	Per ASCE 7-05 (psf)
Floor Live Loads		
Office	**100**	**50**
Corridors	**100**	**100 (first level) 80 (upper levels)**
Mechanical	**150**	**(not provided)**
Stairs	**100**	**100**
Retail	**100**	**100**
Garage Live Load	**50**	**40**
Roof Live Load	**20 (min)**	**20**

TABLE 5.2 - Dead Loads

Load Type	As Designed (psf)
Superstructure Weight	5
Roofing, Ceiling, Misc.	8
Collateral Load (MEP)	7
Total Roof Dead Load	**20**
5 ½" Light Weight Conc. Slab	45
Steel/Joist Framing	10
Ceiling, Misc.	5
MEP	5
Total Floor Dead Load	**65**
6" Metal Studs + Insul + GWB 4" Brick	10 40
Total Exterior Wall Load	**50**
Stairs	30
Stair Landings	40

6. FLOOR SYSTEMS

In the following four chapters, three alternate floor systems are chosen to be analyzed and compared to the existing structural system of this building. The existing system, which is a steel frame system with a composite deck, was analyzed in the first technical report. The findings of that analysis will be reiterated in this report for ease of comparison. The three alternate systems chosen for analysis were a one way skip joist and beam system, a pre-stressed hollow core plank system, and a two way post-tensioned slab system. These systems were chosen based on several relevant design factors that will be elaborated upon later in this report.

For each alternate floor system, a superimposed dead load of 20 psf was used and the live load was taken as 80 psf (except for the post-tensioned slab which was conservatively taken as 100psf). For each floor system, an attempt was made to base the design on the current column grid with a 38’ x 44’ bay used as the representative design bay. The analyses will show, however, that this 38’ x 44’ bay cannot always be accommodated as part of the alternative designs. Lateral loads have not been taken into account for the design of alternate floor systems in this report.

6.1 – Existing System: Steel Framing with Composite Deck

The current structural system in place for the North Shore Equitable Building is a lightweight composite slab supported by steel beams girders and columns. As stated in the structural systems overview, the slab consists of 3 ½” lightweight concrete over a 2” 18 gage composite galvanized metal floor deck (5 ½” total depth). The beams are typically W18x40’s in exterior bays and W21x44’s in interior bays and the girders range in size from W24x62 to W30x116. The typical beam span is 42’ to 44’, spaced at 10’ typically and the girders typically span either 30 or 38 feet. Shear studs (4 ½” length, ¾” diameter) are used to create composite action between the deck and the steel beams.

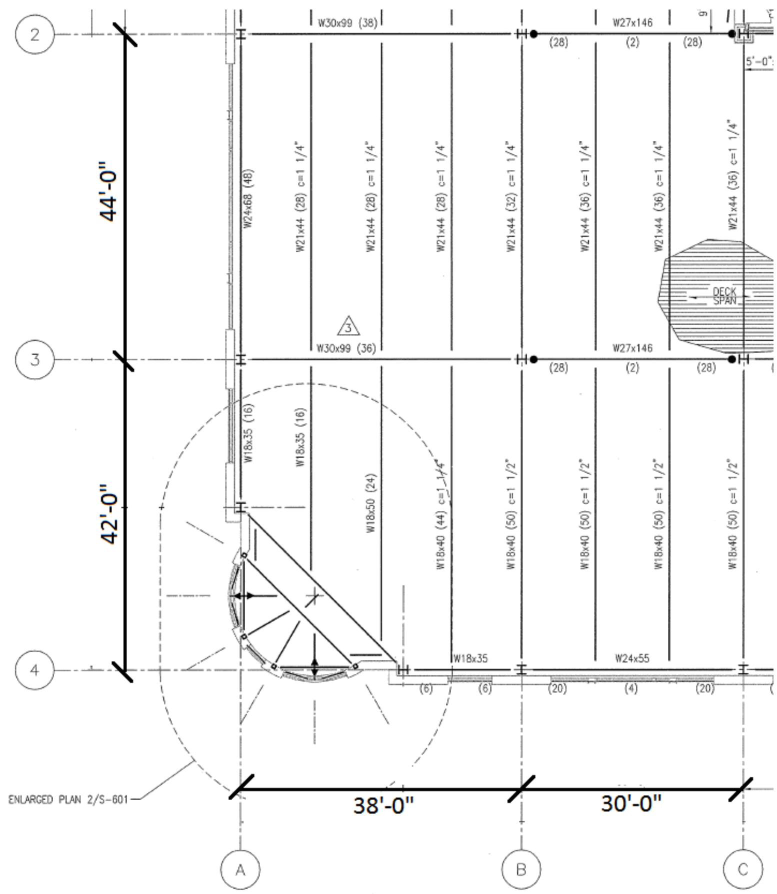


Figure 6 - 1: Partial framing plan showing bay sizes

It is clear why the designer chose a composite steel system. Steel framing systems are relatively easy to design (compared to concrete systems) and changes to the design are easy to accommodate during the design phase. Also, steel frame designs allow for a more open floor plan. A list of the advantages and disadvantages of composite steel systems is given on the next page.

Advantages	
1.	Relatively low building weight compared to other systems
2.	Composite action decreases the necessary member sizes
3.	Easily and quickly erected
4.	Steel is recycled
5.	No formwork is required
6.	Allows more open plans and the option of glazed facades
7.	Changes in design are simple during the design process
8.	Cost is average compared to most systems

Disadvantages	
1.	Vibration damping is minimal
2.	Deflections are larger than with other floor systems
3.	Floor thickness is increased as compared to concrete systems
4.	Requires a more complicated lateral system
5.	Fireproofing must be added (unlike with concrete systems
6.	Member may interfere with mechanical ductwork

W18x40's are the most common beam used in this building, found at over 170 locations at a length of 42 feet and a span of 10 feet. As a result, a W18x40 was chosen for the typical beam spot check. A W24x55 edge girder was chosen for analysis since it is the most prevalent girder size (appearing at 45 locations). The analyses showed that both the beam and the girder selected were able to carry their respective applied loads and meet deflection criteria. For the spot checks, a live load of 50psf was used instead of the overly conservative 100psf live load used by designers. This resulted in a lower size being selected for the edge girders than seen in the design. The typical column analyzed was a W14x211 column on the first level. The results of this check show that this column exhibits inelastic behavior and can carry the axial load both from a yielding and buckling standpoint.

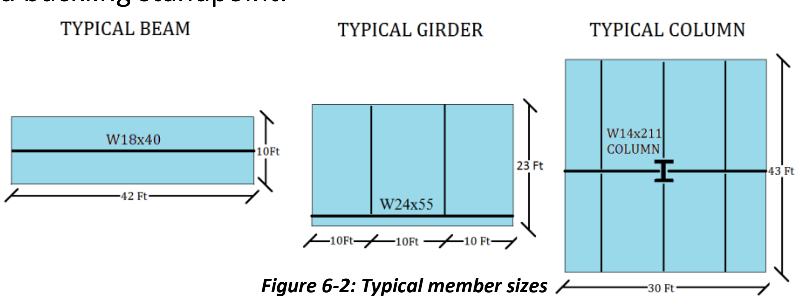


Figure 6-2: Typical member sizes

There are a few disadvantages to this system, but overall, the advantages outweigh the disadvantages and it can be concluded that this system is definitely a viable structural system for the design of this building.

6.2 – Alternate System #1: One Way Skip Joist and Beam System

The first alternate system chosen to be analyzed is a one way wide module skip joist and beam system. This system was chosen for analysis due to its ability to accommodate large spans, which are prevalent in the design of this building. A skip joist system with 53" wide pans was chosen over a system of 30" pans because wide module systems are more economical for large bay sizes. This system also offers inherent vibration resistance which is an especially important design feature considering the light rail transit line designed to pass under the foundation.

Because this system can accommodate large spans, the original column layout was used in this analysis. The slab for this system was taken as a 4 ½" NW concrete slab with a weight of 56.25 psf. reinforced with #3 bars spaced at 12". Using _Concrete Floor Systems: Guide to Estimating and Economizing_ by David Fanella, the bay size was approximated as 30' x 40'. The pan depth, rib width, beam width and column size were then selected from the table shown below in Figure 6-3. Because of the extremely large beam and column sizes selected from this table, the decision was made to approximate the bay size unconservatively as 30' x 40' under the assumptions that the sizes would still work. Hand calculations for the pan, rib, beam and column sizes using a 30' x 44' bay confirmed that the sizes selected would indeed work for a 30' x 44' bay. The pan depth, however, was conservatively chosen as 20" rather than the 16" given in the table. If this system is chosen for further analysis, a 16" pan will be investigated. A torsional analysis was also conducted and the sizes and reinforcement were found to be sufficient for torsion as well. The results of the hand calculations (found in appendix C) are shown in table 6.1.

One-Way Joist – 53" pan					f'_c = 4,000 psi SIDL = 20 psf		
					Slab h = 4½" LL = 100 psf		
Bay Size (ft)	Pan Depth (in.)	Rib Width (in.)	Beam Width (in.)	Square Column Size (in.)	Concrete (ft^3/ft^2)	Reinforcement (psf)	Pan Area (%)
20 × 20	16	7	22	22	0.68	2.35	89
20 × 25	16	7	24	24	0.67	2.43	91
20 × 30	16	7	26	26	0.65	2.51	91
20 × 35	16	7	32	32	0.65	2.76	91
20 × 40	16	7	34	34	0.64	2.95	92
25 × 25	16	7	28	28	0.68	2.60	89
25 × 30	16	7	32	32	0.67	2.66	90
25 × 35	16	7	34	34	0.66	3.10	90
25 × 40	16	7	36	36	0.65	3.52	91
30 × 30	16	7	34	34	0.67	3.03	89
30 × 35	16	7	38	38	0.67	3.24	89
30 × 40	16	7	40	40	0.66	3.53	90
35 × 35	20	7	40	40	0.76	3.27	89
35 × 40	20	7	42	42	0.74	3.48	90
40 × 40	20	7	44	44	0.75	4.01	89
45 × 45	24	7	44	44	0.82	4.10	90
50 × 50	24	7	60	48	0.85	4.99	89

Figure 6 - 3: Design table courtesy of
<u>Concrete Floor Systems: Guide to Estimating and Economizing</u> by David Fanella

Table 6.1 - Hand Calculation Results

	Width	Depth	Top Reinforcement	Bottom Reinforcement	Stirrups
Slab	- -	4.5”	#3 bars @ 12” o.c.		
Skip Joists	53”	20”	4#6 bars	2#10 bars	#3’s @ 8”
Beams	40”	24.5”	10#6 bars	14#6	#3’s @ 6” (2 legs)

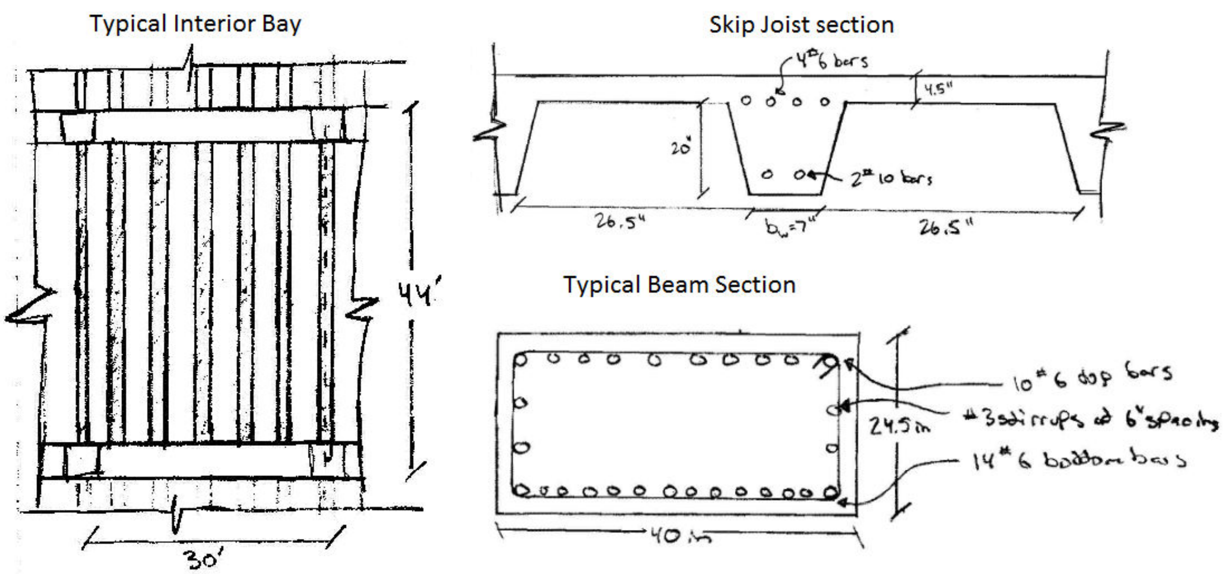


Figure 6 - 4: One way joist and beam system details

Although, the weight increased for the one way system, the floor depth is decreased by over 10”. Just as with a composite steel system, the floor depth of one way skip joist systems is not affected by mechanical equipment since this equipment can be run between the ribs. Also, with the use of concrete, no fireproofing is needed since the concrete itself is fireproof. The cost of this structure would most likely be kept around or even below the cost of the existing system. There is no effect on the original column grid with this system but the columns sizes are greatly increased which will have a negative impact on the building weight and may affect the foundation. The architectural appearance of the building may also be negatively impacted and some interior space is lost due to the increase in column size. For this system, an alternative lateral system will need to be used. This system will most likely consist of masonry shear walls. A general list of advantages and disadvantages for this system is given on the following page.

