Farquhar Park Aquatic Center

York, PA

Technical Report #2

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Structural Option

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October 28, 2009

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Executive Summary

The Pro-Con Structural Study of Alternate Floor Systems discusses the existing floor system of the Farquhar Park Aquatic Center natatorium and three potential alternate floor systems. A typical bay size of 27’-0” x 30’-0” was used for the analysis of each floor system. A superimposed dead load of 15 psf and a live load of 125 psf were applied to each system. No live load reductions were permitted since the 125 psf live load was greater than 100 psf. The existing floor system consists of 12” precast concrete hollow core planks with 2” topping. The planks typically bear on a 12” CMU wall at one end and bear on the bottom flange of a W27x84 girder at the other end. These support conditions were used for the analysis of each alternate floor system. Load tables from Nitterhouse Concrete Products, Inc. were used to determine that the existing hollow core plank system was sufficient to resist the applied gravity loads. The three alternate floor systems that were analyzed included a one-way slab, a non-composite steel frame, and a one-way post-tensioned slab.

The one-way slab was designed according to ACI 318-08 using a one-foot unit width and f'_c of 4,000 psi, which resulted in a 16.5” thick slab with #5 bars @ 5” o.c. The AISC *Steel Construction Manual* was used to size the girder, which resulted in a W24x76. The beams and girders of the non-composite steel frame system were designed using the AISC *Steel Construction Manual*, and the non-composite deck was designed using the *Vulcraft Steel Roof and Floor Deck Catalog*. Steel framing consisted of W16x26 beams spaced 5’-0” o.c., spanning the 27’-0” direction, and framing into a W21x55 girder with 1.3C20 metal deck and a total slab depth of 4.3 inches. The one-way post-tensioned slab was designed to be 13” thick with 32 tendons spanning in the 27’-0” direction. The girder was determined to be a W24x68 using the AISC *Steel Construction Manu*al.

The advantages and disadvantages of each system were discussed, and a system comparison chart was created to present a summary of the pros and cons of each floor system. Overall, the best floor system was determined to be the existing precast concrete hollow core planks. The planks provided a relatively thin and lightweight floor system for the spans and high applied loads. The most viable option for an alternate floor system is the post-tensioned slab. This was the only system that was able to achieve a thinner floor system depth than that provided by the hollow core planks with 2” topping. The post-tensioned slab can be used for long spans and heavy loads, and it provides excellent vibration control. The one-way slab was not feasible due to the thickness of the solid concrete slab and associated high costs. A one-way concrete joist slab system may be further investigated in place of the one-way slab. This system can achieve a very thin floor slab and is cost effective for longer spans and higher loads. The amount of concrete would be drastically reduced from that required for the one-way slab and would hence be a much cheaper system. The non-composite steel frame was also not very feasible because the required girders increased the floor system depth by 11.1 inches. However, this system is lightweight and relatively cheap, and perhaps a composite steel frame would be worth further investigation along with the one-way post-tensioned slab and the one-way concrete joist slab system.

Introduction

The Farquhar Park Aquatic Center is a 37,000 square foot multi-level, state-of-the-art natatorium complex designed by Nutec Design Associates, Inc., a full-service architectural and engineering firm located in York, PA. The facility is located in the city of York and features a 53-foot high natatorium with raised seating, a 12-foot deep indoor swimming pool with diving platforms, a 3,600 square foot single story masonry bath house, and a large outdoor swimming pool, as can be seen in Figure 1. The complex was intended to be used by the YMCA of York, but the original design was never constructed due to cost and budget concerns. The natatorium contains an entry level, a concourse level, and a gallery level. The main entrance opens up into an expansive 24-foot high lobby than spans from one end of the building to the other. The lobby provides access to concessions, men's and women's toilets, and corridors that connect the main lobby to the indoor swimming pool area. The entry level also contains men's and women's lockers and showers, a team room, offices, storage rooms, timer room, utility room, dish room, and trophy display case.

Figure 1 – Arial View of Natatorium Complex

Concrete stairs near the main entrance lead up to the concourse level which houses a mechanical room and a team store. A long precast concrete ramp also connects the ground floor to the second floor. The floor of the concourse level sits about 10 ½' above the ground level and consists of 12" precast hollow core concrete planks, as can be seen in Figure 2. Visitors can overlook the lobby below behind a 3 ½' guardrail. A precast L-shaped concrete balcony spans the entire length of the pool and provides access to the grandstand seating area.

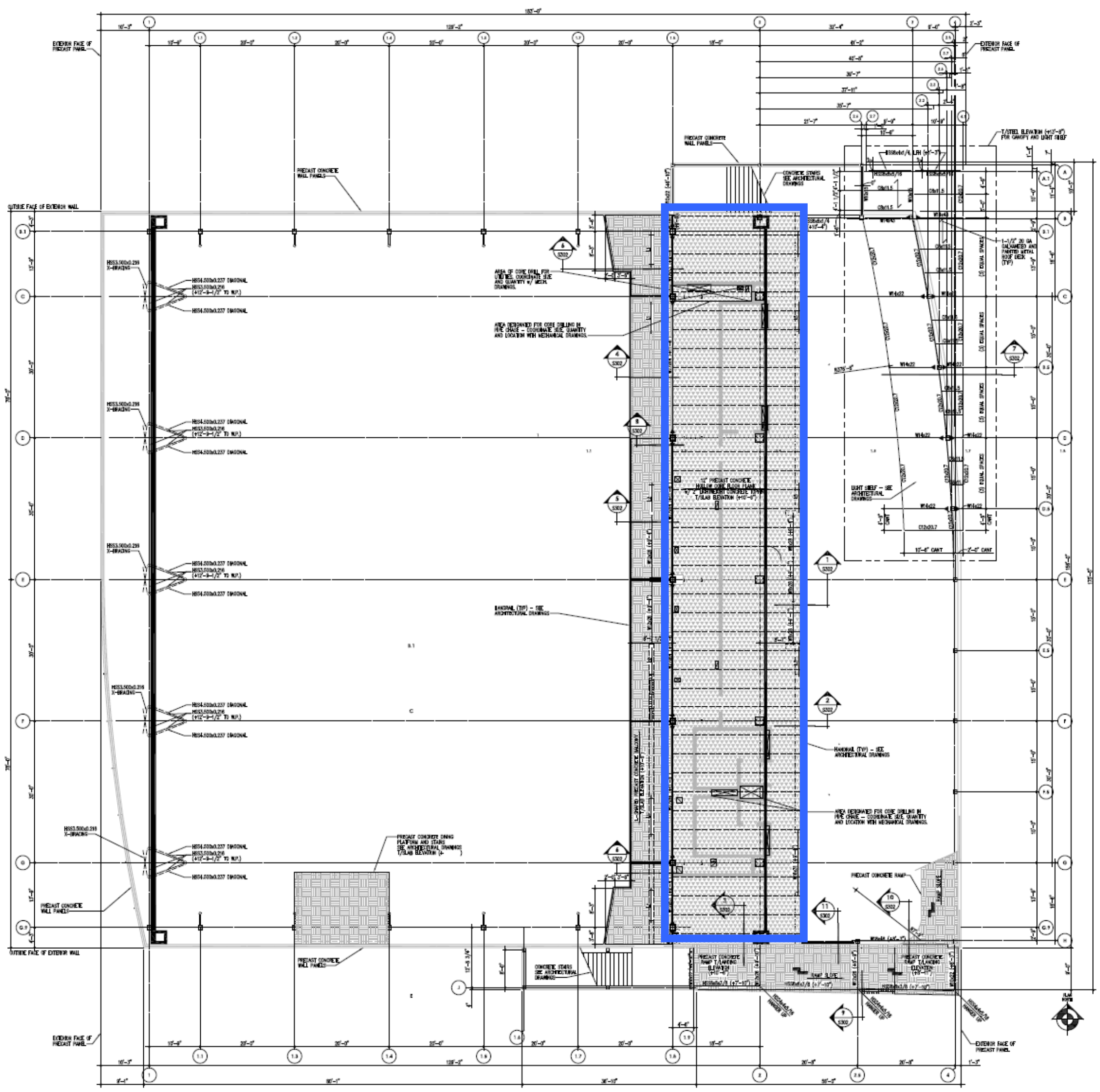


Figure 2 - Concourse Level Framing Plan (12" precast concrete hollow core floor planks are shown in blue – they span 27'-0" and run almost the entire length of the building)

The natatorium's curved roof spans about 130'0" and is supported by large trusses, creating a very open space. The lower roof above the lobby sits about 14' below the lowest point of the curved roof and contains most of the mechanical units. Trusses spaced at 15'-0" on-center support the roof and units. The east-facing and west-facing exterior walls of the natatorium are both slightly curved. At each end of the indoor swimming pool area is a large, curved glazed aluminum curtain wall made of Solera-T glazing. These two curtain walls are each 123'-11" long, 21'-0" tall at their highest points, and 8'-0" tall at their shortest points. Precast concrete panels are primarily used as the façade along with a mix of metal wall panels and glazed curtain walls, as can be seen in Figure 3.

Nutec Design Associates designed the facility to comply with certain LEED credits for the project to achieve LEED Silver Certification. Thermal shading effects were provided by a façade plant climbing system that helped to reduce indoor air temperatures. Another sustainability feature was the natural daylighting provided by the large glass curtain walls enclosing the indoor swimming pool area. Other requirements were related to certain materials and ensuring that they are environmentally friendly.

Figure 3 – View of Main Entrance of Natatorium (showing precast concrete panels, metal wall panels, and glazed curtain walls)

The Pro-Con Structural Study of Alternate Floor Systems examines the existing hollow core concrete plank floor system and three alternate floor systems for the natatorium. Various aspects of each floor system are compared to help determine potential candidates for the structural proposal assignment of AE Senior Thesis. Specific aspects of each system that were taken into account include slab self weight, slab depth, floor system depth, vibration control, architectural impact, constructability, and system cost. All alternate floor systems were designed using a typical 27’-0” x 30’-0” bay.

<u>Structural System Overview</u>

Foundation

The geotechnical evaluation was performed by GTS Technologies, Inc. on September 30, 2005. The study included five boring tests, only one of which hit water and revealed a water level 12'-0" below existing site grades. The recommended allowable bearing pressure from GTS Technologies for compacted structural fill was 2500 psi. A shallow foundation system consisting of isolated spread footings at various depths was used. Most of the foundations were located about 2'-0" below finished floor elevation, however a few along the west side of the natatorium were located about 15'-0" below finished floor elevation in order to get below the pool structure. This can be seen in Figure 4. Footings range in size from 4'-6"x4'-6"x1'-0" up to 19'-0"x19'-0"x2'-0". Larger foundations were required to handle the loads carried by the trusses spanning across the indoor pool.

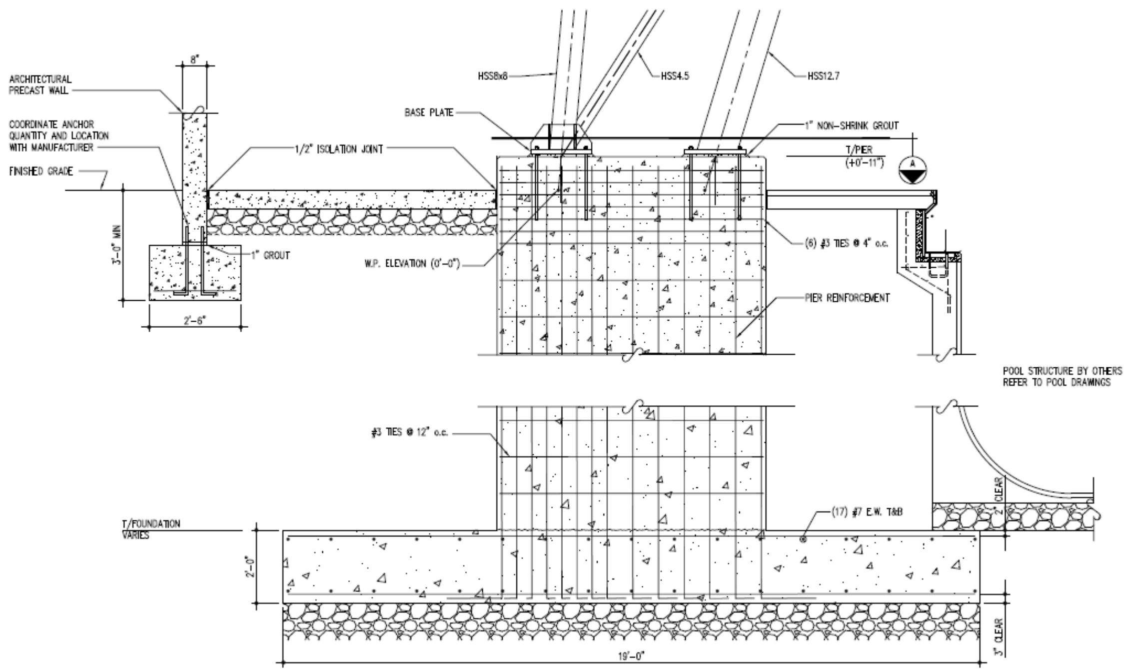


Figure 4 – Detail of Pier Supporting Large Tapered Truss Column

Concrete with a compressive strength of 4,000 psi was used for the footings. Reinforcement in the footings consists of #5, #6, and #7 bars, while reinforcement in the piers consists of #6 and #8 bars, with the #8 bars only being used in the large, deep piers supporting the tapered truss columns. A typical pier detail is shown in Figure 5. Strip footings were 2'-6" wide for interior walls and 2'-0" wide for exterior walls. Geotechnical reports indicate that exterior footings shall be embedded a minimum of 36 inches below final grade for frost protection. Foundations were to be placed on a geotextile layer to minimize the loss of aggregate materials into the subgrade. Due to the proximity of Willis Creek Run and the fact that water was found in one boring test, the geotechnical report suggests that the bottom layer of the pool slab be designed to include a 12-inch No. 57 aggregate drainage layer and pressure release valves to prevent potential floatation due to ground water when the pool is drained.

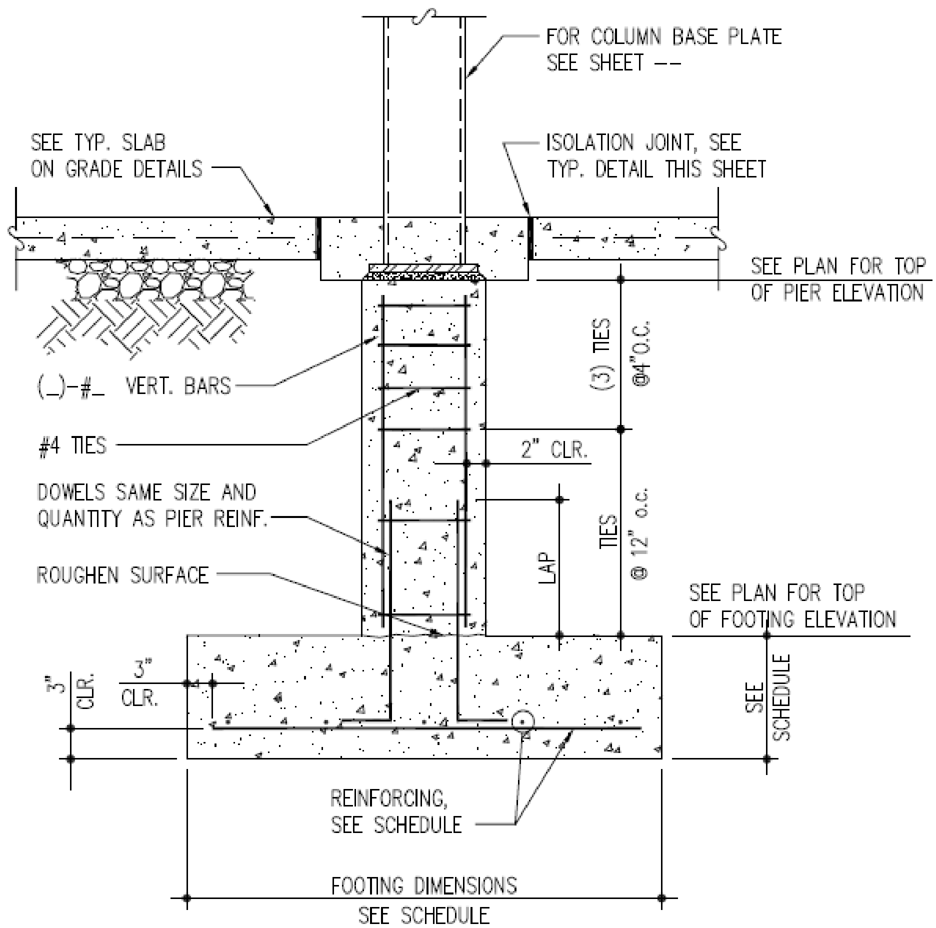


Figure 5 – Typical Pier Detail

Superstructure

The ground floor consists of a 4” concrete slab-on-grade with 6x6 W2.0xW2.0 W.W.F. on 4” crushed stone base and a compressive strength of 4,000 psi. The concession area sits on a recessed concrete slab, and a portion of the floor slab near the pool structure becomes 8” thick with #4 bars at 12” on-center L.W. and #5 bars at 12” on-center S.W. HSS columns in the lobby run along the east wall and support the roof trusses above the lobby. The entry level also contains 12” CMU walls with #5 bars at 32” on-center that are grouted solid full height. These walls enclose parts of the bathrooms, locker rooms, offices, team room, storage rooms, and utility room and are located beneath the grandstand seating area. A floor plan of the entry level is shown in Figure 6. Precast concrete columns help support the 8" precast concrete ramp that runs from the ground floor up to the concourse level. The ramp is also supported by W-shape beams, HSS columns, and hangers.

