Technical Report 2

Nathan McGraw
Structural Option
Faculty Advisor: Dr. Richard Behr
October 19th, 2011

Inova Fairfax Hospital | South Patient Tower

Falls Church, VA

Table of Contents

Executive Summary:

The purpose of Technical Report 2 was to design three alternative floor systems and compare the design results with the existing floor system of the South Patient Tower. Through hand calculations, a typical 29 ft. x 29 ft. bay was analyzed. The systems were then compared based on general conditions (weight, cost impact and depth), architectural impacts, structural impacts (foundation and lateral systems), serviceability requirements (deflection and vibration control) and constructability concerns (schedule related issues). The three systems designed in this report include:

- Post-Tensioned Concrete
- Composite Steel Framing with Composite Steel Deck
- One-way Slab and Beam

The design of the post-tensioned concrete system resulted in a slab thickness of 8 in. with a total thickness of 14 in. around the columns due to the addition of drop panels for punching shear. To achieve this, (39) ½" ϕ 7-wire unbounded tendons were spaced evenly in the North-South direction while in the East-West direction, (24) ½" ϕ 7-wire unbounded tendons were distributed evenly. This system weighed slightly less than the two-way flat slab system leading to a similar foundation plan and has a comparable cost to the original (slightly higher). The positive aspects corresponding to this system were the ability to decrease the depth of the system and its lack of vibrational concerns. The one drawback with a post-tensioned concrete system is the constructability concern. The post-tensioning tendons may lead to some difficulties as well as the fact that the slab cannot be easily cored in the event of future space changes. However, this system remains a feasible option due to the decrease in depth as well as the similar cost breakdown.

A 3 ½ in. normal weight concrete topping on a 2" Vulcraft 2VLI20 composite decks rests on top of W12x22 infill beams spanning the East-West direction with W18x46 girders spanning the North-South direction. Because of the steel construction, this system weighs nearly half as much as the original concrete system. This could lead to a change in the foundation system, but due to the low bearing capacity of the soil, it was determined that the current foundation would have to remain. However, because of the location of the building, the cost of the composite system far exceeded all of the other flooring systems analyzed. Along with cost, the composite system is most economical for higher floor-floor heights due to the increased depth of the member sizes which cannot be achieved in this structure due to the connection with the existing hospital. Also of concern are vibration issues as well as higher deflection values as compared to the concrete systems. On the other hand, because of the ease of construction, this may result in a quicker erection time. In light of the positive schedule impact, the composite steel system is a viable option.

Finally, a 5 in one-way slab with 12 in. x 24 in. infill beams was investigated. The depth of the beams was dictated by the size of the girders (24 in. x 24 in.) for ease of construction and formwork. Due to this increased depth, a higher degree of coordination between the disciplines must take place in order to effectively place the mechanical and electrical equipment. Since it is far easier to have equipment run through the steel beams of a composite system, the one-way slab was ruled out as a possible replacement.

Building Introduction:

Figure 1:
Aerial map from Bing.com showing the location of the building site

As an early phase in the Inova Fairfax Hospital Campus Development Plan, the South Patient Tower will be connected to the existing patient tower (see Figure 1) at all levels above grade including the penthouse. Construction started in the Summer of 2010 and is expected to be completed by Fall 2012 with an overall project cost of around \$76 million. Standing at 175 ft., the 236,000 ft^2 concrete structure consists of 12 stories above grade (excluding the penthouse) with an additional story below grade. A system of auger-cast piles and pile caps are used to support the structure with a soil bearing pressure of 3000 psf.

Along with the physical connection, the architecture of the South Patient Tower shares some similarities with the surrounding campus/hospital buildings. Wilmot/Sanz Architects designed the South Patient Tower as a continuation of the main architectural features of the existing patient tower building while at the same time displaying Inova's commitment to sustainable and functional buildings. Consisting of 174 all-private intensive-care and medical/surgical patient rooms, the floor plans are situated so that the various intensive-care unit specialties correspond to the same level as that of the existing main hospital. In order to meet the patient's specialized needs, workstations will be placed outside of the patient's rooms to maintain privacy while being able to monitor the patients at the same time.

Figure 2:
Exterior rendering showing the circular entrance and precast concrete façade (Provided by Turner Construction)

The façade is largely composed of a smooth finished precast concrete panel as well as a precast concrete panel with a thin brick face (see Figure 2). To add more architectural detail, thin brick soldier courses are used at every story level, starting with the 4th floor and continuing up the building to the 11th floor. The only tangent from the typical architectural pattern occurs on the 5th floor (main mechanical floor) where architectural louvers are used to allow air to exit the building. The first two levels are composed entirely of an aluminum curtain wall system which is also used for the majority of the building's windows. The two main architectural features that stand out along the ground floor of the building are the large two-story rotunda and the canopy covering the main entrance which is constructed from 4 custom steel columns.

The South Patient Tower is attempting to achieve LEED Silver Certification by including numerous sustainable design features (see Figure 3). Inside the patient rooms, the use of low-VOC paints, building materials and furniture will lead to higher indoor air quality. Also, the use of low flow plumbing fixtures and sensors will greatly reduce the water consumption by up to 30%. Outside of the building, native plants that are resistant to drought will surround the building. From the patient rooms, guests will be able to see the green roof and the water cisterns used to capture rain water.

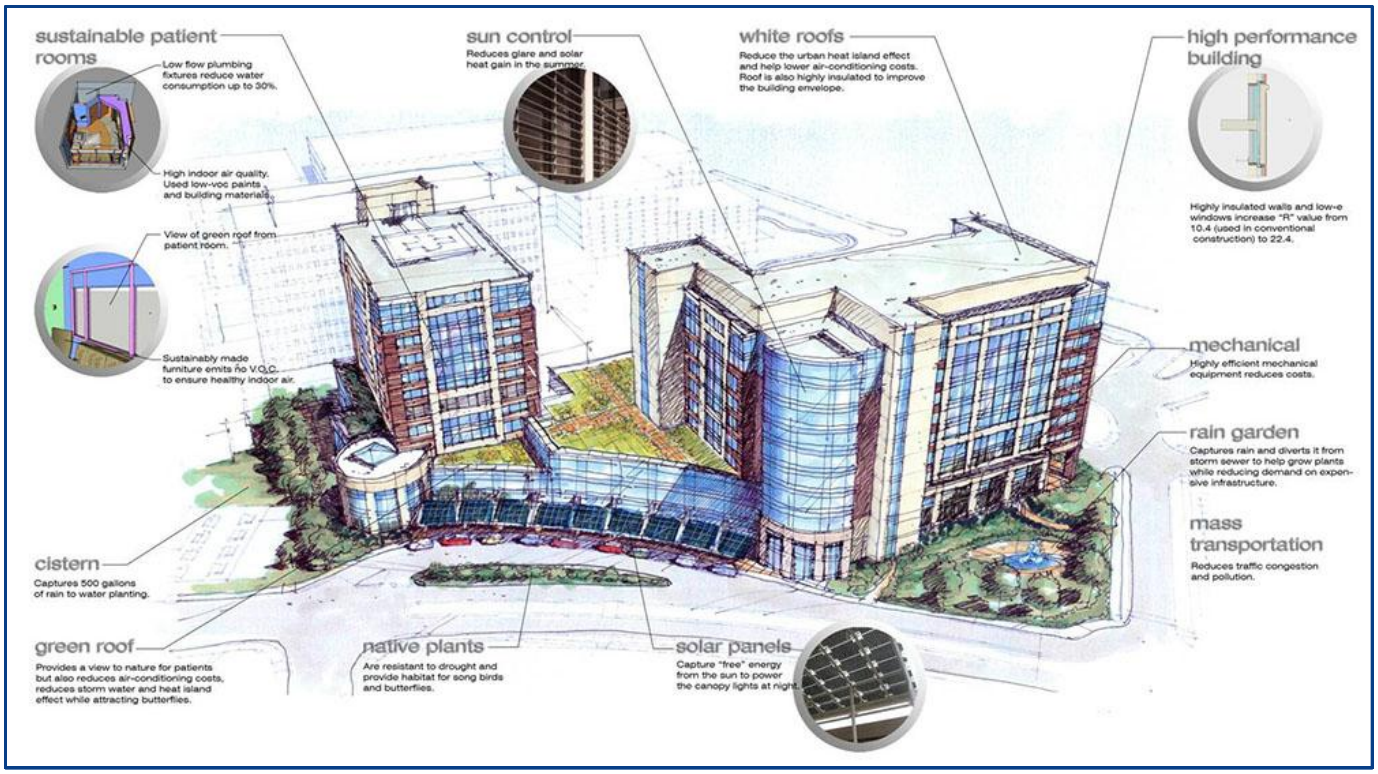


Figure 3:
Sustainability features (rendering provided by Wilmot/Sanz Architects)

Structural Overview:

Foundation:

Schnabel Engineering North performed the geotechnical studies for the South Patient Tower and provided the report in which they explain the site and below-grade conditions. The structural engineers of Cagley & Associates designed the foundation for an undisturbed soil net allowable bearing pressure of 3000 psf. Also given in the geotechnical report are lateral equivalent fluid pressures which are 60 psf/ft of depth for both the braced walls and cantilevered retaining walls. The sliding resistance (friction factor) was found to be 0.30.

In light of the soil conditions, the SPT utilizes a foundation with a system of 16 in. diameter auger-cast piles and pile caps on top of a slab on grade (see Figure 4). Due to higher stresses around the staircase and elevator pit, a large pile cap is situated around each of these areas to help alleviate the stresses on the slab (see Figure 5). The number of piles per pile cap varies throughout the foundation with the most common being 9 and 11.

