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Executive Summary

In the following report, an analysis was performed of three alternate floor systems for the Patient Pavilion. Hand calculations supplemented with computer-aided calculations to determine schematic sizes for each system. Each system was then compared with each other, as well as the existing system, based on the weight, deflections, depth, cost, constructability, and architectural impacts. The existing floor system is composite steel framing with composite steel deck, the alternate systems designed in this report include:

- Two-Way Flat Slab with Drop Panels
- One-Way Concrete Slab with Concrete Beams
- Hollow Core Precast Plank on Steel Framing

The design of the two-way flat slab resulted in a 10" slab with 11'x11'x12.5" drop panels, reinforced with No. 7 bars, the location and span direction dictates the amount of bars used. The advantages of this system include minimal formwork, and no fireproofing needed. Some disadvantages of this system include the difficulty of construction, large deflections, and the need for shear walls for lateral load resistance. The two-way slab system with drop panels is a viable solution to the Patient Pavilion, however with the large deflections and mass of the system it is not the best solution.

The one-way system comprises of a 5" slab spanning 10', framed by a 30' girder spanning column to column with two infill beams at 10' on center. This system is slightly over designed due to trying to increase the constructability and reducing the amount of formwork. The depth of the girder was designed first because it controlled the depth of the beam in order to create redundant formwork. In addition, the girder is 30" wide to be flush with the 30"x30" column further reducing the cost of formwork and the difficulty of constructability. Advantages of this system include low cost of formwork due to redundancy, good constructability, and a good damping response to floor vibrations. Some negative aspects of this system include high over price, increase of floor weight, and larger seismic loads due to the increase in floor weight. The one-way slab with beams system is a viable solution to the Patient Pavilion; however, it is not the most optimal option.

The final system is a hollow core precast plank with steel framing, the Nitterhouse catalog was used to pick a plank size and an 8" plank reinforced with (6) 1/2" Ø 270k tendons with a 2 hour fire rating was chosen. Due to the large span and tributary width of the girders, a W18x175 was designed to be the most optimal. However, if the plank were to sit on top of the girder the total depth of the floor system would be approximately 30". To reduce the height a special connection has been designed to box the W-shape and have the plank sit on an angle the is welded to the face of this boxed in W-shape. The advantages of this system include minimal formwork and fireproofing included however, these are obsolete compared to the disadvantages of this system. The disadvantages include large deflections, difficult construction, special in-shop connections, deep girders, and difficult to renovate due to the prestressed tendons. The hollow core precast plank system was determined to not be a viable solution for the Patient Pavilion.

Introduction

The Patient Pavilion is located in Albany, NY, at the intersection of New Scotland Avenue and Myrtle Avenue, on the eastern end of the existing Albany Medical Center Hospital (AMCH) campus. Constructed as an expansion to the AMCH, the Patient Pavilion utilizes pedestrian bridges to tie into an existing parking structure across New Scotland Avenue, as well as tying into an existing building on the AMCH campus as shown in *Figure 1.*

The Patient Pavilion will retain the architectural style, forms, and materials of downtown Albany and the AMCH campus, as specified in the City of Albany Zoning Ordinance. The façade primarily consists of brick and stone with punched windows and white stone accenting the upper levels. To add emphasis to the pedestrian walkway over New Scotland Avenue, metal paneling and glazed aluminum curtain-walls added an integrated modern look to the traditional façade.

The Patient Pavilion consists of two phases; Phase 1, contains the demolition of an existing building on the AMCH campus, and the construction of a six story medical center see *Figure 2* and Phase 2 is a future four story vertical expansion of the Patient Pavilion see *Figure 3*. The building height of Phase 1 is 75 feet above grade and the vertical expansion of Phase 2 will increase the building height to 145 feet above grade. Due to a small site and large square footage demands, the building cantilevers over the site on the side of New Scotland Avenue, demanding for the design of cantilevered plate girders to support a column load from stories 2-10.

This patient care facility, contributes 229 patient beds, 20 operating rooms, and 1000 new permanent jobs to the AMCH campus. The 348,000 square foot expansion consists of six stories above grade with a four story vertical expansion in the future. Phase 1 construction on the Patient Pavilion began in September of 2010 and projects to finish in June of 2013.

Figure 3 – Phase 2 of the Patient Pavilion; Vertical Expansion

To better understand the terminology used for referring to designated levels, an architectural elevation is provided on the next page.

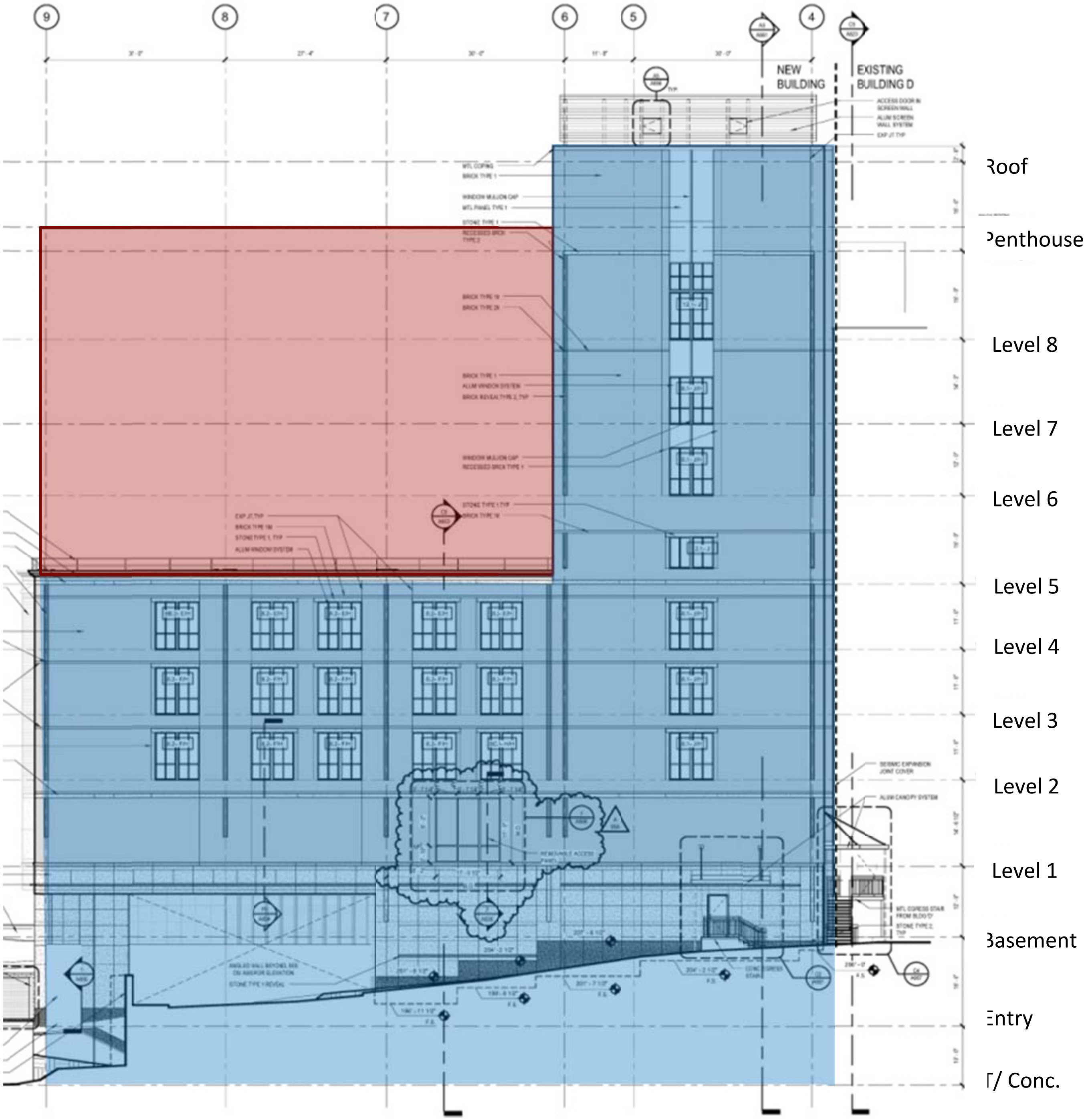


Figure 4 – South Elevation

Phase 1

Phase 2

Figure 5 – Site Plan

New Scotland Avenue

Myrtle Avenue

Structural Overview

The majority of the Patient Pavilion rests on a 36" thick mat foundation, and some piles located near existing buildings. The floor system utilizes composite beams, girders, and slabs to carry the loads derived from ASCE07-02. The lateral forces are collected on the brick non-bearing façade, transfers into the slab and is distributed to the foundation/grade by the integration of braced and moment frames. On the southern end of the site, 62" deep plate girders are utilized to cantilever nine stories over the edge of the site. Multi-story trusses are utilized to carry multiple levels with a large clear span, these are located over the emergency access ramp and at the pedestrian bridge that ties into an existing AMCH building see *Figure 6.*

Foundation

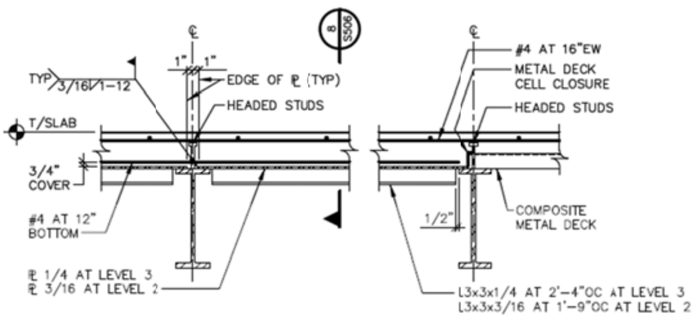
Vernon Hoffman PE Soil and Foundation Engineering supplied the geotechnical report for the Patient Pavilion site. Procedures used were site boring, vane shear testing, pressure testing, and cone testing. Soil testing concluded that foundations must be designed to a net bearing pressure of 3000psf. Design ground water level was reported to be between 4' and 10' throughout the site. After a full analysis of the site, the geotechnical report recommended the building to sit on a mat foundation resting on a controlled fill.

Because of the relatively low allowable soil bearing pressure, the majority of the Patient Pavilion sits on a 36" mat foundation resting on a 4" mud slab with a 12" compacted aggregate base. Alternatively, 20'-0" deep piles are utilized in order to prevent unwanted settlement of the existing buildings. Piles are utilized in place of shallow foundations because piles will control settlements and provide uplift resistance more effectively than shallow foundations.

Foundation walls are utilized along existing building C and along New Scotland Avenue to lessen the demand on the excavation shoring; these walls also serve the purpose of shear walls in the lateral system. Backfilling behind these walls was needed to provide construction access for equipment and materials to build the pile caps and grade beams.

Floor System

The Patient Pavilion utilizes 3”x20ga galvanized composite steel deck with 3 1/2” lightweight topping, reinforced with #4’s at 16” O.C. for shrinkage and temperature, this floor system is typical throughout the levels, unless otherwise noted. On level 2, the floor slab is thickened with a 3” lightweight concrete topping in order to reduce floor vibrations in the operating rooms. The entry level utilizes an 8” lightweight concrete slab on 3 1/2”x16ga composite metal deck because of longer deck spans and larger live loads. In areas where radiation is prevalent, the slabs above and below that level are stiffened with a steel plate anchored to the slab with angles. These plates are located on levels 2 and 3 and their function is to provide a shield from the radiation for adjacent areas, refer to *Figure 7* for radiation slab details.



Typical beam spacing throughout is 10’-0” O.C., creating a 10’-0” deck span requirement, all beams are composite beams, typically W12’s. However, on the Basement Level and Level 2, typical beams range from W16’s to W18’s. Reasons for deeper beams are that the live load requirements on the Entry Level through Level 2 are greater than the other floors. However, the Basement Level and Level 2 utilize deeper beams than the Entry Level and Level 1 due to greater floor-to-floor heights.

Typical beams span 27’-4”, these beams sit on girders that typically span 30’-0”. Girder sizes range from W14’s to W18’s; however, on the Basement Level and Level 2 girder sizes fluctuate from W18’s to W24’s, refer to *Figure 8* for a typical bay on Level 3.