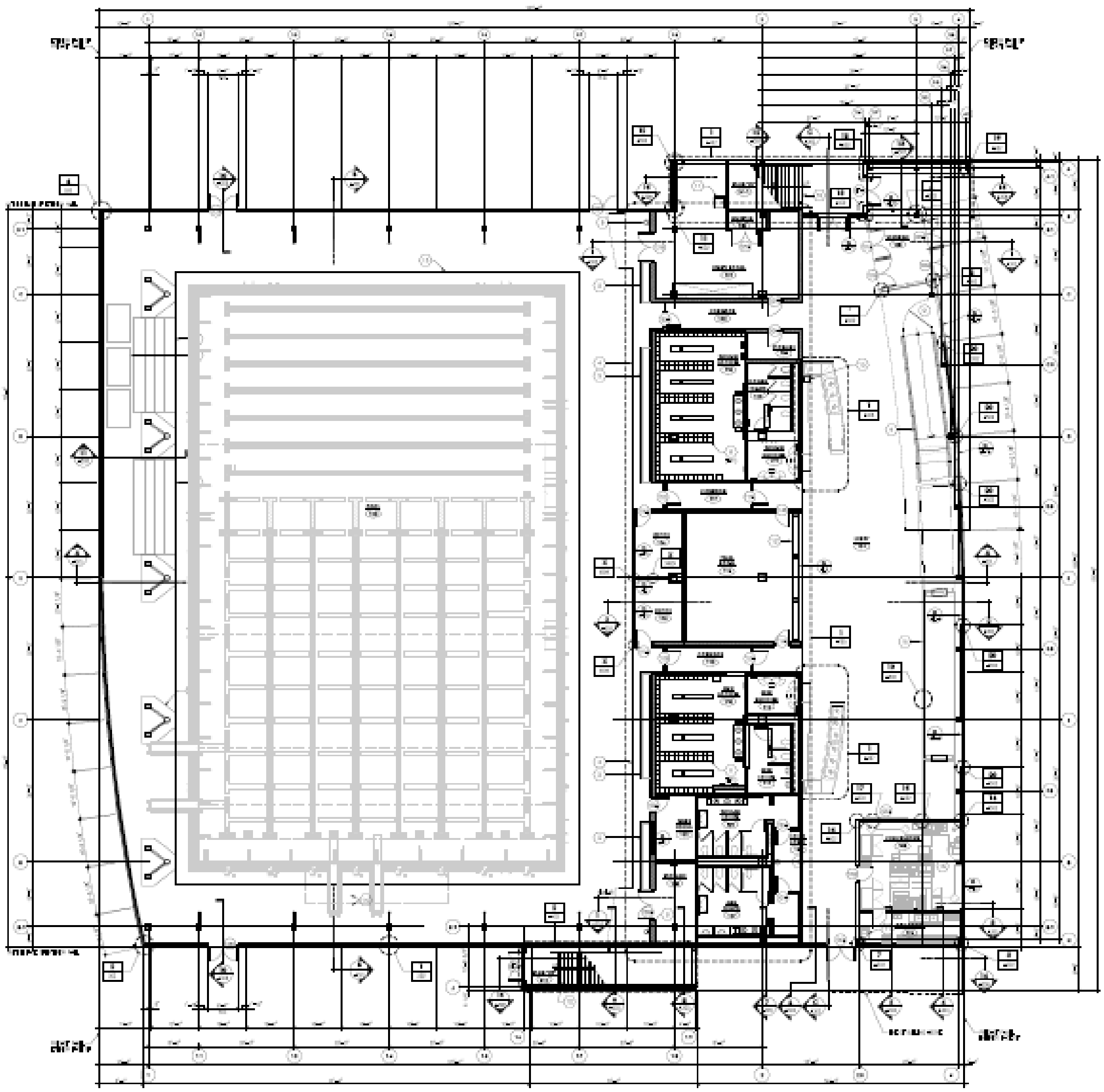


Figure 6 – Entry Level Floor Plan

Triangular HSS trusses spanning 130'-0" support the large curved roof above the indoor swimming pool area and are shown in Figure 7. The columns for these trusses are triangular, tapered, and spaced 30'-0" on center. Both the trusses and the supporting columns are made up of HSS members. Long span deck was used to span between the trusses. The other ends of the large trusses are supported by HSS18x18x5/8 columns. HSS wind column trusses run along the north and south walls in the indoor pool area as well. The trusses are 3'-0" deep and vary in height with the tallest at 51'-2 ¼" above finished floor elevation. The wind column trusses connect into the main roof diaphragm. The rest of the high roof framing primarily consists of HSS6x6 and HSS8x8 members.

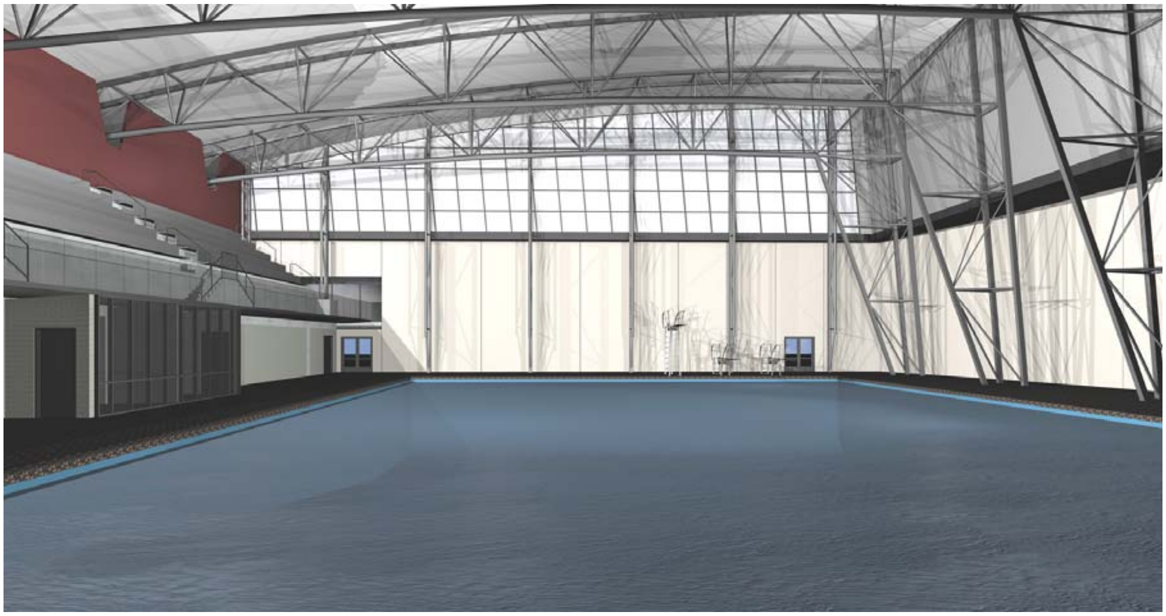


Figure 7 – Rendering of Indoor Pool Area Showing Large Curved Trusses

The precast concrete grandstand seating area that runs from the concourse level to the gallery level is supported by sloped W27x94 beams that frame into the HSS18x18x5/8 members that also support the large curved trusses. The floor system of the concourse level consists of 12" precast concrete hollow core floor planks with 2" lightweight concrete topping, as is shown in Figure 8. Top of slab elevation is 10'-6". The precast concrete balcony is supported by a 12" CMU wall, and additional strength is provided by a 12" beam with two continuous #5 bars. A canopy and light shelf near the main entrance and lobby are slightly higher than the concourse level and are supported by cantilevered W14x22 and W14x43 beams. Additional framing is provided by C8x11.5 beams and curved C12x20.7 beams. Moment connections allow the W14 beams to cantilever from the supporting HSS10x10 columns.

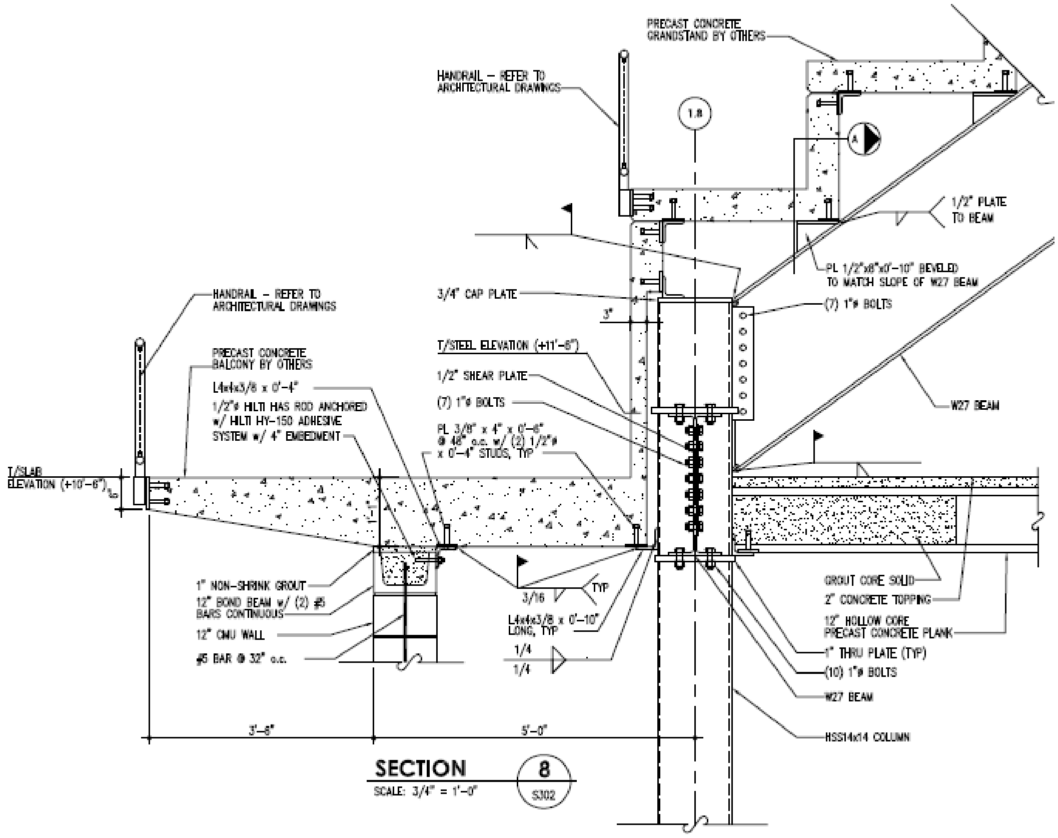


Figure 8 – Section Showing the 12" Hollow Core Precast Concrete Planks, the Precast Concrete Balcony, and the W27x94 Beams Supporting the Concrete Grandstand

The gallery level has HSS roof trusses spanning about 41'-0" and spaced 15'-0" on center (and 2'-5" deep) supporting 6" 18 GA acoustical long span metal roof deck with 18 GA perforated cover and polyencapsulated acoustical batt insulation. The trusses are 2'-5" deep, slightly sloped, and also support the mechanical unit support framing above. The top of steel elevation for the mechanical unit support framing is 28'-0", and the framing consists of W8, W10, and C8 beams.

Lateral System

The large truss columns and mezzanine moment frame take the lateral load in one direction, while the truss columns, a frame between the pool and lobby, and frame at the front of the lobby handle the lateral load in the other direction. Some lateral load from the mezzanine goes into the CMU walls, but the steel moment frame provides most of the lateral support. The wind columns are designed to simply take the wind force and transfer it to the roof diaphragm. A mezzanine level framing plan is shown in Figure 9, and a roof framing plan is shown in Figure 10. The wind columns transfer roughly half the load to the ground or base connection and the other half of the load to the high roof diaphragm. The roof diaphragm transfers the load to the large trusses over the indoor pool, which in turn sends part of the load to the five large braced truss columns and the rest of the load to the mezzanine moment frame system. The large truss columns are laterally braced by HSS3.500x0.216 X-bracing. The two chords of the truss columns are offset by four feet at the base, providing a rather rigid support that can handle high lateral loads. The large trusses and supporting truss columns can be seen in Figure 11, and the wind columns can be seen in Figure 12.

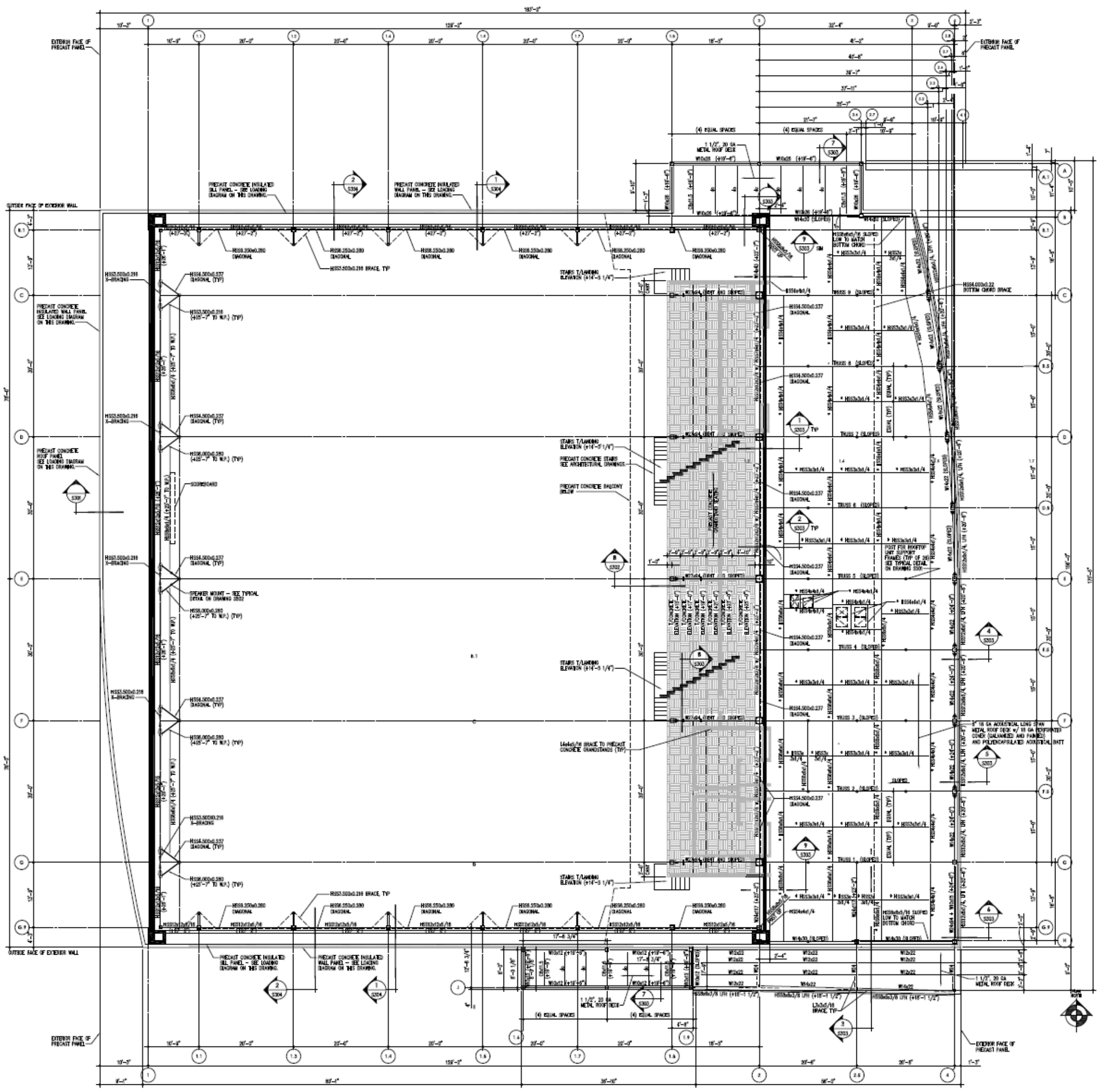


Figure 9 – Gallery/Mezzanine Level Framing Plan (the shaded portion is the grandstand seating area)

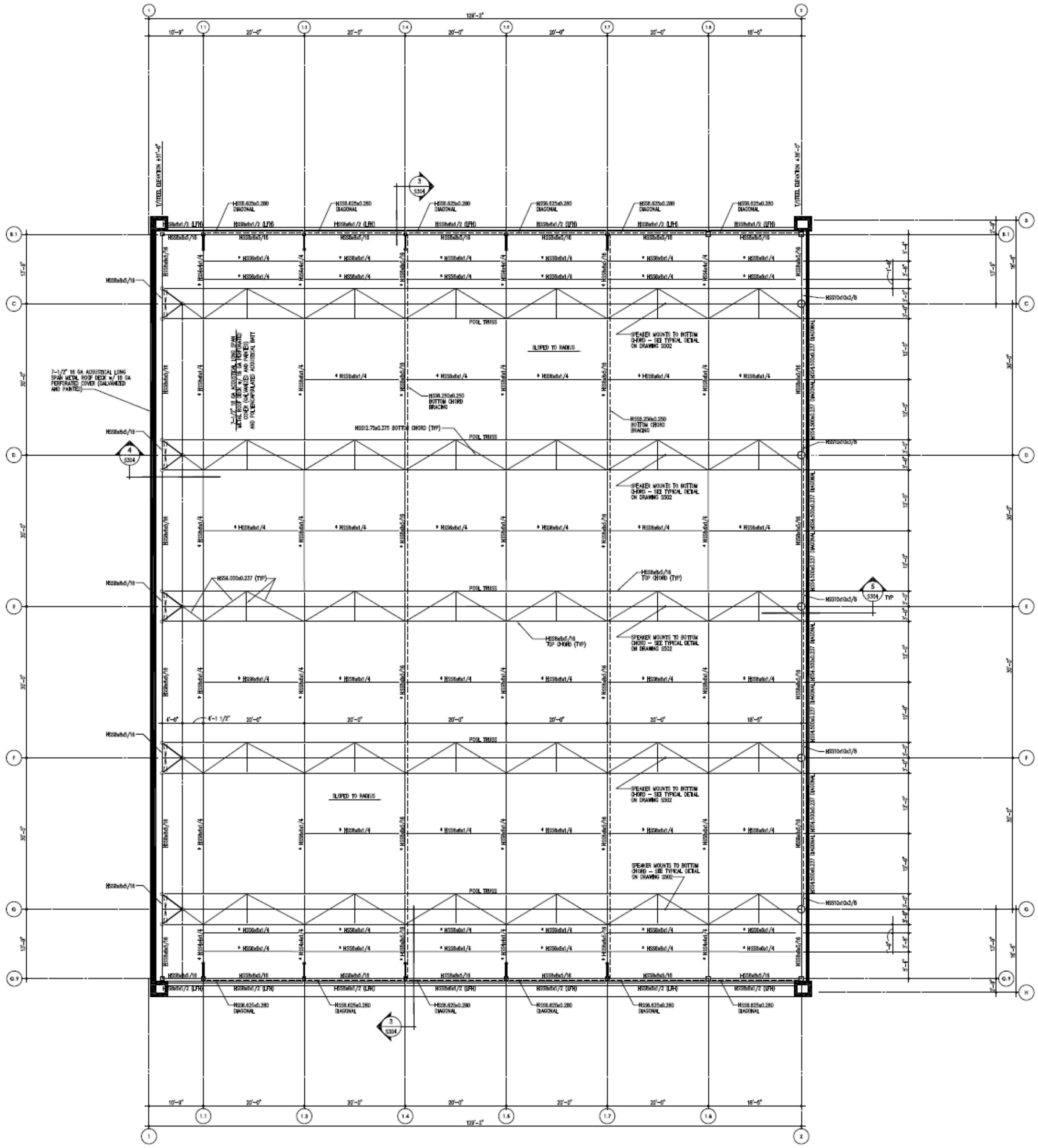


Figure 10 – Roof Framing Plan (including the five large trusses above the pool area and additional framing)