Along with the 5 in. slab on grade, grade beams connect the piles within the foundation footprint. Along the perimeter of the foundation, the SPT makes use of spread and strip footings (see Figure 6). Since the foundation does not cover the entire area of the ground floor, some areas consist of piles and pile caps directly underneath the ground floor slab to support the main entrance and lobby space.

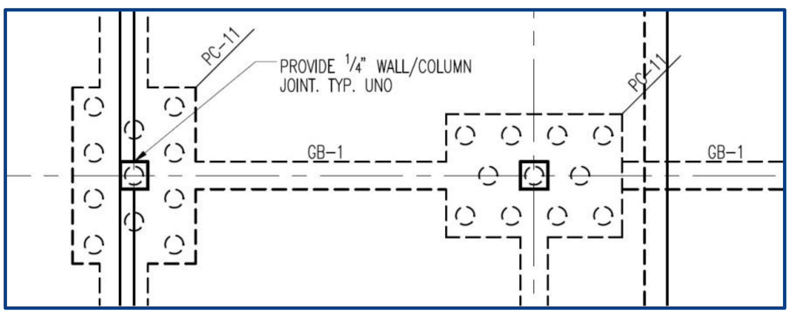


Figure 4:
Typical pile and pile cap

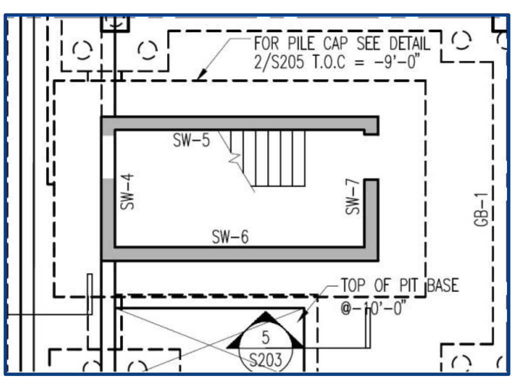


Figure 5:
Pile cap constructed around staircase

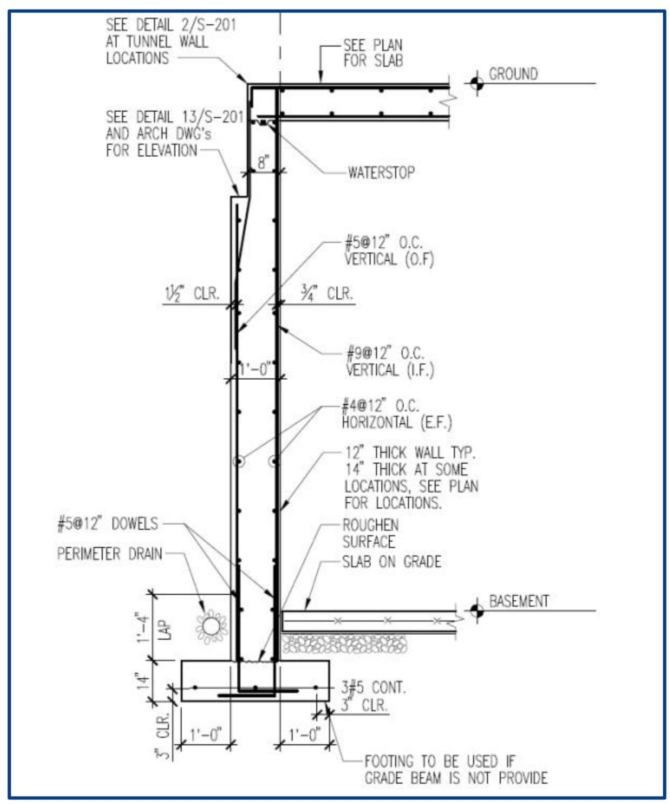


Figure 6:
Spread footing with basement wall

Framing System:

As mentioned in the previous section, the columns follow a pretty regular pattern with a few exceptions. Typically the bay sizes are 29 ft. x 29 ft. with drop panels at every location (see Appendix F for typical floor plans). There are no interior beams but there are a few beams along the perimeter of the building towards the south end of the structure and near the connection to the existing hospital.

The columns are all cast-in-place concrete with the largest column being 30 in. x 30 in. in the basement level. The typical column size is 24 in. x 24 in. and 12 in. x 18 in. (rotated as required to fit the wall thickness). Because of the higher loads located in the columns towards the lower portions of the building, 7000 psi concrete is utilized up to the 5th floor level with the rest of the upper floor columns being 5000 psi concrete. Consisting of mainly #11 reinforcement bars with #4 stirrups, the maximum number of reinforcement bars around a column is 20 with the typical number being 4.

Lateral Systems:

Shear walls and ordinary moment resisting frames make up main lateral force resisting system in the South Patient Tower and are situated throughout the building to best resist the lateral forces in the building. Seven different walls make up the shear wall system which surrounds both the main staircase and the main elevator while the moment frames are situated near the connection and at the far end of the structure (see Figure 7 located on the next page). The shear walls are 12 in. thick and are composed of 5000 psi cast-in-place concrete. Most span from the basement level to the main roof line but the northern core around the elevator shaft extend up the entire 175 ft. height to the top of the penthouse level.

All of the shear walls are connected to the foundation with dowels to properly allow the loads to travel through the walls down to the foundation. These two shear wall cores along with the moment frames help resist lateral loads in both the North-South and East-West direction.

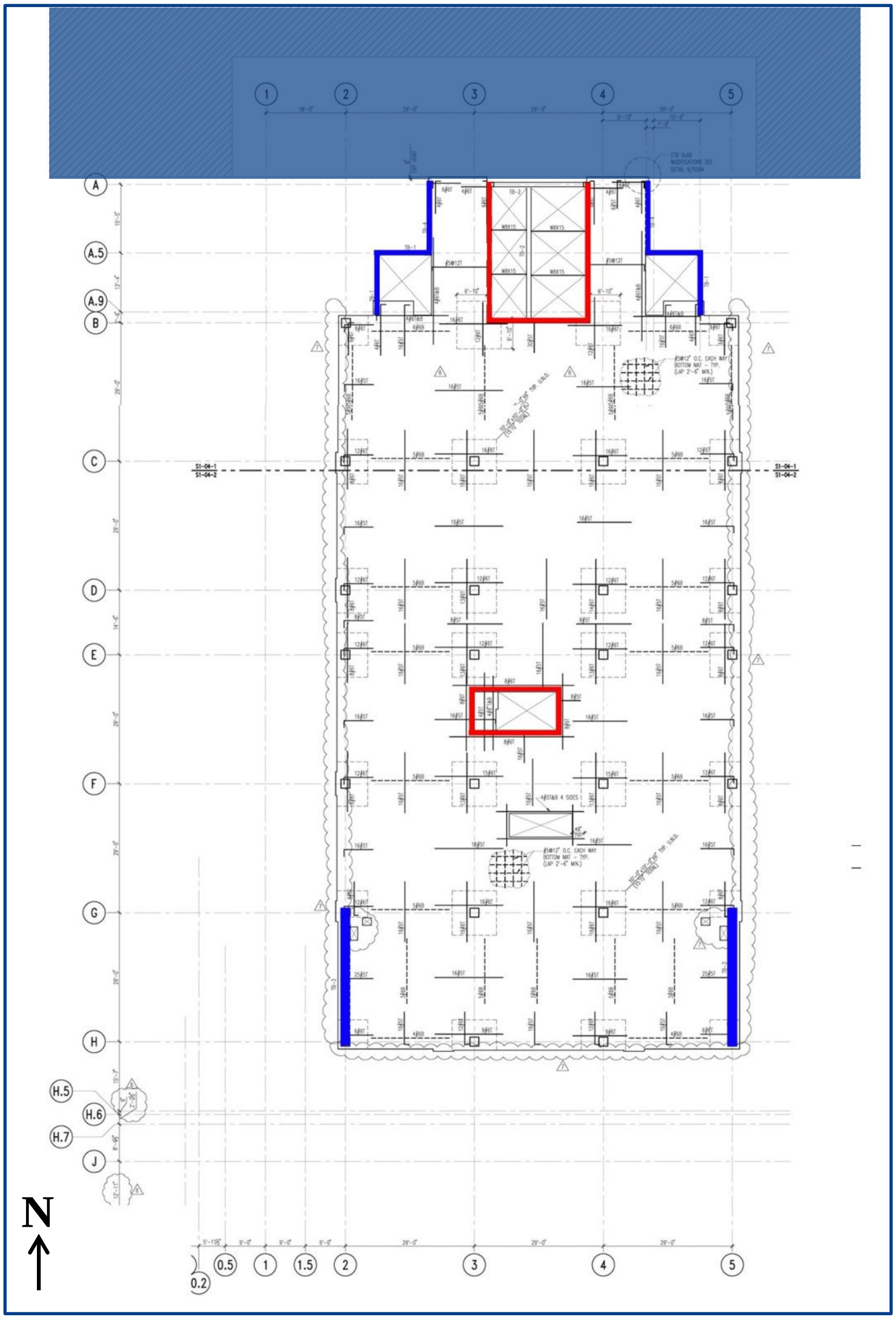


Figure 7:
Shear wall locations shaded in red with the moment frames shaded in blue

Roof System:

In general, there are three different main roof levels (see Figure 8). The roofing system on the 11th floor is comprised mainly of Polyvinyl-Chloride (PVC) roofing situated on top of Composite Polyisocyanurate Board Insulation. This system rests on top of a concrete slab with varying thickness.

Highlighting the 11th floor roof is the pre-engineered aluminum helicopter landing system. Supporting the landing platform is a system of structural steel columns with vibration isolators (see Figure 9).