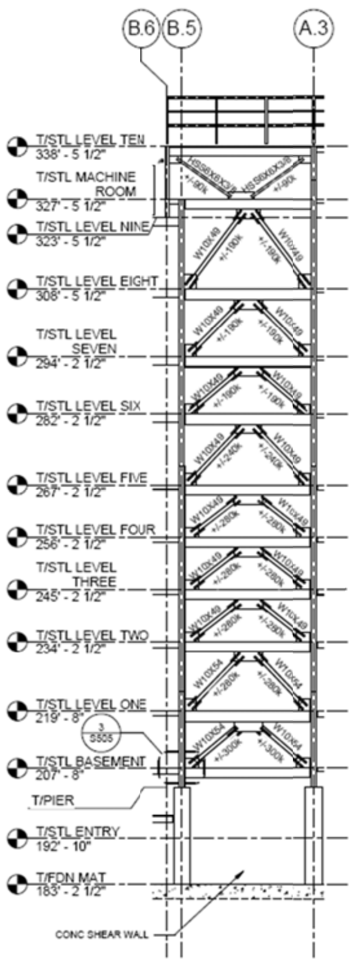
A demand for specialty framing is needed in certain areas in this project; on the southern end of the site, a column is cantilevered 18’ over the edge of the site resting on a 62” plate girder. The pedestrian bridge on the tying into the existing AMCH building spans 83’ over another existing AMCH building. A two-story truss was designed on bottom two levels of this pedestrian bridge, consisting of W10x77’s and W10x100’s.


Match Line

Lateral System

The lateral system for the Patient Pavilion predominantly consists of braced frames, with some moment frames. Within the structure, there are 14 braced frames and 5 moment frames, because of the locations of the braced frames, Chevron bracing is utilized to allow openings for doorways and corridors. See *Figure 8* for a typical braced frame. Figure *7* shows the locations of the braced and moment frames, the location of some braced frames fluctuate from level to level. For instance, braced frame 13 is braced between the Basement Level through Level 2 and above Level 2 is a moment frame.

The braced frames along the western side of the site sit on retaining walls in the basement, which also act as concrete shear walls. A strong connection is required to transfer the shear load from the column into the concrete shear wall, for these connections a 30"x30"x3½" baseplate with a 2" diameter anchor bold anchored 42" into the wall is specified. Diagonal bracing on the lower levels range from W10's to W12's and HSS8x6's to HSS8x8's on the upper levels. Heavier bracing on the lower levels provides a greater resistance to shear, which increases as the force moves down the frame. Columns used in these lateral resisting frames range from W14x43 to W 14x233.



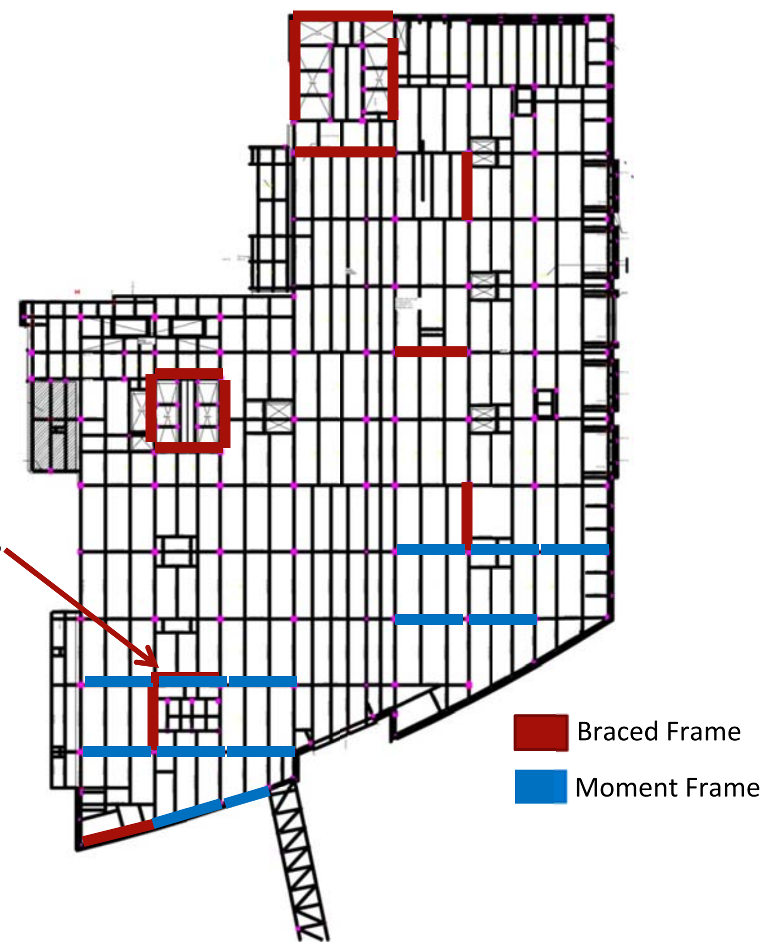


Figure 10 – Typical Braced Frame

Design Codes and Standards

Ryan-Biggs Associates abided by these standards and codes when developing the design of the Patient Pavilion:

- AISC 13th Edition Manual
- AISC Specification 360-05
- 2007 Building Code of New York State (BCNYS)
- Minimum Design Loads for Buildings and Other Structures (ASCE7-02)
- AISC Manual of Steel Construction, Load Resistance Factor Design (LRFD)

Standards and codes utilized for this report:

- AISC 14th Edition Manual
- AISC Specification 360-10
- 2006 International Building Code (IBC 2006)
- Minimum Design Loads for Buildings and Other Structures (ASCE7-05)
- AISC Manual of Steel Construction, Load Resistance Factor Design (LRFD)

Materials

The structural materials designated by the AISC 13th Ed. were used in the design of the Patient Pavilion by Ryan-Biggs; see *Table 1* for the capacities of the large variety of structural elements. The materials were specified on the General Notes page, S001, on the Construction Documents provided via Gilbane Building Company. All steel materials below are according to ASTM standards.

Table 1 – Material Properties

Material Properties		
Material		**Strength**
Rolled Steel	**Grade**	**f_y = ksi**
W Shapes	A 992	50
C, S, M, MC, and HP Shapes	A 36	36
Plates, bars, and angles	A 36	36
HSS pipe	A53 type E or S Grade B	35
Reinforcing Steel	A 615	60
Concrete	**Weight (lb/ft^3)**	**f'_c = psi**
Footings/mat foundation	145	3,000
Interior S.O.G or Slab on Deck	145	3,500
Foundation Walls, Shear walls, Piers, Pile caps, and Grade beams	145	4,000
Exterior S.O.G.	145	4,500
Masonry	**Grade**	**f'_m = psi**
Concrete Block	C 90	2,800
Mortar	C 270 Type S	n/a
Unit Masonry	n/a	2,000
Grout	C 476	2,500
Brick	C 216 type FBS Grade SW	3,000
Welding Electrodes	E70 XX	70 ksi

Loads

In the following tables, dead and live loads that were used to analyze and design the Patient Pavilion are listed as well as the loads used for this thesis. Live loads interpreted by the designer were derived from ASCE7-02, live loads used in this thesis were derived from ASCE 7-05; dead loads were assumed or calculated and verified with specified dead loads on the structural general notes.

Dead Loads

The dead loads listed on the general notes of the structural drawings are listed below in *Table 2.* Upon further analysis shown in *Table 3* and *Table 4*, the assumptions of these loads were verified to be accurate and conservative in some cases. The MEP is larger than typical because in a hospital the MEP weight is to be assumed larger than typical.

Table 2 – Superimposed Dead Loads

Dead Loads (As Shown on General Notes S100)	
Description	**Weight (psf)**
Roof Without Conc. Slab	30
Roof With Conc. Slab	95
Roof Garden	325
Floor	95
Level 9 Mechanical Penthouse	125

Table 3 – Roof without Conc. Slab Verification

Roof Without Conc. Slab Verification (ASCE7-05 and Vulcraft)	
Description	**Weight (psf)**
MEP	12
3”x16ga decking	5
Rigid Insulation (tapered starting at 8”)	.75psf per in thickness=(.75x8x.5)= 12
Total	**29**

Table 4 – Roof with Conc. Slab and Floor Verification

Roof With Conc. Slab and Floor Verification (ASCE7-05 and Vulcraft)	
Description	**Weight (psf)**
MEP	12
3”x20ga Composite Decking	48
Steel Framing	13
Finishes and Partitions	15
Fireproofing	2
Miscellaneous	5
Total	**95**

Live Loads

See *Table 5* for the controlling live load description per each level with the exception of elevator lobbies and stairs. The live loads given on the structural general notes were obtained using ASCE7-02, they were rechecked according to ASCE7-05 and were deemed accurate, see *Table 6.*

Table 5 – Live Loads

Live Loads (As Shown on General Notes S100)	
Description	**Weight (psf)**
Entry	100
Basement	100
Level 1	100
Level 2	100
Level 3	80
Level 4	80
Level 5	80
Level 6	80
Level 7	80
Level 8	80
Level 9 (Mechanical Penthouse)	125
Elevator Lobbies and Stairs	100

Table 6 – Verifying Live Loads per ASCE7-05

Level 1 – Level 2; Verification (ASCE7-05)	
Occupancy	**Weight (psf)**
Assembly Areas – Lobby	100
Hospitals – OR Rooms	60 + Partitions
Hospitals – Patient Rooms	40 + Partitions
Hospitals – Corridors above 1st Floor	80

Existing Floor System

Spot checks were performed on a typical 27'x30' bay located on Level 3; columns J-1, J-2, K-1, and K-2 make up the corners for the bay see *Figure 11*. Spot checks were performed to utilize knowledge learned in previous courses to verify the structural system of the building. Complete hand calculations are located in Appendix A.

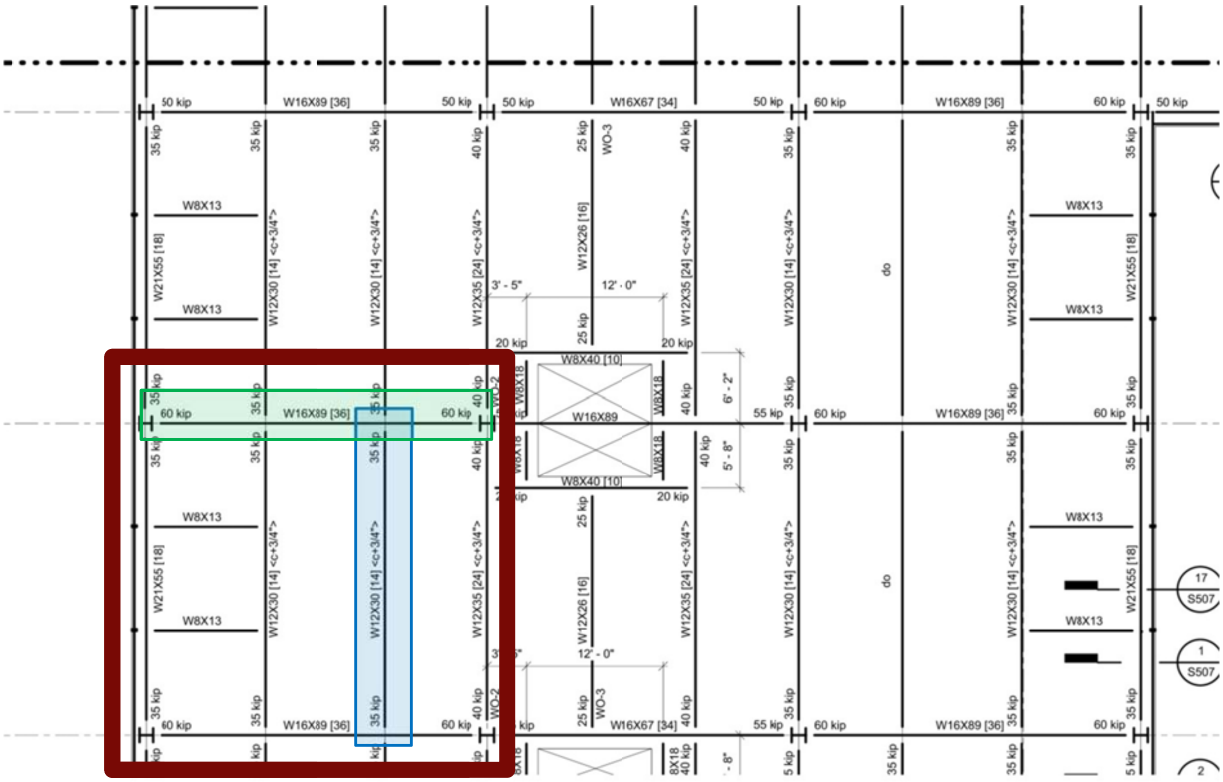


Figure 11 – Typical Bay

General

The existing composite system weighs 60 pounds per square foot (psf), and costs approximately $17.76 per square foot, the second cheapest system. This system was chosen because within the area of Albany, New York steel is cheaper than concrete and most of the contractors are more familiar with steel construction. The cost of the system was derived using RS Means Costs Works.

Decking

Typical floor construction for the Patient Pavilion utilizes a 3"x20ga unshored composite steel deck with 3 1/2" lightweight concrete topping. Using Vulcraft Steel Decking (2008) catalog, the specified deck type is acceptable according to the allowable strengths and unshored lengths for a 3VLI20. The allowable superimposed live load is 157psf, roughly double the live load on Level 3; a possible reason for this is because the live load at the penthouse is 125psf, increasing the demand for strength in the deck. Verifying the fire rating with the 2001 Fire Resistance Directory, a 3 1/2" lightweight concrete topping allows for a 2-hour rating. After verifying the fire rating, the deck size was governed by the fire rating not strength.