Advantages

1.	Economical for large spans up to 40' x 40' bays and heavy loads
2.	Pan voids reduce the dead loads
3.	Overall floor depth doesn't need to be increased to accommodate equipment
4.	Inherent vibration resistance
5.	Provides for maximum flexibility in space planning
6.	Easier future renovations
7.	No additional fireproofing necessary
8.	Easier construction due to faster lead time and simpler connections than steel
9.	Cost will not increase drastically

Disadvantages

1.	Very large column sizes
2.	Less desirable architectural appearance than steel frames
3.	Increase in building weight as compared to steel frame systems
4.	Requires formwork

As can be seen from the analysis, a wide module skip joist and beam system offers several improvements over a composite steel frame system while minimizing negative impacts to the design. This system would make an excellent alternative to the existing system and should be investigated further.

6.3 – Alternate System #2: Precast, Pre-stressed Hollow Core Planks

The second structural system chosen for analysis is a precast, pre-stressed hollow core plank system. This system was attractive given its ability to accommodate longer spans than two way slab systems which is a key design feature of the current grid layout. There is also the potential to decrease the slab weight using this system because of the voids in the planks. Lastly, hollow core planks have low noise transmission which could help reduce noise levels caused by subway trains running below the building.

Using the load tables found on Nitterhouse Concrete Product's website, a 6" deep, 4'0" wide hollow core plank was chosen for analysis. Specifications on this plank can be seen in Figure D-1. It was found that this size plank supports the superimposed dead load of 20 psf and a live load of 80 psf at a length of 20'. This particular size plank would need to be reinforced with 6 - ½" Ø strands to support the loads. Shown below are a hand drawn plan view of typical bays and a section of the plank size chosen. The 20' plank lengths run east to west as shown by the dashed lines in Figure 6-6.

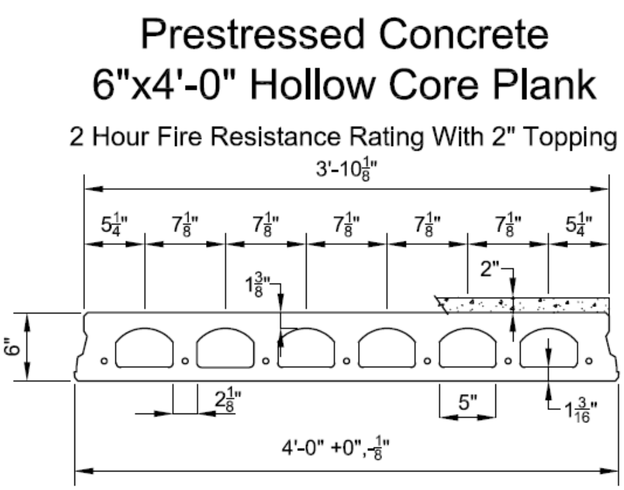


Figure 6 - 5: Image courtesy of Nitterhouse Concrete Products

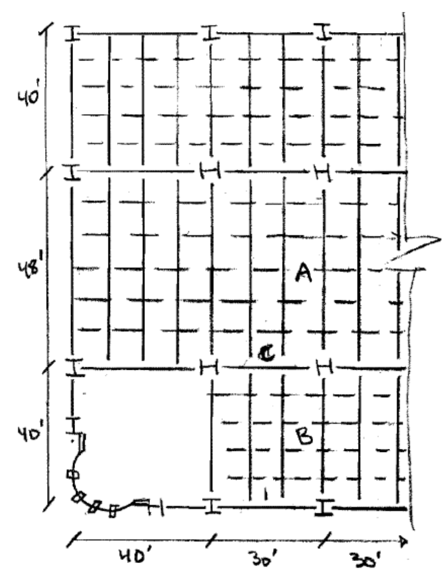


Figure 6 - 6: Partial floor plan showing typical bays

Although the hollow core plank system was chosen with the intent to decrease the building weight, the end result was an increase in weight. The hollow core slab turned out to be slightly heavier than the composite slab (48.75 psf compared to 45 psf). Also, switching from composite beams to non-composite beams resulted in an increase in both beam depth and weight. This caused the floor system depth to increase by 2 ½" as well. The cost of this structure would increase because of the increase in beam size. The increase in weight and member sizes would most likely effect the current foundation design which would have to be rechecked to make sure it was still able to resist applied lateral loads affecting the subway tunnel design. The existing lateral support system, however, would most likely remain intact if hollow core planks were to be used. Also, as with the existing system, fireproofing would be required for the steel members.

Because the planks come in pre manufactured sizes, the bay widths on the eastern and western faces of the building would need to be adjusted from 42' and 44' to 40' and 48'. Also, the bay on the southwest corner of the building (where the turret is located) is not rectangular and would not accommodate prefabricated planks. An alternate structural system would have to be proposed for this particular bay. A general list of pros and cons for this system is shown below.

Advantages

1.	Reduced construction time
2.	Low noise transmission
3.	Accommodation of larger spans than two way slab systems
4.	Low maintenance
5.	No formwork needed

Disadvantages

1.	Increased building weight
2.	Increased floor depth
3.	Increase in cost
4.	Column grid adjustment needed due to prefabricated sizes
5.	Only accommodates rectangular shaped bays
6.	Leveling compound will be needed to compensate for cambering

Hollow core plank systems have numerous disadvantages to them when compared to composite steel systems. The increase in weight and floor depth, combined with the multiple required design adjustments show that this system is not a practical alternative to the existing design. No further investigation is required.

6.4 – Alternate System #3: Two Way Post-tensioned Slab

The third and final structural system evaluated as a potential alternative is a two way post-tensioned flat slab. This system was chosen for analysis based on the fact that post-tensioned flat slabs greatly reduce floor height, simplify formwork, and reduce ceiling finish costs. Going into the evaluation, there were concerns about whether this system would be able to span the large end bays that are present in this building.

Through hand calculations, it was determined that a 13 inch slab reinforced with 7 - ½" ø strands would be necessary to carry the applied loads. It was also determined however that a post-tensioned flat slab system (with or without drop panels) would not be able to span the 44' x 38' end bays without exceeding the allowable compression stress limit. In order to proceed with the analysis, an end bay size of 44' x 30' was assumed (which is also the bay size for a typical interior bay). This could potentially be achieved by adding an 8' cantilever to the end of the edge bays on the west face of the building as shown in Figure 6-7 below. The column lines on the east face of the building cannot be moved due to the designed subway rail line passing under the foundation. The bays would most likely be split in half along the 44' dimension creating two 22' x 40' end bays. Under this assumption, the slab is able to span all other existing bay sizes. The slab would be reinforced using 6 #9 top bars at interior and exterior supports, and #3 bottom bars spaced at 12" o.c. at the midspans of exterior spans. No reinforcement is needed in the positive moment region of interior spans for this design.

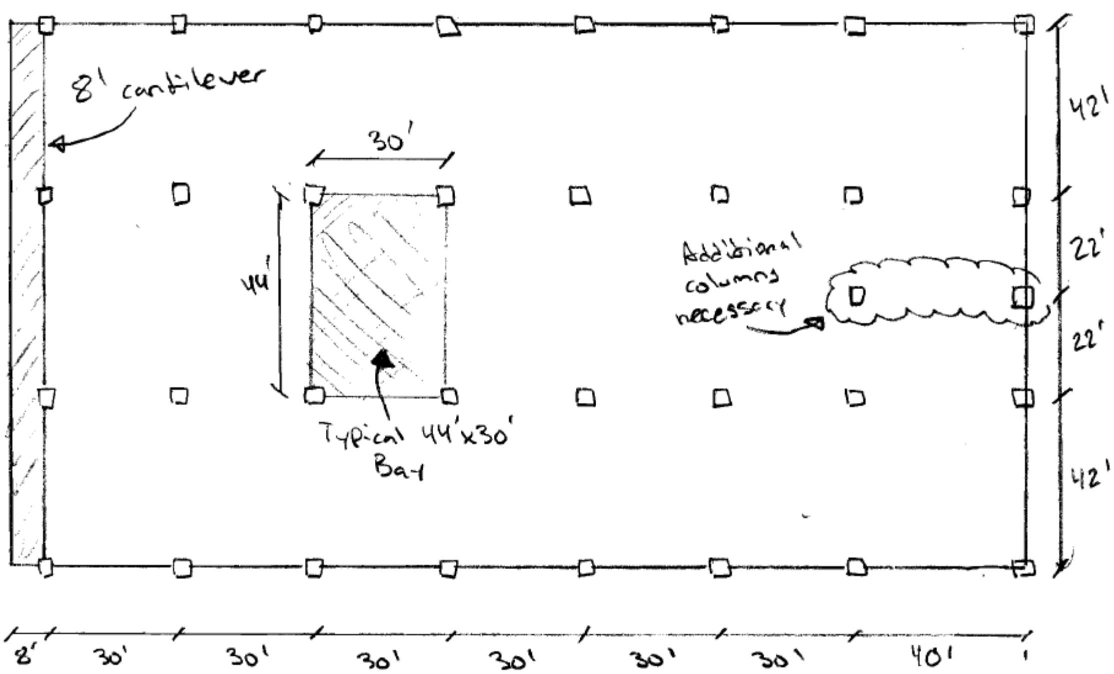


Figure 6 - 7: Two way post-tensioned slab plan

A post-tensioned two way slab offers very few advantages to the design of this building. The floor depth was drastically reduced, but at the cost of weight and the current grid layout. In order to utilize this system, several changes to the grid layout would have to be investigated. The lateral system would need to be redesigned using masonry shear walls most likely. Due to the fact that the slab weight for this system is nearly double that of any other system investigated, this system would have adverse effects on the foundation design. Also, a more experienced construction team would be needed to perform the post-tensioning work and the construction team would be exposed to more dangerous working conditions. There are a few advantages to this system, such as lower costs associated with lower floor heights and no need for fireproofing, but these advantages are negligible compared to the disadvantages mentioned above. The advantages and disadvantages of this system in general are as listed on the next page.

	Advantages
1.	Reduced floor to floor height
2.	Cost savings due to reduced floor height and reduced ceiling finish costs
3.	No need for additional fireproofing

	Disadvantages
1.	Increased building weight
2.	Large bay sizes not accommodated so current column grid is not feasible
3.	Negative impact on foundation due to increased weight
4.	A more experience construction team familiar with post-tensioning would be necessary
5.	Much larger columns than a composite steel system
6.	Potential for punching shear

Based on the advantages and disadvantages above, especially given the fact that several changes would need to be made to the current column grid, a two way post-tensioned slab system is not a feasible alternative to the existing system. No further research of this system is needed.

7. COMPARISON OF SYSTEMS

A comparison chart of the existing system and the three alternate systems is shown below. When choosing which alternate systems to investigate further, extra emphasis was placed on the weight of the floor systems, the floor depth, cost and the impact on the column grid.

Table 7.1 - Structural System Comparison

Factors to consider	Steel with composite deck	One way joist and beam	Pre-stressed hollow core planks	Two way post-tensioned slab
weight of floor deck and beams	55.09 psf	87.15 psf	63.4 psf	162.5 psf
cost	$14.80 per SF	$12.37 per SF	$22.03 per SF	Unsure
floor depth	35.5 in	24.5 in	38 in	13 in
Column size	30 x 30(including GWB)	40 x 40	30 x 30 (including GWB)	40 x 40
Additional fireproofing?	Yes	No	Yes	No
Formwork necessary?	No	Yes	No	Yes
Vibration reduction*	Minimal†	Better	Better	Unsure
ease of construction*	Average†	Quicker and simpler	Quicker and simpler	More Difficult
MEP impact*	N/A	None	Negative	None
column grid impact*	N/A	No change	Negative	Very Negative
foundation impact*	N/A	None	None	Very Negative
lateral system impact*	N/A	Slightly negative	None	Negative
architectural impact*	Positive†	Slightly negative	Neutral	Neutral
Viable alternative?	Yes	Yes	No	No

Note: Costs are obtained from RS Means 2002 and would have to be adjusted for inflation upon further analysis
* evaluation of these categories given for the alternate systems are relative to the existing system
† evaluation of these categories given for the existing system are the baseline for the evaluations given for the alternate systems

8. CONCLUSION

All the floor systems analyzed in this report have advantages and disadvantages. Upon further investigation, it was discovered that some systems are much better for the design this building than others. The existing composite steel system was an excellent choice by the designer given its reasonable cost, low floor weight, and ability to span longer distances. A one way skip joist and beam system also proved to be an excellent design choice. Advantages such as a reduction in floor depth and reduced vibration, all without raising the weight too much or affecting the column grid make it an excellent floor system to be investigated further. The pre-cast hollow core plank and the two way post-tensioned slab systems however proved to be impractical alternatives. The hollow core plank system increased the floor depth and weight, and did not accommodate the current grid layout. The two way post-tensioned slab presents far too many design challenges with respect to the existing column grid to be investigated further. The next step from here will be to focus further investigation of the one way wide module skip joist and beam system.