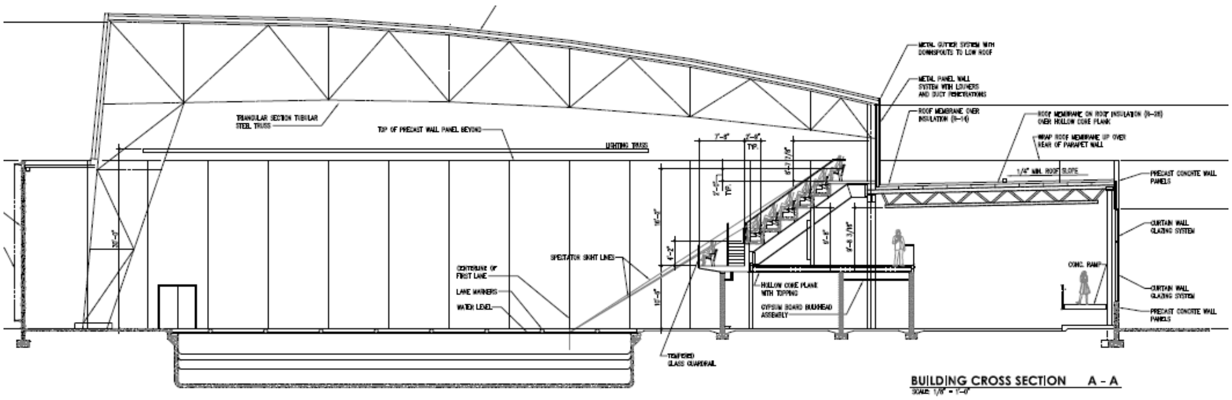


Figure 11 – Cross Section Through Center of Building

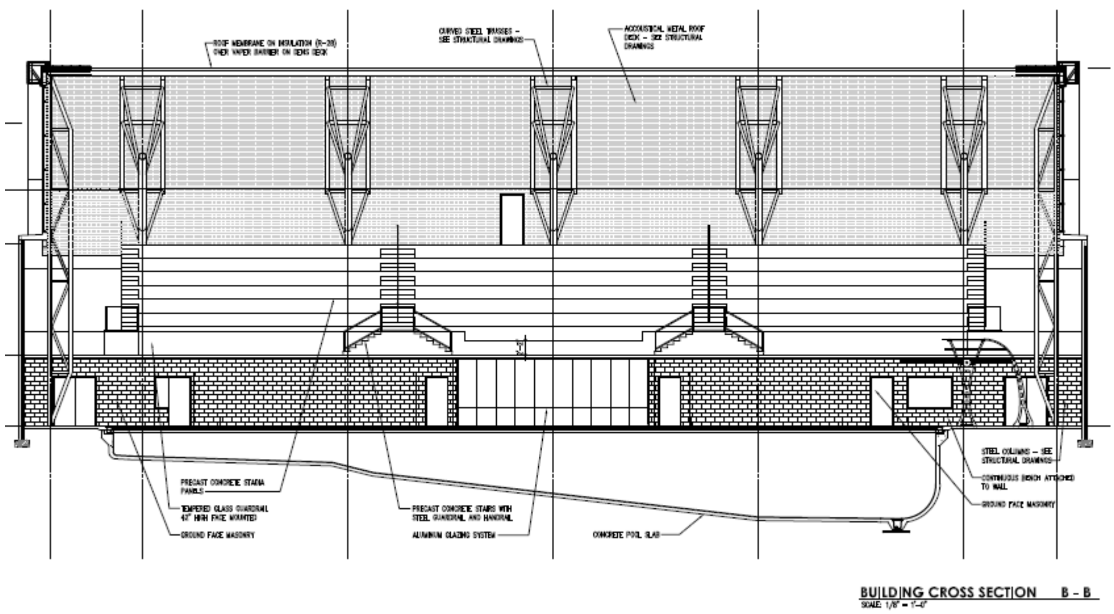


Figure 12 –Cross Section Through Indoor Pool Area Showing the Wind Columns

Codes and Standards

Below is a list of codes and standards applied to the original design and a list of codes that were substituted for Thesis analysis. The codes and standards applied to the original design were noted on Nutec's structural drawings. Also listed is a strength requirement summary of the materials used in the building.

Applied to Original Design:

International Building Code – 2003

"Building Code Requirements for Reinforced Concrete, ACI-318-99", American Concrete Institute

"ACI Manual of Concrete Practice – Parts 1 through 5, 2002", American Concrete Institute

"Manual of Standard Practice", Concrete Reinforcing Steel Institute

"Manual of Steel Construction – Load and Resistance Factor Design", Third Edition, American Institute of Steel Construction (including specification for structural steel buildings, specification for steel hollow structural sections, specification for single-angle members, specification for structural joints using ASTM A325 or A490 bolts, and AISC Code of Standard Practice)

"Hollow Structural Sections Connections Manual", American Institute of Steel Construction

"Detailing for Steel Construction", American Institute of Steel Construction

"Structural Welding Code ANSI/AWS D1.1-98", American Welding Society

"Building Code Requirements for Masonry Structures", (ACI 530-99/ASCE 5-99)

"Specifications for Masonry Structures", (ACI 530.1-99/ASCE 6-99)

Substituted for Thesis Analysis:

International Building Code – 2006

ASCE 7-05

ACI 318-08

Material Strength Requirement Summary:

Cast-in-Place Concrete

Foundations:	4,000 psi
Slabs on Grade:	4,000 psi
Exposed to Freezing:	4,000 psi
Reinforcing Bars:	60 ksi

Structural Steel

Channels, Angles, and Plates:	36 ksi
Wide Flange Shapes:	50 ksi
Structural Tubing (Rectangular):	46 ksi
Structural Tubing (Round):	42 ksi
Structural Pipe:	35 ksi

Masonry

Compressive Strength:	2,000 psi
Reinforcing Bars:	60 ksi

Building Load Summary

Gravity Loads

Nutec Design Associates, Inc., used the 2003 International Building Code and the American Society of Civil Engineers (ASCE) 7-98 to determine gravity loads, while ASCE 7-05 was used to determine the gravity loads in this report. All reported loads are noted in Table 1. Snow load factors using ASCE 7-05 are shown in Table 2, and Table 3 shows a breakdown of the weights of the various components of the building.

Gravity Loads			
Description	*Nutec*	*ASCE 7-05*	*Design Value used for Thesis*
Dead (DL)			
Concrete	145 pcf	150 pcf	150 pcf
Live (LL)			
Roofs	30 psf + Drifted Snow	20 psf	20 psf + Drifted Snow
Grandstands	100 psf	100 psf	100 psf
Ramps, Corridor	100 psf	100 psf	100 psf
Mechanical Rooms	100 psf	?	100 psf
Snow (S)			
Snow	21 psf	23.1 psf	23.1 psf

Table 1 – Building Gravity Loads
*Nutec's roof live load may have conservatively been taken to be 30 psf + drifted snow instead of 20 psf + drifted snow
*Nutec showed a Snow Load Importance Factor of 1.0 on the drawings. Nutec said this was a mistake and that the drawings should have shown a Snow Load Importance Factor of 1.1. The Nutec snow load of 21 psf in Table 1 was taken from the drawings, which incorporated the incorrect Snow Load Importance Factor of 1.0 instead of 1.1. Nutec's values for C_e, C_t, and C_s matched those from ASCE 7-05. Hence, the Nutec snow load and the ASCE 7-05 snow load technically match, but Nutec's drawings do not reflect this and only show a snow load of 21 psf.

Snow Loads	
Ground Snow Load, P_g	30 psf
Snow Exposure Factor, C_e	0.7
Thermal Factor, C_t	1.0
Snow Load Importance Factor, I	1.1
Flat Roof Snow Load, P_f	23.1 psf
Roof Slope Factor, C_s	1.0

Table 2 – Snow Load Factors using ASCE 7-05
*Roof Slope Factor, C_s, was conservatively taken to be 1.0 (Nutec also used C_s = 1.0)

Weights of Building Components		
Large Trusses and Supporting Columns	146.78	kips
Concrete Grandstand	331.52	kips
Concrete Balcony	129.89	kips
Concrete Ramp	107.04	kips
Hollow Core Concrete Planks	315.71	kips
(2) Stairs at Grandstand	28.48	kips
Concrete Stairs by Lobby	41.97	kips
Roofing	242.02	kips
Wind Column Trusses	30.25	kips
Trusses Above Lobby	22.23	kips
Gallery Level Framing (above lobby)	51.75	kips
Mechanical Unit Support Framing	18.92	kips
Mechanical Units	54.50	kips
Interior Walls (Ground Level)	271.77	kips
Interior Walls (Concourse Level)	179.81	kips
Precast Concrete Panels	1577.84	kips
Roofing above Lobby	304.20	kips
Precast Sill by Wind Trusses	66.89	kips
Roofing along Large Trusses	44.02	kips
Roofing along West Edge	59.21	kips
Columns in Lobby	37.22	kips
Sloped Beams Supporting Concrete Seating Area	9.09	kips
TOTAL	4071.12	kips

Table 3 – Weights of Building Components

Floor Systems

Precast Concrete Hollow Core Planks – Existing

Material Properties:

Concrete:	12"x4'-0" hollow core planks with 2" topping
	f'_c = 6,000 psi
	f'_{ci} = 3,500 psi
Topping:	f'_c = 3,000 psi
Tendons:	(7) ½" diameter strands
	f_{pu} = 270,000 psi

Loading:

Dead (Self Weight):	77 psf
2" Topping:	25 psf
Superimposed Dead:	15 psf
Live:	125 psf

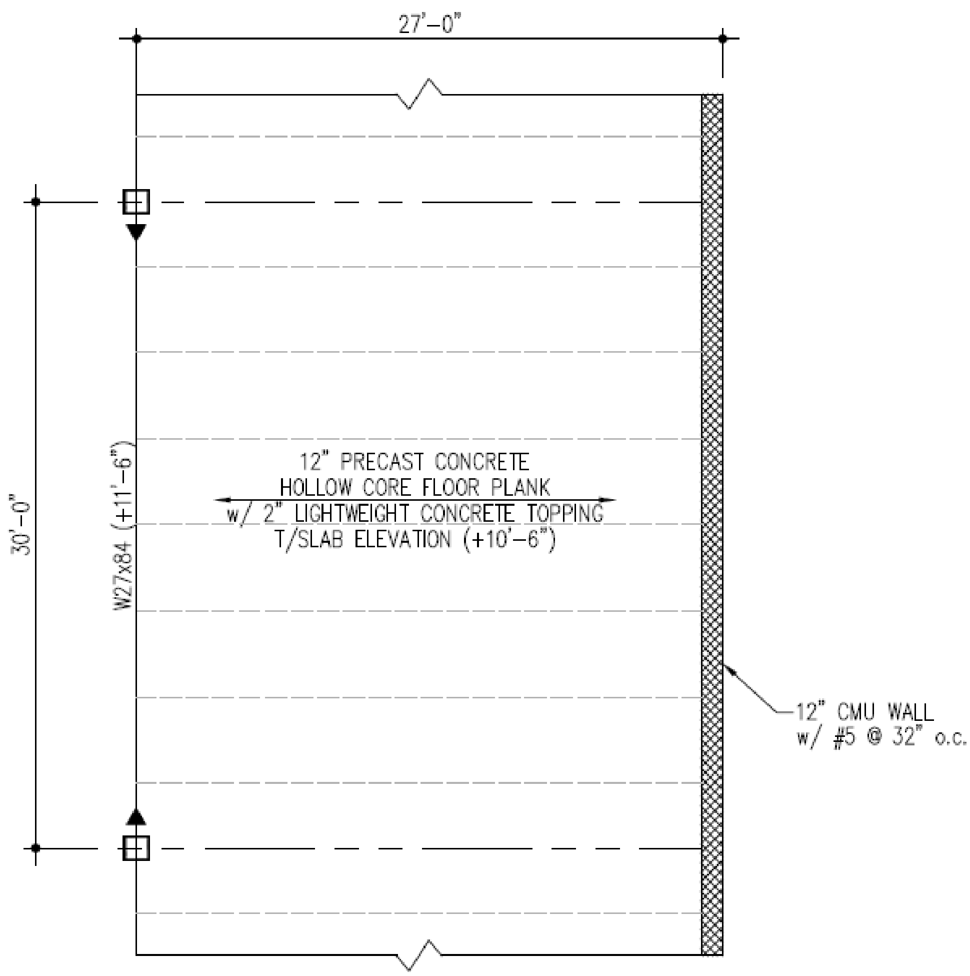


Figure 13 - Precast Concrete Hollow Core Plank Layout

Description:

The existing floor system of the Farquhar Park Aquatic Center consists of 12” precast concrete hollow core planks with 2” topping, as shown in Figure 13. The planks are 4’-0” wide and were manufactured by Nitterhouse Concrete Products, Inc. Load tables from Nitterhouse were used to determine if the hollow core planks provided adequate strength for the applied floor loads and span of 27’-0”. The planks bear on the bottom flange of W27x84 girders on one end and are supported by a 12” CMU wall on the other end. These end supports vary, however, along the length of the wall. Sometimes the hollow core planks are supported by lintels on one end, and some planks are supported by masonry walls on the opposite end of the plank as well. For simplicity of analysis, a typical 27’-0” by 30’-0” bay was chosen with a girder on one end and 12” CMU wall on the other end.

The Nitterhouse load tables for 12”x4’-0” hollow core planks only go down to a span of 32 feet. However, the planks are capable of holding a superimposed service load of 170 psf at a 32-foot span, which is greater than the actual superimposed service load of 165 psf. Therefore, it was determined that the 12” hollow core planks were sufficient for a span of only 27 feet since they were sufficient at an even greater span. The load tables for a 2-hour fire resistance rating were conservatively used even though the drawings for the project show a 0-hour fire resistance rating for floor construction including supporting beams and joists. In addition, it appears that 10”x4’-0” hollow core planks would have worked for the given loads and spans as well. However, the 12” planks may have been used because they fit better geometrically.

Analysis of the W27x84 girder for the 27’-0” x 30’-0” bay was performed using the AISC *Steel Construction Manual* and showed that the girder had more than enough strength to meet load and deflection criteria. However, the girder must also take some load from the precast concrete balcony, hence explaining why the girder appeared to be oversized. These additional loads were not accounted for directly in the analysis of the girder, but the girder was analyzed as though it were a simple span even though the girder had moment connections at both ends. Even with this conservative assumption, the W27x84 girders were found to be more than adequate. Supporting calculations can be found in Appendix A.

The live load of 125 psf matched that on the structural drawings used and was rather high due to the mechanical room that the floor system must support, as can be seen in Figures 14 and 15. No live load reductions were allowed since the live load was greater than 100 psf. A superimposed dead load of 15 psf was also chosen to match that noted on the structural drawings for the project.

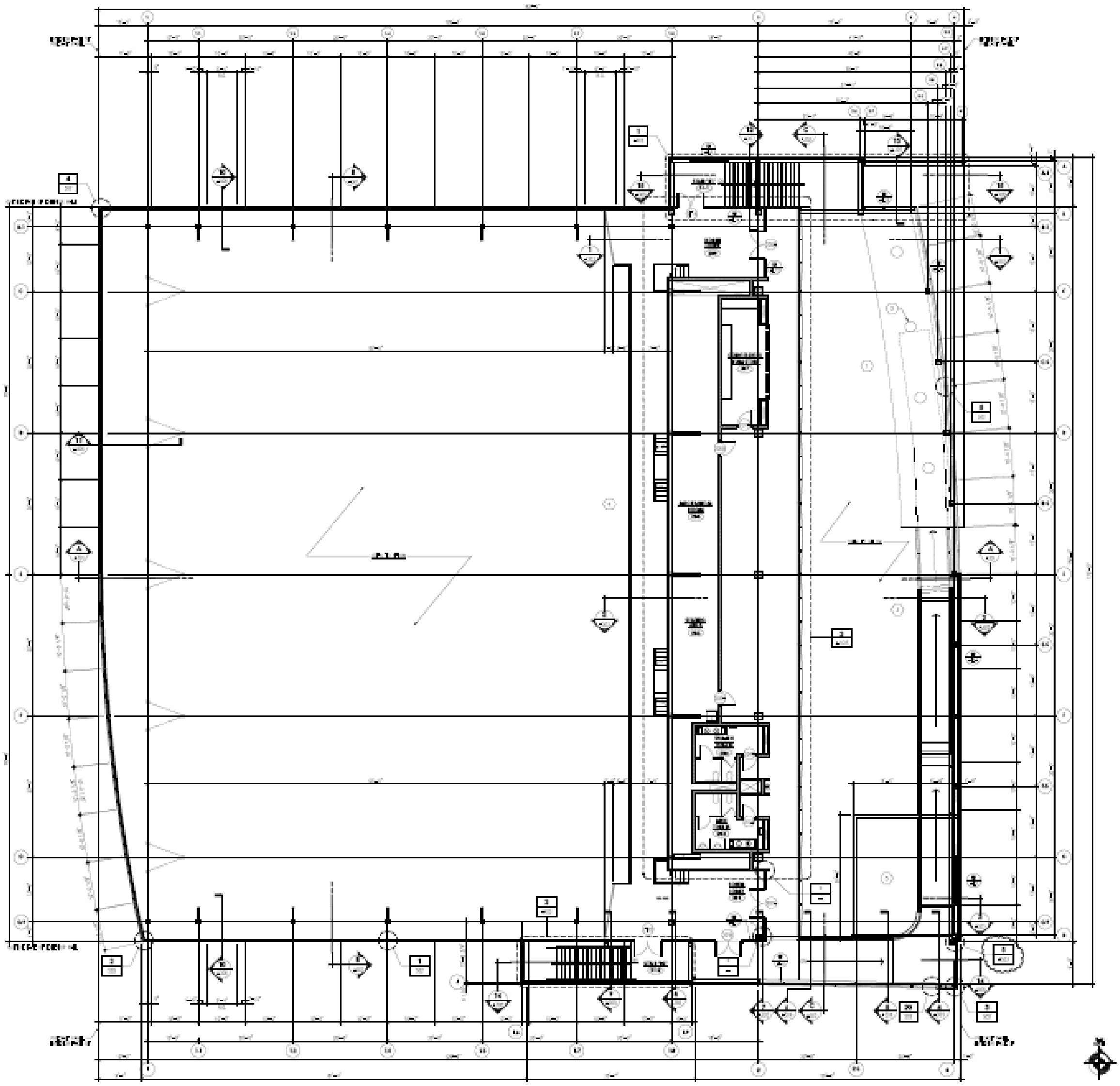


Figure 14 - Room Layout of Concourse Level

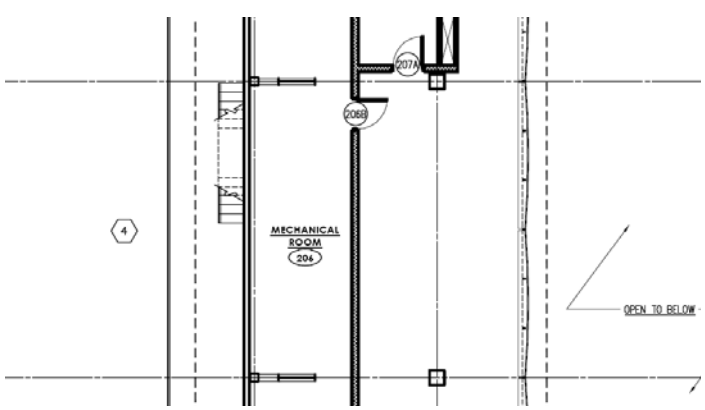


Figure 15 - Mechanical Room on Concourse Level (the HSS columns shown to the right of the mechanical room are only used to support upper levels and do not support the concourse level floor system)

Advantages:

The precast concrete hollow core plank system offers many benefits. A rather shallow floor system was achieved despite the relatively high loads and large spans. Hollow core planks are naturally fire resistant, hence eliminating the need for additional structural fireproofing. Ease of construction allows for fast and efficient erection, which in turns allows for a quicker construction schedule. Lead time for the concrete planks is also relatively short. Hollow core planks provide natural channels for wiring, conduits, and piping, and the planks can be drilled or shot for the installation of dropped ceilings and special lighting fixtures if necessary. This product provides a finished product in the sense that paint or carpet can be directly applied to the floor or ceiling. The floor system is very durable, clean, and low maintenance in addition to being naturally sound-resistant as well. Overall, the hollow core planks are a very cost effective system and achieve a thinner floor system depth than most other floor systems.