Beam

Gravity spot checks were performed on a W12x30 beam highlighted in blue in *Figure 11.* This is the most typical beam used throughout, spanning 27'-0" with a tributary width of 10'-0". Hand calculations as well as Table 3-19 in the AISC were used to calculate strength and deflection for both pre-concrete curing and post concrete curing. The spot check analysis verified that the W12x30 with 14 studs was adequate for the given loading on Level 3.

Girder

The girder analyzed was a W16x89 with 36 studs, as shown highlighted in green in *Figure 11*. Spanning 30'-0" this girder is typical from Level 3 up. The girder was analyzed using a tributary width of 27'-0", the live load was reduced and the distributed load along the girder was transformed into two point loads. The girder was checked for flexural, shear, and unshored strength, as well as wet concrete deflection and live load deflection. After analyzing the girder, it was found that the strength of the girder was approximately 45% larger than the ultimate moment of the floor loads. The reason for a larger capacity could be because the live load in the elevator lobbies is 20psf greater than the rest of the floor, and the girder is typical throughout the building.

Architectural

The low floor-to-floor heights on Level 3 up, restrict the beams and girders from going deeper than 16", this requires heavy sections to resist the factored loads. A heavier topping had to be utilized to meet the fire-rating minimum of 2 hours; also, spray fireproofing had to be added to all structural members.

Structural

The composite system is the most practical for the long spans that are needed for the Patient Pavilion; the structural system should not change for this system.

Serviceability

Maximum deflections were calculated for wet concrete deflection and live load deflection. The deflections for this system are deemed adequate for serviceability with a total deflection of 0.89 inches. A deflection criterion is stringent in hospitals due to operating rooms and intensive care units.

Pros and Cons

Pros:

- Lightweight system
  - Less seismic base shear
  - Smaller foundations
- Low cost per square foot
- Very flexible to meet floor-to-floor height requirements
- Small deflection

Cons:

- Floor vibrations are average
  - Deeper slabs utilized in critical areas to reduce vibrations
- Additional spray fireproofing needed
  - Additional construction cost
  - Delays construction

Two-Way Flat Slab with Drop Panels

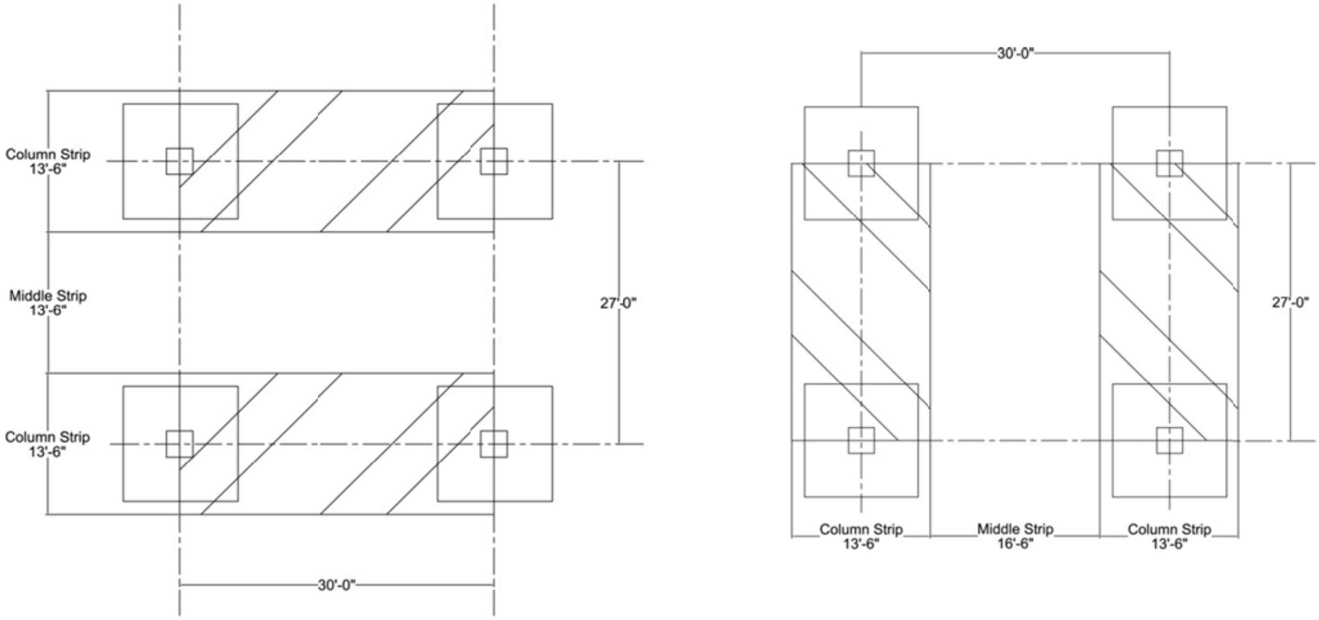


Figure 13 – Long Span

The design of the two-way reinforced flat slab systems is comprised of a 10" inch slab with 5000psi concrete and at the columns a 12 1/2" thick drop panel was utilized to prevent punching shear. Drop panels were sized using ACI318-08 section 13.2.5, resulting in an 11ft x 11ft drop panel 10ft x 10ft, see *Figure 14* for drop panel detail. The two-way slab is reinforced with #6 bars, with spacing fluctuating depending on the location of the bar.

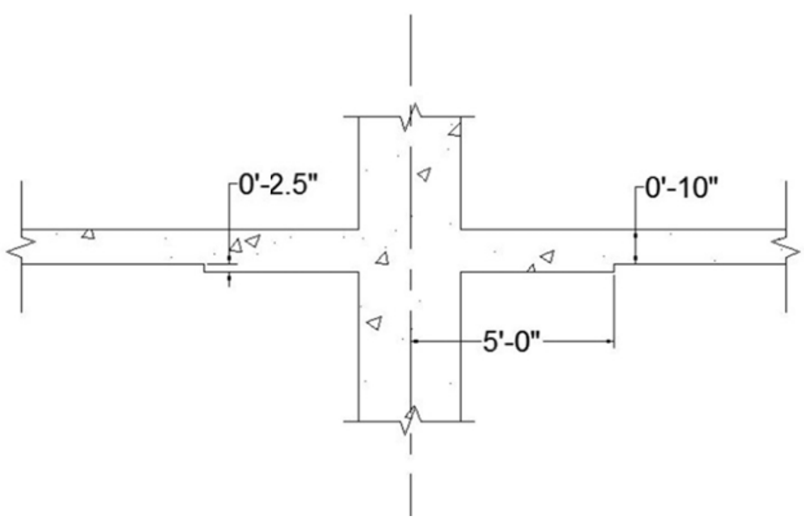


Figure 14 – Drop Panel Detail

Preliminary column sizes were needed in order to proceed with the slab analysis, using spColumn and analyzing a column on Level 3, it was found that a 30"x30" column would be sufficient for the factored axial loading. The Direct Design Method prescribed in ACI318-08 was used to obtain positive and negative bending values for the slab. Minimum slab thickness was determined per Table 9.5(c), using the larger of the two spans, t_{min} was found to be 10". After the moments were calculated in each direction, moments were distributed to column and middle strips, per ACI318-08 section 13.6.4.

General

The two-way flat plate system is one of the more costly of the systems at $20.65 per square foot; this was expected due to the cost of concrete in the Albany, New York area. This is also the heaviest system; however, it has the smallest total system depth, which is an advantage due to small floor-to-floor heights. The increase in weight due to the deep slab would increase the size of the foundations and the lateral resisting systems.

Architectural

The two-way flat slab with drop panels is an ideal system for the floor-to-floor height restrictions on this project allowing for more space for MEP. Fire rating for this system is achieved without any additional fireproofing, therefore reducing ceiling finishing costs in permitted areas. Some architectural spaces would be impeded because of the use of concrete shear walls in the building to resist the lateral forces.

Structural

Although the floor-to-floor heights are easily met with this system, it is not a viable solution structurally due to the total weight of the system, and the larger deflections. However the extra weight to benefit this project specifically because an issue with soil rebounding arose during construction of this building. There was elastic rebounding due to the unloading of the weight of the excavated material, and the extra weight of the system on this subgrade could counteract this uplift.

Serviceability

Long-term deflections were calculated assuming 25% of the live load is dead load after a long time; the deflections met the serviceability standards set by ACE. The long-term deflections were calculated to be 1.19", which would not be ideal for a hospital due to disruptions of ceiling equipment that is finely calibrated.

Pros and Cons

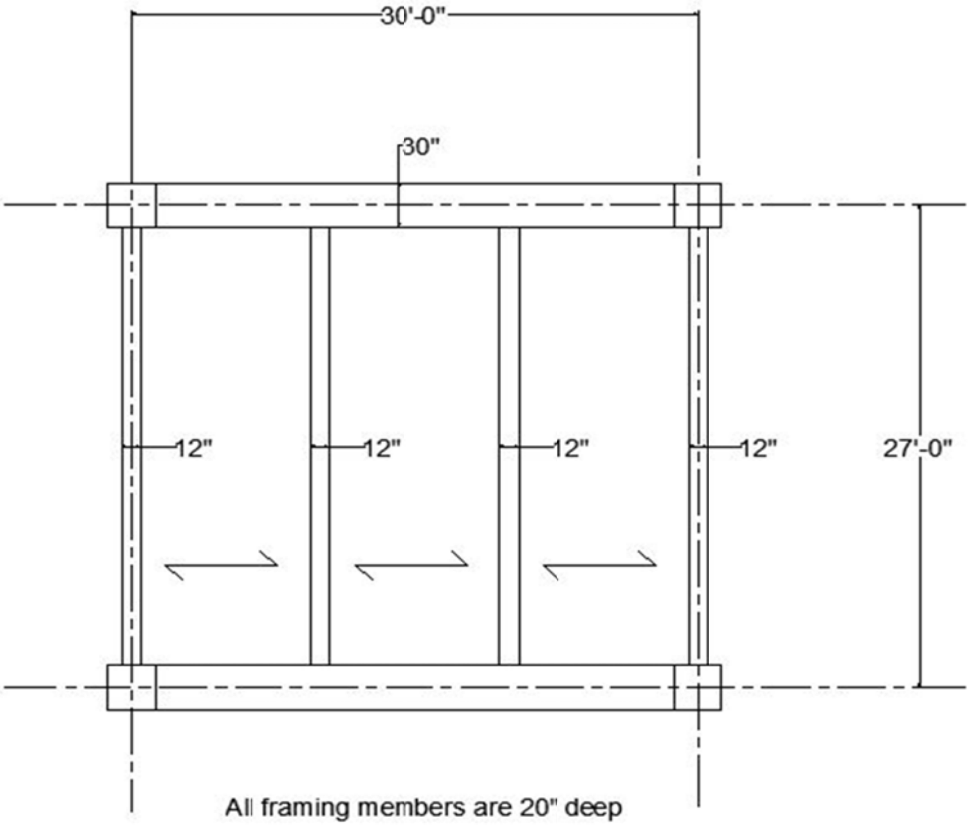
Pros:

- Low floor-to-floor heights
- No fire-proofing required
- Cost saving on ceiling finishes
- Minimal formwork

Cons:

- Large deflections
- Difficult to construct
- Tedious formwork for drop panels
- Longer construction timeline

One-Way Concrete Slab and Beam System



Another alternate floor system is a one-way concrete slab and beam system, the slab depth is 5" and total depth is 20". The same column dimensions of 30"x30" were also used in this system, the compressive strength of concrete is 4000psi, and the beam depths are the same as the girder depths for constructability.

The one-way slab spans 10'-0" between the two-infill beams spanning 27'-0", sitting on a 30" wide by 20" deep girder that spans 30'-0". To prevent tedious formwork between the column face and the girder, the girder is 30" to be flush with the faces of the column. When designing the beams ACI moment coefficients were used to find the negative and positive moments for the span. The moments in the girders were obtained by modeling the girder in STAAD Pro. Refer to Appendix C for hand calculations.

General

The one-way slab and beam system is the most costly system used due to the amount of formwork required compared to a two-way flat slab. The increase in weight will create a greater demand for the size of the foundations as well as the seismic detailing due to an increase in base shear, but the additional mass is better for vibration resistance. The one-way system is obsolete to the one-way composite system because of cost, total deflection, and weight.

Architecture

Fireproofing is not needed in this system because the 2-hour fire rating is obtained by providing a minimum clear cover of 3/4" for slabs and 1 1/2" for beams and girders. The depth of this system is appropriate for the floor-to-floor height limitations, but it would be more efficient if the column spacing were rearranged to reduce the depth of the beam and girders.