9. APPENDICES

APPENDIX A – Plans & Elevations

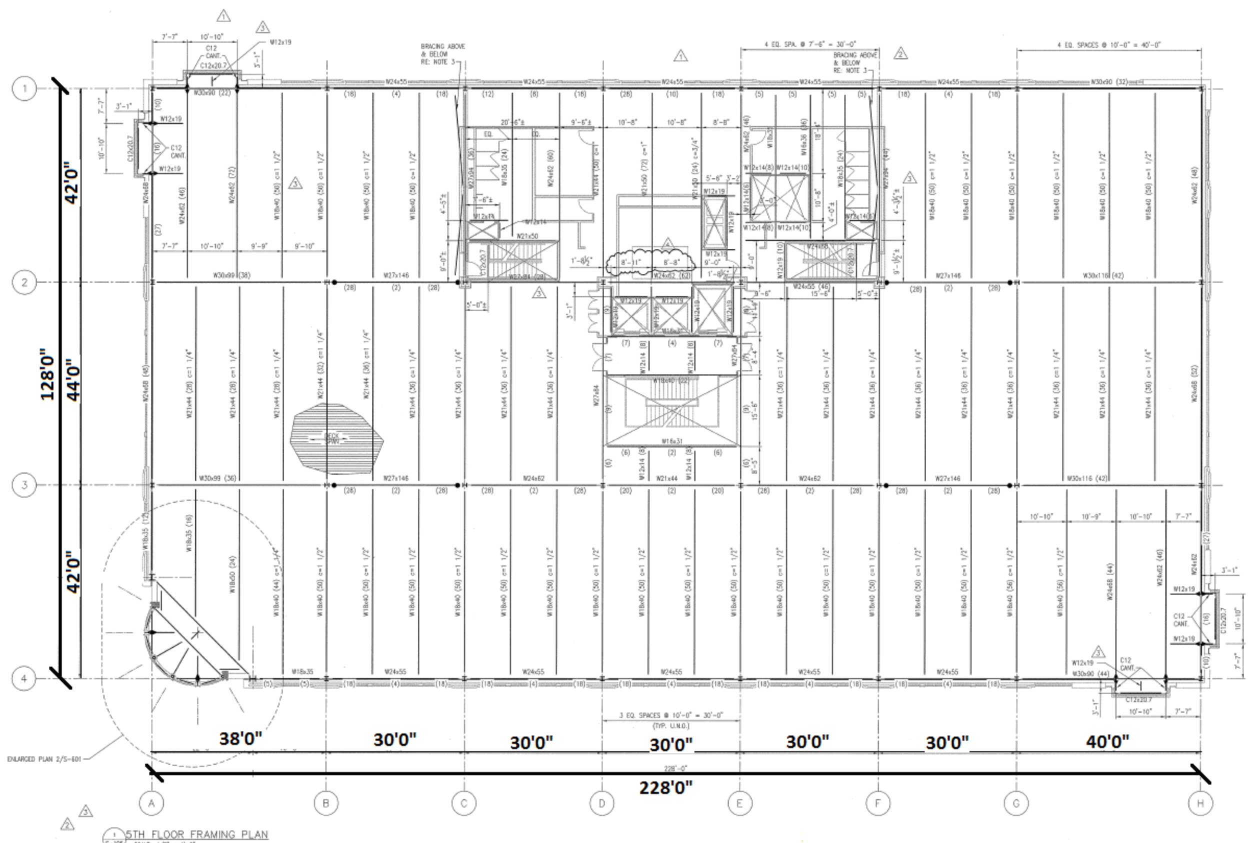


Figure A-1: Typical floor framing plan

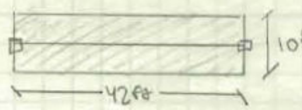
APPENDIX B – Steel Framing Calculations

Typical Beam and Girder spot check | Tech Report # 2 | Page 1 of 4

Typical Beam

(A992 steel), $f'_c = 4000$ psi

$w_D = 55 +$ selfweight
$w_L = 50$ psf
– take self weight to be 5 psf



$LL_{reduction} = L_o\left(.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right)$

$A_T = 10(42) = 420\ ft^2$
$K_{LL} = 2$ (interior beams)

$L = 50\left(0.25 + \frac{15}{\sqrt{2(420)}}\right) = 50(.768) = 38.377$

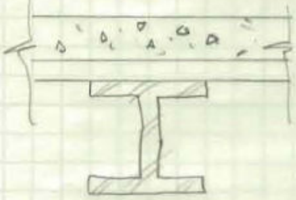
$\sqrt{2(420)} < 400 \Rightarrow$ reduction is OK

$w_u = 1.2 w_D + 1.6 w_L = 1.2(60) + 1.6(38.38) = 133.4$ psf

133.4 psf (10 ft) = 1334 plf = 1.334 klf

$M_u = \frac{w_u L^2}{8} = \frac{1.334(42)^2}{8} = 294.15$ ft·k

* use 2" deck with 3.5" LW conc topping (45psf used in design)



$B_{eff} = \min \left| \begin{array}{l} \frac{42(12)}{8} \times 2 = 126 \\ 10(12) = 120 \end{array} \right. = 120$ in

assume $a = 1 \Rightarrow Y_2 = 5.5 - \frac{a}{2} = 5$

Try W16x31 $\Rightarrow \phi M_n = 325$ ft·k
$\Sigma Q_N = 164^k$
Stud strength $= 17.2^k$

$\frac{164}{17.2} = 9.53 = 10 \Rightarrow$ $\boxed{\text{20 studs on beam}}$

Check unshored strength: W16x31, $\phi_b M_p = 203$ ft·k

* use either $w = 1.4D$ or $w = 1.2D + 1.6L$

$w = \max \left| \begin{array}{l} 1.4D \\ 1.2D + 1.6L \end{array} \right. = \begin{array}{l} 1.4(55(10) + 31) = .812\ klf \\ 1.2(55(10) + 31) + 1.6(20(10)) = 1.017\ klf \end{array}$

$M_u = \frac{1.017(42)^2}{8} = 224$ ft·k > 203 ft·k $\Rightarrow$ No good

Try W18x40: $\phi M_n = 422.5$ ft·k
$\Sigma Q_N = 147^k$
$Q_N = 17.6$

$\frac{147}{17.6} = 8.35 = 9 \Rightarrow$ Use 18 studs on beam

Unshored strength: $\phi_b M_p = 294$ ft·k
$w = 1.2(55(10) + 40) + 1.6(20)(10) = 1.03$ klf
$M_u = \frac{1.03(42)^2}{8} = 227.12$ ft·k < 294 ft·k ✓ ok

Typical Beam and Girder spot check | Tech Report # 2 | Page 2 of 4

check deflection

$W_{LL} = 38.38 \text{psf} = \frac{38.38(10)}{1000} = .384^{klf}$

$I_{LB} = 1070 \text{ in}^4$ $\quad \Delta_{LL} = \frac{5wl^4}{384EI} = \frac{5(.384)(42)^4(1728)}{384(29000)(1070)} < \frac{L}{360}$

$\Delta_{LL} = .866 \text{ in} < \frac{L}{360} = \frac{42(12)}{360} = 1.4$ ✓ ok

check wet concrete deflection

$w_{wc} = 45(10) + 40 = .49 \text{ klf}$

$\Delta_{wc} = \frac{5(.49)(42)^4(1728)}{384(29000)(612)} = 1.93 \text{ in}$ $\quad \frac{L}{240} = \frac{42(12)}{240} = 2.1 \text{ in}$

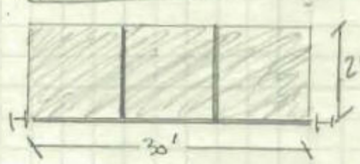
$1.93 \text{ in} < 2.1 \text{ in}$ ✓ ok

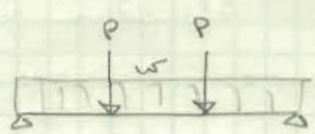
$\boxed{\text{use W18x40 (18)}}$

- as compared to W18x40 (50) in origional design.

Typical Beam and Girder spot check | Tech Report # 2 | Page 3 of 4

Typical Edge Girder





$\text{Area} = (21 + \frac{24}{12})(30) = 690 \text{ ft}^2$

$LL_{red} = L_o(.25 + \frac{15}{\sqrt{A_{T} K_{LL}}}) = 50(.25 + \frac{15}{\sqrt{2(690)}}) = 32.69 \text{ psf}$

$\sqrt{2(690)} < 400 \Rightarrow$ reduction is ok

facade = 50psf with a 13'10" story to story height

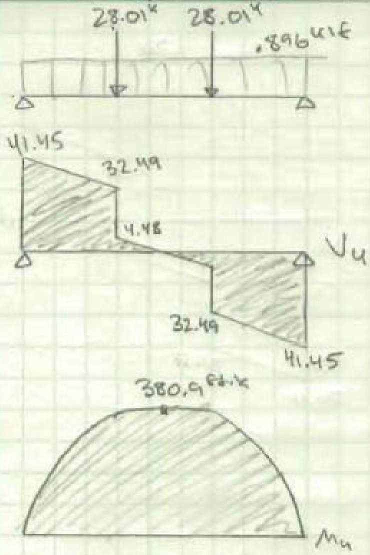
$50 \text{ psf}(13.83 \text{ ft}) = 691.7 \text{ plf} = .692 \text{ klf}$

* from previous calculation: $P = \frac{w_u L}{2} = \frac{1.334(42)}{2} = 28.01^k$

$P_L = \frac{38.38(10)(42)}{1000(2)} = 8.06^k$

$w = 1.2D = 1.2(.692 + \frac{55}{1000}) = 1.2(.747) = .896^{k}/ft$

Typical Beam and Girder spot check | Tech Report # 2 | Page 4 of 4



$M_u = 32.49(10) + (41.45-32.49)(10)(\frac{1}{2}) + (4.48)(5)(\frac{1}{2})$

$M_u = 380.9$ ft·k

$B_{eff} = \min \begin{cases} 26 + \frac{30(12)}{8} = 71'' \\ 26 + 21(12) = 278'' \end{cases}$

$B_{eff} = 71''$

assume $a = 1''$ $\Rightarrow Y_2 = 5.5 - \frac{1}{2} = 5$

$Q_n = 21.5^k$

W18x40: $\phi M_n = 461$ ft·k
$\Sigma Q_n = 210$
# studs = 20
economy = 1400

W21x44: $\phi M_n = 516$
$\Sigma Q_n = 162$
# studs = 16
economy = 1480

W24x55: $\phi M_n = 720$
$\Sigma Q_n = 203$
# studs = 20
economy = 1850

Try W18x40

$\phi_b M_p = 294$ ft·k

* from last calc, beam had unshored load of 1.017 klf

$P = \frac{1.017(42)}{2} = 21.36^k$ $\Rightarrow M_u = 21.36(10) = 213.6$ ft·k

$\boxed{213.6 \text{ ft·k} < 294 \text{ ft·k}}$ ✓ ok for unshored strength

check Live load deflection

From previous page $\Rightarrow P_L = 8.06^k$ $I_{LB} = 1210 \text{ in}^4$

$\Delta_{LL} = \frac{Pl^3}{28EI} = \frac{8.06(30^3)(1728)}{28(29000)(1210)} = .383 \text{ in}$ $\frac{L}{360} = \frac{30(12)}{360} = 1 \text{ in}$

$\boxed{.383 < 1}$ ✓ ok

check wet conc. deflection

from previous beam calc $\Rightarrow w_{wc} = .49$ klf $\Rightarrow P_{wc} = \frac{.49(42)}{2} = 10.29^k$