The main design features of the lower roof level (2nd floor) consist of a vegetated roof system, accent vegetation and concrete roof pavers. Also on the lower roof, a hexagonal skylight covers the circular rotunda (see Figure 10). The slab thickness for the lower roofs (excluding the green roof) varies but is mainly 9 ½ in. while the main roof, which supports higher loads from the mechanical penthouse, is 12 in. thick.

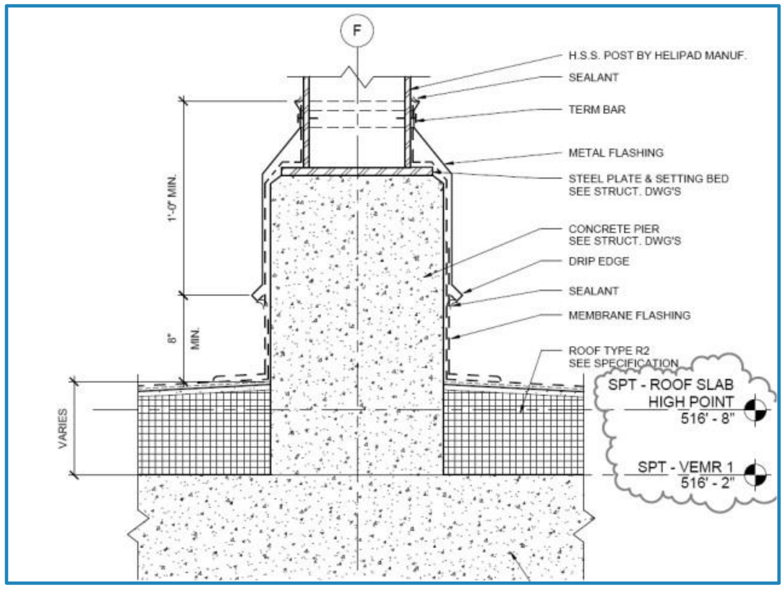
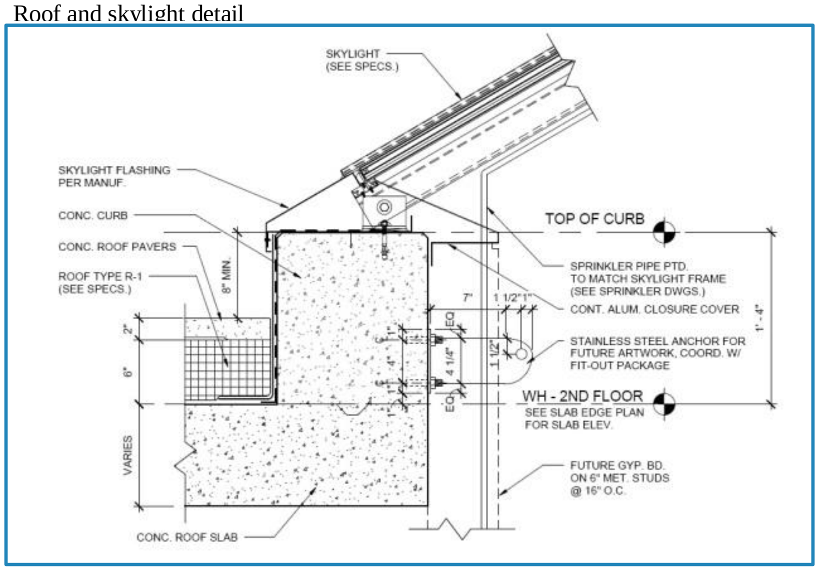


Figure 9:
Helipad support post

Figure 10:
Roof and skylight detail



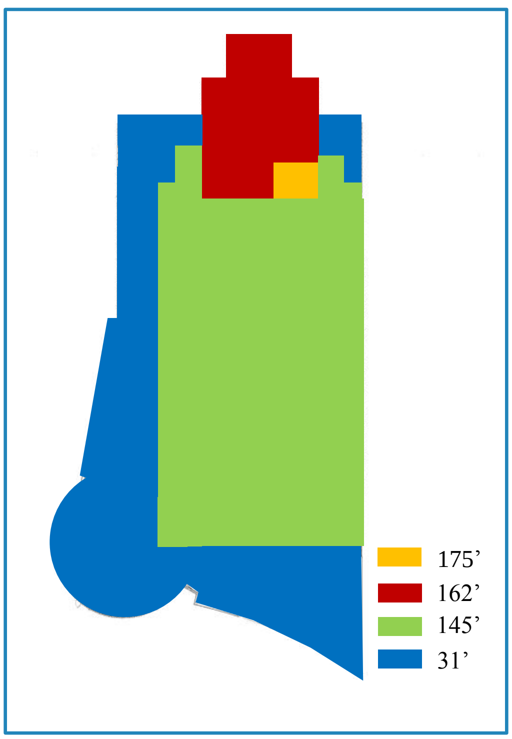


Figure 8:
Showing different roof heights in relation to 0'-0"

Design Codes:

According to Sheet S0-01, the original building was designed to comply with the following codes/standards:

- 2006 International Building Code (IBC 2006)
- 2006 Virginia Uniform Statewide Building Code (Supplement to 2006 IBC)
- Minimum Design Loads for Building and Other Structures (ASCE7-05)
- Building Code Requirements for Structural Concrete (ACI 318-05)
- American Concrete Institute Manual of Concrete Practice – Parts 1 through 5 (ACI)
- Manual of Standard Practice (Concrete Reinforcing Steel Institute)
- Manual of Steel Construction – Allowable Stress Design 9th Edition (American Institute of Steel Construction - AISC)
- Manual of Steel Construction, Volume II, Connections (ASD 9th Edition/LRFD 1st Edition – AISC)
- Detailing for Steel Construction (AISC)
- Structural Welding Code ANSI/DWS D1.1 (American Welding Society – AWS)
- Design Manual for Floor Decks and Roof Decks (Steel Deck Institute – SDI)
- Standard Specifications for Structural Concrete (ACI 301)

Thesis Codes and References:

- 2009 International Building Code
- ASCE 7-05
- ACI 318-08
- AISC Steel Manual - 14th Edition (2010)

Materials Used:

The various kinds of materials and standards used for the construction of the South Patient Tower are listed in Figure 11a and 11b on the following page. All information was derived from Sheet S0-01.

Concrete		
Usage	**Strength (psi)**	**Weight**
Piles	4000	Normal
Pile Caps	5000	Normal
Footings	3000	Normal
Grade Beams	3000	Normal
Foundation Walls	3000	Normal
Shear Walls	5000	Normal
Columns	5000/7000	Normal
Slabs-on-Grade	3500	Normal
Reinforced Slabs LG-L4	5000	Normal
Reinforced Beams LG-L4	5000	Normal
Reinforced Slabs L5-Roof	4000	Normal
Reinforced Beams L5-Roof	4000	Normal
Topping Slabs	3000	Lightweight
Concrete on Steel Deck	3000	Lightweight

Steel		
Type	**Standard**	**Grade**
Wide Flange Shapes and Tees	ASTM A992	50
Round Hollow Structural Shapes	ASTM A992	B (F_y = 35 ksi)
	ASTM 501	F_y = 36 ksi
Square or Rectangular Hollow Structural Shapes	ASTM A500	B (F_y = 46 ksi)
Other Structural Shapes and Plates	ASTM A36	N/A
High Strength Bolts	ASTM A325 N	N/A
Smooth and Threaded Rods	ASTM A572	N/A
Headed Shear Studs	ASTM A108	N/A
Welding Electrodes	AWS A5.1 or A5.5	E70xx
Galvanized Steel Floor Deck	ASTM A653 SS	33

Figure 11a:
Summary of materials used on the SPT project with design standards and strengths

Reinforcement	
Type	**Standard**
Deformed Reinforcing Bars	ASTM A615 (Grade 50)
Weldable Deformed Reinforcing Bars	ASTM A706
Welded Wire Fabric (WWF)	ASTM A185
Epoxy Coated Reinforcing Bars	ASTM A6775
Mechanical Connection Splices	DYIDAG, Lenton, or ACI 318 §12.14.3
Adhesive Reinforcing Bar Doweling Systems	ASTM A621

Miscellaneous	
Type	**Standard/Value**
Cement	ASTM C150 (Type I or II)
Blended Hydraulic Cement	ASTM C595
Aggregates	ASTM C33 (NW) ASTM C330 (LW)
Air Entraining Admixture	ASTM C260
Chemical Admixture	ASTM C494
Grout	ASTM C1107 ($F'_c = 5000$ psi)

Concrete Water Cementitious Ratio	
F'c @ 28 Days (psi)	**W/C (Max)**
$F'_c \leq 3500$	0.55
$3500 < F'_c < 5000$	0.50
$5000 \leq F'_c$	0.45

Figure 11b:
Summary of materials used on the SPT project with design standards and strengths

Gravity Loads:

As part of this technical report, the dead, live and snow loads have all been calculated and compared to the loads listed on the structural drawings. Following the determination of the various loads using ASCE 7-05, several gravity members part of the structural system were checked to verify their adequacy to carry the gravity loads. Detailed calculations for these members can be found in Appendix A.

Dead and Live Loads:

The structural drawings list the superimposed dead loads used by the structural engineers for the design of the gravity members which are summarized in Figure 12.

Superimposed Dead Loads	
Description	**Load**
Floors	20 psf
Standard Roof	20 psf
Main Roof	20 psf

Figure 12:
Summary of superimposed dead loads

Following the confirmation of the superimposed dead loads, these loads along with the weights of the slabs, columns, shear walls, roofs, façade and the drop panels were used to calculate the overall weight of the entire structure. The exterior walls are made up of 5 ½ in. concrete with a ½ in. thin brick face. To simplify calculating the weight of this system, a 6 in. concrete panel was assumed to account for both elements. Figure 13 on the following page shows the overall weight of each floor as well as the complete weight of the entire structure which was found to be approximately 38,600 k.