Structural

The floor weight is significantly increased compared to the one-way composite; this will help floor vibrations around critical areas; however, it will increase the size of foundations and the seismic base shear. Compared to a flat slab, less shear walls will be needed, because moment frames can be utilized to resist the lateral forces, this will create more openings within the building.

Serviceability

Total deflections for the beam were found by using deflection calculations given in Chapter 3 of the AISC 14th Edition, and for the girder, STAAD Pro was utilized to determine the deflections at third points on the girder. Adding the deflection of the girder where the beam rests to the total deflection of the beam gives an approximate total deflection. This deflection is .902 inches, which passes the deflection criteria of L/240 per Section 8.4.2 in ACI318-08. Refer to *Figure 16 and Figure 17* for STAAD deflection results for the girder.

Pros and Cons

Pros:

- No fire-proofing required
- Good vibration resistance

Cons:

- Most costly
- Increase in lateral loads

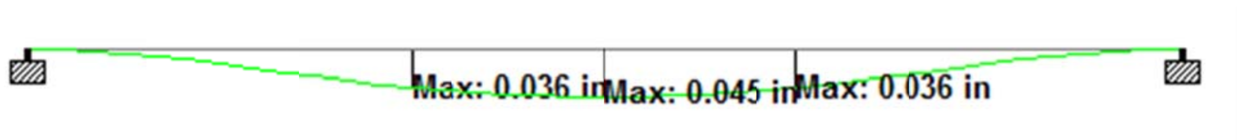


Figure 16 – Live Load Deflection of Girder

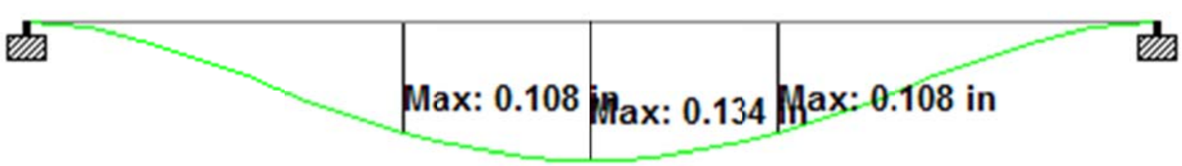


Figure 17 – Total Load Deflection of Girder

Hollow Core Precast Planks with Steel Framing

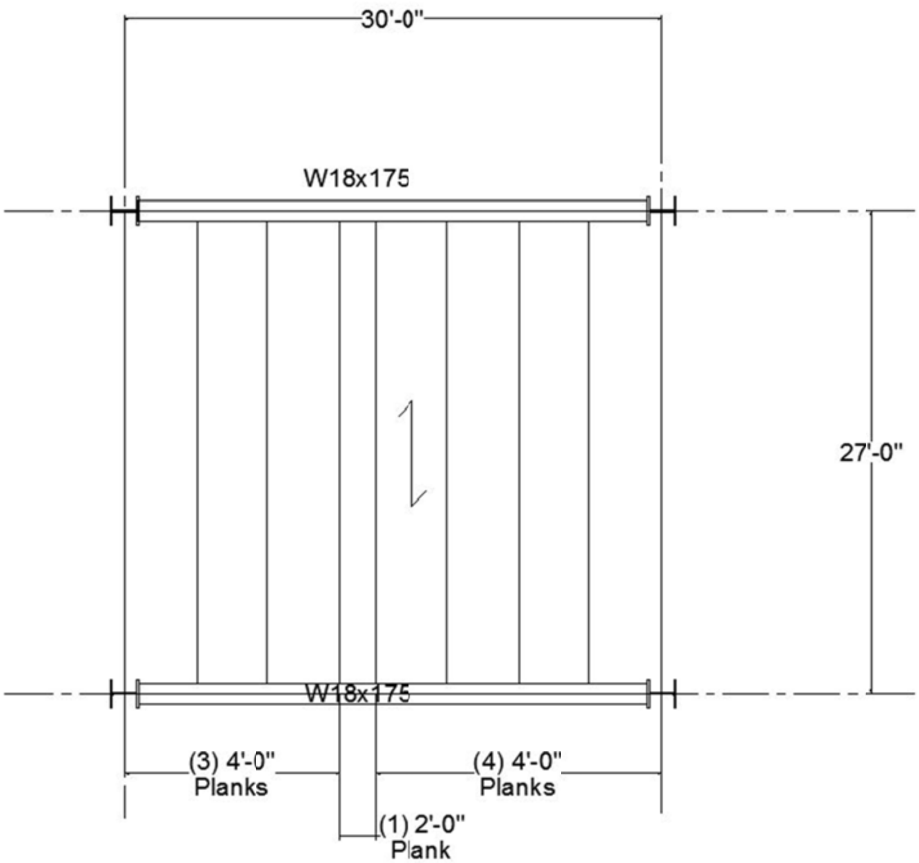


Figure 18 – Frame Layout

Hollow core precast plank with steel framing spanning 27'-0" demanded for an 8" plank and a 2" topping added for floor vibrations and fire-rating requirements. The planks rest on a seated connection consisting of an angle welded to the web of the girder in order to comply with the floor-to-floor height limitations see *Figure 19*.

The hollow core plank was chosen from the Nitterhouse catalog, using an 8"Dx4'-0"W plank with (6) 1/2" ϕ 270kip tendons, see *Figure 20*. Using 6 tendons provides the option of cutting a plank in half without cutting through any of the tendons. Cutting the 4'-0" plank into a 2'-0" plank is necessary because the bay width is 30'-0", (7) 4'-0" planks and (1) 2'-0" plank must be used to accommodate the bay sizes.

General

Hollow core precast plank is the cheapest and relatively lightest; however it is not a viable solution for the Patient Pavilion. The issue with this system is the girder needs to be very deep in order to support the weight of the plank over a large span. Making the girder deeper is the only solution and in turn, this will affect the allowable total system depth.

Architectural

The hollow core plank chosen has a 2-hour fire rating incorporated in its design; however the steel girders would still need fire proofing. Column spacing should be rearranged in order to reduce the span of the steel girder, in turn reducing the depth of the girder.

Structural

The weight of the hollow core plank demands for a heavy deep girder, making this not a viable solution for the Patient Pavilion. The depth of a W18x175 is 20.04 inches with the plank sitting on top of the girder; the total system depth would be 30" deep, which is not acceptable. Using the connection shown in *Figure 19,* a depth of 20" will be achieved and the total depth will be acceptable for floor-to-floor height restrictions.

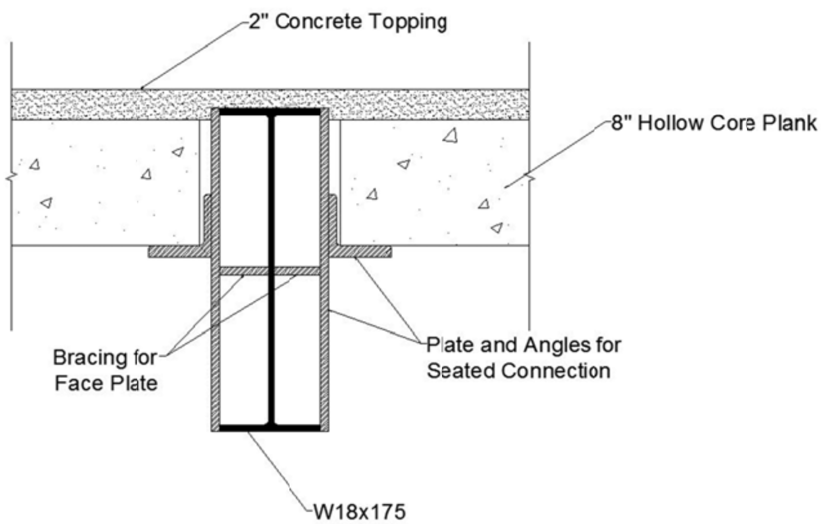


Figure 19 – Connection Detail

Serviceability

The overall deflection of the hollow core plank is very small which is due to the initial camber in the system, however the span of the girder and the weight of the plank results in a large deflection. The total deflection of this system is 0.97 inches, which passes the deflection criteria of L/240.

Pros and Cons

Pros:

- Minimal formwork
- Less weight

Cons:

- Construction difficult
- Heavy girders
- Large deflection
- Slab renovation is difficult

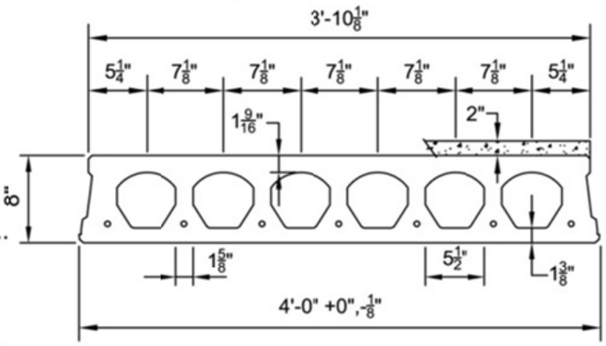
Prestressed Concrete
8"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 301 in.^{2}	Precast b_w = 13.13 in.
I_c = 3134 in.^{4}	Precast S_{bcp} = 616 in.^{3}
Y_{bcp} = 5.09 in.	Topping S_{tct} = 902 in.^{3}
Y_{tcp} = 2.91 in.	Precast S_{tcp} = 1076 in.^{3}
Y_{tct} = 4.91 in.	Precast Wt. = 245 PLF
	Precast Wt. = 61.25 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 = 92.3 k-ft at 60% jacking force
   6-1/2"Ø, 270K = 130.6 k-ft at 60% jacking force
   7-1/2"Ø, 270K = 147.8 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)																		
		17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
4 - 1/2"ø	LOAD (PSF)	280	248	214	185	159	138	118	102	87	74	62	52	42						
6 - 1/2"ø	LOAD (PSF)	366	341	318	299	271	239	211	187	165	146	129	114	101	88	77	67	58	50	42
7 - 1/2"ø	LOAD (PSF)	367	342	320	300	282	265	243	221	202	181	161	144	128	114	101	90	79	70	61

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

8SF2.0T

Figure 20 – Nitterhouse Catalog

Floor System Summary

Table 7 – Alternate Floor System Summary

		Existing	Alternatives		
		One-way Composite	Two-way Flat Slab With Drop Panels	One-way Slab and Beams	Hollow Core Precast Planks with Steel Framing
General	Weight(psf)	60	150	100	95
	Cost ($/SF)	$17.76	$20.65	$21.76	$14.45
	Slab Depth (inches)	6.5	10	5	8
	Total System Depth (inches)	22.5	12.5	20	22
Structural	Lateral System Alterations	N/A	Shear Walls	Shear Walls and Moment Frame Detailing	Braced and Moment Steel Frames
	Foundations	N/A	Yes	Yes	Yes
Architectural	Bay Size Alteration	N/A	Yes	Yes	Possibly
	Fire-Rating	2 hr with Spray fireproofing	2 hr	2 hr	2 hr with spray fireproofing
Serviceability	Max Deflection (Inches)	.89	1.19	.902	.97
	Vibration	Average	Average	Good	Good
Construction	Constructability	Easy	Difficult	Moderate	Difficult
	Formwork	Minimal	Fair	Ample	None
Conclusion	Viable Option	N/A	Yes	Yes	No

Conclusion

The results of this study concluded whether or not each alternate floor system is a viable solution for the Patient Pavilion, as well as comparing them against the existing floor system. Hand calculations supplemented with computer modeling were used to obtain schematic sizes and deflections for each floor system. Comparing the aspects of each system structurally, architecturally, and for constructability helped determine the most viable solution.

The existing composite beam and composite deck system seems to be the most viable solution for the Patient Pavilion. The cost of this system is the second lowest at $17.76, which is expected in the area of Albany because of the high cost of concrete. A composite system requires minimal formwork and is easily constructed, which lowers the overall cost of the system. Because of the large bay sizes, a steel system is the most appropriate due to the flexibility of the beam sizes to meet floor-to-floor height requirements. This system is deemed the most viable solution out of the floor systems analyzed.

The two-way slab system with drop panels was the heaviest of the systems analyzed, however the low slab depth is superb for the floor-to-floor height restrictions. The advantages of this system include minimal formwork, and no fireproofing needed. Some disadvantages of this system include the difficulty of construction, large deflections, and the need for shear walls for lateral load resistance. The two-way slab system with drop panels is a viable solution to the Patient Pavilion, however with the large deflections and weight of the system it is not the best solution.

The one-way slab with beams system was the most expensive system analyzed, however it did have the best deflection control other than the existing composite system. The advantages of this system include low cost of formwork due to redundancy, good constructability, and a good damping response to floor vibrations. Some negative aspects of this system include costly, increase of floor weight, and larger seismic loads due to the increase in floor weight. The one-way slab with beams system is a viable solution to the Patient Pavilion; however, it is not the most optimal option.