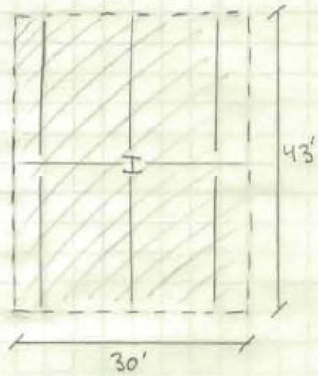
$I_{wc} = 612 \text{ in}^4$ $\Delta_{wc} = \frac{Pl^3}{28EI} = \frac{10.29(30)^3(1728)}{28(29000)(612)} = .966 \text{ in}$ $\frac{L}{240} = 1.5$

$\boxed{.966 \text{ in} < 1.5 \text{ in}}$ ✓ ok

$\boxed{\text{Use W18x40 (20)}}$

Typical column spot check | Tech Report #1 | Page 1 of 1

Analyze W14x211 at first level (A992 steel, LRFD)



$A_{TT} = 43(30) = 1290 \text{ ft}^2$

$k = 4$ (interior column)

$LL_{red} = 0.25 + \frac{15}{\sqrt{kA_{TT}}} = 0.25 + \frac{15}{\sqrt{(4)(5)(1290)}}$

$LL_{red} = 0.343 < 0.4 \Rightarrow$ use 0.4

Floor DL = 65 psf
Roof DL = 20 psf
Roof Snow = 21 psf
Floor LL = 100 psf (corridors)

$P_L = 0.4(100)(5)(1290) = 258^k$

$P_S = 21(1290) = 27.09^k$

$P_D = 20(1290) + 65(5)(1290) = 445.05^k$

$P_u = 1.2P_D + 1.6P_L + 0.5P_S = 1.2(445.05) + 1.6(258) + 0.5(27.09)$

$P_u = 960.405^k$

$A_s = 62 \text{ in}^2$
$I_x = 2660 \text{ in}^4$
$r_x = 6.55 \text{ in}$
$I_y = 1030 \text{ in}^4$
$r_y = 4.07$

Yielding: $P_y = F_y A_s = 50(62) = 3100^k$

$\phi P_y = 0.9(3100) = 2790$

$\boxed{960.4 < 2790 \checkmark \text{ ok}}$

Buckling: y axis controls

$\frac{kL}{r_y} = \frac{(12)(18)}{4.07} = 53.07$

$4.71\sqrt{E/F_y} = 4.71\sqrt{\frac{29000}{50}} = 113.43 > 53.07 \Rightarrow$ inelastic buckling

$F_e = \frac{\pi^2 E}{(\frac{kL}{r})^2} = \frac{\pi^2(29000)}{(53.07)^2} = 101.62 \text{ ksi}$

$F_{cr} = \left[0.658^{F_y/F_e}\right]F_y = \left[0.658^{(50/101.62)}\right]50 = 40.69 \text{ ksi}$

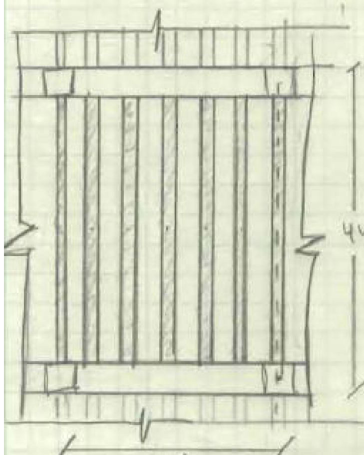
$P_n = 40.69(62) = 2523.04^k \Rightarrow \phi P_n = 0.9(2523.04) = 2271^k$

$\boxed{960.405 < 2271 \checkmark \text{ ok}}$

APPENDIX C – One Way Joist and Beam Calculations

One way skip joist and beam | Tech Report # 2 | Page 1 of 12

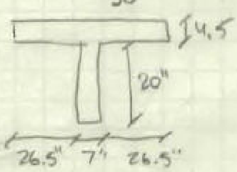
typical interior bay



Approximate as 30x40 bay

→ 53" pan
Slab thickness = 4.5"
Rib width = 7"
Beam width = 40"
Square column size = 40"
Pan depth = 20"
f'_c = 4000 psi
concrete weight = 150 pcf
* sizes estimated using "concrete floor systems" by David Fanella
SDL = 20 psf
LL = 80 psf (upper floor corridors per ASCE 7-05)

Slab self weight = 150 pcf $\left(\frac{4.5}{12}\right)$ = 56.25 psf



Area = 4.5(60) + 7(20) = 382 in² = 2.847 ft²

Joist/slab self weight = 2.847 ft² (150 pcf)
= 398 lb/ft = 85.46 psf

Beam self weight = $\frac{(24.5\text{ in})(40\text{ in})}{144}$ (150 pcf) = 1020.83 lb/ft

Floor slab design

$\boxed{4.5\text{" slab}}$ Slab self weight = 56.25 psf

$w_u = 1.2D + 1.6L \Rightarrow$ DL = 20 psf + 56.25 psf = 81.25 psf = .081 k/ft²
LL = 80 psf = .08 k/ft²

w_u = 1.2(.081) + 1.6(.08) = .2252 ksf

Analyze 12" section of slab: min reinf = .0018 A_g = .0018(4.5)(12)

A_{min} = .0972 in²/ft width ⇒ Try #3 bars @ 12" spacing

$M_u = \frac{w_u \ell_n^2}{10}$ ⇒ with 53" pan ⇒ $\ell_n = \frac{53}{12}$ = 4.42 ft

$M_u = \frac{.2252(4.42)^2}{10} = .4399 \frac{\text{ft}\cdot\text{k}}{\text{ft width}}$

#3 bars: A_s = 0.11 in²

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{.11(60)}{.85(4)(12)}$ = .162 in

One way slab joist and Beam | Tech Report #2 | Page 2 of 12

$M_n = A_s f_y (d - \frac{a}{2}) = .11(60)(2.25 - \frac{.162}{2}) = 14.31 \text{ in}\cdot k = 1.19 \text{ ft}\cdot k$

$\phi M_n = .9(1.19) = 1.071 \text{ ft}\cdot k > 0.687 \text{ ft}\cdot k$ ✓ ok

$\boxed{\text{use \#3 bars @ 12" spacing}}$

Skip Joist loading

.. 20" pan depth

53" pan size = 60" pan module = 5 ft

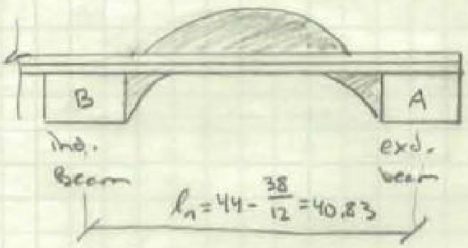
$DL = .081 \text{ ksf } (5 \text{ ft}) = .405 \text{ k/ft}$

rib size + depth $= (7)(20)(150 \text{ pcf})(\frac{\text{ft}^2}{144 \text{ in}^2}) = 145.8 \text{ lb/ft}$

Total $DL = .405 + .146 = .551 \text{ k/ft}$

Total $LL = 80 \text{ psf } (5 \text{ ft}) = 400 \text{ plf} = .4 \text{ klf}$

$w_u = 1.2D + 1.6L = 1.2(.551) + 1.6(.4) = 1.301 \text{ k/ft}$



$M_{uA}^- = \frac{w_u l_n^2}{10} = \frac{1.3(40.83)^2}{10} = 216.72 \text{ ft}\cdot k$

$M_{uB}^- = \frac{w_u l_n^2}{11} = \frac{1.3(40.83)^2}{11} = 197.02 \text{ ft}\cdot k$

$M_u^+ = \frac{w_u l_n^2}{14} = \frac{1.3(40.83)^2}{14} = 154.8 \text{ ft}\cdot k$

reinf depth: $d = (20+4.5) - 2.25 = 22.25 \text{ in}$

Top reinf: $A_s = \frac{M_u}{4d} = \frac{154.8}{4(22.25)} = 1.72 \text{ in}^2 \Rightarrow$ try 4 #6 bars ($A_s = 1.76$)

check if tension controlled: $\rho = \frac{A_s}{bd} = \frac{1.76}{(7)(22.25)} = .0113$

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{1.76(60)}{.85(4)(7)} = 4.44 \text{ in}$ $\quad c = \frac{a}{\beta_1} = \frac{4.44}{.85} = 5.22 \text{ in}$

$\varepsilon_s = \frac{(22.25 - 5.22)}{5.22}(.003) = .0097 > .005 \Rightarrow$ Tension controlled

$\phi M_n = \phi A_s f_y (d - \frac{a}{2}) = .9(1.76)(22.25 - \frac{4.44}{2})(60) = 1903.65 \text{ in}\cdot k$

$\phi M_n = 158.64 \text{ ft}\cdot k > 154.8 \text{ ft}\cdot k$ ✓ ok

$\boxed{\text{use 4 \#6 top bars}}$

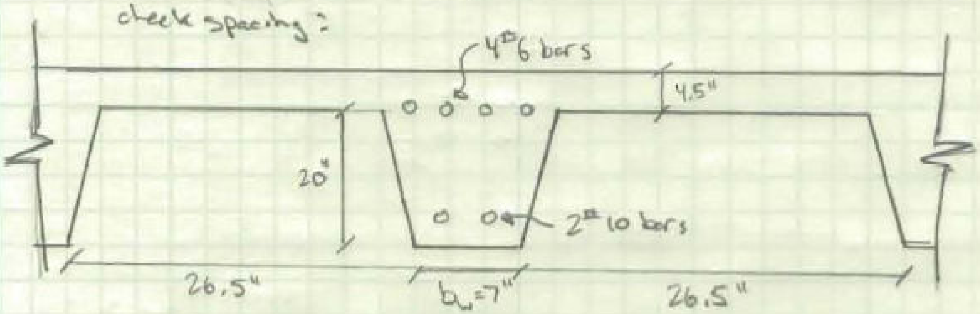
One way Skip Joist and Beam | Tech Report # 2 | Page 3 of 12

Skip joist bottom reinf. $\Rightarrow d = 24.5 - 1.5 - .375 -$

$A_s = \frac{M_u}{4d} = \frac{216.72}{4(22.25)} = 2.44 \text{ in}^2 \Rightarrow$ Try 2 #10 bars, $A_s = 2.54$

$d = 24.5 - 1.5 - .375 - \frac{1.27}{2} = 21.99 \approx 22 \text{ in}$

check spacing?



Treat as T-beam?

$$b_{eff} = \min \begin{vmatrix} b_w + \text{clear span} \\ b_w + 16h_f \\ b_w + \ell/4 \end{vmatrix} = \min \begin{vmatrix} 7 + 60 \\ 7 + 16(4.5) \\ 7 + \frac{40.83(12)}{4} \end{vmatrix}$$

$$= \min \begin{vmatrix} 67 \\ 79 \\ 130 \end{vmatrix} \Rightarrow 67 \text{ controls} \Rightarrow b_{eff} = 67 \text{ in}$$

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{2.54(60)}{.85(4)(67)} = .669 \text{ in}$ $\quad c = \frac{a}{\beta_1} = \frac{.669}{.85} = .787$

check that $a \le h_f \Rightarrow .669 < 4.5$ ✓ ok

$A_{s\,min} = \frac{200 b_w d}{f_y} = \frac{200(7)(22)}{60000} = .513 < 2.54$ ✓ ok

$\varepsilon_s = \frac{.003}{.787}(22 - .787) = .081 > .005 \Longrightarrow$ tension controlled

$\phi M_n = 0.9 A_s f_y (d - \frac{a}{2}) = 0.9(2.54)(60)(22 - \frac{.669}{2}) = 2971.64 \text{ in-k}$

$\phi M_n = 247.64 \text{ ft-k} > 216.72$ ✓ ok

$\boxed{\text{use 2 \# 10 bottom bars}}$

$$V_u = \frac{w_u l_n}{2} = \frac{1.3(40.83)}{2} = 26.54^k$$

$$\phi V_c = \phi 2\sqrt{f'_c}\, b_w d = \frac{0.75(2)\sqrt{4000}\,(7)(22)}{1000} = 14.61^k$$

$$\phi V_s = V_u - \phi V_c = 26.54 - 14.61 = 11.93^k$$

$$\phi V_s = 11.93 = \frac{\phi A_{sv} f_y d}{S_{max}} \quad \text{where } S_{max} = \min \left| \begin{array}{l} d/2 = \frac{22}{2} = 11 \\ 24 \end{array} \right. = 11$$

reinf spacing: $\frac{d}{2} = \frac{22.25}{2} = 11.125 \Rightarrow$ try 10" spacing $\Rightarrow A_{v,min} = \frac{.75(2)\sqrt{4000}\,(7)(10)}{60000}$

$A_{v,min} = .1106 > .11 \Rightarrow$ No good, use 8" spacing

$$11.93 = \frac{\phi A_{sv} f_y d}{S_{max}} = \frac{.75 A_{sv}(60)(22)}{8} \Rightarrow A_{sv} = .096\,in^2$$