A comparison of the live loads used in the SPT and Table 4-1 in ASCE 7-05 resulted in very little differences except when it came to the loads used for the offices as well as the patient floors (see Figure 14). The offices were all designed for 60 + 20 psf partition loading, which is 10 psf over the value given in Table 4-1. This could be due to the fact that offices are located on floors with patient rooms and corridors which both have a total live load of 80 psf. To be conservative, the project engineer probably just used 80 psf to be on the safe side. One other difference in live load occurred with the patient floor levels. According to ASCE, the minimum live load for hospital patient floors is 40 psf + partitions. However, the engineers for the SPT used 60 psf + partitions. A possible explanation for the increased load could be attributed to the future needs of individualized patients. Because certain patients may need different equipment, the exact load is uncertain. Therefore, the more conservative value of 60 psf was chosen. Calculations involving the patient floors will use 60 psf + 20 psf for partitions for this report and future reports.

Live loads for both the café and the roof were not given, but a live load of 80 psf was assumed for the café. Since the main roof utilizes a helicopter landing system, the specification for the system indicated a minimum live load of 100 psf and therefore will be used. Because the green roof will be accessible, a live load of 100 psf will be used for the lower vegetated roofs.

Weight Per Level		
Level	**Area (ft^2)**	**Weight (kips)**
Ground	25513	N/A
1st	25513	4393
2nd	11649	2418
3rd	17958	3902
4th	16571	3011
5th	16571	3285
6th	16571	3078
7th	16571	3011
8th	16571	3011
9th	16571	3011
10th	16571	3011
11th	16571	3066
Penthouse/Roof	16571	3383
		38578

Figure 13:
Distribution of weight per floor level

Live Loads			
Space	**Design Live Load (psf)**	**ASCE 7-05 Live Load (psf)**	**Notes**
Assembly Areas	100 (U)	100	N/A
Corridors	100	100 (first floor) ; 80 psf above	Based on both "Corridors" and "Hospitals" Section
Patient Floors	60 + 20	60 + 20	Based on "Hospitals - Operating Rooms, Laboratories"
Lobbies	100	100	N/A
Marquess and Canopies	75	75	N/A
Mechanical Rooms	150 (U)	N/A	N/A
Offices	60 + 20	50 + 20	Office Load + Partition Load
Stairs and Exitways	100 (U)	100	N/A
Café	N/A	80	N/A
Roof	N/A	100	Based on Future Helicopter Landing System

Figure 14:
Comparison of live loads

Snow Loads:

Following the procedure outlined in Chapter 7 of ASCE 7-05 and using the snow load maps, the roof snow load and drift values were obtained. The factors used to calculate the flat roof snow load are summarized in Figure 15. A flat roof snow load of 21 psf was calculated which matched the structural drawings. Due to the different roof heights, drift was considered at multiple locations. A summary of the snow and drift calculations and results can be found in Figure 16.

Flat Roof Snow Load Calculations	
Variable	**Value**
Ground Snow Load - p_g (psf)	25
Exposure Factor - C_e	1
Temperature Factor - C_t	1
Importance Factor - I	1.2
Flat Roof Snow Load - p_f (psf)	**21**

Figure 15:
Summary of roof snow load values

Snow Drift Load Calculations								
Roof Levels	**Windward**				**Leeward**			
	L_u (ft)	**h_d (ft)**	**p_d (psf)**	**w_d (ft)**	**L_u (ft)**	**h_d (ft)**	**p_d (psf)**	**w_d (ft)**
1 and 2	39.83	1.55	26.80	6.22	175.33	4.35	75.10	17.42
2 and 3	159.5	3.13	53.98	12.52	46.33	2.26	38.92	9.03
2 and 4	159.5	3.13	53.98	12.52	31.33	1.80	31.00	7.19
1 and 3	37.33	1.50	25.82	5.99	50.17	2.36	40.67	9.43
3 and 4	19.33	0.98	16.91	3.92	30.83	1.78	30.70	7.12

Figure 16:
Summary of roof snow drift calculations

Floor Systems:

The bay sizes of the South Patient Tower are relatively regular with very few variations from the typical 29 ft. x 29 ft. size. On the ground floor, the bay sizes vary somewhat from the norm due to the various architectural details situated near the atrium/front entrance.

The main objective of this technical report was to analyze the existing two-way flat slab system, and then design three other systems. For ease of comparison, all of the frames were analyzed with the same typical interior bay (29 ft. x 29 ft.) spanning column lines C and D in the North-South direction and between 3 and 4 in the East-West Direction. All four systems were then compared on a various items ranging from cost per square foot to constructability concerns.

Two-Way Flat Slab with Drop Panels (Existing Floor System):

The elevated floors of the South Patient Tower are comprised of a 9 ½ in. two-way flat slab. A drop panel is located at every column location in order to prevent punching shear as well as to increase the thickness of the slab to help with the moment carrying capacity of the slab near the columns. The typical size for the drop panel is 10 ft. x10 ft. x 6 in.

For the ground floor through the 4th floor, 5000 psi concrete is used for construction of the two-way slab while the upper floors use a 4000 psi concrete. The one exception to the 9 ½ in. slab is the mechanical floor (5th floor). Because of the higher load imposed by the mechanical equipment over the entire floor, the slab was designed accordingly and bumped up to 10 ½ in.

Reinforcement for the two-way slab system is comprised of both top and bottom steel. The typical bottom reinforcement consists of #5@12 in. o.c. each way (see Figure 17 and 18 for reinforcement details). Additional bottom reinforcement is listed on the drawings wherever needed as well as top reinforcement which is located in areas of negative moments (mainly around the columns and between column lines depending which direction the frame of interest is going). With a fairly simple column layout, the two-way slab system has a span of 29 ft. in both directions for the most part.

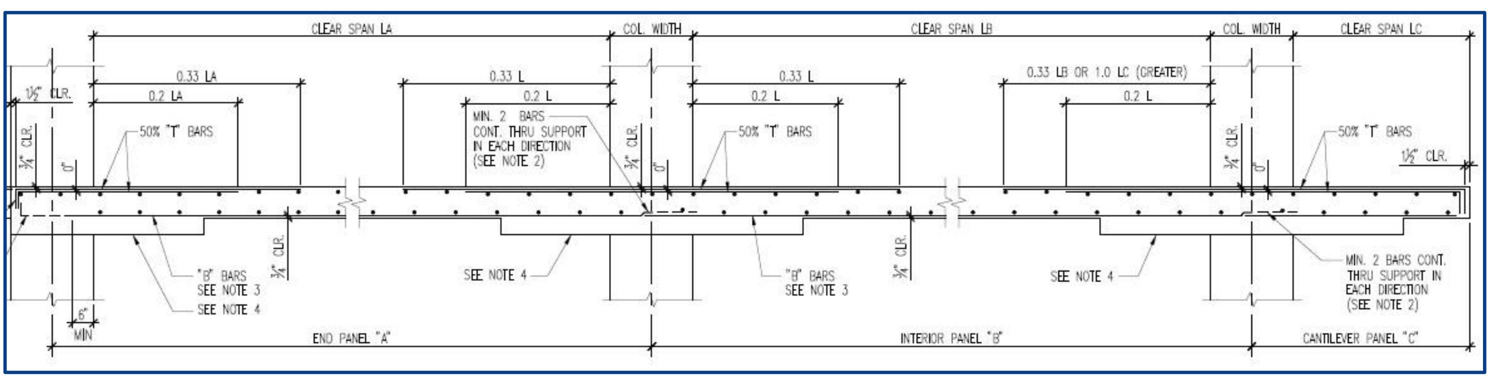


Figure 17:
Typical column strip reinforcement and placement

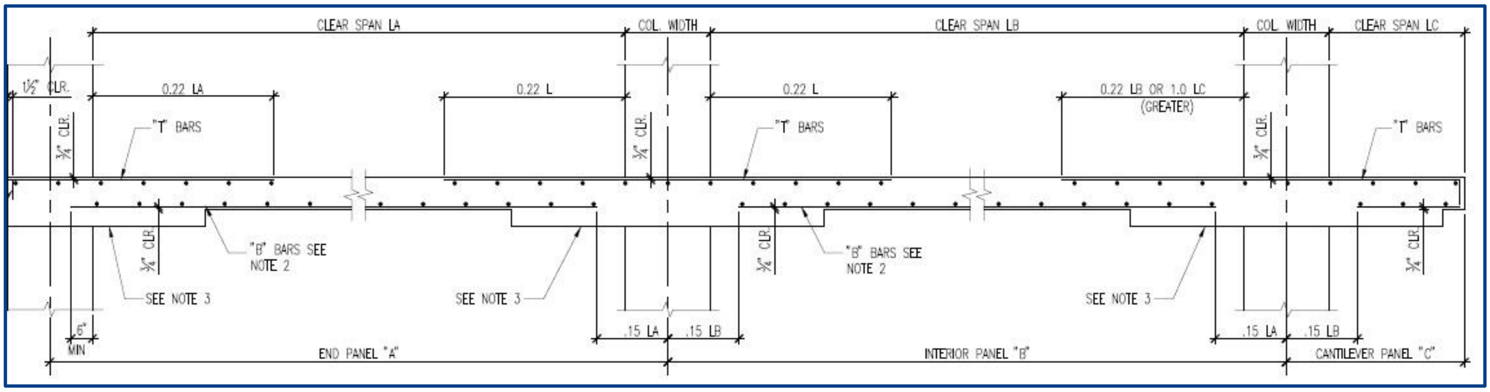


Figure 18:
Typical middle strip reinforcement and placement

General:
The two-way flat slab system was found to weigh 118.75 pounds per square foot (psf) which served as a baseline to compare to the other three flooring systems. At approximately $16.32/SF, this is the least expensive system when compared to the others. This cost is an assemblies estimate based on data from RS Means CostWorks which includes material (including formwork), labor, and surface treatments. Cost breakdowns for each of the systems can be found in Appendix E. With the addition of the drop panels, the total depth of the system totals 15.5 in. The plenum depth throughout the South Patient Tower averages 36", so the two-way flat slab system leaves plenty of room for the large mechanical ductwork needed for hospitals.