The hollow core precast plank system is the cheapest of all the systems used, however this does not include the additional cost of the special connection. The advantages of this system include minimal formwork and fireproofingis not needed; however, these are obsolete compared to the disadvantages of this system. The disadvantages include large deflections, difficult construction, special in-shop connections, deep girders, and difficult to renovate due to the prestressed tendons. The hollow core precast plank system was determined to not be a viable solution for the Patient Pavilion.

Appendix A: Existing System Hand Calculations

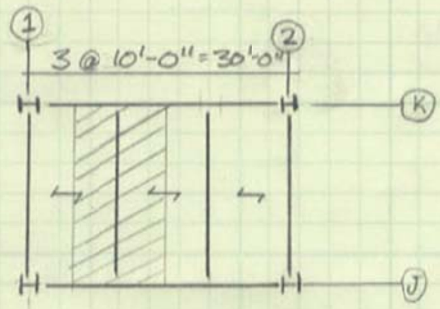
THOMAS KLEINOSKY | GRAVITY CHECKS | Page 1 of 10

FLOOR CONSTRUCTION

3" x 20GA COMPOSITE DECK
3½" LW CONCRETE

$t = 3 + 3\frac{1}{2} = 6\frac{1}{2}"$

$f'_c = 3500\,psi$



Typical Bay

FLOOR LOADS

DL = 95 psf
LL = 80 psf

DECKING (PER VULCRAFT 2008)

UNSHORED LENGTH

3VLI20 w/ 3½" topping - 3 span condition

13'-3"

13'-3" > 10'-0" ∴ OK ✓

Super Imposed live loads

allowable = 149 psf @ 10'-0" Clear span

149 psf > 80 psf ∴ OK ✓

Fire Resistance

From the 2001 Fire Resistance Directory

3½" LW Conc. Allows a 2hr Fire rating

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Beam Checks:

W 12×30 [14] <c + 3/4">

Tributary width = 10'-0"
Span = 27'-4"

W12×30 Properties

$F_y = 50 ksi$
$I_x = 238 in^3$
$A = 8.79 in^2$

Loads

LL = 80 psf → non-reducable
SDL = 47 psf
DL = 48 psf → Slab + Deck Self wt.

$w_u = 1.2D + 1.6L$

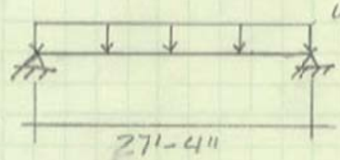
Dead

$(47+48) \times 10/1000 = 0.95 klf$

Live

$\left(\frac{80 psf}{1000}\right) \times 10 ft = 0.8 klf$

$w_u = 1.2(.95) + 1.6(0.8) = 2.42 klf$ + Beam Self wt.



$w_u = 2.42 klf$

$M_u = \frac{2.42(27.33^2)}{8} = 225 k\text{-}ft$

$V_u = \frac{2.42(27.33)}{2} = 33^k$ → drawings call out 35^k ∴ ok ✓

$b_{eff} = \min \begin{cases} 2 \times \frac{Span}{8} = \frac{2(27.33)}{8} \times 12 = 82 in. \rightarrow \text{Controls} \\ 2 \times \frac{1}{2} \text{ dist to adjacent beam} = 2 \times \frac{10}{2} \times 12 = 120 in. \end{cases}$

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For 3/4" ϕ studs + 3500psi conc.

$Q_n = 17.2^k$ (table 3-21)

assume $a \approx 1$in

$Y_2 = 6.5 - 1\text{in}/2 = 6$in

$\phi M_n = 244$ k-ft > 225 k-ft ∴ ok ✓

$\Sigma Q_n = 110^k$

number of studs $= \frac{\Sigma Q_n}{Q_n} = \frac{110^k}{17.2} = 6.4$ stud/side

$6.4 \times 2 = 12.8$ studs → use 14 studs

$\phi V_n = 95.9^k$ (table 3-6)

$95.9^k > 33^k$ ∴ ok ✓

Check unshored length:

W12x30 → $\phi M_p = 162$ k-ft

Construction Loads = 20psf

$w_u = 1.2D + 1.6L$

$w_u = 1.2(48\text{psf} \times 10\text{ft}) + 1.2(30\text{plf}) + 1.6(20\text{psf} \times 10\text{ft})$

$= 0.932$ klf

$M_u = \frac{0.932(27.33^2)}{8} = 87$ k-ft

$\phi M_p > M_u$ ∴ ok ✓

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Check wet concrete deflection:

$$w_{wc} = (10 \times 48) + 30 = 0.51 \text{ klf}$$

$$\Delta_{wc} = \frac{5 w l^4}{384 EI} = \frac{5(.51)(27.33)^4(1728)}{384(29000)(238)} = 0.93 \text{ in}$$

$$\Delta_{wc_{max}} = \frac{l}{240} = \frac{27.33(12)}{240} = 1.4 \text{ in.}$$

$$0.93 \text{ in} < 1.4 \text{ in} \therefore OK \checkmark$$

Check Live Load Deflection (Δ_{LL}):

$$w_{LL} = 80 \text{ psf} \times 10 \text{ ft} = 0.8 \text{ klf}$$

(table 3-20) $I_{LB} = 498 \text{ in}^4$

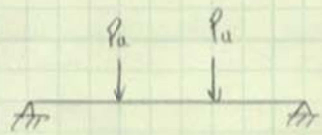
$$\Delta_{LL} = \frac{5(0.8)(27.33^4)(1728)}{384(29000)(498)} = 0.695 \text{ in.}$$

$$\Delta_{LL_{max}} = \frac{l}{360} = \frac{27.33 \times 12}{360} = 0.911 \text{ in.}$$

$$\Delta_{LL} < \Delta_{LL_{max}} \therefore OK \checkmark$$

$$\boxed{\text{Use a W12x30 [14]}}$$

Composite Girder: W16×89 [36]



Properties:
$F_y = 50$ ksi
$I_x = 1300$ in³
$A = 26.2$ in²
$t_f = 0.89$ in
$b_f = 10.4$ in

$P_u = P_{u_{DEAD}} + P_{u_{LIVE}}$

$L_{red} = 0.25 + \frac{15}{\sqrt{2(819.9)}} = 0.62$

Dead: $P_{u_D} = (95\text{psf} \times 10 \times 27.33) = 26^k$

Live: $P_{u_L} = (80\text{psf} \times 10\text{ft} \times 27.33\text{ft}) \times 0.62 = 13.6^k$

$P_u = 1.2(26) + 1.6(13.6) = 53^k$

$V_u = 53^k$ $M_u = 53^k \times 10\text{ft} = 530\text{ k-ft}$

Find b_{eff}

$b_{eff} = \min \begin{cases} 2 \times \frac{\text{Span}}{8} = 2 \times \frac{30}{8} \times 12 = 90\text{in.} \\ \text{Clear Span} = \cancel{27.33 \times 12} = \end{cases}$

Find ΣQ_n

For Girder, $Q_n = \min \begin{cases} R_g R_p A_{sc} F_u = 1(.75)(\frac{.75}{2})^2 \pi (65) = 21.5^k \\ .5 A_{sc} \sqrt{f'_c E_c} = .5(\frac{.75}{2})^2 \times \pi \sqrt{3.5(2160)} \\ = 19.2^k \rightarrow \text{Use} \end{cases}$

$\Sigma Q_n = 19.2^k \times \frac{36 \text{ studs}}{2} = 346^k$

Is it Partially composite?

$V'_c = 0.85(3.5)(10)(6.5) = 1740^k$
$V'_s = 26.2(50) = 1310^k$

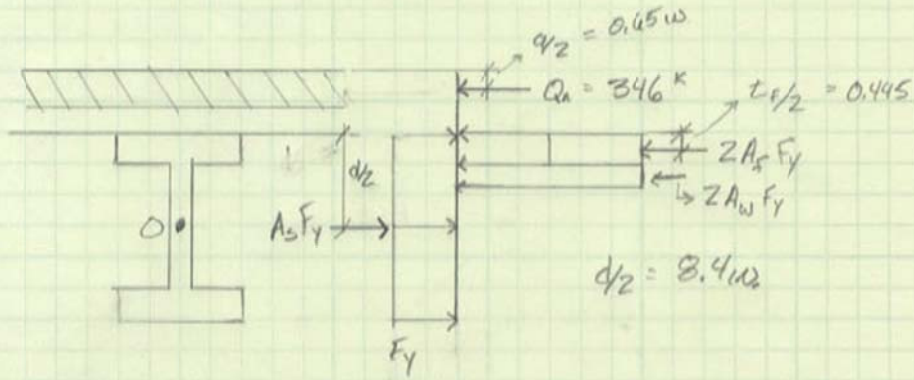
$\Sigma Q_n = 346 \le \begin{cases} 1740^k \\ 1310^k \end{cases}$ Yes, it's Partially Composite

Find a

$$a = \frac{346}{.85(3.5)(90)} = 1.3\text{ in}$$

Find PNA

$$PNA = \frac{1310 - 346}{2(50)(10.4)} = 0.93\text{ in} > 0.89\text{ in} \therefore \text{PNA in web}$$



Force in Web $= A_sF_y - \Sigma Q_n - 2A_fF_y$

$$= 1310 - 346 - 2(10.4 \times .875)(50) = 54^k$$

$$2A_wF_y = 2(t_f)y(F_y) = 2(0.5)y(50) = 54^k$$

$$y = 1.08\text{ in}$$

① $$\Sigma M_{n_o} = \left[8.4 - 0.875 - \left(\frac{1.08}{2}\right)\right] \times (2 \times 54) + \left[8.4 - \left(\frac{.875}{2}\right)\right] \times (2 \times 0.875 \times 10.4 \times 50) + \left[8.4 + (6.5 - 0.65)\right] \times 346 = 12931\text{ k-in}$$

$$\phi M_n = 0.9\left(\frac{12931}{12}\right) = 969\text{ k-ft}$$

969 k-ft > 530 k-ft ∴ OK ✓

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CHECK UNSHORED LENGTH:

$W16 \times 89 \rightarrow \phi M_p = 657 \text{ ft-k}$

$w_u = 1.2(48\text{psf} \times 27.33) + 1.2(89\text{plf}) + 1.6(20\text{psf} \times 27.33)$

$= 2.6 \text{ klf}$

$M_u = \frac{2.6(30^2)}{8} = 292.5 \text{ ft-k} \therefore \text{ok} \checkmark$

Check wet concrete deflection:

$w_{wc} = (48 \times 27.33) + 89 = 1.4 \text{ klf}$

$\Delta_{wc} = \frac{5(1.4)(30)^4(1728)}{384(29000)(1300)} = 0.382 \text{ in}$

$\Delta_{wc_{max}} = \frac{l}{240} = \frac{30(12)}{240} = 1.5 \text{ in} \therefore \text{OK}$

Check Δ_{LL}:

$\Delta_{LL} = \frac{5(0.8)(30^4)(1728)}{384(29000)(1300)} = 0.386 \text{ in.}$

$\Delta_{LL_{max}} = \frac{30(12)}{360} = 1 \text{ in} \therefore \underline{\text{OK}}$

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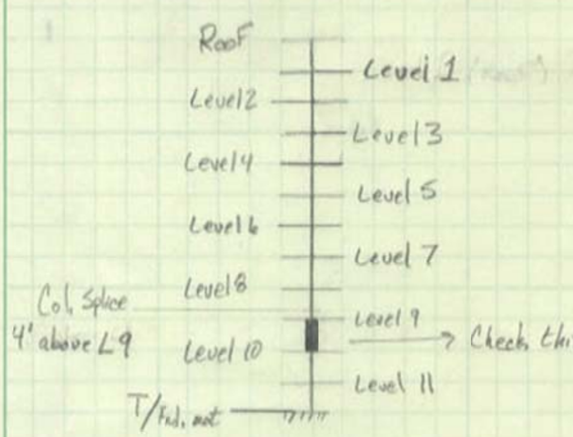
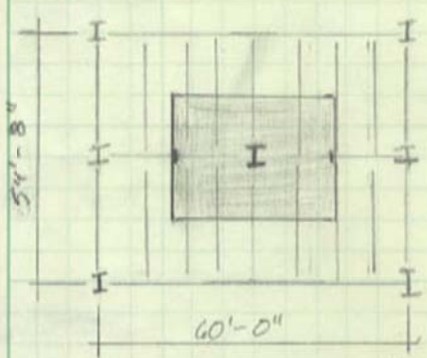
Column Check

Interior Column → J-4



W14x120 is from splice to roof.