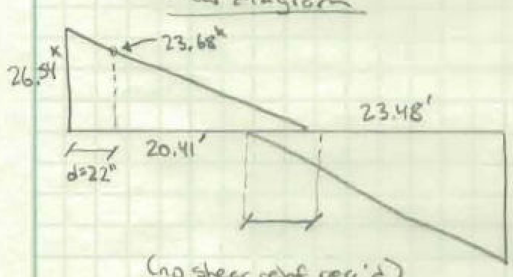
$\boxed{\text{Use \#3 stirrups at 8" spacing}}$

check that $\phi(V_c + V_s) \geq V_u$

$$\phi V_c + \phi V_s = 14.61 + \frac{.75(.11)(60)(22)}{6} = 32.76 > 26.54 \quad ✓ \text{ ok}$$

Shear at face of support $= 1.15 \frac{w_u l_n}{2} = 1.15 \frac{(1.3)(40.83)}{2} = 30.52^k$

Shear diagram



Design shear a length "d" from support:

$$V_u = 26.54 - 1.3\left(\frac{22}{12}\right) = 23.68$$

reinforcing not required:

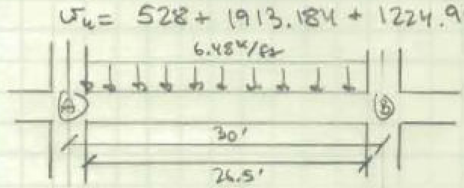
$$\phi V_n = 0.5\phi V_c = \frac{14.61}{2} = 7.305$$

$$\frac{7.305}{1.3} = 5.62' \text{ at center of joist}$$

40" Edge Beam check

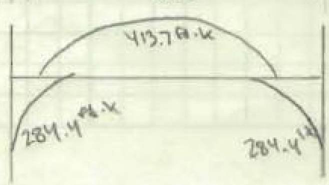
$$w_u = 1.2(20)(22) + 1.2(85.41)\left(22 - \frac{40}{12}\right) + 1.2(1020.83) + 1.6(80)(22)$$

$$w_u = 528 + 1913.184 + 1224.99 + 2816 = 6482.174 = 6.48^k/ft$$



$$M_{uA}^- = \frac{w_u l_n^2}{16} = \frac{6.48(26.5)^2}{16} = 284.4\ ft\cdot k$$

$$M_{uB}^- = \frac{w_u l_n^2}{16} = 284.4\ ft\cdot k$$



$$M_u^+ = \frac{w_u l_n^2}{11} = 413.7\ ft\cdot k$$

Top reinf. at interior support

$A_s = \frac{M_u}{4d} = \frac{284.4}{4(22)} = 3.23 \text{ in}^2$ Try 8 #6 bars ($A_s = 3.52$)
use bar yield strength $f_y = 60$ ksi

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{(3.52)(60)}{.85(4)(40)} = 1.55$

$c = \frac{a}{\beta_1} = \frac{1.55}{0.85} = 1.82 \text{ in}$ $\Rightarrow$ $.375d = .375(22) = 8.25$

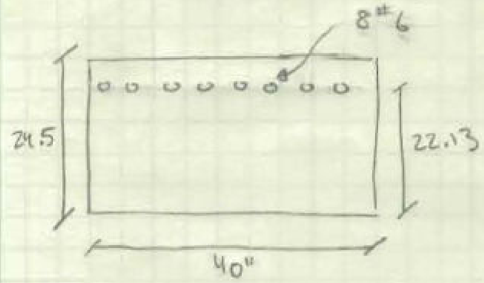
1.82 in < 8.25 in therefore => tension controlled, $\varepsilon_t = 0.005$, ϕ 0.9

$d = 24.5 - 1.5 - .375 - \frac{1}{2} = 22.13 \text{ in}$

$\phi M_n = A_s f_y (d - \frac{a}{2}) = 0.9(3.52)(60)(22.13 - \frac{1.82}{2}) = 4033.5$ in·k

$\phi M_n = 336.12$ ft·k

check that $\phi M_n > M_u$ $\Rightarrow$ $336.12 > 284.4$ ✓ ok



$\boxed{\text{Use 8 \#6 top bars}}$

$d = 2(2) + 8(.75) + 7(2) = 24 < b = 40"$ ✓ ok

Bottom Reinf. at midspan

$\frac{M_u}{4d} = \frac{413.7}{4(22)} = 4.70 \text{ in}^2$ Try 12 #6 bars : $A_s = 5.28 \text{ in}^2$
spacing: $2(2) + 12(.75) + 11(2) = 35 < 40$ ✓ ok

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{5.28(60)}{.85(4)(40)} = 2.33 \text{ in}$ $c = \frac{a}{\beta_1} = \frac{2.33}{.85} = 2.74$

$.375d = .375(22) = 8.25 > 2.74$ Therefore => tension controlled

$d = 24.5 - 1.5 - .375 - \frac{1.75}{2} = 22.25 \text{ in}$

$\phi M_n = A_s f_y (d - \frac{a}{2}) = .9(5.28)(60)(22.25 - \frac{2.33}{2}) = 6011.76$ in·k

$\phi M_n = 500.98$ ft·k > 413.7 ft·k ✓ ok

$\boxed{\text{use 12 \#6 bottom bars}}$

Bar detailing

Top reinf at interior support

$L_d = 62(d_b) = 62(.75) = 46.5$ in

1/3 of reinf (8 bars) = 3 bars with embed length = min $\begin{vmatrix} d \\ 12d_b = \\ L_n/16 \end{vmatrix}$ min $\begin{vmatrix} 22.25 \\ 19 \\ 19.88 \end{vmatrix}$

cutoff to 3 #6 bars ($A_s = 1.32$ in^2)

$a = \dfrac{1.32(60)}{0.85(4)(40)} = .582$ in

$\phi M_n = 0.9(1.32)(60)(22.25 - \frac{.582}{2}) = 1565.24$ in·k $= 130.43$ ft·k

$-130.4 = w_u(\frac{x}{2})(L-x) - M_1 - \frac{(M_2-M_1)}{L_n} x = 6.48(\frac{x}{2})(26.5-x) - 284 - \frac{(284-284)}{26.5} x$

$-130.4 = 85.86x - 3.24x^2 - 284 - 0$

$3.24x^2 - 85.86x + 153.6 = 0 \Rightarrow \dfrac{85.86 \pm \sqrt{85.86^2 - 4(3.24)(153.6)}}{2(3.24)}$

$\dfrac{85.86 \pm 73.36}{6.48} = 13.25 \pm 11.32 = 1.93 \Rightarrow 24'' + 22.25'' = 46.25 < 62''$

$\boxed{\text{cutoff at } 5'2'' \text{ from column face}}$

cutoff to 0 #8 bars

$3.24x^2 - 85.86x + 284 = 0 \rightarrow \dfrac{85.86 \pm \sqrt{85.86^2 - 4(3.24)(284)}}{2(3.24)}$

$x = \dfrac{85.86 \pm 60.76}{6.48} = 13.25 \pm 9.38 = 3.874'$

$\max \begin{vmatrix} 3.874 + \frac{22.25}{12} \\ 1.32 + \frac{46.5}{12} \end{vmatrix} = \max \begin{vmatrix} 5.73 \\ 5.20 \end{vmatrix} = 5.73 = 5'9''$

$\boxed{\text{cutoff at } 5'9'' \text{ from column face}}$

Bottom Reinforcement detailing

$L_d = 47.4(.75) = 35.6$

1/4(12) = 3 bars extend 6" into support

cutoff to (3) #6, $A_s = 1.32$ in^2

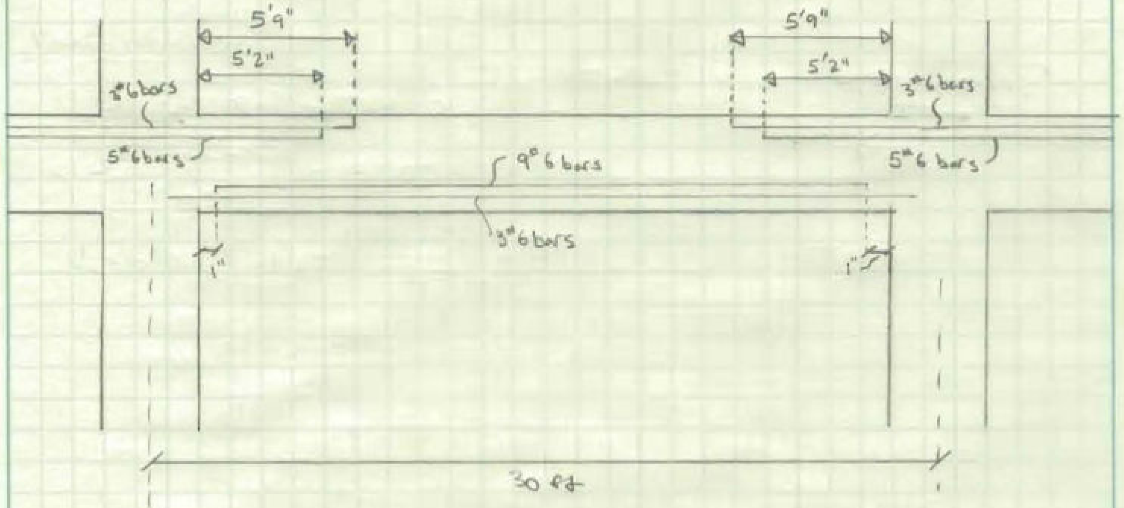
$a = \dfrac{1.32(60)}{.85(4)(40)} = .582$ in $\quad \phi M_n = 130.4$ ft·k (see above calcs)

$3.24x^2 - 85.86x + 153.6 = 0 \Rightarrow x = 13.25 \pm 11.32 = 24.57, 1.93$

from interior support

cutoff at $1.93 - \frac{22.25}{12} = .076$ $\boxed{= 0'1'' \text{ from face of interior support}}$

$26.5 - (24.57 + \frac{22.25}{12}) = .076$ $\boxed{= 0'1'' \text{ from ext. col. face}}$



Shear reinforcing for girder

V_u at face of first interior support $= 1.15 \frac{w_u l_n}{2} = 1.15 \frac{(6.48)(26.5)}{2} = 98.74^k$

V_u at all other supports $= \frac{w_u l_n}{2} = \frac{6.48(26.5)}{2} = 85.86^k$

V_u at "d" from face $\Rightarrow 85.86 - 6.48^k/ft \left(\frac{22.25}{12}\right) = 73.85^k$

Shear reinf req'd? $\Rightarrow \phi V_c = .75(2)\sqrt{f'_c}\, b_w d = .75(2)\sqrt{4000}(40)(22.25)$

$\phi V_c = 84.43^k \Rightarrow \phi V_n = \frac{1}{2}\phi V_c = 42.22^k$

Reinf. not required within $\frac{42.22^k}{6.48^k/ft} = 6.515$ ft at center of beam

Spacing $= \frac{d}{2} = \frac{22.25}{2} = 11.13 \Rightarrow$ try 10" spacing

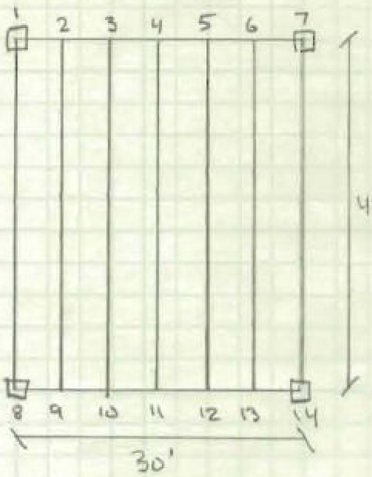
$A_{v,min} = \frac{0.75\sqrt{4000}(40)(10)}{60000} = .316\ in^2 > .22 \Rightarrow$ No good

$\frac{.22}{.316}(10) = 6.96 \Rightarrow$ try 6" spacing

$A_{v,min} = \frac{0.75\sqrt{4000}(40)(6)}{60000} = .1897\ in^2 < .22$

$\boxed{\text{Use 2 legs, \#3 stirrups, 6'' spacing}}$

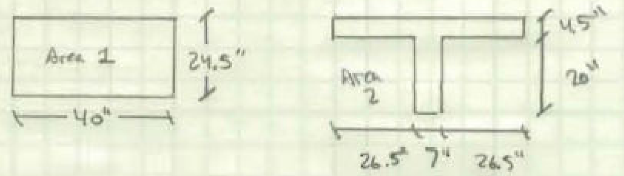
Torsional Analysis



$E = 3600$ ksi

$G = 1565$ ksi (assume $\nu = 0.15$)

$K_T = \frac{1}{3}\sum x^3 y\left(1 - 0.63\left(\frac{x}{y}\right)\right)$



$K_{T1} = \frac{1}{3}(24.5)^3(40)\left(1 - .63\left(\frac{24.5}{40}\right)\right) = 120419 \text{ in}^4$