Architectural:
This system has a minimum of the required 2 hour fire rating and because the original building was designed around this flooring system, there are no additional architectural impacts.

Structural:
The pile/pile cap foundation and the main lateral force resisting system were designed for this system and are unchanged should this system remain. A summary of the reinforcement calculated for the middle and column strips can be seen in Figure 19. Because of the square bay, the reinforcement needed in the other direction will be the same as that shown. A complete set of calculations can be found in Appendix A.

Serviceability:
The maximum deflection for the two-way flat slab system was calculated by first finding the immediate deflection due to total dead load and live load. Next, the additional deflection after a long period of time due the total dead load was calculated. Deflections were then compared to limits laid forth in ACI 318-08 (both live load and total deflection after partitions). The maximum deflection for this system was 1.10 in., which was a conservative value based on the deflection after a long period due to the total dead load and 25% of the live load. Vibration analyses were not performed for this report, but general research was performed on how the

systems behave for vibration. Due to the mass and stiffness of the concrete slab, the system behaves quite well and possesses very few vibrational concerns.

Construction:

This system requires no additional fireproofing since the system already achieves the minimum 2 hour fire rating. Because of the simplicity of the two-way flat slab and the redundancy of the drop panels (all drop panels are the same height), this system does not require multiple types of crews on site and therefore has very few constructability concerns.

System Pros and Cons:

Pros:

- Low cost per square foot
- Floor depths allow for adequate space to place mechanical and electrical equipment
- No vibration concerns
- Ease of construction

Cons:

- Relatively heavy (higher seismic forces)
- Deflection control (relatively high)

Although the system is relatively heavy, the two-way flat slab performs well in most of the categories. Due to the nature of the building and the vibration characteristics of the floor system as well as the other pros, it is easy to see why this system was chosen for the South Patient Tower.

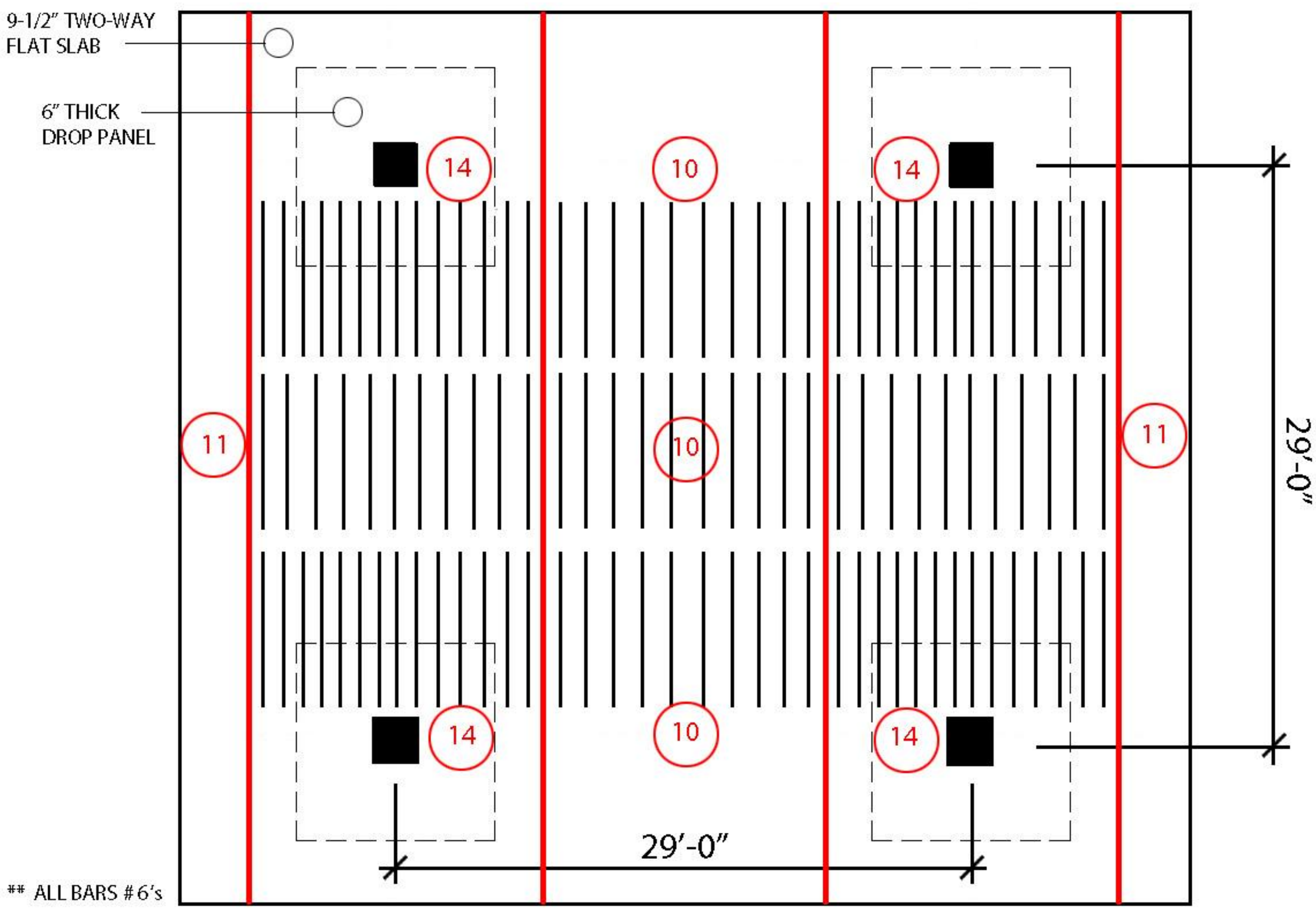


Figure 19:
Calculated reinforcement for column and middle strips in the North-South

Post-Tension Concrete (Flat Slab with Drop Panels):

The post-tensioned design was chosen to reduce the depth of the original flooring system, as well as to decrease the weight. Reducing the depth could be of importance mainly to allow for more space for mechanical equipment. The design was performed by hand calculations (which can be found in Appendix B) based on a design example published in *Prestressed Concrete: A Fundamental Approach* (4th Edition), written by Edward G. Nawy.

The calculations resulted in an 8 in. thick flat slab. The post-tensioning required was (39) ½" ϕ 7-wire unbounded tendons in the North-South direction with (24) ½" ϕ 7-wire unbounded tendons in the East-West direction (Figure 20).

General:

With the reduction in the slab thickness, the post-tensioned flooring system only weighs 100 pounds per square foot (18.75 psf less than the two-way flat slab). The cost for this flooring system basically equates to the two-way flat slab, but is slightly more expensive at $16.82/SF. In terms of floor depth, the post-tensioned system does slightly better than the original system. A flat plate was considered, but due to the large punching shear values obtained, drop panels were needed to resist both the shear and the larger moments located at the columns. The same size drop panels from the two-way system were used with the post-tensioned design (10 ft. x 10ft. x 6 in.). Because of the drop panels, the total depth of this system results in 14 in.

Architectural:

This system achieves the minimum fire rating from cover requirements of the draped tendons and the incorporated eccentricity maintains the 2 hour fire rating. Because of a similar depth to the system, no major architectural changes will occur.

Structural:

With a reduced weight in the flooring system, the foundation likely could experience some changes. However, due to the minimal changes, the foundation would likely remain pile and pile caps. This system would also have very minimal changes to the lateral system, since shear walls and ordinary reinforced moment resisting frames are the most sensible choices.

Serviceability:

Deflections were calculated focusing mainly on live load. Since the dead load was balanced in both directions from the eccentricity of the tendon profile, only the live load deflection would need to be calculated. The deflection resulted in values well below the two-way flat slab and well below the maximum allowable deflection per ACI. Similar to the original system, vibrations are not of a concern due to the mass and stiffness of the slab.

Construction:

No additional fire proofing is required to achieve the minimum fire rating. One concern with construction revolves around the placement of the tendons. The crew must be familiar with post-tensioned construction to complete the project timely, and at the same pace as the two-way flat slab. If the crew is accustomed with this system, then the schedule should remain similar to the original.

System Pros and Cons:

Pros:

- Less weight
- Cost comparable to the original system
- Less floor depth
- No vibration concerns

Cons:

- Added construction difficulties due to post-tensioning
- Difficult to add holes after concrete is poured due to tendons

The post-tension system compares relatively quite well with the original system. However, the constructability issues with the slight increase in price may pose a problem, but the system remains feasible for the South Patient Tower.