Disadvantages:

One of the disadvantages of a hollow core plank system is that the planks are only available in units of a certain width. The system seems to work best with floor geometries that fit the size of the planks. An unusual floor layout may eliminate the possibility of using a hollow core plank floor system. This is not really a problem with the Farquhar Park Aquatic Center although the 30-foot bay width is not evenly divisible by the 4-foot wide planks. It appears that 3’-0” wide planks were used at each end of the 30-foot bays to achieve the 30-foot dimension. Hollow core planks may also require more upfront planning, and plank vibration control in unknown at this time.

Deep girders required to support the hollow core planks can also increase the floor-to-ceiling height of some floor systems and hence have a negative architectural impact. However, this is not a problem with the natatorium since the hollow core planks bear on the bottom flange of the W27x84 girder, as can be seen in Figure 16. Therefore, the overall depth of the floor system is basically just the thickness of the hollow core planks and topping.

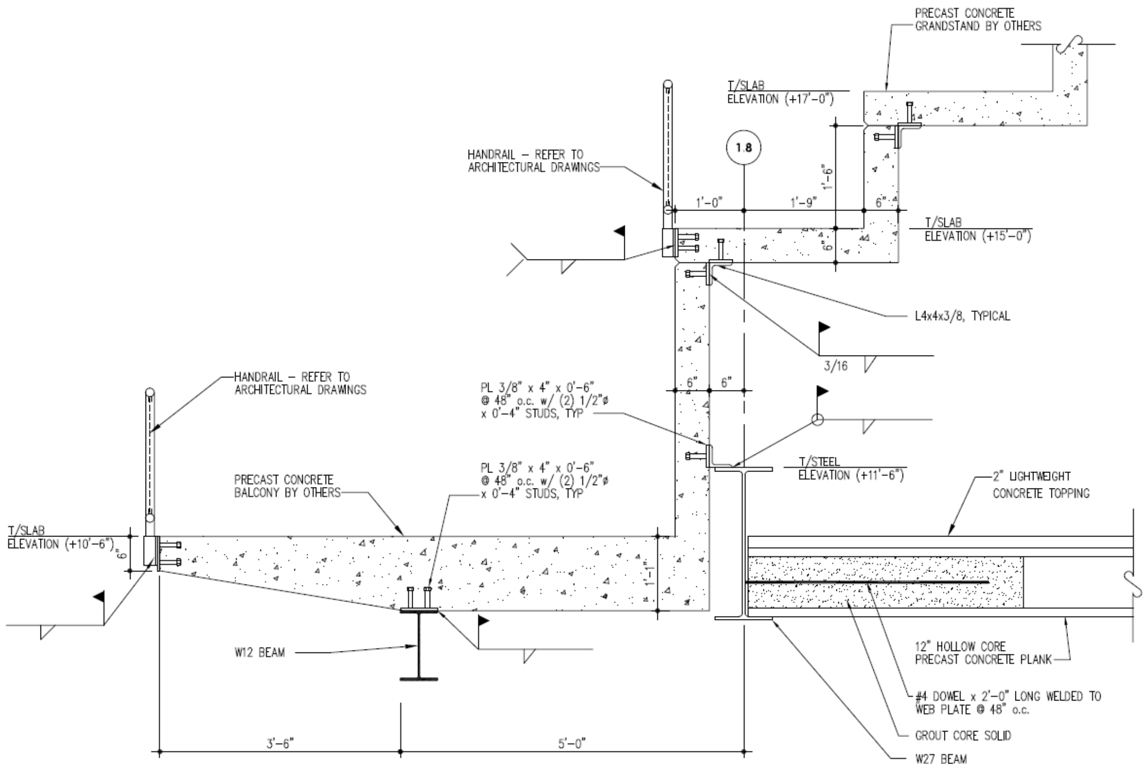


Figure 16 – Section showing 12" hollow core precast concrete planks bearing on bottom flange of W27x84 girder

Feasibility:

The hollow core plank system works very well for the high loads and spans of the floor system layout. Achieving a shallower floor system depth with other types of floor systems is rather difficult, making the hollow core planks seem like the best option for use in the natatorium.

One-Way Concrete Slab: Option #1

Material Properties:

Concrete:	16.5" slab (NWC)
	f'_c = 4,000 psi
Reinforcement:	f_y = 60,000 psi

Loading :

Dead (Self Weight) :	206.25 psf
Superimposed Dead:	15 psf
Live:	125 psf

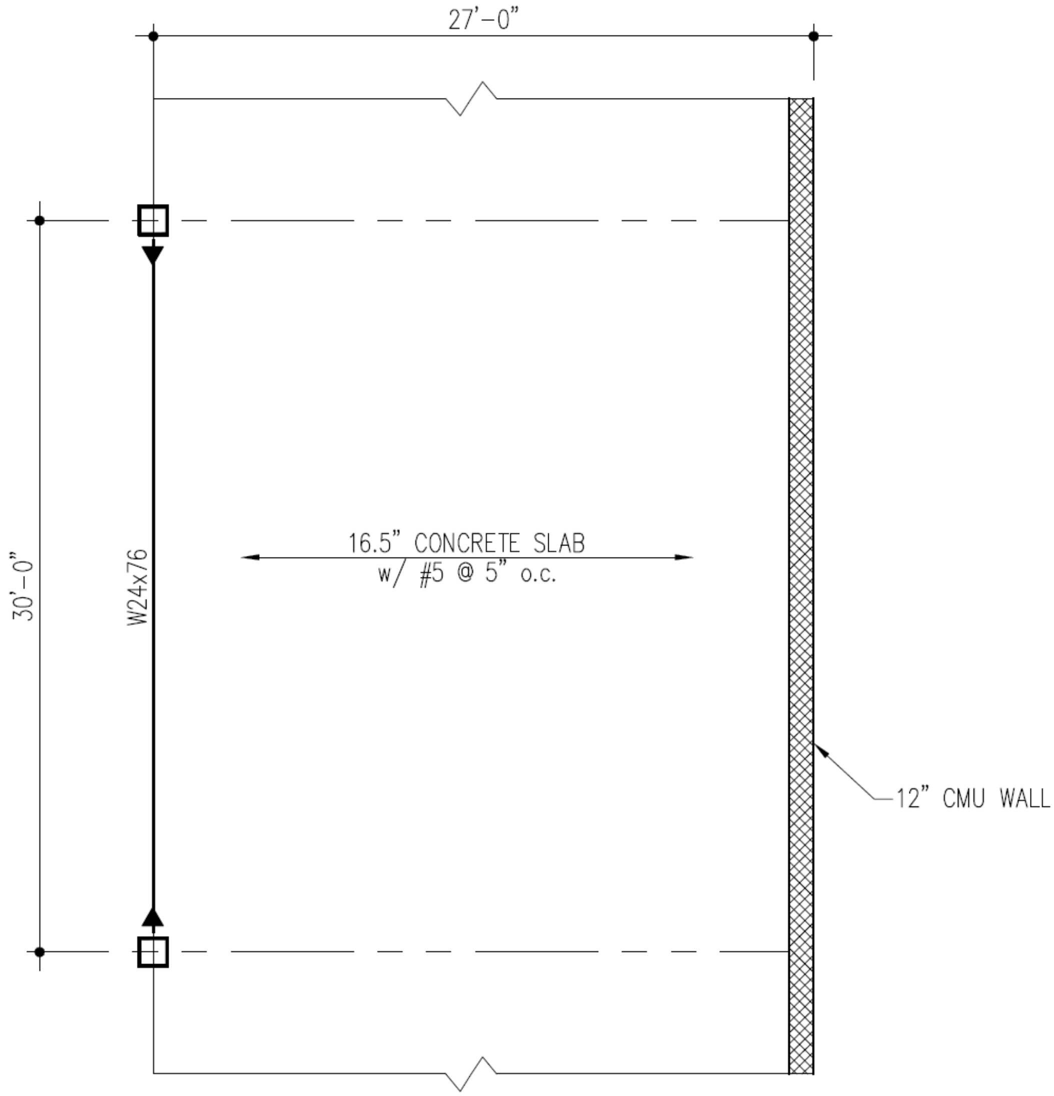


Figure 17 - One-Way Concrete Slab Layout

Description:

This one-way slab system was designed using a typical 27'-0" x 30'-0" bay, which can be seen in Figure 17. The slab spans 27'-0" and is supported by a steel girder on one end and by a 12" CMU wall on the other end, which seems to be the most common support conditions for the existing floor system. The one-way slab was designed according to ACI 318-08 using a 1-foot unit width and f'_c of 4,000 psi, which resulted in a 16.5" deep slab with #5 bars @ 5" o.c. No live load reductions were permitted since the live load of 125 psf is greater than 100 psf. Using a higher f'_c may have allowed the required slab thickness to decrease.

The AISC *Steel Construction Manual* was used to size the girder. The girder was designed as simply supported although the existing floor system shows moment connections at the corresponding girder. Treating the girder as simply supported was a conservative approach. Additional loads from the concrete balcony on the girder were not taken into account and may describe the size differences between this W24x76 girder and the W27x84 girder used in the existing floor system. Supporting calculations can be found in Appendix B.

Using one-way floor systems made the most sense with the floor layout of the natatorium. The floor basically spans from one support to the other and runs along the entire length of the building, as shown in Figure 2. Therefore, it did not make sense to try any two-way floor systems.

Advantages:

A one-way slab system is relatively simple to erect and does not require complex formwork. Since the slab is flat along the entire span, the cost of formwork is fairly low. The lead time for a one-way slab system is also relatively short. The slab provides a natural fire resistance, hence eliminating the need for extra fire protection and thus saving money. One-way slabs also offer an exposed flat ceiling upon which finishes can be easily applied. They are fairly common systems and do not require special expertise in the field as is required with, for example, post-tensioned systems. It seems that the one-way slab system would have a minimal impact on the lateral system of the natatorium, which is primarily a steel moment frame. In addition, the large depth of the required girder may not negatively affect the overall depth of the floor system if the slab bears on the bottom flange of the girder, as was done with the existing floor system and is shown in Figure 16.

Disadvantages:

The main disadvantage with this one-way slab system is the amount of material required. The 16.5" thick solid slab requires a large amount of concrete and hence becomes a very expensive system. Footings may have to be resized due to the much heavier floor system. The self-weight of the one-way slab is 206.25 psf, whereas the self-weight of the hollow core planks is 77 psf + 25 psf for the 2" topping. Hence, the self weight of the one-way slab is roughly twice that of the hollow core planks. It is difficult to get a relatively thin one-way slab with such a high live load, which was due to the mechanical room. Time is also required for formwork, pouring of the concrete, and curing of the concrete, which can lengthen the construction schedule. In addition, the overall depth of the floor system is much deeper than that of the existing system due to the size of the girders required. However, if the slab bears on the bottom flange of the beam, the depth of the one-way slab system will effectively be just the 16.5" depth of the slab. This is only 2.5" deeper than the existing 14" floor system depth (12" hollow core planks + 2" topping).

Feasibility:

Overall, a one-way slab system does not seem very feasible for this project. The excessive amount of material and associated high costs outweigh the benefits of this system. Adjusting the value of f'_c, however, may permit using a thinner slab and may be investigated further in the future. If the slab bears on the bottom flange of the girder, as was done in the existing floor system, the overall depth of the one-way slab system is fairly close to the 14" depth of the hollow core plank system. A one-way concrete joist slab system may be investigated further. These systems allow for a much thinner slab and are suitable for high loads and long spans. Although the floor system depth may increase due to the depth of the joists, a huge amount of material would be saved. The floor-to-ceiling height in the Farquhar Park Aquatic Center is not as critical of a design aspect as it would be in, say, a multi-story residential building. The only main upper floor of the building is at the concourse level, and it only really supports the mechanical room, a concession/team store, and restrooms. Overall, a one-way slab would probably work better at lower loads and shorter spans. In addition to a one-way concrete joist system, another possibility would be to span beams in the 27' direction and have a one-way slab span the other direction across a shorter span length.

Non-Composite Steel Frame: Option #2

Material Properties:

Concrete:	4.3” slab (3” topping)
	f'_c = 3,000 psi
Steel:	f_y = 50,000 psi
Reinforcement:	f_y = 60,000 psi
Metal Deck:	1.3C20 (3-span)

Loading:

Dead (Self Weight):	46 psf
Superimposed Dead:	15 psf
Live:	125 psf

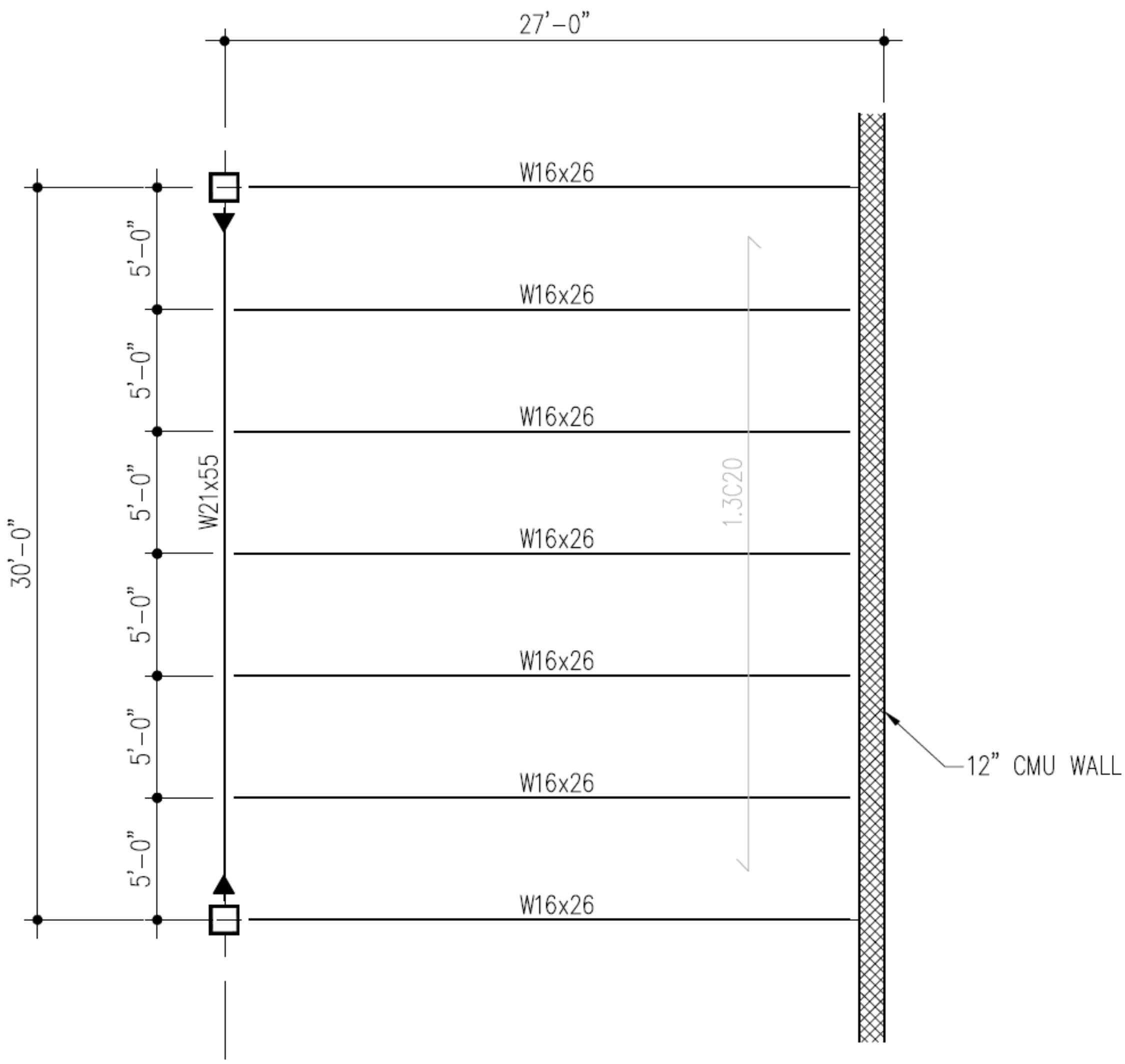


Figure 18 - Non-Composite Steel Layout

Description:

The non-composite steel frame system was designed using a typical bay of 27'-0" x 30'-0". Beams were evenly spaced at 5' on center spanning the 27'-0" direction, and beams and girders were designed using the AISC *Steel Construction Manual*. Member sizes can be seen in Figure 18. The Vulcraft 1.3C20 non-composite deck has a maximum construction clear span of 7'-11", which is greater than the 5'-0" clear span between beams. The corresponding total slab depth is 4.3 inches. The allowable uniform load on the 1.3C20 deck is sufficient for the applied loads. Columns were not yet designed at this stage.

Live load reductions were not permitted since the live load of 125 psf exceeded 100 psf. In addition, the additional load from the concrete balcony was not taken into consideration when designing the girder. However, the girder was conservatively designed as a simply supported member even though the existing floor system uses moment connections where the girders frame into the HSS columns. Supporting calculations for the slab and steel members can be found in Appendix C.