→ Check this column, it's on Basement Level, W14x193



$$\text{Tributary Area} = \left(\frac{60'-0''}{2}\right)\left(\frac{54'-8''}{2}\right) = 820\,ft^2$$

$$\text{Influence Area} = 60' \times 54'-8'' = 3280\,ft^2$$

ϕP_n for a W14x193 @ 15 ft

$\phi P_n = 2210^k$

Loads

Dead

Floors = 95 psf
Roof = 30 psf
Level 1 = 125 psf

Live

Level 11 - 8 = 100 psf
Level 7 - 2 = 80 psf
Level 1 = 125 psf
Roof = 40 psf

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Column Loads

Roof:

$P_D = 30psf \times 820ft^2 = 24.6^k$

$P_L = 40psf \times 820ft^2 = 32.8^k$

Penthouse:

$P_D = P_L = 125psf \times 820ft^2 = 102.5^k$

Level 2:

$P_D = 95psf \times 820ft^2 = 77.9^k$

$u_{red} = .25 + \frac{15}{\sqrt{3280}} = 0.51$

$P_L = .51(80 \times 820) = 33.4^k$

Level 3

$P_D = 95psf \times 820 = 77.9^k$

$u_{red} = .25 + \frac{15}{\sqrt{2(3280)}} = 0.44$

$P_L = .44(80 \times 820) = 28.9^k$

Level 4

$P_D = 95 \times 820 = 77.9^k$

$u_{red} = .25 + \frac{15}{\sqrt{3(3280)}} = 0.25 \rightarrow 0.4$

$P_L = 0.4(80 \times 820) = 26.2^k$

Level 5

$P_D = 77.9^k$

$P_L = 0.4(80 \times 820) = 26.2^k$

Level 6 & 7

$P_D = 77.9^k$

$P_L = 26.2^k$

Level 8 & 9

$P_D = 77.9^k$

$P_L = (100psf \times 820) = 82^k$

Total

$P_D = 750.3^k \qquad P_L = 466.4$

$$\boxed{P_u = 1.2(750) + 1.6(466) = 1650^k < 2210^k \therefore \text{ OK } \checkmark}$$

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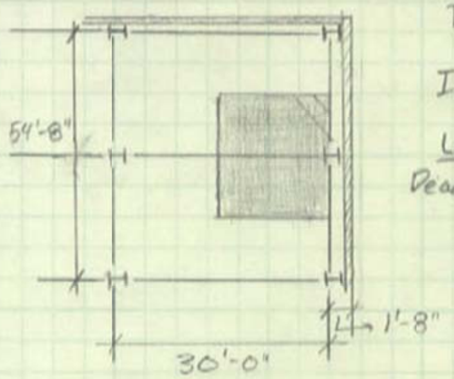
Exterior Column – 8-9



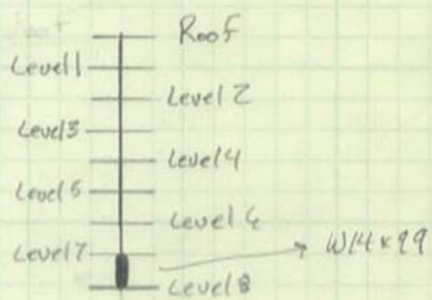
Tributary Area = 27'-4" × 16'-8" = 456 sq ft.

Influence Area = 54'-8" × 31'-8" = 1731 sq ft

Loads

Dead:
- Floor = 95 psf
- Roof = 30 psf
- Level 1 = 125 psf
- Facade = 48 psf

Live:
- Level 11-8 = 100 psf
- Level 7-2 = 80 psf
- Level 1 = 125 psf
- Roof = 40 psf



Dead Loads

$P_{Roof} = 30 \times 456 = 13.7^k$

$P_{Floors} = 95 \times 456 = 43.3^k$

$P_{wall} = 89.25ft \times (54'\text{-}8") \times 48psf$
$= 234.2^k$ total

$P_{Level 1} = 125psf \times 456 = 57^k$

Live Loads

Level	LL_{red}	P_L
Roof	N/A	$40 \times 456 = 18.2^k$
Level 1	N/A	$125 \times 456 = 57^k$
Level 2	$.25 + 15/\sqrt{1731} = 0.61$	$0.61(80 \times 456) = 22.2^k$
Level 3	$.25 + 15/\sqrt{(2)1731} = 0.5$	$0.5(80 \times 456) = 18.2^k$
Level 4	$.25 + 15/\sqrt{(3)(1731)} = 0.46$	$0.46(80 \times 456) = 16.8^k$
Level 5	$.25 + 15/\sqrt{(4)(1731)} = 0.43$	$0.43(80 \times 456) = 15.7^k$
Level 6	$.25 + 15/\sqrt{(5)(1731)} = 0.41$	$0.41(80 \times 456) = 15.0^k$
Level 7	0.40	$0.4(80 \times 456) = 14.6^k$
	Total	177.7^k

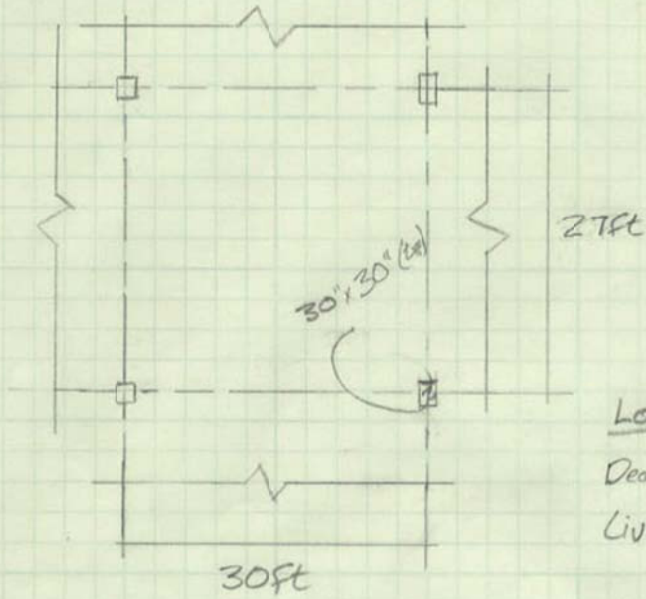
$P_u = 1.2(13.7) + 1.2(43.3 \times 6) + 1.2(57) + 1.2(234.2) + 1.6(177.7^k)$

$P_u = 962^k$

W14×99 @ 11ft $\rightarrow \phi P_n = 1190^k > 962^k \therefore$ OK ✓

Appendix B: Flat Slab with Drop Panels

2-way Flate Slab



<u>Assume:</u>

$f'_c = 5000\text{psi}$

$f_y = 60\text{ ksi}$

<u>Loads</u>

Dead = 34psf + slab

Live = 80psf

<u>Minimum Slab thickness</u>

(ACI 318-08) $t_{min} = \frac{\ell_n}{36} = \frac{(30\times12)-(\frac{30}{2}\times2)}{36} = 9.2\text{in}$

Use a 10in slab.

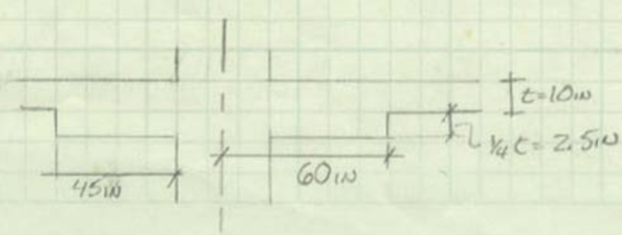
Slab Self weight $= \frac{10}{12} \times 150 = 125\text{psf}$

$w_u = 1.2(125+34)+1.6(80) = 320\text{psf}$

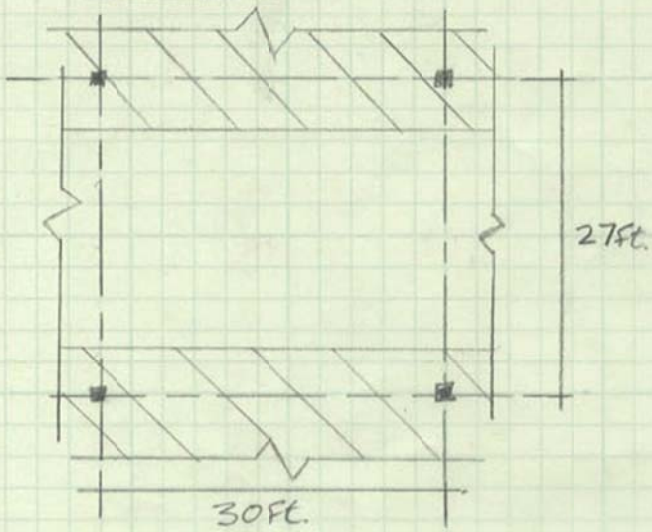
<u>Size of drop Panels</u>

$\ell/6$ or greater from ℄ of Column

$\frac{\ell}{6} = \frac{30\times12}{6} = 60\text{in} \rightarrow$ minimum



Long Span



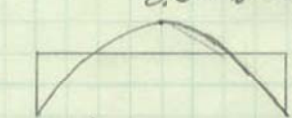
1/2 Column Strip $= \frac{27\times12}{4} = 81$ in

Middle Strip $= (27\times12) - (2\times81)$

$= 162$ in.

$$M_o = \frac{0.32(27)\left(30 - \frac{30}{12}\right)^2}{8}$$

$$M_o = 817 \text{ ft-k}$$



Positive Moment $\rightarrow M_o = 286$ ft-k $+ \alpha_1 \frac{l_2}{l_1} = 0$

60% to Column Strip $= 0.6(286) = 172$ ft-k

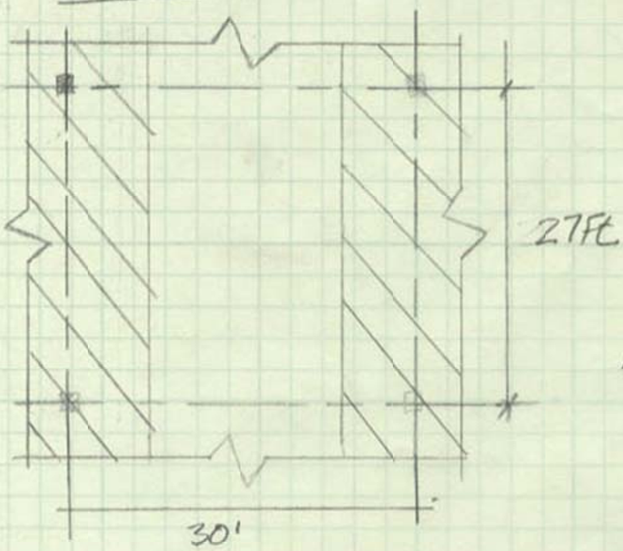
40% to Middle Strip $= 0.4(286) = 114$ ft-k

Negative Moment $\rightarrow M_o = 528$ ft-k $+ \alpha_1 \frac{l_2}{l_1}$

75% to Column Strip $= 0.75(528) = -396$ ft-k

25% to Middle Strip $= 0.25(528) = -132$ ft-k

Short Span



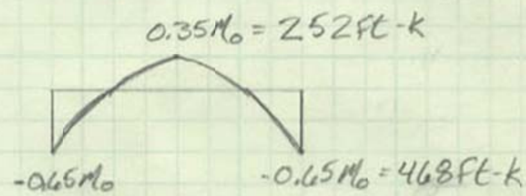
$\frac{1}{2}$ Column Strip = 81in
Middle Strip = $30 \times 12 - (81 \times 2)$
$= 198$in.

$$M_o = \frac{w_u l_2 (l_n^2)}{8}$$

$$= \frac{0.32(30)(27-\frac{30}{12})^2}{8}$$

$M_o = 720$ft-k

*Use direct design to distribute moments



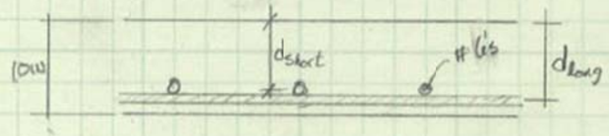
Positive Moments → $M_o = 252$ft-k & $\alpha_1 \frac{l_2}{l_1} = 0$

60% to Column Strip = 0.6(252) = 152ft-k
40% to Middle Strip = 0.4(252) = 100ft-k

Negative Moments → $M_o = 468$ft-k + $\alpha_1 \frac{l_2}{l_1} = 0$

75% to Column Strip = 0.75(468) = -351ft-k
25% to Middle strip = 0.25(468) = -117ft-k

Reinforcement design - Long Span



$d_{short} = 10 - 0.75 - \frac{0.75}{2} = 8.875 in$

	Full column strip		Middle Strip	
	M^+	M^-	M^+	M^-
M_u (ft-k)	172	-396	114	-132
Width of strip (in)	162	162	162	162
effective depth (in)	8.875	8.875	8.875	8.875
$\frac{M_u \times 12}{b}$ (k-in/in)	12.7	29.3	8.44	9.8
$\frac{M_u}{\phi}$	191.1	-440	126.7	-146.7
$R = \frac{M_n}{bd^2}$	177.1	-413.8	119.2	-138
ρ	0.00302	0.0073	0.00202	0.0023
$A_s = \rho bd$	4.34	10.5	2.9	3.31
$A_{s_{min}} = 0.0018bt$	2.92	2.92	$\boxed{2.92}$	2.92
$N = \frac{A_s}{0.44}$	$\boxed{10}$	$\boxed{24}$	8	8
$N_{min} = \frac{b}{2t}$	9	9	$\boxed{9}$	$\boxed{9}$
Spacing	14.5in	6in	18in	18in

Reinforcement design → Long Span

Drop Panel

$d_{long} = 12.5 - .75 - \frac{0.75}{2} = 11.4$ in.