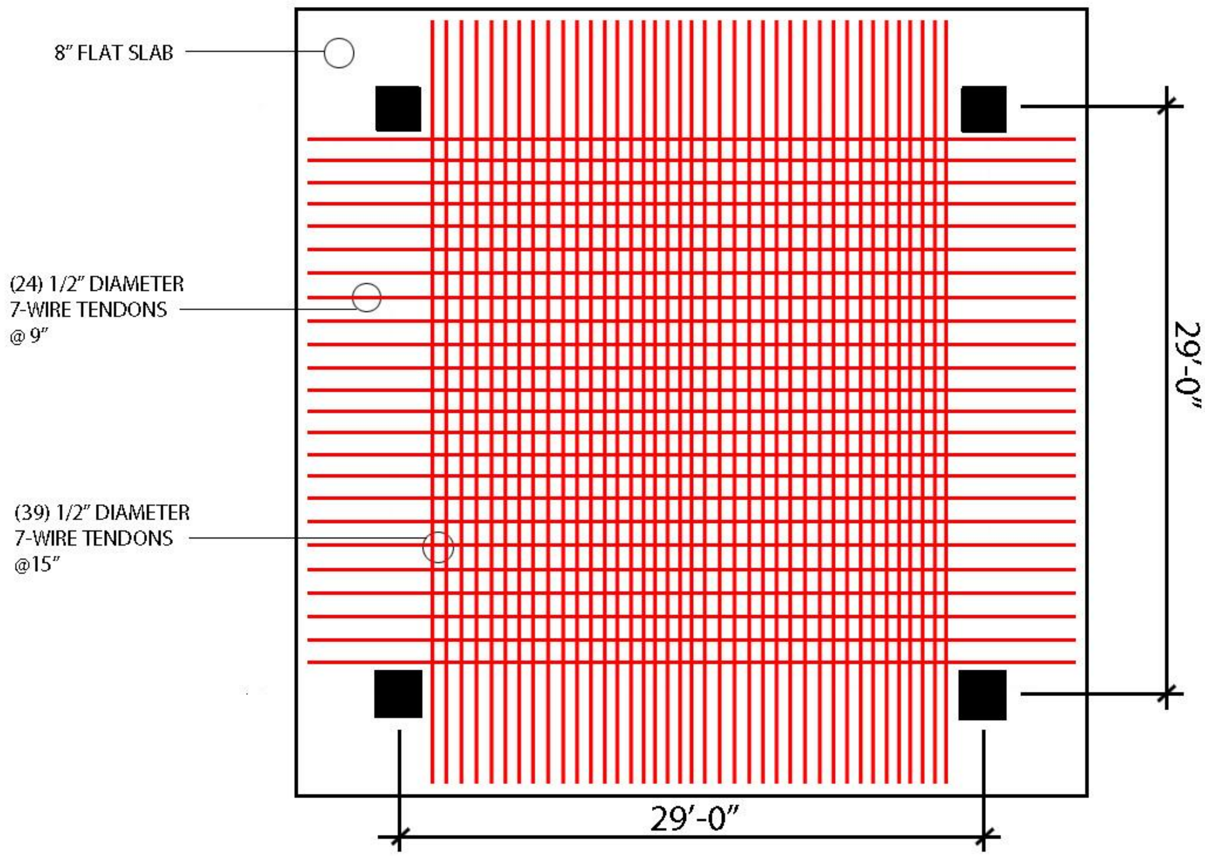


Figure 20:
Calculated number of tendons in each direction

Composite Steel:

The next system designed was a composite steel system. Calculated beam and girder sizes along with the required camber and number of shear studs can be seen in Figure 21 (hand calculations can be found in Appendix C for the entire system composite system). The beams are situated beneath a 2VLI20 Vulcraft composite deck along with a 3 ½ in. normal weight concrete topping.

The selection process for the beam and girder revolved around depth. The goal was to minimize the depth of the members to help increase the amount of space for mechanical/electrical equipment. Since unshored strength usually dictated the member size, both the beam and girder sizes had to be increased in order to prevent shoring. The main reason behind this upsizing is due to the economical disadvantages of having to shore the beam and girder costing both time and money.

General:

With a 5 ½ in. total thickness (deck plus the topping), the system was found to weigh approximately 61 pounds per square foot. This weight is significantly lower than both the post-tensioned and the two-way flat slab systems. However, the cost corresponding to the composite system reaches the maximum value of any of the floor systems at $20.37/SF. This is most likely due to the increased labor costs of having multiple crews (both concrete and steel). Another difference with the previous concrete systems is the floor depth. Under the beams, the total depth is 17.5 in., and the distance below the girder is 23.5 in. This will decrease the available space for the ductwork as well as increase the construction issue of coordination between disciplines. Also, in order to obtain a 2 hour fire rating, it was decided to spray fireproofing on the underside of the deck system instead of increasing to a 4 ¼ in. concrete topping. The spay fireproofing brings the composite system to the required 2 hour rating.

Architectural:

Due to the increase in depth of the system, the drop ceiling may need to be lowered slightly to incorporate all of the equipment. Since the floor heights cannot be altered in any way, the increased depth may pose a problem. However, since holes can be punched out of the beams, mechanical and electrical equipment may not need as much space below the beam/girder system.

Structural:

Since this system weighs considerably less than both of the systems already discussed, the foundation system has the possibility to be reduced significantly. However, the structural engineers designed the foundation for a bearing pressure of 3000 psf. Under normal circumstances, the foundation could be designed for lesser loads and redesigned using spread and strip footings for the entire foundation. Due to the low bearing pressure, piles and pile caps remain a better option. Therefore, it seems as though the concrete piles would be impractical for a structural steel frame due to the grossly excessive capacity of the piles and pile caps. An alternative foundation system could consist of micropiles, but the design of these was not considered in this technical report.

The lateral system for the composite steel floor system would have to change, but could be easily changed to a dual system consisting of braced frames and moment frames, or a configuration consisting entirely of one of the aforementioned. This was not considered in this analysis, but would have to be investigated if composite steel were to be the flooring system for the South Patient Tower.

Serviceability:

The maximum deflection for the composite steel flooring system was found to be the highest out of all of the systems considered. This deflection was found by adding the deflection of the girder to the deflection caused by the beam. The camber on both of the members helps out to a degree, but deflection is still an issue with the steel construction. Although the deflection ended up being the highest for this system, the minimum deflection requirements (wet concrete, total load and live load) were met. Although no vibration calculations were performed, vibration definitely remains a concern for any steel construction. A further investigation would be carried out should this system be chosen.

Construction:

The added spray fireproofing to achieve the required fire rating could be costly as well as impact the schedule. However, steel erection tends to be quicker than the casting of concrete, and therefore the use of a steel superstructure could vastly decrease the schedule significantly. Other than spray fireproofing, the flooring system is typical and possesses no other major constructability concerns.

System Pros and Cons:

Pros:

- Less weight (a decrease in seismic loads and a potential to reduce the foundation)
- Quicker erection time
- Ease of construction

Cons:

- Higher cost
- Works better for higher floor to floor heights (more economical)
- Vibration and deflection concerns

Although the system is costly compared to the concrete systems, the quicker erection time could be beneficial and merits further investigation. However, the vibration and deflection concerns could pose a problem and would have to be evaluated further in order use a composite steel structure.

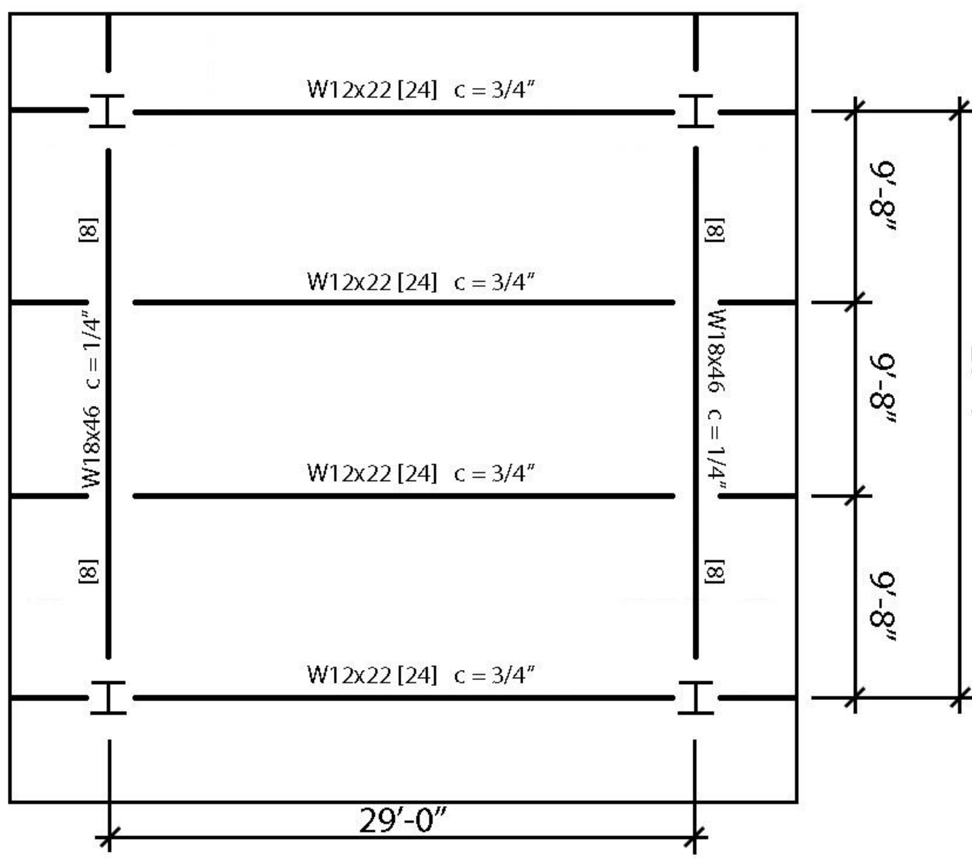


Figure 21:
Calculated member sizes, shear studs, and camber for typical panel

One-Way Slab and Beams:

The one-way slab with beams was chosen after careful consideration and evaluation of several other possible floor systems. The process started with looking at hollow-core planks. After considering this system, it was determined that the architectural changes involved in making the bay sizes modular to fit the dimensions of the plank would be uneconomical. Another steel structure with concrete floors investigated was Girder-Slab. However, it was found that in order to use the D-beam, one dimension of the bay would have to be cut in half. Again, this architectural change was too drastic and too costly to be considered as a viable floor system. Finally, joists were looked into as an alternative floor system; however the vibration concerns prevented this floor system from being a possible replacement.