Advantages:

The non-composite steel frame system offers several benefits. One of the main advantages of the system is that it is much lighter than the possible concrete slab systems, and footings would not have to be resized. The overall cost of the system would be much lower than that of the concrete slab systems as well since very thick slabs are required for this floor system layout and applied loads. Lower costs would also be achieved since no shear studs are required. No formwork is required with the non-composite steel frame system, which results in a reduced labor cost. The system is rather quick and simple to erect, and there is no need for shoring due to the maximum construction clear span. The non-composite steel frame system would have a minimal impact on the natatorium's lateral system since the main lateral force resisting system is steel moment frames provided partly by the large trusses over the pool and other members. In addition, other systems in the building can take advantage of the dropped ceiling provided by the system.

Disadvantages:

One of the main disadvantages of the non-composite steel frame system is that the floor system depth would be much deeper than that provided by the existing hollow core planks. The W16x26 beams plus the deck and concrete create a floor system depth of 20.0 inches. If the top of the W21x55 girder is at the same elevation as the W16x26 beams, then the overall floor system depth would be further increased to 25.1" to account for the girder. Another main disadvantage is that the system would require additional fireproofing to achieve a 2-hour fire rating, which would increase costs. However, the drawings for the Farquhar Park Aquatic Center specify a 0-hour fire resistance rating for floor construction, so fireproofing may not be required. The non-composite steel frame system provides relatively poor vibration control. In addition, this system would require a longer lead time for the fabrication, detailing, and transportation of the steel.

Feasibility:

Overall, this system does not seem very feasible due to the large increase in floor system depth. Shallower beams and girders could be used to achieve the same floor system depth as that provided by the hollow core planks, but the steel members would be very heavy and the system would not be very cost efficient. Even trying steel joists would most likely increase the floor system depth. However, the fact that maintaining a certain floor-to-ceiling height is not a huge concern with the natatorium and that basically no fireproofing is required due to the 0-hour fire resistance rating for floor construction for this building makes the non-composite steel frame system seem fairly attractive. Plus, the system is much lighter and cheaper than the one-way slab system and uses much less concrete. Also, another option to investigate would be a composite steel system.

One-Way Post-Tensioned Slab: Option #3

Material Properties:

Concrete:	13" slab (NWC)
	f'_c = 5,000 psi
	f'_{ci} = 3,000 psi
Tendons:	Unbonded tendons
	½" diameter, 7-wire strands
	A_{pt} = 0.153 in^2
	f_{pu} = 270 ksi
Reinforcement:	f_y = 60,000 psi

Loading :

Dead (Self Weight) :	162.5 psf
Superimposed Dead:	15 psf
Live:	125 psf

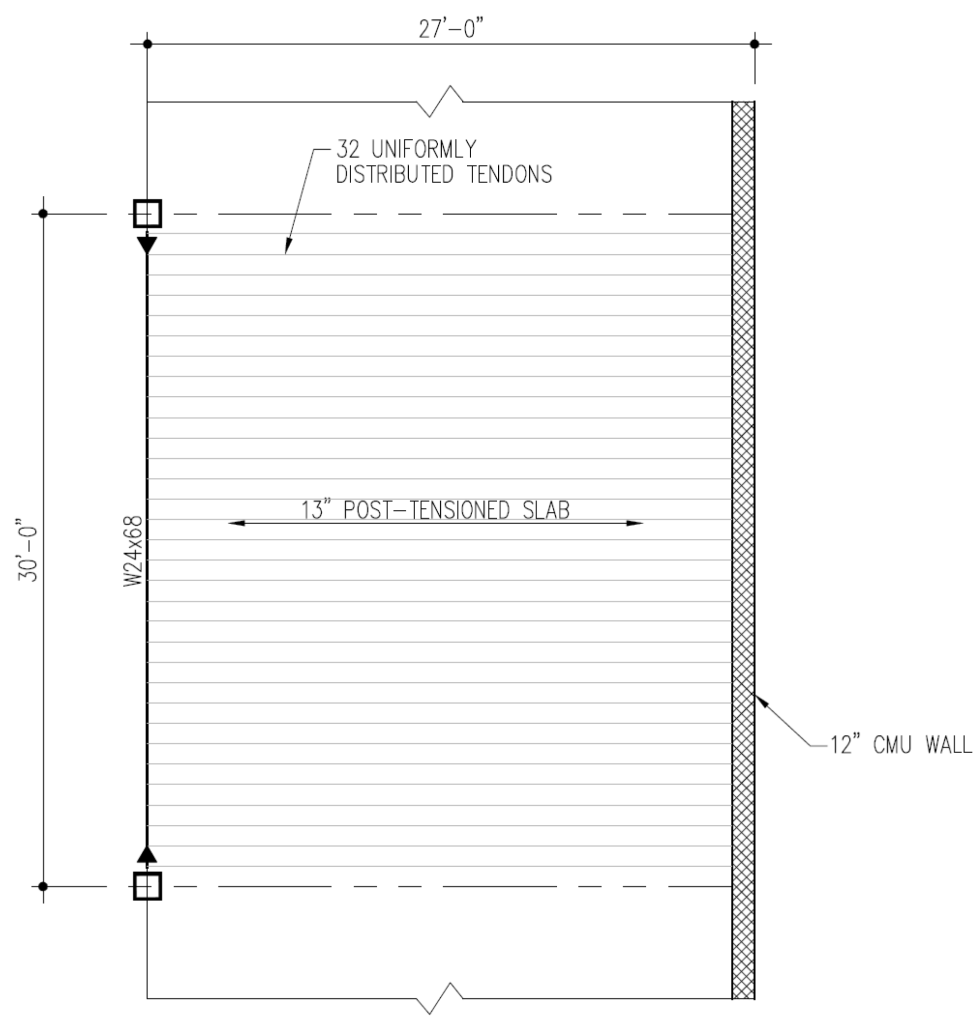


Figure 19 - One-Way Post-Tensioned Layout

Description:

A one-way post-tensioned slab was designed for a typical 27’-0” x 30’-0” bay, as can be seen in Figure 19. A design example provided by the Portland Cement Association was followed, along with additional information from Richard Apple’s presentation “Post-Tensioned Concrete: Practical Applications.” The slab was designed to span in the 27’-0” direction. An initial span/depth ratio of 40 was used, which is typical for one-way solid slabs with simple spans. The resulting initial slab depth was 8.5 inches. However, during the design the slab thickness had to be increased to 13 inches in order for the system to work for the bay size and applied loads. Thirty-two tendons, each providing 26.6 kips, were required in the 27’-0” direction. Additional reinforcing was also required and was provided by #5 @ 4” o.c. at bottom of midspan and (18) #4 at top at exterior supports.

The girder was designed using the AISC *Steel Construction Manual*. Additional loads from the concrete balcony on the girder were not taken into consideration. However, the girder was conservatively designed as a simply supported beam whereas the existing system shows moment connections at the ends of the corresponding girders.

No live load deflections were allowed since the live load of 125 psf was greater than 100 psf. Also, one of the initial steps required a design for Class C which states that stresses at service loads shall be calculated using the cracked transformed section. However, for simplicity of calculations, gross section properties were used for the design. Plus, deflection was not accounted for in the design, and further study is required for deflection calculations. Design calculations can be found in Appendix D.

Advantages:

One of the greatest advantages of this system is the relatively thin floor system depth achieved, especially for the high live loads applied to the floor. The 13” slab depth is actually thinner than the existing 14” floor system depth by 1 inch. This is the only system that allowed a thinner floor system depth to work. The thinner slab would save concrete and hence reduce material costs as well. The post-tensioned system also offers great vibration control. This system would have minor, if any, effects on the lateral system of the building since the main lateral force resisting system is a steel moment frame consisting of the large trusses over the pool and other members. Plus, no additional fireproofing would be required since the concrete slab is naturally fire resistant. This system provides a very high strength for long spans and seems to be the best possible option for an alternate floor system.

Disadvantages:

Along with the outstanding advantages provided by this system, it also has its drawbacks. The main disadvantage of the post-tensioned system is that it is potentially dangerous and much caution must be taken in the field during construction. Special expertise is required with the erection of post-tensioned systems. Workers must also be careful if cutting into the concrete slab because they may cut a tendon. This system also requires formwork and shoring, which costs both time and money. Plus, the self weight of the slab may require the footings to be resized.

Feasibility:

The post-tensioned system seems to be the most viable option as an alternate floor system. No other system was able to actually achieve a thinner floor system depth than that provided by the hollow core planks, especially with the high live load applied to the floor. The thinner slab will also save in material costs of concrete. More research and investigation into this system will be performed. There are many variables involved with post-tensioned design that affect the resulting slab thickness including the tendon profile, the value of eccentricity, percentage of the self weight of the slab to use for target load balances, and the number of tendons to use. With more experience, an even thinner slab and more cost effective solution may be able to be achieved.

System Comparison:

Comparison Criteria	Existing System Precast Concrete Hollow Core Planks	Option #1 One-Way Slab	Option #2 Non-Composite Steel Frame	Option #3 One-Way Post-Tensioned Slab
Slab Self Weight	77 psf	206.25 psf	46 psf	162.5 psf
Slab Depth	12"	16.5"	4.3"	13"
System Depth	14"	16.5"	25.1"	13"
Vibration Control	Further Study Required	Good	Poor	Excellent
Fire Rating	2 hour	1.5 - 2 hour	1.5 - 2 hour	2 hour
Fire Protection	None	None	Spray	None
Architectural Impact	Existing	Negative: Reduces Floor-to-Ceiling Height	Negative: Reduces Floor-to-Ceiling Height	Positive: Increases Floor-to-Ceiling Height
Constructability	Easy	Medium	Easy	Difficult
Formwork	No	Yes	No	Yes
Lead Time	Long (for Steel Framing)	Short	Long	Short
System Cost	$13.46/SF	$32.89/SF	$12.72/SF	$28.85/SF
Feasibility	Yes	No	Possibly	Yes

Table 4 – System Comparison Summary

Notes:
*Drawings specify a 0-hour fire resistance rating for floor construction
*System costs estimated using RS Mean Building Construction Cost Data and RS Means Square Foot Costs

Conclusion:

Overall, it is evident that the best floor system for the Farquhar Park Aquatic Center natatorium was the existing precast concrete hollow core plank system with 2” topping. The hollow core planks offered a relatively thin and light weight floor system for the high applied loads and spans. The estimated cost of this system was less than half as much as the estimated cost of the one-way slab, as can been seen in Table 4. It even appeared that a thinner hollow core plank system could have been used. Achieving an equivalent floor system depth in a cost effective manner using an alternate floor system was very difficult. The high live load, and the fact that no live load reductions were allowed since the live load was greater than 100 psf, somewhat limited the possible options for an alternate floor system. Most systems that were initially investigated required an increased floor system depth. Hence, a post-tensioned system was investigated to see if achieving a thinner floor system depth was possible.

The post-tensioned system seems like the best candidate for an alternate floor system. This was the only system that was, in fact, actually able to achieve a thinner floor system depth than that provided by the hollow core planks with 2” topping. Post-tensioned systems are excellent for long spans and high loads, and they provide great vibration control. The one-way slab was not a very feasible option simply due to the large required slab thickness and corresponding cost of concrete. The floor system depth provided by the one-way slab was actually only 2.5” deeper than the 14” floor system depth provided by the hollow core planks with 2” topping. However, the one-way slab was solid whereas the hollow core planks had a large percentage of concrete removed, making the planks much lighter and cheaper. The non-composite steel system, on the other hand, has potential as a possible alternate floor system. This system is relatively cheap and lightweight, especially when compared to the cost of the one-way slab.

The post-tensioned system will be further investigated. Despite the potential dangers present during the construction process, the advantages provided by the post-tensioned system make it the best option for an alternate floor system. A non-composite steel system may also prove to be beneficial, and a composite steel system may be investigated as well. In addition, a one-way concrete joist slab system seems to provide several benefits and may also be investigated. A very thin slab may be able to be achieved, which would save a great deal of money in material costs. The depth of the joists may increase the floor system depth, however, the floor-to-ceiling height is not an extremely critical design aspect in the natatorium. This system is often a cost effective solution for higher loads and longer spans.

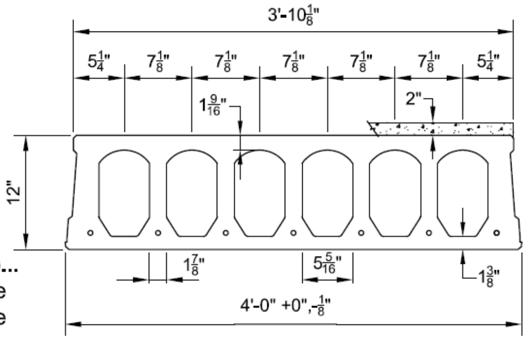
Appendix A – Precast Concrete Hollow Core Planks

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

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 361 in.^{2}	Precast b_w = 14.25 in.
I_c = 7840 in.^{4}	Precast S_{bcp} = 1081 in.^{3}
Y_{bcp} = 7.26 in.	Topping S_{tct} = 1644 in.^{3}
Y_{tcp} = 4.74 in.	Precast S_{tcp} = 1653 in.^{3}
Y_{tct} = 6.74 in.	Precast Wt. = 308 PLF
	Precast Wt. = 77.00 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 and 0.6"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   6-1/2"Ø, 270K = 205.4 k-ft at 60% jacking force
   7-1/2"Ø, 270K = 235.4 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. All load values are controlled by ultimate flexural strength or fire endurance limits.
14. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
15. 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)																		
		32	33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48	48	50
6 - 1/2"ø	LOAD (PSF)	133	119	107	95	84	74	65	56	49	41	34								
7 - 1/2"ø	LOAD (PSF)	170	154	139	125	113	101	91	81	72	63	56	48	42						

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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.

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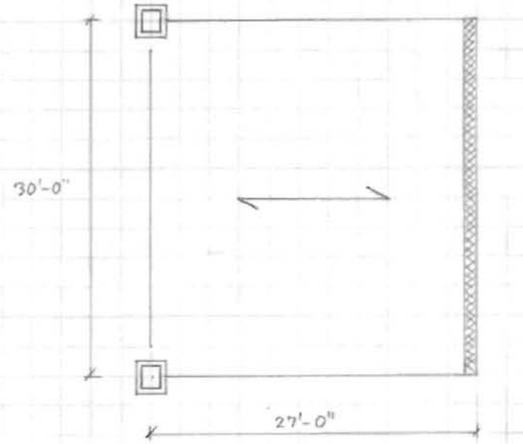
Precast Concrete Hollow Core Planks



$f'_c = 6000$ psi

$f'_{ci} = 3500$ psi

· Loads

Superimposed DL = 15 psf

Self Weight of Precast Planks = 77 psf

2" Topping DL: 25 psf

LL = 125 psf

Superimposed Service Load = 15 psf + 25 psf + 125 psf = 165 psf

· Load Tables from Nitterhouse Concrete Products

12" x 4'-0" Hollow Core Planks with 2" Topping were used (manufactured by Nitterhouse Concrete Products)

Use load table for 2-hour fire resistance rating → most conservative (versus 1-hour fire resistance rating)

* Drawings actually specify a 0-hour fire resistance rating for floor construction (allowed by code)

(7) ½" Φ strands, Low-Relaxation

$f_{pu} = 270{,}000$ psi

Span = 27'

Load Table only goes down to a span of 32'

At span = 32' → 170 psf > 165 psf

∴ OK for a span of 27'

· Check Girder → Checked as simply supported to be conservative (existing has moment connections at ends)

$W_u = 1.2D + 1.6L = (1.2)(15\text{ psf} + 25\text{ psf} + 77\text{ psf}) + (1.6)(125\text{ psf}) = 340.4\text{ psf}$

$W_u = (340.4\text{ psf})\left(\frac{27'}{2}\right) = 4595.4\text{ lb/ft} = 4.595\text{ k/ft}$

$M_u = \frac{w_u L^2}{8} = \frac{(4.595\text{ k/ft})(30')^2}{8} = 516.938\text{ 'k}$

W27x84 → Assume fully braced → $\phi M_n = 915\text{ 'k} > M_u = 516.938\text{ 'k}$ ∴ OK

$\Delta_{LL} = \frac{5 w_L L^4}{384 EI} \leq \frac{L}{360} = \frac{(30')(12\text{ in/ft})}{360} = 1.0''$

$w_L = (125\text{ psf})\left(\frac{27'}{2}\right) = 1687.5\text{ lb/ft} = 1.6875\text{ k/ft}$

$\Delta_{LL} = \frac{(5)(1.6875\text{ k/ft})(30')^4(1728)}{(384)(29000\text{ ksi})(2850\text{ in}^4)} = 0.372'' < 1.0''$ ∴ OK

$\Delta_{TL} = \frac{5 w_{TL} L^4}{384 EI} \leq \frac{L}{240} = \frac{(30')(12\text{ in/ft})}{240} = 1.5''$

$W_{TL} = (15\text{ psf} + 25\text{ psf} + 77\text{ psf} + 125\text{ psf})\left(\frac{27'}{2}\right) = 3267\text{ lb/ft} = 3.267\text{ k/ft}$

$\Delta_{TL} = \frac{(5)(3.267\text{ k/ft})(30')^4(1728)}{(384)(29000\text{ ksi})(2850\text{ in}^4)} = 0.720'' < 1.5''$ ∴ OK

· Check Bearing on 12" CMU Wall

Bearing Area $= (30')\left(\frac{12''}{12\text{ in/ft}}\right) = 30\text{ ft}^2 = 4320\text{ in}^2$

$P_u = (340.4\text{ psf})\left(\frac{27'}{2}\right)(30') = 137{,}862\text{ lb} = 137.862^k$

Bearing Capacity $= \phi(0.85 f'_c A_b) = (0.65)[(0.85)(2000\text{ psi})(4320\text{ in}^2)] =$
↑ for masonry is 2000 psi (specified on drawings)

$= 4{,}773{,}600\text{ lb} = 4773.6^k > 137.182^k$ ∴ OK

· Hollow Core Plank Deflection Check

$\Delta_{LL} = \frac{5 w_L L^4}{384 EI} = \frac{(5)[(0.125\text{ ksf})(4')](27')^4(1728)}{(384)(29000\text{ ksi})(7840\text{ in}^4)} = 0.0263'' \leq \frac{L}{360} = \frac{(27')(12\text{ in/ft})}{360} = 0.9''$ ∴ OK

$\Delta_{TL} = \frac{5 w_{TL} L^4}{384 EI} = \frac{(5)[(0.015\text{ ksf} + 0.077\text{ ksf} + 0.025\text{ ksf} + 0.125\text{ ksf})(4')](27')^4(1728)}{(384)(29000\text{ ksi})(7840\text{ in}^4)} = 0.0509''$