Column Strip

	M^-
M_n	396 ft-k
Width	120 in
d	11.4 in.
M_u	440
R	338.6
ρ	0.00589
$A_s = \rho bd$	8.06 in^2
$A_{s,min} = 0.0018bh$	2.7 in^2
N	$\boxed{19}$
$N_{min} = \frac{b}{2t}$	5
Spacing	$\boxed{6\text{ in.}}$

Short Span

$d_{short} = 10 - .75 - .75 - .75/2 = 8.125_{in}$

	full column strip		Middle Strip	
	M^+	M^-	M^+	M^-
M_u (ft-k)	152	-351	100	-117
Width of strip	162	162	198	198
effective depth	8.125	8.125	8.125	8.125
$\frac{M_u \times 12}{b}$ (k-in/in)	11.3	26	7.4	8.7
$M_n = \frac{M_u}{\phi}$	168.9	390	111.1	130
$R = \frac{M_u}{bd^2}$	189.5	437.6	102	119.3
ρ	0.0032	0.0077	0.0017	0.00202
$A_s = \rho bd$	$\boxed{4.25}$	$\boxed{10.15}$	2.75	3.25
$A_{min} = 0.00186t$	2.92	2.92	$\boxed{3.56}$	$\boxed{3.56}$
$N = \frac{A_s}{0.44}$	$\boxed{10}$	$\boxed{24}$	9	9
$N_{min} = \frac{b}{2t}$	9	9	$\boxed{10}$	$\boxed{10}$

Short Span

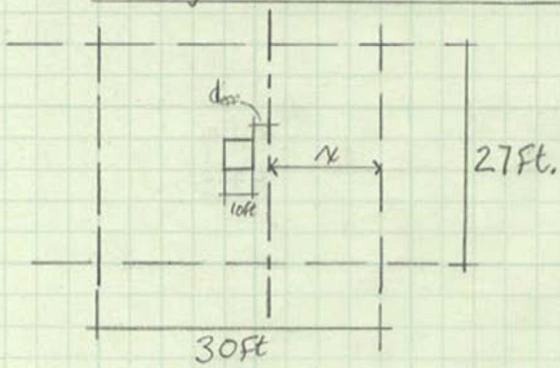
Drop Panel

$d_{short} = 12.5 - .75 - .75 - .75/2 = 10.625 in$

Column Strip

	M^-
M_n	-351
Width	120
d	10.625
M_u	390
R	345.5
ρ	0.0060
$A_s = \rho bd$	$7.65 in^2$
$A_{s min} = 0.0018 bt$	$2.7 in^2$
N	$\boxed{18}$
$N_{min} = \frac{b}{2t}$	5

Punching Shear – Wide Beam Action



$d_{eff} = 11$ in.

$x = 15 - \frac{11}{12} - \frac{10}{2} = 9.08$ ft

$V_u = \omega_u \times Area = 0.320 \times (27 \times 9.08) = 78.5^K$

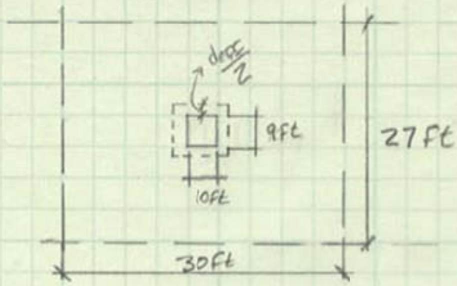
$V_n = 2\sqrt{f'_c}\, b_w d$

$= 2\sqrt{5000} \times (27 \times 12) \times (11) = 504^K$

$\phi V_n = 0.75(504) = 378^K$

$78.5^K < 378^K \therefore$ OK

Punching Shear → two way action



$d_{eff} = \frac{11.375 + 10.625}{2}$

$d_{eff} = 11\text{IN}$

$V_u = W_u \times Area = 0.320\left[(30\times 27) - \left(9+\frac{11}{12}\right)\left(10+\frac{11}{12}\right)\right]$

$V_u = 104^k$

$\phi V_c = \left(\frac{\alpha_s}{b_o/d} + 2\right)\sqrt{f'c}\, b_o d$

$b_o = 2\left[\left(10+\frac{11}{12}\right) + \left(9+\frac{11}{12}\right)\right] \times 12 = 500\text{in}$

$\frac{b_o}{d} = \frac{500}{11} = 45.5$

$\alpha_s = 40$ for interior Column

$V_c = \left(\frac{40}{45.5} + 2\right)\sqrt{5000} \times 500 \times 11 = 1120^k$

$\phi V_c = 1120 \times 0.75 = 840^k$

Deflection

Service Loads

$w_D = 34 + 125 = 159\text{psf}$
$w_L = 80\text{psf}$
$w_{const} = 20\text{psf}$

Assume:

67.5% moment is in column strip
32.5% moment is in middle strip

Column Strip along lines

$$I_g = \frac{(\ell_2/2)(t^3)}{12} = \frac{\frac{27\times 12}{2}(10^3)}{12} = 13500\text{ in}^4$$

$$E = 57000\sqrt{f'_c} = 57000\sqrt{5000} = 4030\text{ ksi}$$

Loads:

$w_D = 159\text{psf} \times 27\text{ft} = 4.3\text{klf} \times 0.675 = 2.9\text{klf}$

$w_L = 80\text{psf} \times 27\text{ft} = 2.2\text{klf} \times 0.675 = 1.5\text{klf}$

$w_{const} = 20\text{psf} \times 27\text{ft} = 0.54\text{klf} \times 0.675 = 0.36\text{klf}$

$$\Delta_{D_{max}} = 0.0026\frac{w_D \ell^4}{E I_g} = 0.0026\frac{2.9(30^4)}{4030(13500)} \times 12^3 = 0.193\text{ in.}$$

$$\Delta_{L_{max}} = 0.0048\frac{w_L \ell^4}{E I_g} = 0.0048\frac{1.5(30^4)}{4030(13500)} \times 12^3 = 0.185\text{ in}$$

$$\Delta_{long\ term} = 3(0.193 + 0.25(0.185)) = 0.718\text{ in}$$

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Deflection

Middle Strip

$$I_g = \frac{(198)(10)^3}{12} = 16{,}500\,in^4$$

$$\omega_D = 159 \times 30 \times 0.325 = 1.6\,klf$$

$$\omega_L = 80 \times 30 \times 0.325 = 0.78\,klf$$

$$\Delta_{D_{max}} = \frac{0.0026(1.6)(27^4)}{4030(16500)} \times 12^3 = 0.0575\,in.$$

$$\Delta_{L_{max}} = \frac{0.0048(0.78)(27^4)}{4030(16500)} \times 12^3 = 0.0517\,in$$

$$\Delta_{Long} = 3(0.0575 + 0.25(0.0517)) = 0.211\,in$$

Immediate Live Load deflection

$$\Delta_L = 0.185 + 0.0517 = 0.236\,in$$

$$\text{ACI Code Limit} = \frac{l}{360} = \frac{30 \times 12}{360} = 1.0\,in \therefore OK$$

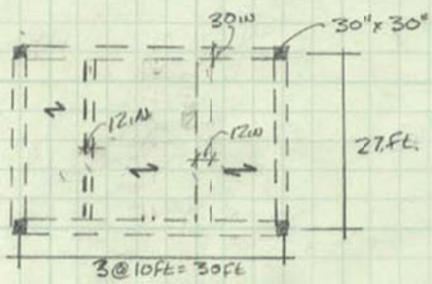
Check total deflection after attachment of Partitions

$$\Delta_{max} = 0.1(0.193 + 0.0575) + 0.236 + (0.718 + 0.211)$$

$$\Delta_{max} = 1.19\,in$$

$$\text{ACI Code Limit} = \frac{l}{240} = \frac{30 \times 12}{240} = 1.5\,in \therefore OK$$

One-Way Slab with Beams

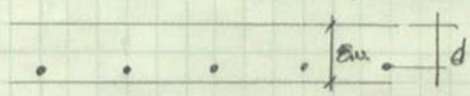


$F'_c = 4000\text{psi}$
$F_y = 60\text{ksi}$

Minimum Slab Thickness Per ACI 9.5(a)

Exterior Bay $= \frac{l}{24} = \frac{10 \times 12}{24} = 5.0\text{in}$

Use 8" Slab with #4s



$d = 5 - 0.75 - \frac{0.5}{2} = 4\text{in.}$

Slab Self weight $= 5/12 \times 150 = 62.5\text{psf}$

$w_D = 62.5 + 34 = 96.5\text{psf}$

$w_L = 80\text{psf}$

$w_u = 1.2(96.5) + 1.6(80) = 244\text{psf}$

Max Negative Moment $\Rightarrow M_u = -\frac{w_u l_n^2}{11} = -\frac{0.244(10 - \frac{12}{12})^2}{11} = 1.8\text{k-ft}$

Reinforcement

$A_s = \frac{M_u}{\phi F_y(jd)}$ & $jd = 0.95d$

$= \frac{1.8 \times 12}{0.9(60)(.95)(4)} = 0.105\ \text{in}^2/\text{ft}$

$a = \frac{.105(60)}{.85(4)(12)} = 0.154\text{in} \rightarrow \phi = 0.9$

$A_s \geq \frac{1.8 \times 12}{0.9(60)(4 - \frac{.154}{2})} = 0.101\ \text{in}^2/\text{ft}$ ∴ use #4 @ 12" OC.

$\rho = \frac{0.2}{12 \times 7} = 0.0023$

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Shear Check

$$V_u = \frac{1.5 w_u l_n}{2} = \frac{1.5(0.299)(10 - \frac{12}{12})}{2} = 1.65 \text{ k/ft width}$$

$$\phi V_c = 0.75(2\lambda\sqrt{f'_c}\, b_w d)$$

$$= 0.75(2(1))\sqrt{4000}\,(12)(4) = 4.5 \text{ k/ft width}$$

$$A_{s,min} = 0.0018bh = 0.173 \text{ in}^2 \therefore A_s = 0.189 \text{ in}^2 \text{ Governs}$$

Reinforcement Spacing

$$s = 15\left(\frac{40000}{f_s}\right) - 2.5C_c \le 12\left(\frac{40000}{f_s}\right)$$

$$s = 15\left(\frac{40000}{(2/3)40000}\right) - 2.5(.75) = 13.125 \le 12 \text{ in} \rightarrow \text{Governs!}$$

Shrinkage and temperature

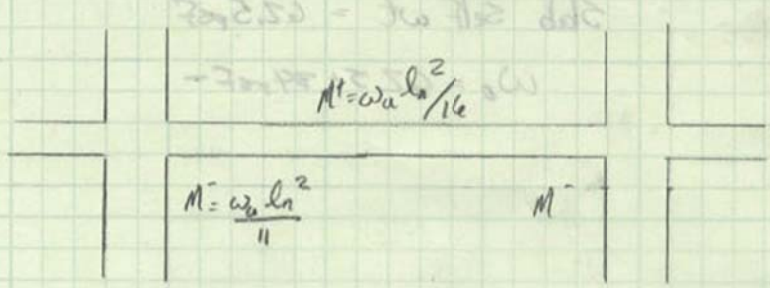
$$A_{s(S+T)} = 0.0018(12)(8) = 0.1728 \text{ in}^2 \rightarrow \text{use \#4s}$$

Max Spacing ≤ min | 5h = 5(8) = 40 in
| 18 in → Governs

Slab Summary:

- 5 in Slab with #4 Bars @ 12 in O.C. For top + Bottom Flexural steel
- No. 4 Bars @ 18 in For Shrinkage + temperature

Beam Design



Assume: - Girders are 30" wide, same width as columns
- Beam depth is the same as girder depth to reduce the amount of formwork
- Beams are 12in wide