$K_{T2} = \frac{1}{3}(7^3)(20)\left(1 - .63\left(\frac{7}{20}\right)\right) + \frac{1}{3}(4.5)^3(60)\left(1 - .63\left(\frac{4.5}{60}\right)\right) = 3519 \text{ in}^4$

$I_1 = \frac{bh^3}{12} = \frac{40(24.5)^3}{12} = 49021 \text{ in}^4$

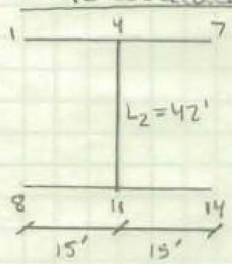
$\bar{Y} = \frac{\sum Ad}{\sum A} = \frac{7(20)(10) + (4.5)(60)(22.25)}{(7)(20) + (4.5)(60)} = 18.07 \text{ in}$

$I_2 = \frac{bh^3}{12} + Ad^2 = \frac{7(20)^3}{12} + (7)(20)(18.07-10)^2 + \frac{60(4.5)^3}{12} + (4.5)(60)(22.25-18.07)^2$

$I_2 = 4666.67 + 9117.5 + 455.63 + 4717.55 = 18958 \text{ in}^4$

Slope deflection equation for joist 4-11

$w_u = .2252 \text{ ksf}(5.00) = 1.13 \text{ k/ft}$



Joint 4: $\left(\frac{4EI_2}{L_2} + \frac{2GK_{T1}}{L_{8-11}}\right)\theta_{4-11} + \frac{2EI_2}{L_2}\theta_{11-4} - \frac{w_u L_2^2}{12} = 0$

$\left[\frac{4(3600)(18958)}{42(12)} + \frac{2(1565)(120419)}{15(12)}\right]\theta_{4-11} + \left[\frac{2(3600)(18958)}{42(12)}\right] - \frac{(1.13)(42)^2}{12} \cdot 12 = 0$

$2635610\,\theta_{4-11} + 270829\,\theta_{11-4} = 1993.32$

Joint 11: $2635610\,\theta_{11-4} + 270829\,\theta_{4-11} = -1993.32$ (because same girder size)

$\theta_{4-11}(2364781) + \theta_{11-4}(-2364781) = 3986.64$

$\theta_{4-11} = \theta_{11-4} + \frac{3986.64}{2364781} = \theta_{11-4} + .00169$

$2635610(\theta_{11-4} + .00169) + 270829\,\theta_{11-4} = 1993.32$

$2635610\,\theta_{11-4} + 270829\,\theta_{11-4} = 1993.32 - 4454.18 = -2460.86$

$\theta_{11-4} = -8.467 \times 10^{-4}$ rad one way slab joist and beam | Tech Report # 2 | Page 9 of 12

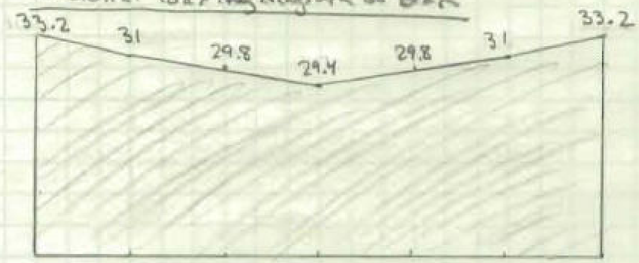
$\Theta_{4-11} = -8.467\times10^{-4} + .00169 = 8.467\times10^{-4}$

$M_{4-11} = \frac{4EI_2}{L_2}\Theta_{4-11} + \frac{2EI_2}{L_2}\Theta_{11-4} - \frac{w_u L^2}{12}$

$M_{4-11} = \frac{4(3600)(18958)}{(42\times12)}(8.467\times10^{-4}) + \frac{2(3600)(18958)}{(42)(12)}(-8.467\times10^{-4}) - (1.13)(42)^2$

$M_{4-11} = 458.621 - 229.31 - 1993.32 = -1764$ in.k $= -147$ ft-k

Torsional loading diagram of beam

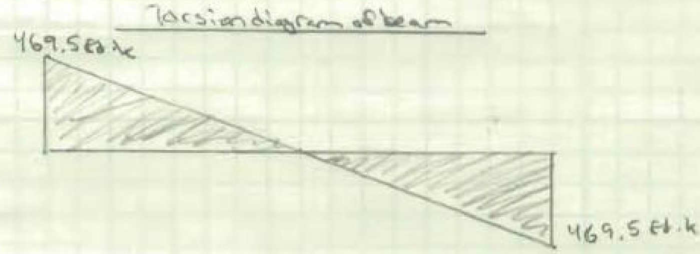


* Approximate Area as two triangles and a rectangle.

$A = 2(\tfrac{1}{2})(15)(33.2-29.4) + (29.4)(30)$

$A = 939 \Rightarrow \frac{939}{2} = \boxed{469.5\text{ ft-k}}$

Torsion diagram of beam



Design beam for full factored torque:

Shear at distance "d" from support: $\frac{40}{2} + 22.25 = 42.25$

$V_u = 85.86 - 6.48\left(\frac{42.25}{12}\right) = 63.05^k$

Torque at distance "d" from support: $T_u = 469.5 - \frac{42.25}{12}\left(\frac{33.2}{3}\right) = 430.54^{\text{ k-ft}}$

Threshold torsion: $A_{cp} = (40)(24.5) = 980\text{ in}^2$
$P_{cp} = (2)(40+24.5) = 129\text{ in}$

$\phi\sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right) = 0.75\sqrt{4000}\left(\frac{980^2}{129}\right) = 29.43\text{ ft-k} < 430.54\text{ ft-k}$

$\Rightarrow$ therefore, we must design for torsion.

check maximum torsional strength

$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u P_h}{1.7A_{oh}^2}\right)^2} \le \phi\left(\frac{V_c}{b_w d} + 8\sqrt{f'_c}\right)$

* use #4 stirrups

$P_h = 2(40-2(1.5)-2(.25)) + 2(24.5-2(1.5)-2(.25)) = 115 \text{ in}$

$A_{oh} = (40-3.5)(24.5-3.5) = 766.5 \text{ in}^2$

$V_c = 2\sqrt{4000}(40)(24.5) = 123.96^k$

$$\sqrt{\left(\frac{V_u}{b_w d}\right)^2 + \left(\frac{T_u P_h}{1.7 A_{oh}^2}\right)^2} = \sqrt{\left(\frac{63.05}{40(24.5)}\right)^2 + \left(\frac{430.54(115)(12)}{1.7(766.5)^2}\right)^2} = .5983 \text{ k/in}^2$$

$\phi(2\sqrt{f'_c} + 8\sqrt{f'_c}) = \phi(10\sqrt{f'_c}) = .75(10)\sqrt{4000} = .4743 \text{ k/in}^2$

$.5983 > .4743 \Rightarrow$ Beam not sufficiently large, must use reduced torsion.

Reduced torsion $= 4 \times$ threshold torque $= 4(29.43) = 117.72^k$

Recheck: $\sqrt{.0643^2 + \left(\frac{117.72(115)(12)}{1.7(766.5)^2}\right)^2} = \sqrt{.0643^2 + .1626^2} = .1749 \text{ k/in}^2$

★ $\boxed{.1749 < .4743 \checkmark \text{ ok} \Rightarrow \text{design beam for } 117.72^k \text{ torque}}$

Area of stirrups required to resist T_u:

$T_n = \frac{T_u}{.75} = \frac{430.54}{.75} = 574.05 \text{ ft·k}$ $\qquad \frac{A_T}{s} \geq \frac{T_n}{2 A_o f_y \cot\theta}$ $\qquad$ * Assume $\theta = 45°$

$A_o = 0.85 A_{oh} = 651.53 \text{ in}^2$ $\qquad \frac{A_T}{s} \geq \frac{574.05}{2(651.53)(60)(1)} = .0073 \frac{\text{in}^2}{\text{in}}$

minimum required: $\frac{50 b_w}{f_y} = \frac{50(40)}{60000} = .033 \frac{\text{in}^2}{\text{in}} > .0073 \Rightarrow$ controls

$$S_{max} \leq \begin{cases} 12 \text{ in} = 12 \text{ in} \\ P_h/8 = 14.38 \\ d/2 = 11.13 \end{cases}$$

for #3 stirrups: $s < \frac{2(.11)}{.033} = 6.67 \text{ in}$

for #4 stirrups: $s < \frac{2(.2)}{.033} = 12.12 \text{ in}$

$\boxed{\text{use \#3 stirrups at 6 in spacing}}$

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Longitudinal Reinf: $A_L = \left(\frac{A_t}{s}\right)(P_h)\left(\frac{f_{yt}}{f_{yl}}\right)\cot^2\theta \Rightarrow$ try $\theta = 30$

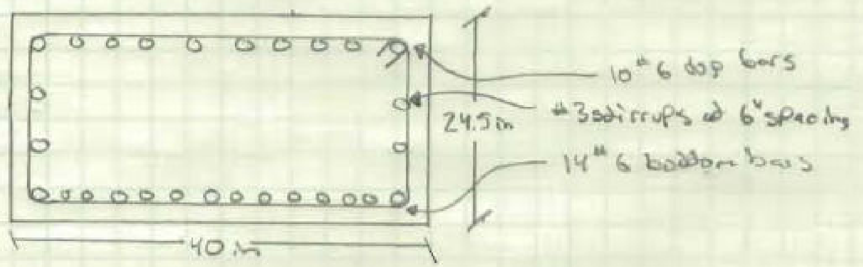
$A_L = .0073(115)(1)(3) = 2.52 \text{ in}^2$ in addition to flexural steel

$$A_{L,min} = \frac{5\sqrt{f'c}\,A_{cp}}{f_{yl}} - \frac{A_t}{s}P_h\left(\frac{f_{yt}}{f_{yl}}\right) = \frac{5\sqrt{4000}\,(980)}{60000} - \left(\frac{.11}{6}\right)(115)(1)$$

$A_{L,min} = 3.05 \text{ in}^2 \Rightarrow$ controls $\qquad \frac{3.05}{.44} = 6.93 \Rightarrow$ add 8 additional #6 bars

Spacing $\Rightarrow 24.5 - 3.05(2) = 18.4 > 12 \Rightarrow$ middle bars are required

* add 2 bars on top, 2 on bottom and 2 on each side



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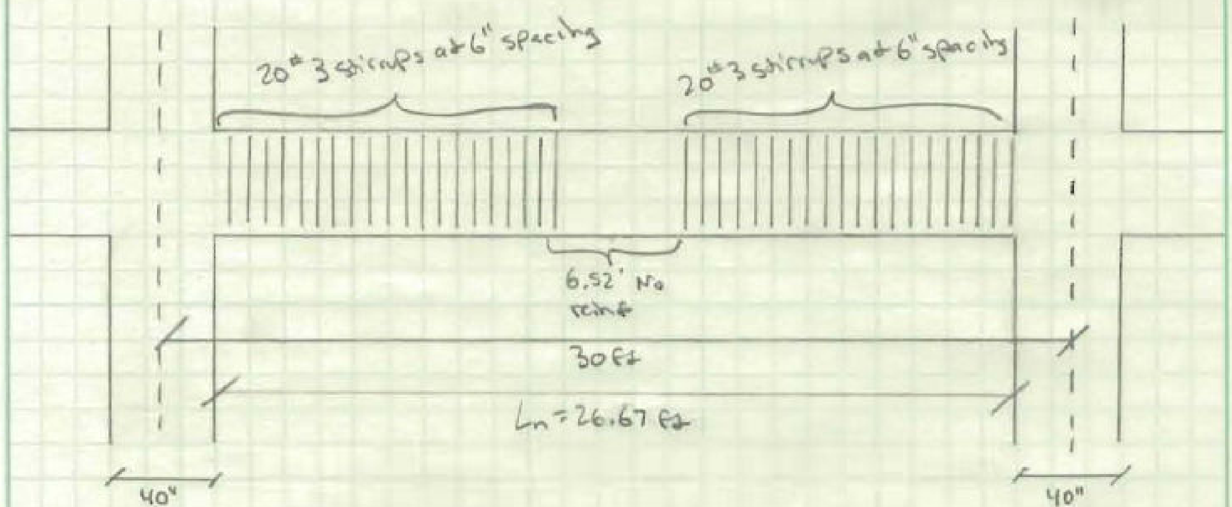


Slope deflection calculations for Torsionally loaded edge girder

E=	3600 ksi	a= $(4EI_2/L_2) + (GKT_1/L_{1a}) + (GKT_1/L_{1b})$
G=	1565 ksi	b= $(2EI_2/L_2)$
KT_1=	120419 in^4	c= $(2wuL_2^2)$
KT_2=	3519 in^4	$\Theta_{1-8} = (c/2 - (ac/a-b))/(a+b) + (c/(a-b))$
I_1=	49021 in^4	$\Theta_{8-1} = (c/2 - (ac/a-b))/(a+b)$
I_2=	18958 in^4	