$\leq \frac{L}{240} = \frac{(27')(12\text{ in/ft})}{240} = 1.35''$ ∴ OK

- It appears that a thinner hollow-core plank could have been used:

Using load tables from Nitterhouse Concrete Products

10" x 4'-0" Hollow Core Plank with 2" topping

2-hour fire rating

(7) $\frac{1}{2}$" ϕ strands

Span = 27' → 222 psf > 165 psf ∴ OK

8" x 4'-0" Hollow Core Plank with 2" topping

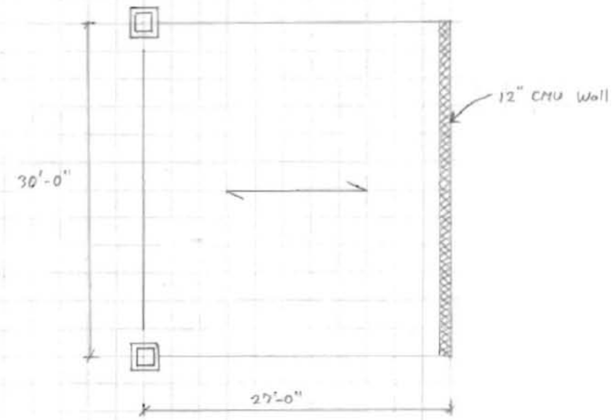
2-hour fire rating

(7) $\frac{1}{2}$" ϕ strands

Span = 27' → 161 psf < 165 psf ∴ Not OK

Appendix B – One-Way Concrete Slab

One-Way Concrete Slab



$f'_c = 4{,}000$ psi

$f_y = 60$ ksi

• Loads:

Superimposed DL = 15 psf

LL = 125 psf

• ACI 318-08 Table 9.5 (a) → Simply Supported Slab

$h_{min} = \frac{\ell}{20} = \frac{(27')(12 \text{ in/ft})}{20} = 16.2"$

Try $h = 16.5"$

• Factored Loads:

$w_L = 125$ psf

$w_{sw} = \left(\frac{16.5"}{12 \text{ in/ft}}\right)(150 \text{ pcf}) = 206.25$ psf

$w_D = 15 \text{ psf} + 206.25 \text{ psf} = 221.25$ psf

$w_u = 1.2D + 1.6L = (1.2)(221.25 \text{ psf}) + (1.6)(125 \text{ psf}) = 465.5$ psf ← Governs

$w_u = 1.4D = (1.4)(221.25 \text{ psf}) = 309.75$ psf

Per one-foot width → $w_u = (465.5 \text{ psf})(1') = 465.5 \text{ lb/ft} = 0.4655 \text{ k/ft}$

· Design Moment:

$$M_u = \frac{w_u L^2}{8} = \frac{(0.4655\ ^{k}/ft)(27')^2}{8} = 42.419\ ^{'k} \text{ per 1-ft width of slab}$$

· Estimate A_s:

Assume #5 bars $\rightarrow$ $d = 16.5'' - 0.75'' - \frac{5}{16}'' = 15.438''$

$$A_s = \frac{M_u}{4d} = \frac{42.419\ ^{'k}}{(4)(15.438'')} = 0.689\ ^{in^2}/ft$$

Provide #5 @ 5" o.c. $\rightarrow$ $A_s = (0.31\ in^2)\left(\frac{12\ ^{in}/ft}{5''}\right) = 0.744\ ^{in^2}/ft > 0.689\ ^{in^2}/ft$

· Check $\phi M_n > M_u$

Assume $f_s > f_y$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.744\ in^2)(60\ ksi)}{(0.85)(4\ ksi)(12'')} = 1.094''$$

$$c = \frac{a}{\beta_1} = \frac{1.094''}{0.85} = 1.287''$$

$$f_s = \frac{\varepsilon_u}{c}(d-c) = \left(\frac{0.003\ ^{in}/in}{1.287''}\right)(15.438'' - 1.287'') = 0.03299\ ^{in}/in > 0.005\ ^{in}/in$$

$\therefore$ Tension Controlled $\rightarrow$ $\phi = 0.9$

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right) = (0.9)(0.744\ in^2)(60\ ksi)\left(15.438'' - \frac{1.094''}{2}\right)\Big/_{12\ ^{in}/ft} = 49.853\ ^{'k}$$

$\phi M_n = 49.853\ ^{'k} > M_u = 42.419\ ^{'k}$ $\therefore$ OK

· Temperature and Shrinkage Reinforcement (ACI 318-08 §7.12.2.1

$A_t = 0.0018\,bh = (0.0018)(12'')(16.5'') = 0.356\ ^{in^2}/ft$

Use #5 @ 10" o.c. $\rightarrow$ $A_t = (0.31\ in^2)\left(\frac{12\ ^{in}/ft}{10''}\right) = 0.372\ ^{in^2}/ft > 0.356\ ^{in^2}/ft$ $\therefore$ OK

· Crack Control

$s \leq 15 - 2.5\,c_c = 15'' - (2.5)(0.75'') = 13.125''$

$s = 5'' < 13.125''$ $\therefore$ OK

Use slab thickness = 16.5" with #5 @ 5" o.c.

· Slab Deflection

OK since $h = 16.5'' > \frac{l}{20} = \frac{(27')(12\ ^{in}/ft)}{20} = 16.2''$ (ACI 318-08, Table 9.5(a))

· Design Girder → Design as simply supported to be conservative (existing has moment connections at ends)

$w_u = 1.2D + 1.6L = (1.2)(15\text{ psf} + 206.25\text{ psf}) + (1.6)(125\text{ psf}) = 465.5\text{ psf}$

$w_u = (465.5\text{ psf})\left(\frac{27'}{2}\right) = 6284.25\text{ lb/ft} = 6.2843\text{ k/ft}$

$V_u = \frac{(6.2843\text{ k/ft})(30')}{2} = 94.265^k$

$M_u = \frac{w_u L^2}{8} = \frac{(6.2843\text{ k/ft})(30')^2}{8} = 706.984^{\text{'k}}$

Assume fully braced

$Z_{req} = \frac{M_u}{\phi F_y} = \frac{(706.984^{\text{'k}})(12\text{ in/ft})}{(0.9)(50\text{ ksi})} = 188.529\text{ in}^3$

Try W24x76 ($Z_x = 200\text{ in}^3 > 183.529\text{ in}^3 \therefore OK$)

$\phi V_n = 316^k > V_u = 94.265^k \therefore OK$

$\Delta_{LL} = \frac{5 w_L L^4}{384EI} \le \frac{L}{360} = \frac{(30')(12\text{ in/ft})}{360} = 1.0''$

$w_L = (125\text{ psf})\left(\frac{27'}{2}\right) = 1687.5\text{ lb/ft} = 1.6875\text{ k/ft}$

$I_{req} = \frac{(5)(1.6875\text{ k/ft})(30')^4(1728)}{(384)(29000\text{ ksi})(1.0'')} = 1060.51\text{ in}^4$

W24x76 is OK ($I_x = 2100\text{ in}^4 > 1060.51\text{ in}^4 \therefore OK$)

$\Delta_{TL} = \frac{5 w_{TL} L^4}{384EI} \le \frac{L}{240} = \frac{(30')(12\text{ in/ft})}{240} = 1.5''$

$w_{TL} = (15\text{ psf} + 206.25\text{ psf} + 125\text{ psf})\left(\frac{27'}{2}\right) = 4674.375\text{ lb/ft} = 4.6744\text{ k/ft}$

$I_{req} = \frac{(5)(4.6744\text{ k/ft})(30')^4(1728)}{(384)(29000\text{ ksi})(1.5'')} = 1958.41\text{ in}^4$

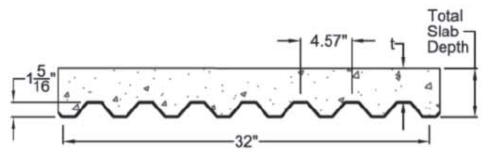
W24x76 is OK ($I_x = 2100\text{ in}^4 > 1958.41\text{ in}^4 \therefore OK$)

$\boxed{\text{Use W24x76 for girder}}$

Appendix C – Non-Composite Steel Frame

VULCRAFT

1.3 C, CSV CONFORM



NON-COMPOSITE

MAXIMUM CONSTRUCTION CLEAR SPANS (S.D.I. CRITERIA)

Total Slab Depth	DECK	WEIGHT PSF	NW CONCRETE N=9 145 PCF 1 SPAN	NW CONCRETE 2 SPAN	NW CONCRETE 3 SPAN	WEIGHT PSF	LW CONCRETE N=14 110 PCF 1 SPAN	LW CONCRETE 2 SPAN	LW CONCRETE 3 SPAN
3.3 (t=2.00)	1.3C26	33	4-6	5-11	6-0	25	4-10	6-4	6-5
	1.3C24	34	5-6	7-4	7-5	26	6-0	7-11	8-0
	1.3C22	34	6-4	8-3	8-3	26	6-11	8-10	9-0
	1.3C20	34	7-1	8-9	8-9	26	7-9	9-7	9-7
3.8 (t=2.50)	1.3C26	39	4-3	5-7	5-8	30	4-7	6-1	6-2
	1.3C24	40	5-3	6-11	7-0	30	5-8	7-7	7-8
	1.3C22	40	6-0	7-10	7-10	31	6-7	8-6	8-6
	1.3C20	40	6-9	8-4	8-4	31	7-4	9-1	9-1
4.3 (t=3.00)	1.3C26	45	4-1	5-5	5-5	35	4-5	5-10	5-11
	1.3C24	46	5-0	6-8	6-9	35	5-5	7-3	7-4
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	35	6-3	8-2	8-2
	1.3C20	46	6-5	7-11	7-11	36	7-0	8-8	8-8
4.8 (t=3.50)	1.3C26	51	3-11	5-2	5-3	39	4-3	5-8	5-8
	1.3C24	52	4-9	6-4	6-5	40	5-3	6-11	7-0
	1.3C22	52	5-5	7-2	7-2	40	6-0	7-10	7-10
	1.3C20	52	6-1	7-7	7-7	40	6-9	8-4	8-4
5.3 (t=4.00)	1.3C26	57	3-9	5-0	5-1	44	4-1	5-5	5-6
	1.3C24	58	4-7	6-2	6-2	44	5-0	6-9	6-10
	1.3C22	58	5-3	6-11	6-11	44	5-9	7-6	7-6
	1.3C20	58	5-10	7-4	7-4	45	6-6	8-0	8-0
5.8 (t=4.50)	1.3C26	63	3-8	4-9	4-11	48	4-0	5-3	5-4
	1.3C24	64	4-5	5-11	6-0	49	4-10	6-6	6-7
	1.3C22	64	5-1	6-8	6-8	49	5-7	7-3	7-3
	1.3C20	64	5-8	7-1	7-1	49	6-3	7-9	7-9
6.3 (t=5.00)	1.3C26	69	3-7	4-5	4-9	53	3-10	5-2	5-2
	1.3C24	70	4-4	5-9	5-10	53	4-8	6-4	6-5
	1.3C22	70	4-11	6-6	6-6	54	5-4	7-1	7-1
	1.3C20	70	5-6	6-11	6-11	54	6-0	7-6	7-6

REINFORCED CONCRETE SLAB ALLOWABLE LOADS

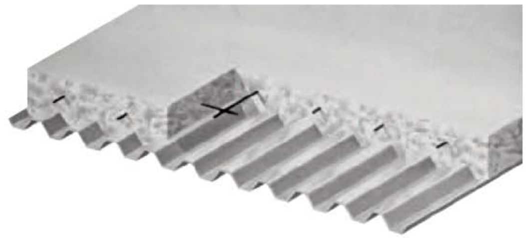
Slab Depth	REINFORCEMENT W.W.F.	As	Superimposed Uniform Load (psf) — 3 Span Condition, Clear Span (ft.-in.) 4-0	4-6	5-0	5-6	6-0	6-6	7-0	7-6	8-0	8-6	9-0
3.3	6X6-W1.4XW1.4	0.028*	71	56									
	6X6-W2.1XW2.1	0.042	105	83									
(t=2.00)	6X6-W2.9XW2.9	0.058	142	113									
3.8	6X6-W2.1XW2.1	0.042*	133	105	85	70							
	6X6-W2.9XW2.9	0.058	181	143	116	96							
(t=2.50)	4X4-W2.9XW2.9	0.087	265	209	169	140							
4.3	6X6-W2.1XW2.1	0.042*	161	127	**156**	**129**	**108**	**92**	**79**				
	6X6-W2.9XW2.9	0.058*	219	173	**209**	**173**	**145**	**124**	**107**				
(t=3.00)	4X4-W2.9XW2.9	0.087	322	255	**309**	**255**	**215**	**183**	**158**				
4.8	6X6-W2.1XW2.1	0.042*	188	149	**191**	**158**	**133**	**113**	**98**	**85**			
	6X6-W2.9XW2.9	0.058*	258	204	**258**	**213**	**179**	**153**	**132**	**115**			
(t=3.50)	4X4-W2.9XW2.9	0.087	380	300	**383**	**316**	**266**	**226**	**195**	**170**			
5.3	6X6-W2.9XW2.9	0.058*	296	234	**299**	**247**	**208**	**177**	**153**				
	4X4-W2.9XW2.9	0.087	400	346	**400**	**364**	**306**	**260**	**225**				
(t=4.00)	4X4-W4.0XW4.0	0.120	400	400	**400**	**400**	**400**	**347**	**299**				
5.8	6X6-W2.9XW2.9	0.058*	334	264	**336**	**278**	**233**	**199**	**172**				
	4X4-W2.9XW2.9	0.087*	400	391	**400**	**400**	**344**	**293**	**253**				
(t=4.50)	4X4-W4.0XW4.0	0.120	400	400	**400**	**400**	**400**	**392**	**338**				
6.3	6X6-W2.9XW2.9	0.058*	373	295	**373**	**308**	**259**	**221**					
	4X4-W2.9XW2.9	0.087*	400	400	**400**	**400**	**382**	**326**					
(t=5.00)	4X4-W4.0XW4.0	0.120	400	400	**400**	**400**	**400**	**400**					

NOTES:
1. * As does not meet A.C.I. criterion for temperature and shrinkage.
2. Recommended conform types are based upon S.D.I. criteria and normal weight concrete.
3. Superimposed loads are based upon three span conditions and A.C.I. moment coefficients.
4. Load values for single span and double spans are to be reduced.
5. Vulcraft's painted or galvanized form deck can be considered as permanent support in most building applications. See page 23.
   If uncoated form deck is used, deduct the weight of the slab from the allowable superimposed uniform loads.
6. Superimposed load values shown in bold type require that mesh be draped. See page 23.

VULCRAFT

SLAB INFORMATION

Total Slab Depth, in.	Theo. Concrete Volume Yd³ / 100 ft²	ft³ / ft²	Recommended Welded Wire Fabric
3.3	0.82	0.221	6x6 - W1.4xW1.4
3.8	0.97	0.263	6x6 - W1.4xW1.4
4.3	1.13	0.304	6x6 - W1.4xW1.4
4.55	1.20	0.325	6x6 - W1.4xW1.4
4.8	1.28	0.346	6x6 - W2.1xW2.1
5.3	1.44	0.388	6x6 - W2.1xW2.1
5.55	1.51	0.408	6x6 - W2.1xW2.1
5.8	1.59	0.429	6x6 - W2.1xW2.1



SECTION PROPERTIES

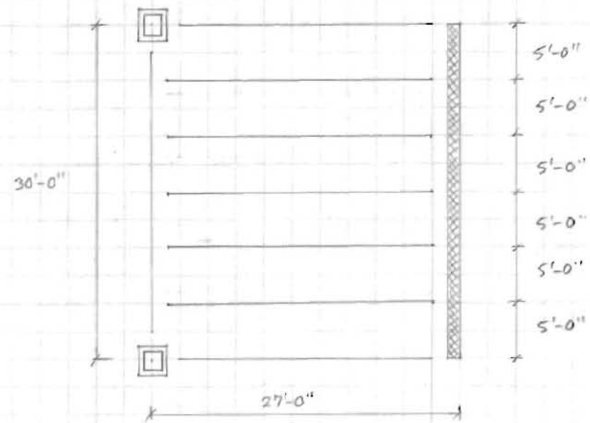
Deck Type	Design Thickness in.	Deck Weight psf	Section Properties I_p in⁴/ft	I_n in⁴/ft	S_p in³/ft	S_n in³/ft	V_a lbs/ft	F_y ksi
1.3C26	0.0179	0.99	0.070	0.069	0.097	0.098	1940	60
1.3C24	0.0239	1.33	0.093	0.093	0.132	0.132	3458	60
1.3C22	0.0295	1.62	0.115	0.115	0.163	0.162	4789	60
1.3C20	0.0358	1.97	0.140	0.140	0.197	0.197	5727	60

NON-COMPOSITE

ALLOWABLE UNIFORM LOAD (PSF)