Girder depth

$$\frac{l}{18} < h < \frac{l}{12} \rightarrow 20in < h < 30in$$

Use $h = 20\,in$

$$\text{Beam Self weight} = \frac{12in \times 20in}{144\,in^2/ft^2} \times 150\,lb/ft^3 = 250\,plf$$

$$\text{Slab Self weight} = \frac{5in}{12\,in/ft} \times 150\,lb/ft^3 = 62.5\,psf$$

$$w_d = 34\,psf + 62.5\,psf + \text{Beam Self weight}$$

$$w_L = 80\,psf$$

Live Load Reduce

Beams: $LL_r = 0.25 + \frac{15}{\sqrt{(16 \times 21) \times 2}} = 0.89$

Girder: $LL_r = 0.25 + \frac{15}{\sqrt{(27 \times 30) \times 2}} = 0.62$

w_u → for Beams

$$w_u = 1.2[(62.5 + 34) \times 9] + 1.6(0.89 \times 80 \times 10) = 2.2\,klf$$

$$M_u^+ = \frac{2.2(24.5^2)}{16} = 82.5\,k\text{-}ft + \frac{250(24.5^2)(1.2)}{16} = 94\,k\text{-}ft$$

$$M_u^- = \frac{2.2(24.5^2)}{11} = 120\,ft\text{-}k + \frac{.25(24.5^2)(1.2)}{11} = -137\,ft\text{-}k$$

$$V_u = \frac{2.2(24.5)}{2} = 26.95\,k + \frac{.25(24.5)(1.2)}{2} = 31\,k$$

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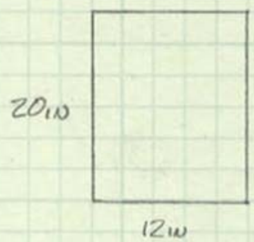
Beam Design At Midspan → $M_u = 94$ ft-k



Assume: $d = 18$in

$A_s = \frac{M_u}{4d} = \frac{94}{4(17.5)} = 1.34 \text{ in}^2$

Use (2) #8 $A_s = 1.58 \text{ in}^2$

$\rho \leq 0.0125 \quad \rho = \frac{1.58}{12 \times 17.5} = 0.007 \therefore$ OK

$d_{actual} = 20 - 1.5 - 0.5 - \frac{1}{2} = 17.5$in

↳ stirrup

$b_{eff} = \min \begin{cases} 1/4 \text{ Span} = 1/4(27\text{ft}) = 81\text{in} \rightarrow \text{Governs} \\ b_w + 16h_f = 12 + 16(8) = 92\text{in} \\ b_w + \text{Clear} = 12 + 10 \times 12 = 132\text{in} \end{cases}$

T-Bm behavoir

$M_{u_{T\text{-}Bm}} = \phi 0.85 f'_c b_{eff} h_f (d - h_f/2) = 0.9(0.85)(4)(81)(5)(17.5 - 5/2)$

$M_{u_{T\text{-}Bm}} = 1550$ ft-k $> M_u \therefore$ Rectangular

$a = \frac{A_s f_y}{0.85 b f'_c} = \frac{1.58(60)}{0.85(4)(12)} = 2.32$in.

$c = \frac{2.32}{0.85} = 2.73$

$\epsilon_s = \frac{0.003}{2.73}(17.5 - 2.73) = 0.016 > 0.005 \therefore \phi = 0.9$

$\phi M_n = \frac{0.9(1.58)(60)(17.5 - \frac{2.32}{2})}{12} = 116 \text{ ft-k} > 94 \text{ ft-k} \therefore$ ok

Spacing

$S = 12 - 1.5(2) - .5(2) - 2(1) - 2(1) = 2\text{in} \therefore$ OK

(stirrups)

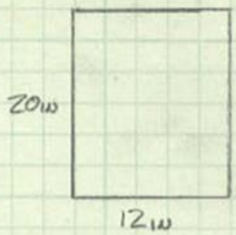
$A_{s_{min}} = \min \begin{cases} \frac{3\sqrt{f'_c}\, bd}{f_y} = \frac{3\sqrt{4000}(12)(17.5)}{60000} = 0.67\text{in}^2 < 1.58\text{in}^2 \therefore \text{OK} \\ \frac{200bd}{f_y} = \frac{200(12)(17.5)}{60000} = 0.7\text{in}^2 \quad \text{OK} \end{cases}$

Shear → use #4 @ 12in O.C.

$\phi V_n = \phi(V_c + V_s) = 0.75\left[2\sqrt{4000}(12)(17.5) + (2)(.2)(40000)\left(\frac{17.5}{12}\right)\right]$

$\phi V_n = 46.1^k > 31^k \therefore$ OK

Beam Design at Supports → $M_u = -137\,\text{ft-k}$



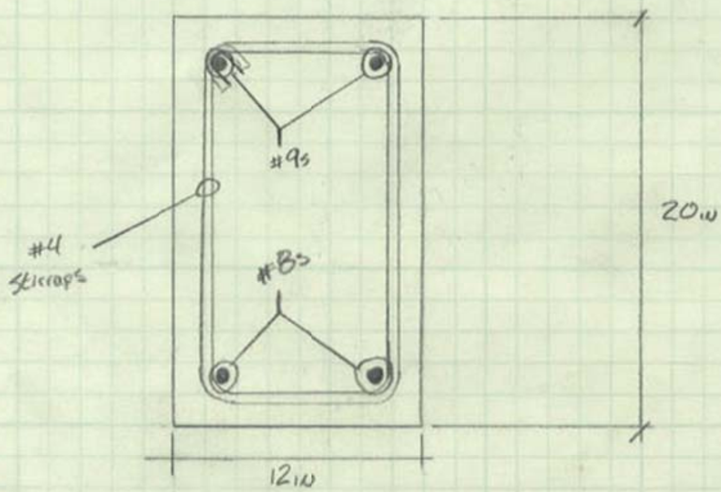
$$A_s = \frac{-137}{4(17.5)} = 1.96\,\text{in}^2 \therefore \text{use } (2)\ \#9s$$

$$\rho = \frac{1.96}{12(17.5)} = 0.009 \le 0.0125 \checkmark$$

$$a = \frac{(2)(60)}{0.85(4)(12)} = 2.94\,\text{in} \therefore c = 2.94/.85 = 3.46$$

$$\varepsilon_s = \frac{0.003}{3.46}(17.5 - 3.46) = 0.0121 > 0.005 \therefore \phi = 0.9$$

$$\phi M_n = 0.9(2)(60)\left(17.5 - \frac{2.94}{2}\right) = 144\,\text{ft-k} > 137\,\text{ft-k} \therefore \underline{OK}$$



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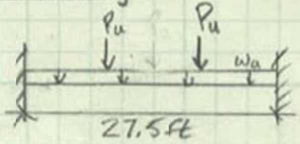
Girder Design



$$w_u = \left[\left(\frac{30}{12} \times \frac{20}{12}\right) \times 0.15\right] \times 1.2 = 0.75 \text{ klf}$$

$$P_u = \left[1.2(.(62.5+34)\times 9) + 1.2(250) + 1.6(.62 \times 80 \times 10)\right] \times \frac{24.5 \text{ ft}}{2} = 26.1^k$$

Using Staad to analyze the moments

$M^+ = 96$ k-ft

$M^- = 232$ k-ft

$b_{eff} = \min \begin{cases} 1/4 \text{ Span} = 1/4 \times 30 \times 12 = 90 \text{ in.} \rightarrow \text{Governs} \\ b_w + 16h_f = 30 + 16 \times 5 = 110 \text{ in} \end{cases}$

Midspan →

T-BM behavior:

$$M_{u_{T\text{-}BM}} = 0.9(0.85)(4)(90)(5)(17.5 - 2.5) = 1721 \text{ ft-k} \gg 130 \text{ ft-k}$$

∴ treat as a rect BM

$$A_s = \frac{M_u}{4d} = \frac{196}{4(17.5)} = 1.37 \text{ in}^2 \therefore \text{ use (2) \#9s}$$

$$a = \frac{2(60)}{0.85(4)(30)} = 1.18 \text{ in}$$

$$c = \frac{1.18}{0.85} = 1.39 \text{ in}$$

$$\varepsilon_s = \frac{0.003}{1.39}(17.5 - 1.39) = 0.035 > 0.005 \therefore \phi = 0.9$$

$$\phi M_n = 0.9(2.0)(60)(17.5 - \tfrac{1.18}{2}) = 152 \text{ ft-k} > 96 \text{ ft-k} \therefore \text{ok}$$

Moment @ Support → $M_u = -232$ ft-k

$$A_s = \frac{232}{17.5(4)} = 3.31 \text{ in}^2 \therefore \text{ use (5) \#8s}$$

$$a = \frac{3.95(60)}{0.85(4)(30)} = 2.32 \rightarrow c = 2.32/.85 = 2.73$$

$$\varepsilon_s = \frac{0.003}{2.73}(17.5 - 2.73) = 0.016 > 0.005 \therefore \phi = 0.9$$

$$\phi M_n = 0.9(3.95)(60)(17.5 - \tfrac{2.32}{2}) = 290 \text{ ft-k} > 232 \therefore \text{ok}$$

Girder Shear Check using #4 @ 12in OC.

$$V_u = \frac{0.625 \times 27.5}{2} + \frac{26.1}{2} = 21.6^k$$

$$\phi V_n = 0.75\left[2\sqrt{4000}(30)(17.5) + 2(.2)(60{,}000)\left(\frac{17.5}{12}\right)\right]$$

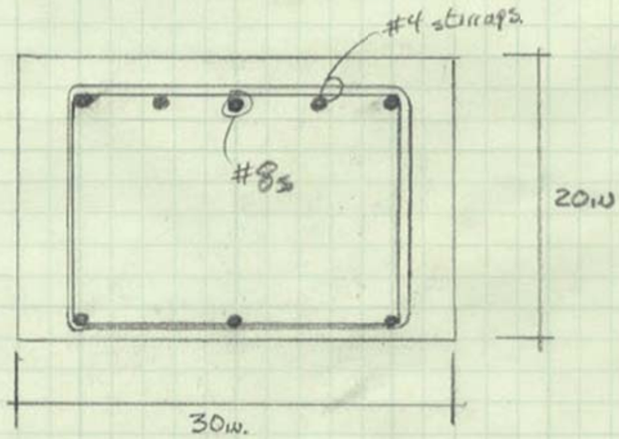
$$\phi V_n = 76^k >> 21.6^k$$

Max Spacing

$$S = 15\left(\frac{40000}{f_{ps}}\right) - 2.5C_c = 15 - 2.5(1.5) = 11.25in$$

$$\leq 12\left(\frac{40000}{f_{ps}}\right) = 12in \rightarrow \text{Governs}$$

Must use (3) #8s top & bottom to meet maximum spacing



$$A_{s_{min}} = \frac{3\sqrt{f'_c}\,bd}{f_y} = \frac{3\sqrt{4000}(30)(17.5)}{60000} = 1.7in^2$$

$1.7 < 2in^2$ & $3.95in^2$ ∴ OK

Deflection Checks:

Beam:

$I = 1/12\,(12)(20^3) = 8000\,in^3$

$E = 57000\sqrt{4000} = 3.61 \times 10^6\ ksi$

$\Delta_{LL} = \dfrac{5(80 \times 10)(27^4)1728}{384(3.61 \times 10^6)(8000)} = 0.33\,in$

$\Delta_{TL} = \dfrac{5\left[(42.5+34)(9)+250+(80 \times 10)\right]27^4 \times 1728}{384(3.61 \times 10^6)(8000)} = 0.794$

ACI Code Limitations

$\Delta_{LL} = \dfrac{\ell}{360} = 0.9\,in > 0.33\,in \therefore$ OK

$\Delta_{TL} = \dfrac{\ell}{240} = 1.35\,in > 0.79\,in \therefore$ OK

Girder: Done In Stadd

$\Delta_{LL} = 0.045\,in$

$\Delta_{TL} = 0.134\,in$

Appendix D: Hollow Core Precast Plank on Steel Framing

Hollow Core Plank

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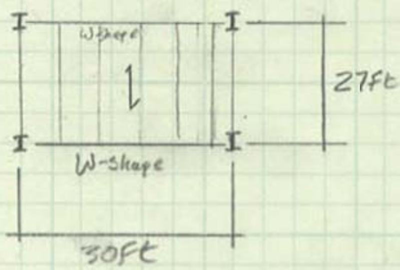

Loads

SDL = 34 psf

Live = 80 psf

w_{Tot} = 114 psf

Choose a Plank from Nitterhouse

Use a 8" Hollowcore with 2in topping

* Because Bay widths are not a factor of 4ft, a 2ft wide plank must be used, 7(4) + 2 = 30ft ∴ Choose a plank with even # of strands

↳ # planks @ 4ft

Use (6) ½" Φ Strand Pattern → allowable psf @ 27ft = 129 psf

Check Deflection

$$\Delta_{LL} = \frac{5(0.08)(4)(27)^4 \times 1728}{384(29000)(3134)} = 0.04 \text{ in.}$$

$$\Delta_{TL} = \frac{5(0.08+0.034+0.025+0.06125)\times 4 \times 27^4 \times 1728}{384(29000)(3134)} = 0.11 \text{ in}$$

Beam Design

$w_D = 1.2(34 + 61.25 + 25) = 145$ psf

$w_L = 1.6(80 \times (.25 + \frac{15}{\sqrt{2(30)(27)}})) = 79.7$ psf

Tributary Width = 27ft

Designed Beam using EnerCalc

Use a W18×175

$\Delta_{LL} = 0.24$ in.

$\Delta_{TL} = 0.86$ in