Joint 1	L_2	42	**Joint 3**	L_2	42	**Joint 5**	L_2	42	**Joint 7**	L_2	42
	L_{1a}	0.01		L_{1a}	10		L_{1a}	20		L_{1a}	29.99
	L_{1b}	29.99		L_{1b}	20		L_{1b}	10		L_{1b}	0.01
	a	157152977		a	2897354		a	2897354		a	157152977
	b	270829		b	270829		b	270829		b	270829
	c	3987		c	3987		c	3987		c	3987
	Θ_{1-8}	0.000001		Θ_{3-10}	0.000759		Θ_{5-12}	0.000759		Θ_{7-14}	0.000001
	Θ_{8-1}	-0.000001		Θ_{10-3}	-0.000759		Θ_{12-5}	-0.000759		Θ_{14-7}	-0.000001
	M_{1-8}	**-166.08**		M_{3-10}	**-148.98**		M_{5-12}	**-148.98**		M_{7-14}	**-166.08**
Joint 2	L_2	42	**Joint 4**	L_2	42	**Joint 6**	L_2	42			
	L_{1a}	5		L_{1a}	15		L_{1a}	25			
	L_{1b}	25		L_{1b}	15		L_{1b}	5			
	a	4310772		a	2635610		a	4310772			
	b	270829		b	270829		b	270829			
	c	3987		c	3987		c	3987			
	Θ_{2-9}	0.000493		Θ_{4-11}	0.000843		Θ_{6-13}	0.000493			
	Θ_{9-2}	-0.000493		Θ_{11-4}	-0.000843		Θ_{13-6}	-0.000493			
	M_{2-9}	**-154.97**		M_{4-11}	**-147.09**		M_{6-13}	**-154.97**			

Joint	Bending in rib at exterior support (Ft.-	Torsional loading on edge girder (Ft.-k/Ft.)
1	-166.08	-33.2163
2	-154.97	-30.9949
3	-148.98	-29.7964
4	-147.09	-29.4172
5	-148.98	-29.7964
6	-154.97	-30.9949
7	-166.08	-33.2163

APPENDIX D – Hollow Core Plank Calculations

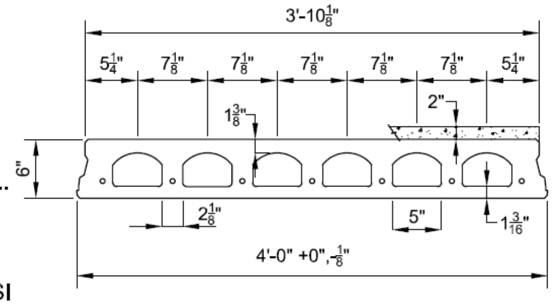
Prestressed Concrete
6"x4'-0" Hollow Core Plank

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

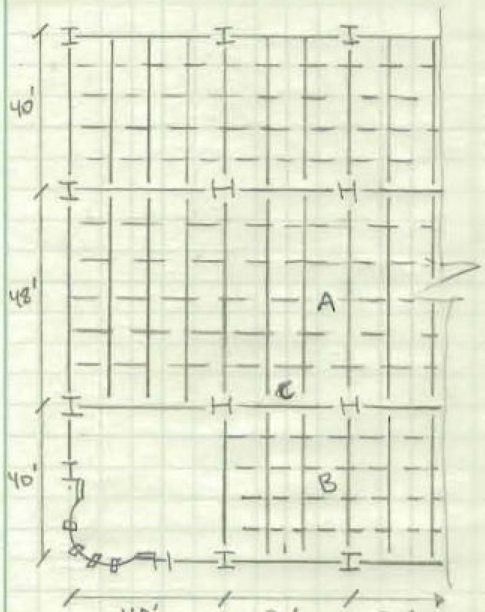
This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads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

11/03/08

6F2.0T

Figure D-1: Table courtesy of Nitterhouse Concrete Products

Precast, Prestressed Hollow core plank	Tech Report #2	Page 1 of 3



Try a 6" x 4'0" hollow core plank with 2" topping

- self weight = 48.75 psf

Loads: SDL = 20 psf
plank self weight = 48.75 psf
LL = 80 psf (corridors)

service = 1.2(20) + 1.6(80) = 152 psf

w_u = 1.2 (68.75) + (1.6)(80) = 210.5 psf

select Designation 6 1/2" φ strands

152 psf < 215 psf (for 30 ft span)

ϕM_n = 92.6 ft·k

Check beam and girder design

- Allowed deflections ⇒ $\Delta_{LL} = L/480$, $\Delta_{TL} = L/240$ (ACI 318.08 Table 9.5 b)

Beam B design

Span = 48'
Tributary width = 10'

$$\Delta_{LL} = \frac{5wL^4}{384EI} = \frac{5(.08\times10)(48)^4(1728)}{384(29000)I_{req}} = \frac{L}{480} = \frac{48(12)}{480} = 1.2$$

$I_{req} = 2746 \text{ in}^4$

$$\Delta_{TL} = \frac{5wL^4}{384EI} = \frac{5(.2105)(10)(48)^4(1728)}{384(29000)I_{req}} = \frac{L}{240} = \frac{48(12)}{240} = 2.4$$

$I_{req} = 3612 \text{ in}^4$ ⇐ controls

$$M_u = \frac{.2105(10)(48)^2}{8} = 606.24 \text{ ft·k}$$

$\boxed{\text{use W30x99} \Rightarrow I_x = 3990 \text{ in}^4 > 3612 \text{ in}^4,\ M_u = 1170 \text{ ft·k} > 606.24 \text{ ft·k}}$

- as opposed to W21x44 in original design

Precast, Prestressed Hollow core plank | Tech Report # 2 | Page 2 of 3

Beam B design

Span = 40'
tributary width = 10'

$$\Delta_{LL} = \frac{5wL^4}{384EI} = \frac{5(.8)(40)^4(1728)}{384(29000)(I_{req})} = \frac{L}{480} = \frac{40(12)}{480} = 1$$

$$I_{req} = 1589 \text{ in}^4$$

controls

$$\Delta_{TL} = \frac{5wL^4}{384EI} = \frac{5(.2105)(10)(40)^4(1728)}{384(29000)(I_{req})} = \frac{L}{240} = 2 \quad I_{req} = 2091 \text{ in}^4$$

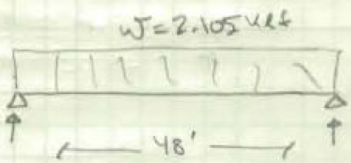
$$M_u = \frac{.2105(10)(40)^2}{8} = 421 \text{ ft-k}$$

$$\boxed{\text{Use W24x76} \Rightarrow I_x = 2100 \text{ in}^4 > 2091 \text{ in}^4 \quad M_u = 750 \text{ ft-k} > 421 \text{ ft-k}}$$

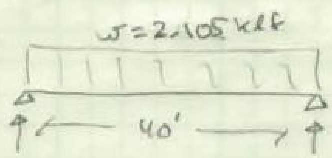
- as opposed to W18x40 in origional design

* note: A serviceability deflection limit of 1 in would most likely be used by the designers as opposed to the deflection limits allowed by ACI 318.08 which would further increase the beam sizes.

Beam A



Beam B



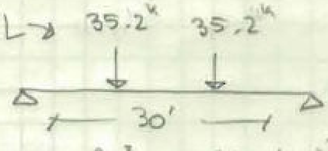
$$TL \Rightarrow \frac{2.105(48)}{2} = 50.52^k \qquad TL \Rightarrow \frac{2.105(40)}{2} = 42.1^k$$

$$LL \Rightarrow \frac{0.8(48)}{2} = 19.2^k \qquad LL \Rightarrow \frac{0.8(40)}{2} = 16^k$$

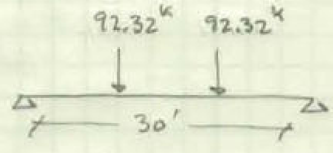
Girder C design

For TL ⇒ $P = 50.22 + 42.1 = 92.32^k$
For LL ⇒ $P = 19.2 + 16 = 35.2^k$

LL →



TL →



$$\Delta_{LL} = \frac{PL^3}{28EI} = \frac{35.2(30)^3(1728)}{28(29000)I_{req}} = \frac{L}{480} = \frac{30(12)}{480} = .75 \Rightarrow I_{req} = 2697 \text{ in}^4$$

Precast, Prestressed Hollow Core plank	Tech Report # 2	Page 3 of 3

$$\Delta_{TL} = \frac{PL^3}{28EI} = \frac{92.32(30)^3(1728)}{28(29000)I_{req}} = \frac{30(12)}{240} = 1.5\text{ in} \qquad I_{req} = 3536.4\text{ in}^4$$

$$M_u = Pa = 92.32(10) = 923.2\text{ ft-k}$$

$$\boxed{\text{use } W30\times90 \Rightarrow I_x = 3610\text{ in}^4 > 3536.4\text{ in}^4 \quad M_u = 1060\text{ ft-k} > 923.2\text{ ft-k}}$$

- as opposed to W27x146 in original design

Result: floor slab thickness increases by 2.5 inches
(8 - 5.5) = 2.5 in

Beam depth increases by 9 inches
(30 - 21) = 9 in

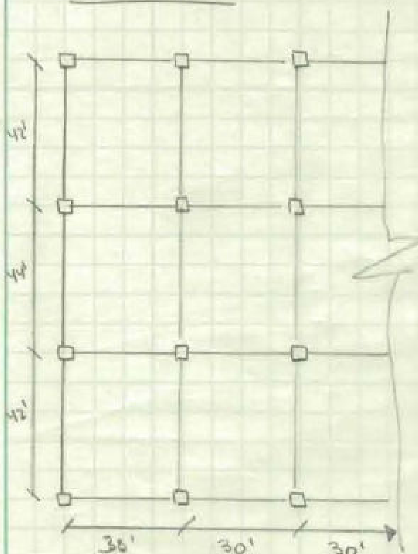
Re-estimation of floor weight: hollow core slab = 48.75 psf
Adjusted framing weight = 14.65 psf

Adjusted floor weight = 63.4 psf

APPENDIX E – Two Way Post-tensioned Slab Calculations

Two way Post tensioned slab | Tech Report # 2 | Page 1 of 7

Grid layout



Loads and Materials

Framing DL = self weight
SDL = 20 psf
LL = 80 psf (corridors)

$LL_{red} = L_o\left(.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) =$

$K_{LL} = 1.0$ $A_T = 30 \times 44 = 1320 ft^2$

$LL = 80\left(.25 + \frac{15}{\sqrt{1(1320)}}\right) = .663(80) = 53 psf$

use NW conc: 150 psf

$f'_c = 5000 psi$ $f'_{ci} = 3000 psi$
$f_y = 60 ksi$

⇒ use unbonded tendons: 1/2" ϕ 7 wire strands

$A = .153 in^2$ $f_{pu} = 270 ksi$

Estimated prestress loss = 15 ksi
$f_{se} = 0.7f_{pu} - loss = 0.7(270) - 15 = 174 ksi$
$P_{eff} = Af_{se} = .153(174) = 26.6 ksi$

Determine preliminary slab thickness

Assume $L/h = 45$ ⇒ max span = 44 ft

$h = L/45 = 44/45 = .977 ft = 11.73 in$ ⇒ $\boxed{\text{Try 12" slab}}$

Slab self weight = 12"(150 pcf) = $\boxed{150 psf}$

* per 18.3.3, A prestressed two way slab system shall be designed as class U with $f_t \leq 6\sqrt{f'c}$

Design of East-West Interior Frame

Span = 30 ft
width = 44 ft

$A = bh = (44)(12)(12) = 6336 in^2$

$S = \frac{bh^2}{6} = \frac{(44)(12)(12)^2}{6} = 12672 in^3$

Design parameters

Stresses in conc immediately after prestress transfer:
$f'_{ci} = 3000 psi$
compression = $0.6 f'_{ci} = 1800 psi$
Tensile = $3\sqrt{f'_{ci}} = 3\sqrt{3000} = 164.32 psi$

Two way Post-tensioned slab	Tech Report # 2	Page 2 of 7

Stresses in conc at service loads:

$f'_c = 5000$ psi

Compression $= 0.45f'_c = 0.45(5000) = 2250$ psi

Tension $= 6\sqrt{f'_c} = 6\sqrt{5000} = 424.3$ psi

Average precompression limits: $P/A = 125$ psi (min)
300 psi (max) ACI 18.12.4

Target load balances: 60% of Dead Load (self weight)

$$0.6(150) = 90 \text{ psf}$$

cover requirements to meet 2hr fire rating:

Restrained slabs $\Rightarrow$ 0.75 in

Unrestrained slabs $\Rightarrow$ 1.50 in (bottom)
0.75 in (top)

Tendon profile

C.G. locations: (from bottom of slab)	
Exterior support:	4 in
Interior support:	7 in
Interior span, bottom:	1 in
End span, bottom:	1.75 in

$a_{int} = 7 - 1 = 6$ in $\qquad a_{end} = \frac{(4+7)}{2} - 1.75 = 3.75$ in

Prestress force required to balance 60% self weight

$w_b = .6(w_{DL}) = .6(150)(44) = 3960$ plf $= 3.96$ klf

force needed to counteract load in end bay:

$$P = \frac{w_b L^2}{8a_{int}} = \frac{3.96(38)^2(12)}{8(3.75)} = 2287.3^k$$

$$\# \text{ of tendons} = \frac{2287.3}{26.6} = 85.99 \Rightarrow \text{use } 86 \text{ tendons}$$

$P_{actual} = (\#\text{tendons})(P_{eff}) = (86)(26.6) = 2287.6^k$

Adjusted load: $\frac{P_{act}}{P} = \frac{2287.6}{2287.3}(3.96) = 3.96$ klf

Actual precompression stress: $\frac{P_{actual}}{A} = \frac{2287.6(1000)}{6336} = 361.4^k$

$$\boxed{361.4^{ksi} > 300^{ksi} \text{ No Good}}$$

38' x 44' Bay size is too large

Recheck for 30x44 exterior bay (assume 8' extra is cantelivered)

$A = bh = 44(12)(12) = 6336 \text{ in}^2$

$w_b = .6(w_{DL}) = .6(150)(44) = 3960 \text{ plf} = 3.96 \text{ klf}$

Force needed to counteract load in end bay:

$$P = \frac{w_b L^2}{8 a_{end}} = \frac{3.96(30)^2(12)}{8(3.75)} = 1425.6^k$$

$$\# \text{ of tendons} = \frac{1425.6^k}{26.6} = 53.59 = 54 \text{ tendons}$$

$P_{actual} = 26.6(54) = 1436.4^{k} \Rightarrow$ Adjusted load $= \frac{P_{act}}{P} = 3.98^{klf}$

Actual precompression stress: $\frac{P_{actual}}{A} = \frac{1436.4(1000)}{6336} = 226.7^k$

$\boxed{\begin{array}{l} 226.7^{ksi} < 300^{ksi} \ \checkmark \text{ ok} \\ \text{This works for } 44\times30 \text{ end bay} \end{array}}$

check interior span force

$w_b = 3.96 \text{ klf}$ $\quad P = \frac{w_b L^2}{8 a_{int}} = \frac{3.96(30)^2(12)}{8(6)} = 891^k$

$891^k < 1425^k \Rightarrow$ less force is required in the center bay

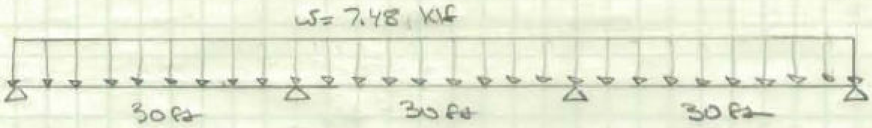
$w_b = \frac{1425^k(8)(6)}{12(30)^2} = 6.33^{klf}$ $\quad w_{DL} = \frac{170(44)}{1000} = 7.48^{klf}$

$\frac{w_b}{w_{DL}} = \frac{6.33}{7.48} = .846 = 84.6\% < 100\%$ ✓ ok

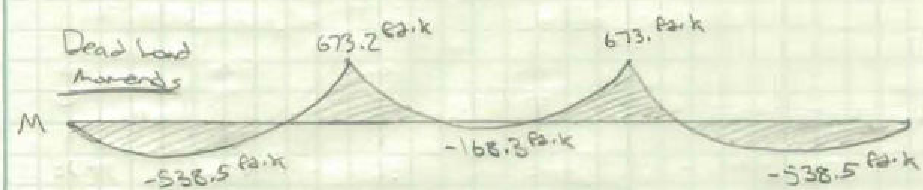
East/west frame: $P_{eff} = 1425^k$

check slab stresses

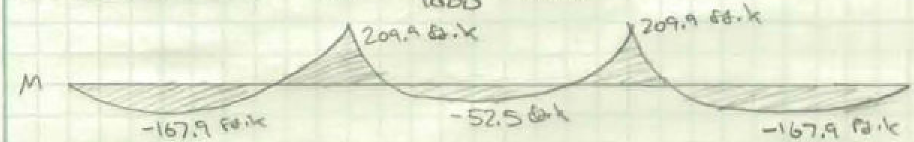
$$w_{DL} = \frac{(170)(44\text{ ft})}{1000} = 7.48 \text{ klf}$$



Dead Load Moments



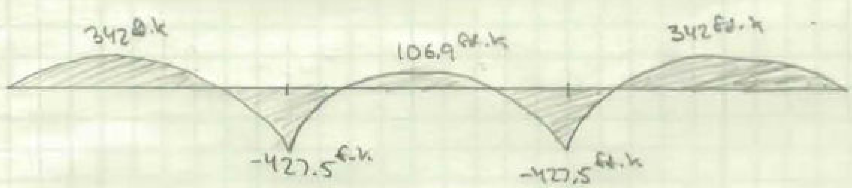
Live Load Moments: $w_{LL} = \frac{53(44)}{1000} = 2.332$ klf



Balanced Load Average:

$$w_b = \frac{3.96(2)+6.37}{3} = 4.75$$

Total Balancing Moment, M_{bal}: $w_b = -4.75$ (Avg)



Stage 1: Stresses immediately after Jacking

Midspan stresses
Int. Span

$$f_{top} = [(-168.3+106.9)(12)(1000)] / (12672) - 226.7$$

$= -58.14 - 226.7 = -284.84 < .6 f'_{ci} = 1800$ psi ✓ ok

$$f_{bot} = [(168.3-106.9)(12)(1000)] / (12672) - 226.7$$

$= 58.14 - 226.7 = -168.56 > 3\sqrt{f'_{ci}} = 164$ psi NO GOOD

* increase slab depth to 13 in ⇒ Self weight = 162.5, A = 6864, S = 14872

Two way Post-tensioned slab	Tech Report #2	Page 5 of 7

End span

$f_{top} = [(-538.5 + 342)(12)(1000)] / 14872 - 226.7$
$= -385.25 < 1800\ psi$ ✓ ok

$f_{bot} = [(538.5 - 342)(12)(1000)] / 14872 - 226.7$
$= -68.14 < 164\ psi$ ✓ ok

Support stresses

$f_{top} = [(673.2 - 427.5)(12)(1000)] / 14872 - 226.7 = -28.44 < \sqrt{3} f'_{ci} = 164\ psi$ ✓ ok

$f_{bot} = [(-673.2 + 427.5)(12)(1000)] / 14872 - 226.7 = -424.95 < 1800$ ✓ ok

Stage 2: Stresses at service load (DL + LL + PT)

Midspan stresses

Interior span

$f_{top} = [(-168.3 - 52.5 + 106.9)(12)(1000)] / 14872 - 226.7 = -406.4 < .45 f'_c = 2250\ psi$
✓ ok

$f_{bot} = [(168.3 + 52.5 - 106.9)(12)(1000)] / 14872 - 226.7 = -134.8 < 6\sqrt{f'_c} = 424\ psi$ ✓ ok

End span

$f_{top} = [(-538.5 - 167.9 + 342)(12)(1000)] / 14872 - 226.7 = -520.73 < 2250$ ✓ ok

$f_{bot} = [(538.5 + 167.9 - 342)(12)(1000)] / 14872 - 226.7 = 67.33 < 424$ ✓ ok

Support stresses

$f_{top} = [(673.2 + 209.9 - 427.5)(12)(1000)] / 14872 - 226.7 = 140.92 < 424$ ✓ ok

$f_{bot} = [(-673.2 - 209.9 + 427.5)(12)(1000)] / 14872 - 226.7 = -594.3 < 2250$ ✓ ok

Ultimate strength

Determine factored moments: $M_1 = P(e)$

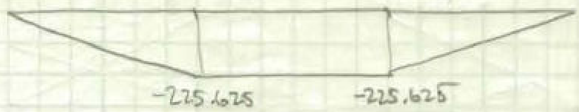
$e = 0$ at the exterior support
$e = 5.5$ at the interior supports

N.A. to center of tendon

$M_1 = \frac{(1425)(5.5)}{12} = 653.125$ ft-k

Two way Post-tensioned Slab | Tech Report # 2 | Page 6 of 7

$M_{sec} = M_{bal} - M_1 = 427.5 - 653.125 = -225.625$ ft-k at interior supports



$M_u = 1.2 M_{DL} + 1.6 M_{LL} + 1.0 M_{sec}$

At midspan: $M_u = 1.2(-538.5) + 1.6(-167.9) + 1.0(-112.83) = -1027.67$ ft-k

At support: $M_u = 1.2(673.2) + 1.6(209.9) + 1.0(-225.65) = 918.03$ ft-k

Determine minimum bonded reinforcement

Positive moment region:

Interior span: $f_t = -134.8$ psi $< 2\sqrt{f'_c} = 2\sqrt{5000} = 141$ psi
no positive reinforcement required.

Exterior span: $f_t = 67.33 < 2\sqrt{f'_c} = 2\sqrt{5000} = 141$ psi
Minimum positive reinf. required.

$$y = \frac{f_t(h)}{(f_t + f_c)} = \frac{67.33(13)}{(67.33 + 520.73)} = 1.49 \text{ in}$$

$$N_c = \frac{M_{DL+LL}}{S}(.5)(y)(\ell_2)(12) = \frac{538.5 + 167.9}{14872}(.5)(1.49)(44)(12)$$

$N_c = 18.68^k$

$$A_{s,min} = \frac{N_c}{0.5 f_y} = \frac{18.68}{0.5(60)} = .623 \text{ in}^2$$

$A_{s,min} = \frac{.623}{44} = .014$ in²/ft => $\boxed{\text{use \#3 at 12" o.c. on bottom}}$

Negative moment region:

$A_{s,min} = 0.00075 A_{cf}$

Interior supports: $A_{cf} = (13)(12)(44) = 6864$ in²

$A_{s,min} = .00075(6864) = 5.148$ in²

$\boxed{\text{use 6 \#9 top bars (6.00 in}^2)}$

Exterior supports: $A_{cf} = 13(44)(12) = 6864$ in²

$A_{s,min} = .00075(6864) = 5.148$ in²

$\boxed{\text{use 6 \#9 top bars}}$

* Must span minimum of 1/6 the clear span on each side of support
At least 4 bars required in each direction.
Place top bars within $1.5h = 19.5$ in away from the face of the support on each side.
Maximum spacing is 12" ⇒ use 12" spacing

Check minimum reinf to see if it meets ultimate strength

$M_n = (A_s f_y + A_{ps} f_{ps})(d - \frac{a}{2})$ ⇒ d = effective depth

$A_{ps} = A_{tendons}(\# \text{ tendons}) = 0.153(54) = 8.262 \text{ in}^2$

$f_{ps} = f_{se} + 10000 + \left(\frac{f'_c bd}{300 A_{ps}}\right) = 174000 + 10000 + \left(\frac{5000(44)(12)(d)}{(300)(8.262)}\right)$

$f_{ps} = 184000 + (1065.1)d \qquad a = \frac{(A_s f_y + A_{ps} f_{ps})}{0.85 f'_c b}$

at supports: $d = 13" - 3/4" - 1/4" = 12"$

$f_{ps} = 184000 + (1065.1)(12) = 196781.3$ psi

$a = \frac{(6)(60) + (8.262)(196.7)}{0.85(5)(44)(12)} = .885$ in

$\phi M_n = 0.9\left[(6)(60) + (8.262)(196.7)\right](12 - \frac{.885}{2}) = 20649$ in·k

$\phi M_n = 1720.74$ ft·k > 918.03 ft·k ✓ ok

use 6 #9 top bars at interior supports use 6 #9 top bars at exterior supports

At midspan: $d = 13" - 1\frac{1}{2}" - 1/4" = 11.25$ in

$f_{ps} = 184000 + (1065.1)(11.25) = 195982$ psi

$a = \frac{(.623)(60) + (8.262)(195.9)}{.85(5)(44)(12)} = .738$ in

$\phi M_n = 0.9\left[(.623)(60) + (8.262)(195.9)\right](11.25 - \frac{.738}{2})$

$\phi M_n = 16216$ in·k $= 1351.3$ ft·k

1351.3 ft·k > 1027.67 ft·k ✓ ok

use #3 bars at 12" o.c on bottom