TYPE NO.	NO. OF SPANS	DESIGN CRITERIA	CLEAR SPAN (ft-in) 4-0	4-6	5-0	5-6	6-0	6-6	7-0	7-6	8-0	8-6	9-0	9-6	10-0
1.3C26	1	Fb = 36,000	145	115	93	77	65	55	47	41	36	32	29	26	23
		Defl. = l/240	72	50	37	28	21	17	13	11	9	7	6	5	5
		Defl. = l/180	96	67	49	37	28	22	18	15	12	10	8	7	6
	2	Fb = 36,000	144	114	93	77	65	55	48	42	37	32	29	26	23
		Defl. = l/240	172	121	88	66	51	40	32	26	21	18	15	13	11
		Defl. = l/180	229	161	117	88	68	53	43	35	29	24	20	17	15
	3	Fb = 36,000	179	142	115	96	81	69	59	52	46	40	36	32	29
		Defl. = l/240	134	94	69	52	40	31	25	20	17	14	12	10	9
		Defl. = l/180	179	126	92	69	53	42	33	27	22	19	16	13	11
1.3C24	1	Fb = 36,000	198	156	126	105	88	75	65	56	49	44	39	35	32
		Defl. = l/240	95	67	49	37	28	22	18	14	12	10	8	7	6
		Defl. = l/180	127	89	65	49	38	30	24	19	16	13	11	9	8
	2	Fb = 36,000	196	155	126	104	87	75	64	56	49	44	39	35	32
		Defl. = l/240	230	161	118	88	68	54	43	35	29	24	20	17	15
		Defl. = l/180	306	215	157	118	91	71	57	46	38	32	27	23	20
	3	Fb = 36,000	243	193	157	130	109	93	80	70	62	55	49	44	39
		Defl. = l/240	180	126	92	69	53	42	34	27	22	19	16	13	12
		Defl. = l/180	240	168	123	92	71	56	45	36	30	25	21	18	15
1.3C22	1	Fb = 36,000	244	193	156	129	108	92	80	69	61	54	48	43	39
		Defl. = l/240	118	83	60	45	35	27	22	18	15	12	10	9	8
		Defl. = l/180	157	110	81	61	47	37	29	24	20	16	14	12	10
	2	Fb = 36,000	241	190	154	128	107	92	79	69	61	54	48	43	39
		Defl. = l/240	284	199	145	109	84	66	53	43	36	30	25	21	18
		Defl. = l/180	379	266	194	146	112	88	71	57	47	39	33	28	24
	3	Fb = 36,000	300	237	193	159	134	114	99	86	76	67	60	54	48
		Defl. = l/240	222	156	114	86	66	52	41	34	28	23	20	17	14
		Defl. = l/180	296	208	152	114	88	69	55	45	37	31	26	22	19
1.3C20	1	Fb = 36,000	295	233	189	156	131	112	96	84	74	65	58	52	47
		Defl. = l/240	144	101	74	55	43	33	27	22	18	15	13	11	9
		Defl. = l/180	192	135	98	74	57	45	36	29	24	20	17	14	12
	2	Fb = 36,000	292	232	188	155	131	111	96	84	74	65	58	52	47
		Defl. = l/240	346	243	177	133	102	81	65	52	43	36	30	26	22
		Defl. = l/180	[illegible]	[illegible]	[illegible]	177	137	107	86	70	58	48	40	34	30
	3	Fb = 36,000	364	289	234	194	163	139	120	105	92	81	73	65	59
		Defl. = l/240	271	190	139	104	80	63	50	41	34	28	24	20	17
		Defl. = l/180	361	253	185	139	107	84	67	55	45	38	32	27	23

Non- Composite Steel



- Loads

Superimposed Dead Load = 15 psf

LL = 125 psf

- Steel Deck → Vulcraft

1.3 C_1 CSV Conform

Reinforced Concrete Slab Allowable Load = 15 psf + 125 psf = 140 psf

Clear Span = 5'-0"

Slab Depth = 4.3" (t = 3.00")

3-Span (use for 3 or more spans) → 1.3C20

1.3 C20 → Maximum Construction Clear Span = 7'-11" > 5'-0" ∴ OK

Weight = 46 psf

Allowable Uniform Load = 15 psf + 46 psf + 125 psf = 186 psf

1.3C20 → No. of Spans = 3 → Clear Span = 5'-0"

F_b = 36,000, 20 GA, Allowable Load = 234 psf > 186 psf ∴ OK

$\frac{L}{240}$, 20 GA, Allowable Load = 139 psf > 125 psf ∴ OK

$\frac{L}{360}$, 20 GA, Allowable Load = 185 psf > 46 psf ∴ OK

Use 1.3C20 Deck (1.3C, CSV Conform) with 4.3" slab depth

Other possible decks to use that would work:

- 1.5C20 with 4.5" slab depth → Max. Construction Clear Span = 8'-3"
- 2C22 with 5" slab depth → Max. Construction Clear Span = 8'-11"

$\boxed{\text{Use 1.3C20 Deck (1.3 C, CSV Conform) with 4.3" slab depth}}$

since smaller slab depth, and not much difference in gauge compared to the 22GA deck

- Beams

$W_u = 1.2D + 1.6L = (1.2)(15\text{ psf} + 46\text{ psf}) + (1.6)(125\text{ psf}) = 273.2\text{ psf}$

$w_u = (273.2\text{ psf})(5') = 1366\text{ lb/ft} = 1.366\text{ k/ft}$

$V_u = \frac{(1.366\text{ k/ft})(27')}{2} = 18.441^k$

$M_u = \frac{w_u L^2}{8} = \frac{(1.366\text{ k/ft})(27')^2}{8} = 124.477^{'k}$

Assume fully braced

$Z_{req} = \frac{M_u}{\phi F_y} = \frac{(124.477'^k)(12\text{ in/ft})}{(0.9)(50\text{ ksi})} = 33.194\text{ in}^3$

Try W14x22 ($Z_x = 33.2\text{ in}^3 > 33.194\text{ in}^3$)

$\Delta_{LL} = \frac{5 w_L L^4}{384EI} \leq \frac{L}{360} = \frac{(27')(12\text{ in/ft})}{360} = 0.9''$

$w_L = (125\text{ psf})(5') = 625\text{ lb/ft} = 0.625\text{ k/ft}$

$I_{req} = \frac{(5)(0.625\text{ k/ft})(27')^4(1728)}{(384)(29000\text{ ksi})(0.9'')} = 286.34\text{ in}^4$

W14x22 doesn't work ($I_x = 199\text{ in}^4 < 286.34\text{ in}^4$)

Try W16x26 ($I_x = 301\text{ in}^4 > 286.34\text{ in}^4$)

$\Delta_{TL} = \frac{5 w_{TL} L^4}{384EI} \leq \frac{L}{240} = \frac{(27')(12\text{ in/ft})}{240} = 1.35''$

self weight of W16x26

$w_{TL} = (15\text{ psf} + 46\text{ psf} + 125\text{ psf})(5') + 26\text{ lb/ft} = 956\text{ lb/ft} = 0.956\text{ k/ft}$

$I_{req} = \frac{(5)(0.956\text{ k/ft})(27')^4(1728)}{(384)(29000\text{ ksi})(1.35'')} = 291.99\text{ in}^4 < 301\text{ in}^4$

∴ W16x26 is OK

Recheck Z_{req} including self weight of W16x26

$$W_u = 1.2D + 1.6L = 1.2\left[(15\ psf + 46\ psf)(5') + 26\ ^{lb/ft}\right] + 1.6\left[(125\ psf)(5')\right] =$$

$$= 1397.2\ ^{lb/ft} = 1.3972\ ^{k/ft}$$

$$V_u = \frac{(1.3972\ ^{k/ft})(27')}{2} = 18.862^k < \phi V_n = 106^k \ \therefore \ OK$$

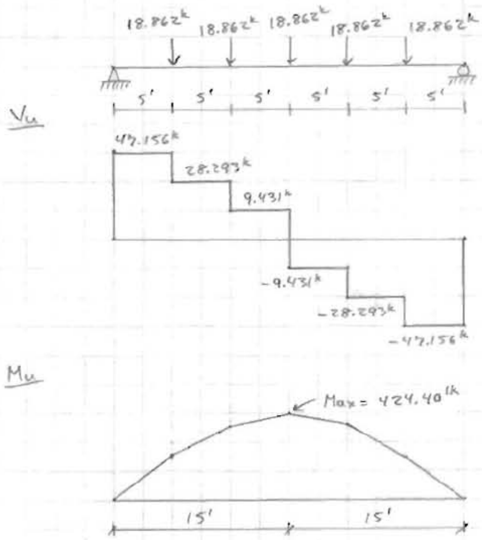
$$M_u = \frac{W_u L^2}{8} = \frac{(1.3972\ ^{k/ft})(27')^2}{8} = 127.320^{'k}$$

Assume fully braced

$$Z_{req} = \frac{M_u}{\phi F_y} = \frac{(127.320^{'k})(12\ ^{in/ft})}{(0.9)(50\ ksi)} = 33.952\ in^3 < 44.2\ in^3 \ \therefore \ OK$$

$\boxed{\text{Use W16 x 26 for beams}}$

· Girder → Designed as simply supported to be conservative (existing has moment connections at ends)



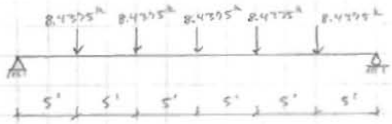
Assume fully braced

$$Z_{req} = \frac{M_u}{\phi F_y} = \frac{(424.40^{'k})(12\ ^{in/ft})}{(0.9)(50\ ksi)} = 113.173\ in^3$$

Try W21x55 $(Z_x = 126\ in^3 > 113.173\ in^3)$

$$\Delta_{LL} \leq \frac{L}{360} = \frac{(30')(12\ ^{in/ft})}{360} = 1.0''$$

$$P_L = \frac{[(125\ psf)(5')](27')}{2} = 8437.5\ lb = 8.4375^k$$

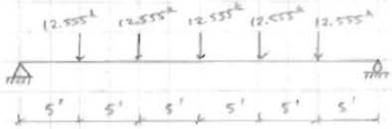


W21x55

From SAP 2000 $\rightarrow \Delta_{L,max} = 0.9358'' < 1.0'' \therefore$ OK

$$\Delta_{TL} \leq \frac{L}{240} = \frac{(30')(12\ in/ft)}{240} = 1.5''$$

$$P_{TL} = \frac{[(15\ psf + 46\ psf + 125\ psf)(5')](27')}{2} = 12555.0\ lb = 12.555^k$$



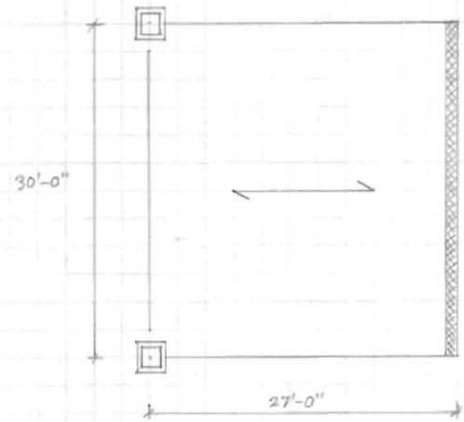
W21x55

From SAP2000 $\rightarrow \Delta_{TL,max} = 1.3924'' < 1.5'' \therefore$ OK

$\boxed{\text{Use W21x55 for girder}}$

Appendix D – One-Way Post-Tensioned Slab

One-Way Post-Tensioned



- Loads

  Framing Dead Load = selfweight

  Superimposed Dead Load = 15 psf

  Live Load = 125 psf

  * No live load reductions since LL = 125 psf > 100 psf

- Materials

  Concrete: Normal Weight 150 pcf

  $f'_c = 5{,}000$ psi

  $f'_{ci} = 3{,}000$ psi

  Rebar: $f_y = 60{,}000$ psi

  PT: Unbonded tendons

  $\frac{1}{2}$" ϕ, 7-wire strands, $A = 0.153$ in^2

  $f_{pu} = 270$ ksi

  Estimated prestress losses = 15 ksi (ACI 18.6)

  $f_{se} = (0.7)(270 \text{ ksi}) - 15 \text{ ksi} = 174 \text{ ksi}$ (ACI 18.5.1)

$P_{eff} = (A)(f_{se}) = (0.153\ in^2)(174\ ksi) = 26.6$ kips/tendon

- Determine Preliminary Slab Thickness

Start with $\frac{L}{h} = 40$ ← typical for one-way solid slabs, simple spans, floor (from presentation by Apple)

Span = 27'

$$h = \frac{(27')(12\ in/ft)}{40} = 8.1''$$

Try h = 8.5" preliminary slab thickness

- Loading

$DL = \text{Selfweight} = \left(\frac{8.5''}{12\ in/ft}\right)(150\ pcf) = 106.25\ psf$

Superimposed DL = 15 psf

LL = 125 psf (no live load reductions)

Design of Frame

Use Equivalent Frame Method, ACI 13.7 (excluding sections 13.7.7.4-5)

Total bay width between centerlines = 30'

No pattern loading for simplicity of hand calculations

- Calculate Section Properties

ACI 318-08 → 18.3.3

$f_t = \frac{6M}{bh^2}$

Calculated at service loads

$w = 106.25\ psf + 15\ psf + 125\ psf = 246.25\ psf$

$w = (246.25\ psf)(30') = 7387.5\ lb/ft = 7.3875\ k/ft$

$$M = \frac{wL^2}{8} = \frac{(7.3875\ k/ft)(27')^2}{8} = 673.186^{k}$$

$$f_t = \frac{6M}{bh^2} = \frac{(6)(673.186^{k})(12\ in/ft)}{[(30')(12\ in/ft)](8.5'')^2} = 1.863\ ksi$$

$1.863\ ksi > 12\sqrt{f'_c} = 12\sqrt{5000\ psi} = 848.528\ psi$

∴ Class C → However, will use gross section properties for simplicity of hand calculations

- Set Design Parameters

Allowable Stresses

At time of jacking

$f'_{ci} = 3000$ psi

Compression $= 0.60 f'_{ci} = (0.60)(3000 \text{ psi}) = 1800$ psi

Tension $= 3\sqrt{f'_{ci}} = 3\sqrt{3000} = 164.32$ psi

At service loads

$f'_c = 5000$ psi

Compression $= 0.45 f'_c = (0.45)(5000 \text{ psi}) = 2250$ psi

Tension $= 6\sqrt{f'_c} = 6\sqrt{5000 \text{ psi}} = 424.26$ psi

Average precompression limits:

$\frac{P}{A} = 125$ psi min. (ACI 18.12.4)

$= 300$ psi max.

Target Load Balances

60%–80% of DL (Selfweight) for slabs (good approximation for hand calculations)

Use $0.75 w_{DL} = (0.75)(106.25 \text{ psf}) = 79.69$ psi

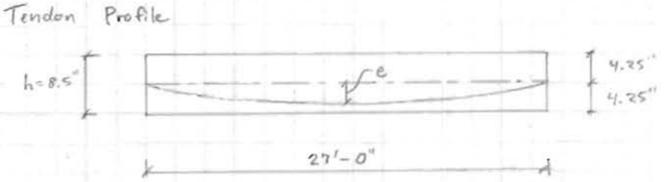
Cover Requirements (2-hour fire rating, assume carbonate aggregate), IBC 2003

Restrained slabs $= \frac{3}{4}$" bottom

Unrestrained slabs $= 1\frac{1}{2}$" bottom

$= \frac{3}{4}$" top

- Tendon Profile



Tendon Ordinate	Tendon (CG) Location*
Exterior Support - Anchor	4.25"
Bottom of Span	1.75"

(CG) = center of gravity
* Measure from bottom of slab

$e = 4.25" - 1.75" = 2.5"$

- Prestress Force Required to Balance 75% of selfweight DL

$w_b = (0.75)(w_{DL}) = (0.75)[(106.25\ psf)(30')] = 2390.625\ ^{lb}/_{ft} = 2.391\ ^{k}/_{ft}$

Force needed in tendons to counteract the load in the end bay:

$P = \frac{w_b L^2}{8e} = \frac{(2.391\ k/ft)(27')^2}{(8)\left(\frac{2.5"}{12"/ft}\right)} = 1045.659^k$

- Check Precompression Allowance

Determine number of tendons to achieve 1045.659^k

$\#\ tendons = \frac{1045.659^k}{26.6^k/tendon} = 39.3$ tendons

Try 39 tendons

Actual force for bonded tendons

$P_{actual} = (39\ tendons)(26.6^k/tendon) = 1038.26^k$

The balanced load is slightly adjusted

$w_b = \left(\frac{1038.26^k}{1045.66^k}\right)(2.391\ k/ft) = 2.374\ ^k/_{ft}$

Determine actual precompression stress

$A = (b)(h) = [(30')(12\ in/ft)](8.25") = 2970\ in^2$

$\frac{P_{actual}}{A} = \frac{1038.26^k}{2970\ in^2} = 0.3496\ ksi = 349.6\ psi > 125\ psi\ min\ \therefore$ OK

$> 300\ psi\ max\ \therefore$ Not OK

Increase Slab Thickness → Try $h = 11"$

DL (Self Weight) $= \left(\frac{11"}{12\ in/ft}\right)(150\ pcf) = 137.5\ psf$

- Calculate Section Properties

$w = 137.5\ psf + 15\ psf + 125\ psf = 277.5\ psf = 0.2775\ ksf$

$M = \frac{wL^2}{8} = \frac{[(0.2775\ ksf)(1')](27')^2}{8} = 25.287^{lk}$

$f_t = \frac{6M}{bh^2} = \frac{(6)(25.287'^k)(12\ ^{in}/_{ft})}{(12'')(11'')^2} = 1.2539\ ksi = 1253.9\ psi$

$1253.9\ psi > 12\sqrt{f'_c} = 12\sqrt{5000\ psi} = 848.528\ psi$

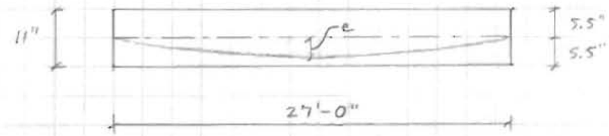
$\therefore$ Class C $\rightarrow$ However, will use gross section properties for simplicity of hand calculations

· Target Load Balances

60% – 80% of DL (Selfweight)

Use $0.75\, w_{DL} = (0.75)(137.5\ psf) = 103.125\ psf$

· Tendon Profile



$e = 5.5'' - 1.75'' = 3.75''$

$w_b = (0.75)\left[(137.5\ psf)(30')\right] = 3093.75\ ^{lb}/_{ft} = 3.0938\ ^{k}/_{ft}$

$P = \frac{w_b L^2}{8e} = \frac{(3.0938\ ^k/_{ft})(27')^2}{(8)\left(\frac{3.75''}{12\ ^{in}/_{ft}}\right)} = 902.138^k$

· Check Precompression Allowance

$\#\ tendons = \frac{902.138^k}{26.6\ ^k/_{tendon}} = 33.9\ tendons$

Try 34 tendons

$P_{actual} = (34\ tendons)(26.6^k/tendon) = 904.40^k$

$w_b = \left(\frac{904.40^k}{902.138^k}\right)(3.0938\ ^k/_{ft}) = 3.102\ ^k/_{ft}$

$\frac{P_{actual}}{A} = \frac{904.40^k}{\left[(30')(12\ ^{in}/_{ft})\right](11'')} = 0.22838\ ksi = 228.38\ psi > 125\ psi\ min\ \therefore OK$

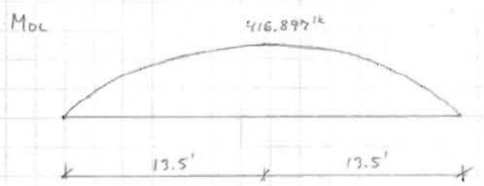
$< 300\ psi\ max\ \therefore OK$

· Check Slab Stresses

DL Moments

$w_{DL} = (137.5\ psf + 15\ psf)(30')/1000\ ^{lb}/_{kip} = 4.575\ ^k/_{ft}$

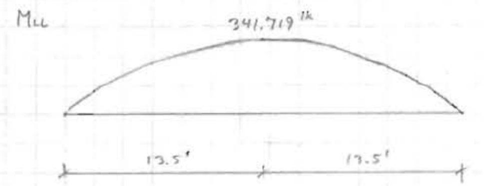
$M_{DL} = \frac{w_{DL} L^2}{8} = \frac{(4.575\ ^k/_{ft})(27')^2}{8} = 416.897'^k$