The final design alternative consists on a 5 in. one-way slab with two infill beams, each 12 in. x 24 in. The girder sizes ended up being 24 in. x 24 in. for constructability purposes. The one-way slab was designed using ACI 318-08 Table 9.5(a). All of the dimensions for a typical interior panel can be found in Figure 22 and the hand calculations are located in Appendix D.

General:
This system falls in the middle of the weights calculated for the various floor systems with a total weight of 104 pounds per square foot. The cost is slightly more than both the two-way flat slab and the post-tensioned systems with a cost of $18.53/SF. This falls below the cost for the composite steel structure but higher than the other concrete systems primarily due to the increased formwork needed. No additional fireproofing is required as the system already meets the 2 hour fire rating. The biggest concern with this system is the depth. The depth of this system (including the slab and beam/girder) comes to 24 in. This is the largest out of any of the systems analyzed. This could pose a major problem when dealing with the placement of the mechanical and electrical equipment due to the locations of the beams and girders.

Architectural:
The system may have major architectural changes when dealing with the depth issue. Because of the large depth of the system, the floors may need to be increased which cannot happen due to the connection to the existing hospital. With a depth of 24 in., this leaves only 12 in. below the beams and girders for equipment which is not nearly enough space needed for a hospital and with the added beams and girders, equipment would have to weave around these obstacles. Serious architectural changes would have to occur.

Structural:
Structure wise, this system acts very similar to the two previous concrete systems and no major changes would occur to the structural system. The lateral system could remain a duel system between shear walls and ordinary moment resisting frames. Because the weight of the floor system remains similar to the original system, no changes to the foundation would be needed.

<u>Serviceability:</u>

Deflections for this system were calculated by combining the total load deflection from both the beam and girder. The deflection (1.18") ended up being higher compared to the two-way flat slab due to the unconservative approach in finding this system's displacements. Because the beam and girder were assumed to be simply supported, a higher deflection was actually calculated than if a more accurate method were used. Vibration is also not of huge concern due to the large mass of the slab with the additional stiffness provided by the beam and the girder.

<u>Construction:</u>

The beam and girder were both designed with the same depth for constructability concerns. This was done to help ease some of the constructability problems involved with all of the formwork. The formwork for both the beam and girder can be formed at once and all poured at the same time to help keep the schedule very similar to the original system.

<u>System Pros and Cons:</u>

Pros:

- Relatively cheap
- No vibration issues

Cons:

- Relatively heavy
- Depth issues
- Coordination of trades/disciplines needed to effectively place mechanical/electrical equipment

The one-way slab with beams compares to the original system in weight and cost (slightly more expensive), but constructability issues and depth issues prevent this system from being an adequate replacement to the original flooring system.

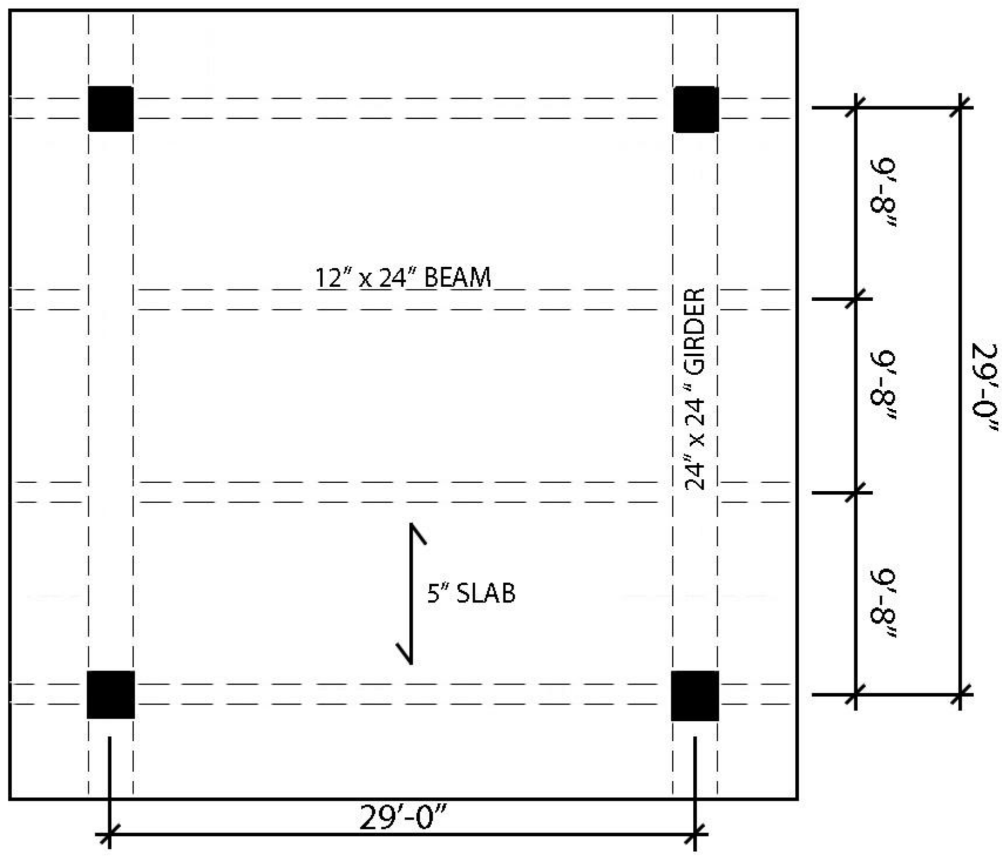


Figure 22:
Calculated member sizes for one-way slab with beams

Summary of Systems:

Figure 23 summarizes the results discussed in the preceding sections in a tabular format.

Consideration		System			
		Two-Way Flat Slab (with drop panels)	Post-Tensioned Concrete	Composite Steel	One-Way Slab and Beam
General	Weight (psf)	118.75	100	61	105
General	Cost ($/SF)	$16.32	$16.82	$20.37	$18.53
General	Floor Depth (inches)	15.5	14	23.5	24
Architectural	Fire Rating	2 hr	2 hr	2 hr	2 hr
Architectural	Other Impacts	No architectural impacts	No architectural impacts	Increased depth may pose problem (mech. could run through beams)	Leaves very little room for mech. equipment
Structural	Foundation Impact	Existing pile and pile caps	May slightly reduce required foundation	May reduce required foundation	May slightly reduce required foundation
Structural	Lateral System Impact	Existing shear walls and MRF	Shear walls would remain	Steel braced/ moment frames	Existing shear walls and MRF
Serviceability	Maximum Deflection (inches)	1.1	0.239	1.33	1.18
Serviceability	Vibration Control	Very Good	Very Good	Average	Very Good
Construction	Additional Fire Protection Required	None	None	Spray-on for beam/deck	None
Construction	Schedule Impact	N/A	May slightly increase schedule due to tendons	May reduce construction schedule	May slightly increase schedule due to increase formwork
Construction	Constructability	Easy	Medium	Easy	Medium
Feasible?		N/A	Yes	Yes	No

Figure 23:
Summary of flooring systems

Conclusion:

Technical Report 2 analyzed the original floor system and compared it to three other floor systems, all of which were designed in this technical report. Each system design was carried out on a typical interior bay/panel. Then, a comparison of the systems was performed based on factors including cost, weight, architecture impacts (mainly depth concerns) and structural impact on lateral and foundation systems as well as others. It was desirable to reduce the weight of the building, while at the same time keeping the same floor to floor heights with the least amount of structural depth.

The existing two-way flat slab was the least costly system, but also the heaviest system. Because of the redundancy involved in this floor system, the constructability aspect is relatively easy. Due to the location of the structure along with the costs associated with this system, it was verified to be a very sensible choice for the South Patient Tower.

Out of all of the alternatives, the post-tensioned concrete system was most comparable to the original system. With this system, the building weight decreased as well as the total depth. This system would cause very little architectural impacts on the current building. Although the cost is slightly more expensive ($0.50/SF), the decreased depth justifies the extra cost. The one major drawback with the post-tensioned system is the additional construction difficulty associated with the placement of the tendons and the post-tensioning process as well as the lack of adaptability to future changes. However, the advantages for this system supersede the drawbacks, and is therefore a viable option.

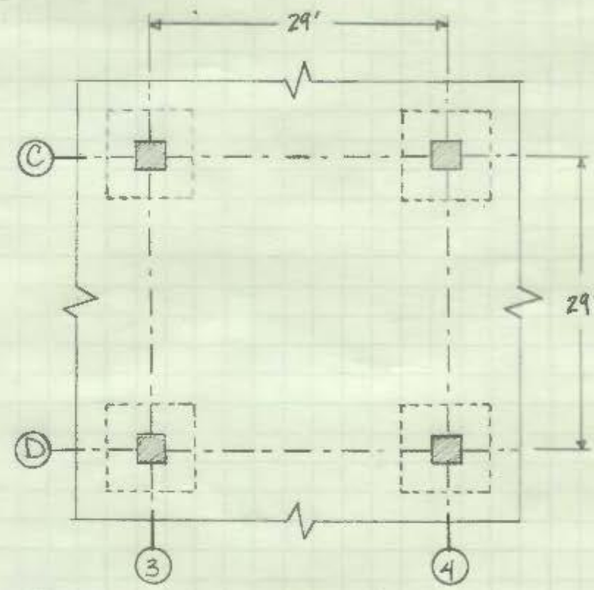
Composite steel was found to be the most expensive floor system analyzed, but also the lightest system by a wide margin. Because the steel composite system is more economical for higher floor to floor heights, the increased depth of the members may not be suitable for the South Patient Tower. Despite these concerns, the system has a great deal of flexibility and may drastically reduce the schedule. It can utilize either a braced frame or moment frame lateral system (or even a combination of the two). For these reasons, the composite steel structural is a feasible option.