LL Moments

$W_{LL} = (125\ psf)(30')/1000\ lb/kip = 3.75\ k/ft$

$M_{LL} = \frac{W_{LL} L^2}{8} = \frac{(3.75\ k/ft)(27')^2}{8} = 341.719\ 'k$



Total Balancing Moments

$W_b = 3.102\ k/ft$

$M_b = \frac{W_b L^2}{8} = \frac{(3.102\ k/ft)(27')^2}{8} = 282.625\ 'k$

Stage 1: Stresses immediately after jacking (DL + PT)

Midspan Stresses

$f_{top} = \frac{(-M_{DL} + M_{bal})}{S} - \frac{P}{A}$

$f_{bot} = \frac{(+M_{DL} - M_b)}{S} - \frac{P}{A}$

$S = \frac{bh^2}{6} = \frac{[(30')(12\ in/ft)](11")^2}{6} = 7260\ in^3$

$f_{top} = \frac{(-416.897'k + 282.625'k)(12\ in/ft)(1000\ lb/kip)}{7260\ in^3} - 228.38\ psi$

$= -450.32\ psi$ compression $< 0.60 f'_{ci} = (0.60)(3000\ psi) = 1800\ psi$ ∴ OK

$f_{bot} = \frac{(416.897'k - 282.625'k)(12)(1000)}{7260\ in^3} - 228.38\ psi$

$= -6.45\ psi$ compression $< 0.60 f'_{ci} = 1800\ psi$ ∴ OK

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

$f_{top} = \frac{(-416.897'k - 341.719'k + 282.625'k)(12)(1000)}{7260\ in^3} - 228.38\ psi$

$= -1015.14\ psi$ compression $< 0.45 f'_c = (0.45)(5000\ psi) = 2250\ psi$ ∴ OK

$$f_{bot} = \frac{(416.897^{1k} + 341.719^{1k} - 282.625^{1k})(12)(1000)}{7260\ in^3} - 228.38\ psi$$

$$= 558.38\ psi\ \text{tension} > 6\sqrt{f'_c} = 6\sqrt{5000\ psi} = 424.26\ psi \quad \therefore \text{ Not OK}$$

Increase Slab Thickness → Try $h = 12''$

$$DL\ (\text{Self Weight}) = \left(\frac{12''}{12\ in/ft}\right)(150\ pcf) = 150\ psf$$

- Calculate Section Properties

$$w = 150\ psf + 15\ psf + 125\ psf = 290\ psf = 0.290\ ksf$$

$$M = \frac{wL^2}{8} = \frac{[(0.290\ ksf)(1')](27')^2}{8} = 26.426^{1k}$$

$$f_t = \frac{6M}{bh^2} = \frac{(6)(26.426^{1k})(12\ in/ft)}{(12'')(12'')^2} = 1.1011\ ksi = 1101.1\ psi$$

$$1101.1\ psi > 12\sqrt{f'_c} = 12\sqrt{5000\ psi} = 848.528\ psi$$

∴ Class C → However, will use gross section properties for simplicity of hand calculations

- Target Load Balances

Use $0.75 w_{DL} = (0.75)(150\ psf) = 112.5\ psf$

- Tendon Profile

$$e = \left(\frac{12''}{2}\right) - 1.75'' = 4.25''$$

$$w_b = (0.75)[(150\ psf)(30')] = 3375\ lb/ft = 3.375\ k/ft$$

$$P = \frac{w_b L^2}{8e} = \frac{(3.375\ k/ft)(27')^2}{(8)\left(\frac{4.25''}{12\ in/ft}\right)} = 868.368^k$$

- Check Precompression Allowance

$$\#\ \text{tendons} = \frac{868.368^k}{26.6^k/\text{tendon}} = 32.6\ \text{tendons}$$

Try 33 tendons

$$P_{actual} = (33\ \text{tendons})(26.6^k/\text{tendon}) = 877.80^k$$

$$w_b = \left(\frac{877.80^k}{868.368^k}\right)(3.375\ k/ft) = 3.412\ k/ft$$

$$\frac{P_{actual}}{A} = \frac{877.80^k}{[(30')(12\ in/ft)](12'')} = 0.20319\ ksi = 203.19\ psi > 125\ psi\ \text{min} \quad \therefore \text{ OK}$$

$$< 300\ psi\ \text{max} \quad \therefore \text{ OK}$$

• Check Slab Stresses

DL Moments

$w_{DL} = (150\text{ psf} + 15\text{ psf})(30')/1000\ \text{lb/kip} = 4.950\ \text{k/ft}$

$M_{DL} = \frac{w_{DL}L^2}{8} = \frac{(4.950\ \text{k/ft})(27')^2}{8} = 451.069\ \text{'k}$

LL Moments

$w_{LL} = (125\text{ psf})(30')/1000\ \text{lb/kip} = 3.75\ \text{k/ft}$

$M_{LL} = \frac{w_{LL}L^2}{8} = \frac{(3.75\ \text{k/ft})(27')^2}{8} = 341.719\ \text{'k}$

Total Balancing Moments

$w_b = 3.375\ \text{k/ft}$

$M_b = \frac{w_bL^2}{8} = \frac{(3.412\ \text{k/ft})(27')^2}{8} = 310.888\ \text{'k}$

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

$S = \frac{bh^2}{6} = \frac{[(30')(12\ \text{in/ft})](12'')^2}{6} = 8640\ \text{in}^3$

$f_{bot} = \frac{(451.069\ \text{'k} + 341.719\ \text{'k} - 310.888\ \text{'k})(12)(1000)}{8640\ \text{in}^3} - 203.19\ \text{psi}$

$= 466.12\ \text{psi tension} > 6\sqrt{f'_c} = 6\sqrt{5000\ \text{psi}} = 424.26\ \text{psi}$ ∴ Not OK

If increase f'_c to 6000 psi:

$466.12\ \text{psi tension} > 6\sqrt{f'_c} = 6\sqrt{6000\ \text{psi}} = 464.76\ \text{psi}$ ∴ Not OK

Increase Slab Thickness → Try $h = 13''$

DL (Self Weight) $= \left(\frac{13''}{12\ \text{in/ft}}\right)(150\ \text{pcf}) = 162.5\ \text{psf}$

• Calculate Section Properties

$w = 162.5\ \text{psf} + 15\ \text{psf} + 125\ \text{psf} = 302.5\ \text{psf} = 0.3025\ \text{ksf}$

$M = \frac{wL^2}{8} = \frac{[(0.3025\ \text{ksf})(1')](27')^2}{8} = 27.565\ \text{'k}$

$f_t = \frac{6M}{bh^2} = \frac{(6)(27.565\ \text{'k})(12\ \text{in/ft})}{(12'')(13'')^2} = 0.9787\ \text{ksi} = 978.7\ \text{psi}$

$978.7\ \text{psi} > 12\sqrt{f'_c} = 12\sqrt{5000\ \text{psi}} = 848.528\ \text{psi}$

∴ Class C → However, will use gross section properties for simplicity of hand calculations

- Target Load Balances

  Use $0.75 w_{DL} = (0.75)(162.5 \text{ psf}) = 121.875 \text{ psf}$

- Tendon Profile

  $e = \left(\frac{13''}{2}\right) - 1.75'' = 4.75''$

  $w_b = (0.75)\left[(162.5 \text{ psf})(30')\right] = 3656.25 \text{ lb/ft} = 3.656 \text{ k/ft}$

  $P = \frac{w_b L^2}{8e} = \frac{(3.656 \text{ k/ft})(27')^2}{(8)\left(\frac{4.75''}{12 \text{ in/ft}}\right)} = 841.707^k$

- Check Precompression Allowance

  $\# \text{ tendons} = \frac{841.707^k}{26.6 \text{ k/tendon}} = 31.6 \text{ tendons}$

  Try 32 tendons

  $P_{actual} = (32 \text{ tendons})(26.6^k/\text{tendon}) = 851.20^k$

  $w_b = \left(\frac{851.20^k}{841.707^k}\right)(3.656 \text{ k/ft}) = 3.697 \text{ k/ft}$

  $\frac{P_{actual}}{A} = \frac{851.20^k}{[(30')(12 \text{ in/ft})](13'')} = 0.18188 \text{ ksf} = 181.88 \text{ psf}$ > 125 psi min ∴ OK

  < 300 psi max ∴ OK

- Check Slab Stresses

  DL Moments

  $w_{DL} = (162.5 \text{ psf} + 15 \text{ psf})(30')/1000 \text{ lb/kip} = 5.325 \text{ k/ft}$

  $M_{DL} = \frac{w_{DL} L^2}{8} = \frac{(5.325 \text{ k/ft})(27')^2}{8} = 485.241^{'k}$

  LL Moments

  $w_{LL} = (125 \text{ psf})(30')/1000 \text{ lb/kip} = 3.75 \text{ k/ft}$

  $M_{LL} = \frac{w_{LL} L^2}{8} = \frac{(3.75 \text{ k/ft})(27')^2}{8} = 341.719^{'k}$

  Total Balancing Moments

  $w_b = 3.697 \text{ k/ft}$

  $M_b = \frac{w_b L^2}{8} = \frac{(3.697 \text{ k/ft})(27')^2}{8} = 336.933^{'k}$

Stage 1 : Stresses immediately after jacking (DL + PT)

Midspan Stresses

$$S = \frac{bh^2}{6} = \frac{[(30')(12\ in/ft)](13")^2}{6} = 10140\ in^3$$

$$f_{top} = \frac{(-485.241^{k} + 336.933^{k})(12)(1000)}{10140\ in^3} - 181.88\ psi$$

$$= -357.39\ psi\ \text{compression} < 0.60 f'_{ci} = (0.60)(3000\ psi) = 1800\ psi \quad \therefore OK$$

$$f_{bot} = \frac{(485.241^{k} - 336.933^{k})(12)(1000)}{10140\ in^3} - 181.88\ psi$$

$$= -6.37\ psi\ \text{compression} < 0.60 f'_{ci} = 1800\ psi \quad \therefore OK$$

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

Midspan Stresses

$$f_{top} = \frac{(-M_{DL} - M_{LL} + M_{bal})}{S} - \frac{P}{A}$$

$$f_{bot} = \frac{(+M_{DL} + M_{LL} - M_{bal})}{S} - \frac{P}{A}$$

$$f_{top} = \frac{(-485.241^{k} - 341.719^{k} + 336.933^{k})(12)(1000)}{10140\ in^3} - 181.88\ psi$$

$$= -761.79\ psi\ \text{compression} < 0.45 f'_c = (0.45)(5000\ psi) = 2250\ psi \quad \therefore OK$$

$$f_{bot} = \frac{(485.241^{k} + 341.719^{k} - 336.933^{k})(12)(1000)}{10140\ in^3} - 181.88\ psi$$

$$= 398.03\ psi > 6\sqrt{f'_c} = 6\sqrt{5000\ psi} = 424.26\ psi \quad \therefore OK$$

All stresses are within the permissible code limits

- Ultimate Strength

Determine factored moments

The primary post-tensioning moments, M_1, vary along the length of the span

$$M_1 = (P)(e)$$

$e = 0$ at the exterior support

$e = \frac{13"}{2} - 1.75" = 4.75"$ at midspan

$$M_1 = (851.2^{k})\left(\frac{4.75"}{12\ in/ft}\right) = 336.933^{k}$$

Secondary Post-Tensioning Moments, M_{sec}

$M_{sec} = 0^{k}$ (since simply supported) ← From presentation by Apple

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

At midspan: $M_u = (1.2)(485.241^{'k}) + (1.6)(341.719^{'k}) + (1.0)(0^{'k}) = 1129.039^{'k}$

- Determine minimum bonded reinforcement

Positive moment region

$f_t = 398.032$ psi $> 2\sqrt{f'_c} = (2)\sqrt{5000 \text{ psi}} = 141.42$ psi

Minimum positive reinforcement required (ACI 18.9.3.2)

$y = \frac{f_t}{(f_t + f_c)}(h) = \left(\frac{398.032 \text{ psi}}{398.032 \text{ psi} + 761.793 \text{ psi}}\right)(13'') = 4.461''$

$N_c = \left(\frac{M_{DD+LL}}{S}\right)(0.5)(y)(\ell_2) = \left[\frac{(485.241^{'k} + 341.719^{'k})(12 \text{ in/ft})}{10140 \text{ in}^3}\right](0.5)(4.461'')[(30')(12 \text{ in/ft})]$

$= 785.903^k$

$A_{s,min} = \frac{N_c}{0.5 f_y} = \frac{785.903^k}{(0.5)(60 \text{ ksi})} = 26.197 \text{ in}^2$

Distribute the positive moment reinforcement uniformly across the slab-beam width and as close as practicable to the extreme tension fiber

$A_{s,min} = \frac{26.197 \text{ in}^2}{30 \text{ ft}} = 0.873 \text{ in}^2/\text{ft}$

$\boxed{\text{Use \#5 @ 4'' o.c. bottom}} = (0.31 \text{ in}^2)\left(\frac{12 \text{ in/ft}}{4 \text{ in}}\right) = 0.930 \text{ in}^2/\text{ft} > 0.873 \text{ in}^2/\text{ft} \therefore \text{OK}$

Minimum length shall be $\frac{1}{3}$ clear span and centered in positive moment region (ACI 18.9.4.1)

Negative moment region

Supports:

$A_{s,min} = 0.00075 A_{cf}$ (ACI 18.9.3.3)

$A_{cf} = \max \begin{cases} (13'')\left[\frac{(27')(12 \text{ in/ft})}{2}\right] = 2106 \text{ in}^2 \\ (13'')[(30')(12 \text{ in/ft})] = 4680 \text{ in}^2 \end{cases}$

$A_{s,min} = (0.00075)(4680 \text{ in}^2) = 3.51 \text{ in}^2$

#4 bars $\rightarrow \frac{3.51 \text{ in}^2}{0.20 \text{ in}^2/\text{bar}} = 17.55$

$\boxed{\text{Use (18) \#4 top}}$ $(A_s = (18 \text{ bars})(0.20 \text{ in}^2/\text{bar}) = 3.60 \text{ in}^2 > 3.51 \text{ in}^2 \therefore \text{OK})$

• Check minimum reinforcement → if it is sufficient for ultimate strength

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

d = effective depth

$A_{ps} = (0.153\ in^2/tendon)(32\ tendons) = 4.896\ in^2$

$f_{ps} = f_{se} + 10{,}000 + \frac{f'_c b d}{100 A_{ps}}$ for slabs with $\frac{L}{h} \leq 35$

$\frac{L}{h} = \frac{(27')(12\ in/ft)}{13''} = 24.923 < 35$

$= 174{,}000\ psi + 10{,}000 + \frac{(5000\ psi)[(30')(12\ in/ft)](d)}{(100)(4.896\ in^2)}$

$= 184{,}000 + (3676.471)(d)$

$a = \frac{A_s f_y + A_{ps} f_{ps}}{0.85 f'_c b}$

When reinforcement is provided to meet ultimate strength requirements, the minimum lengths must also conform to the provision of ACI 318 Chapter 12 (ACI 18.9.4.3)

At midspan:

$d = 13'' - 1\frac{1}{2}'' - \frac{1}{4}'' = 11\frac{1}{4}'' = 11.25''$ ← #4 bar

$f_{ps} = 184{,}000\ psi + (3676.471)(11.25'') = 225360.29\ psi = 225.360\ ksi$

$a = \frac{[(26.197\ in^2)(60\ ksi) + (4.896\ in^2)(225.360\ ksi)]}{(0.85)(5\ ksi)[(30')(12\ in/ft)]} = 1.748''$

$\phi M_n = (0.9)[(26.197\ in^2)(60\ ksi) + (4.896\ in^2)(225.360\ ksi)](11.25'' - \frac{1.748''}{2})/12\ in/ft$

$= 2081.777^{\,k} > 1129.039^{\,k}$ ∴ OK

Use 13" slab with 32 tendons spanning the 27' direction, #5 @ 4" o.c. bottom (positive moment region) and (18) #4 top (negative moment region)

* Deflection check of slab ignored at this point (further study required)

Girder → Design as simply supported to be conservative (existing has moment connections at ends)

$w_u = 1.2D + 1.6L = (1.2)(162.5\ psf + 15\ psf) + (1.6)(125\ psf) = 413\ psf$

$w_u = (413\ psf)(\frac{27'}{2}) = 5575.5\ lb/ft = 5.5755\ k/ft$

$M_u = \frac{w_u L^2}{8} = \frac{(5.5755\ k/ft)(30')^2}{8} = 627.244^{\,k}$

Assume fully braced

$$Z_{req} = \frac{M}{\phi F_y} = \frac{(627.244^{k'})(12\ in/ft)}{(0.9)(50\ ksi)} = 167.265\ in^3$$

Try W24x68 ($Z_x = 177\ in^3 > 167.265\ in^3 \therefore OK$)

$$\Delta_{LL} = \frac{5 w_L L^4}{384EI} \le \frac{L}{360} = \frac{(30')(12\ in/ft)}{360} = 1.0''$$

$$w_L = (125\ psf)\left(\frac{27'}{2}\right) = 1687.5\ lb/ft = 1.6875\ k/ft$$

$$I_{req} = \frac{(5)(1.6875\ k/ft)(30')^4(1728)}{(384)(29000\ ksi)(1.0'')} = 1060.51\ in^4$$

W24x68 is OK ($I_x = 1830\ in^4 > 1060.51\ in^4 \therefore OK$)

$$\Delta_{TL} = \frac{5 w_{TL} L^4}{384EI} \le \frac{L}{240} = \frac{(30')(12\ in/ft)}{240} = 1.5''$$

$$w_{TL} = (162.5\ psf + 15\ psf + 125\ psf)\left(\frac{27'}{2}\right) = 4083.75\ lb/ft = 4.0838\ k/ft$$

$$I_{req} = \frac{(5)(4.0838\ k/ft)(30')^4(1728)}{(384)(29000\ ksi)(1.5'')} = 1710.95\ in^4$$

W24x68 is OK ($I_x = 1830\ in^4 > 1710.95\ in^4 \therefore OK$)

$\boxed{\text{Use W24x68 for girder}}$

- Check Bearing on 12" CMU Wall

Bearing Area $= (30')\left(\frac{12''}{12\ in/ft}\right) = 30\ ft^2 = 4320\ in^2$

$P_u = (413\ psf)\left(\frac{27'}{2}\right)(30') = 167,265\ lb = 167.265^k$

Bearing Capacity $= \phi(0.85 f'_c A_b) = (0.65)[(0.85)(2000\ psi)(4320\ in^2)] =$

for masonry is 2000 psi (specified on drawings)

$= 4,773,600\ lb = 4773.6^k > 167.265^k \therefore OK$