The only system that was not found to be a reasonable replacement was the 5 in. one-way slab with beams. Because the depth of the system was the highest out of all of the systems designed, this left very few plenum space for the large mechanical and electrical equipment throughout the entire building. Although the building weight was slightly reduced compared to the original system, the increased formwork needed for the beam/girder system hiked the price up making this system the most expensive out of the three concrete systems analyzed in this technical report. Because of the increased difficulty in the coordination and placement of ceiling equipment, this system was rejected and will no longer be considered as a viable alternative.

Appendix A: Existing Two-Way Flat Slab Calculations

(TECH 2)

NATHAN McGRAW | EXISTING-TWO-WAY FLAT SLAB | PAGE 1 OF 15



COLUMN SIZES = 24" x 24"

LIVE LOAD = 80 PSF

F_y = 60 KSI

F'_c = 4000 PSI

1 TYPICAL INTERIOR PANEL
FLOORS 6 - 10

USING ACI 318-08

TABLE 9.5 (C) : Minimum thickness of slabs without interior beams and with drop panels

For $f_y = 60{,}000$ psi $\Rightarrow \frac{l_n}{36} = \frac{(29' - 2') \times 12"}{36} = 9.0"$

*TO BE CONSERVATIVE, THE STRUCTURAL ENGINEERS USED A SLAB THICKNESS t = 9.5". THE FOLLOWING CALCULATIONS WILL USE t = 9.5" TO REMAIN CONSISTENT

DIRECT DESIGN METHOD:

1. 3 Continuous spans in each direction ✓
2. Panel ratio ≤ 2

   $\frac{l_2}{l_1} = \frac{29'}{29'} = 1.0 < 2$ ✓
3. $l_1 \geq 2/3\, l_2$ ✓
4. Can't have a column offset of more than 10% of length ✓

(TECH 2)

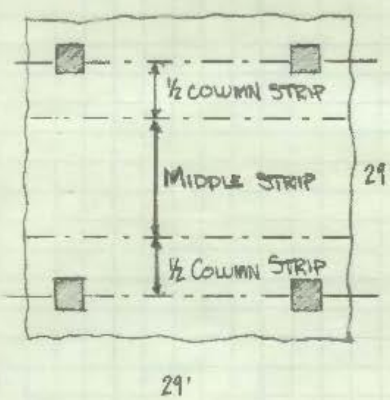
NATHAN M^cGRAW | EXISTING - TWO-WAY FLAT SLAB | PAGE 2 OF 15

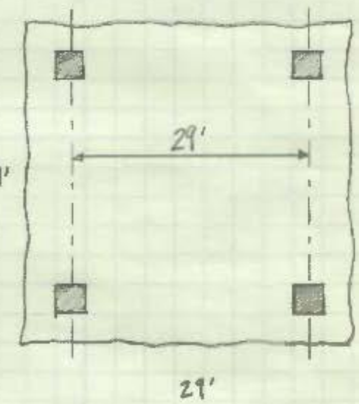
5. $W_L \leq 2W_D$

$$W_D = (9.5"/12")(150\ lb/ft^3) = 118.75\ psf$$
$$W_{DSI} = 20\ psf$$

$$W_L = 80\ psf \leq 2(118.75 + 20) = 277.5\ psf = W_D \quad ✓$$

OK TO USE DIRECT DESIGN METHOD





2 FRAME A

3 FRAME B

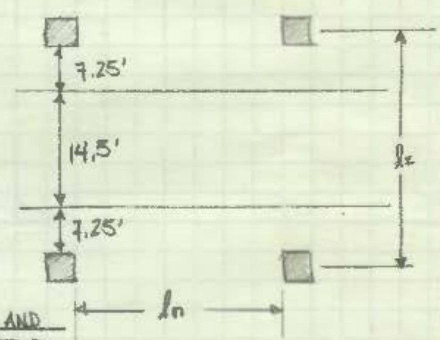
* SINCE FRAME A AND FRAME B HAVE THE SAME DIMENSION, ONLY NEED TO DESIGN ONE COLUMN STRIP AND ONE MOMENT STRIP FOR ONE FRAME

½ COLUMN STRIP = $(29'/4) = 7.25'$

MIDDLE STRIP = $(29'/2) = 14.5'$

MOMENT : $M_o = \dfrac{W_u \ell_2 \ell_n^2}{8}$

$W_u = 1.2D + 1.6L$
$W_u = 1.2(138.75\ psf) + 1.6(80\ psf)$
$W_u = 294.5\ psf$



4 COLUMN AND MIDDLE STRIP

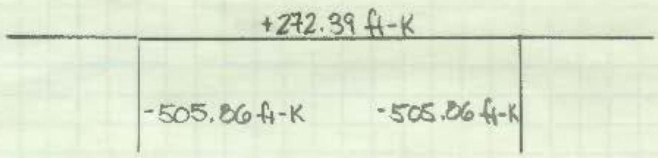
NATHAN McGRAW | (TECH 2) EXISTING - TWO-WAY FLAT SLAB | PAGE 3 OF 15

$$M_o = \frac{(294.5\text{ psf})(29')(29' - 2')^2}{8} = 778.25\text{ ft-K}$$

USING ACI 318-08 §13.6.3.3: EXTERIOR EDGE FULL RESTRAINED
INTERIOR NEGATIVE FACTORED MOMENT = $0.65\,M_o$
POSITIVE FACTORED MOMENT = $0.35\,M_o$

INTERIOR NEGATIVE MOMENT = $0.65\,M_o = 0.65(778.25) = 505.86$ ft-K

POSITIVE FACTORED MOMENT = $0.35(778.25) = 272.39$ ft-K



(5) NEGATIVE AND POSITIVE MOMENTS

DISTRIBUTION OF MOMENTS: (ACI 318-08 §13.6.4)

$\alpha_1 = 0 \Rightarrow$ No interior beams

$\ell_2/\ell_1 = 29'/29' = 1.0$

$\alpha_1(\ell_2/\ell_1) = 0$

$\beta_T = 0 \Rightarrow$ No edge beams

1) NEGATIVE MOMENT @ INTERIOR SUPPORT = 75%

2) POSITIVE MOMENT OF INTERIOR PANEL = 60%

NATHAN MCGRAW | (TECH 2) EXISTING - TWO-WAY FLAT SLAB | PAGE 4 OF 15

-505.86 → 75% to column strip = -379.40 ft-K → 100% to slab
-505.86 → 25% to middle strip = -126.46 ft-K

+272.39 → 60% to column strip = 163.43 ft-K → 100% to slab
+272.39 → 40% to middle strip = 108.96 ft-K

SUMMARY:

FRAME B	TOTAL WIDTH = 29'	COLUMN STRIP = 14.5'	MIDDLE STRIP = 14.5'
TOTAL MOMENT	-505.86	+272.39	-505.86
MOMENT IN COLUMN STRIP SLAB	-379.40	+163.43	-379.40
MOMENT IN MIDDLE STRIP SLAB	-126.46	+108.96	-126.46

(TECH 2)
NATHAN MCGRAW | EXISTING-TWO-WAY FLAT SLAB | PAGE 5 OF 15

MIDDLE STRIP

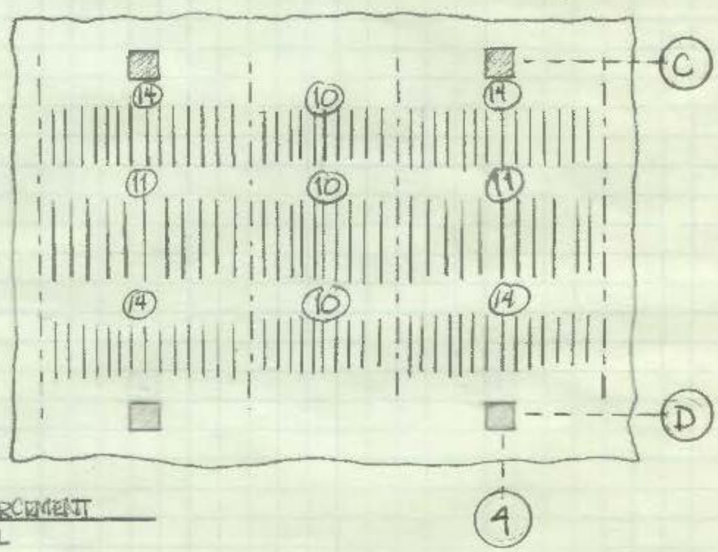
DESCRIPTION:	INTERIOR SPAN M^-	M^+
1) MOMENT M_u (K-ft)	-126.46	+108.96
2) WIDTH OF COLUMN STRIP	174"	174"
3) EFFECTIVE DEPTH	8.375"	8.375"
4) $M_n = M_u/\phi$	-140.51	121.07
5) $R = \frac{M_n \times 12000}{bd^2}$	138.16	119.04
6) ρ (TABLE A.5a NILSON)	0.00235	0.00202
7) $A_s = \rho bd$	3.42	2.94
8) $A_{s,min} = 0.0018bt$	2.9754	2.9754
9) $N = \frac{\text{Larger of 7 or 8}}{0.44}$	7.77 = 8	6.76 = 7
10) $N_{min} = \frac{\text{width of strip}}{2t}$	9.16 = 10	9.16 = 10

(TECH 2)

NATHAN M'GRAW | EXISTING - TWO WAY FLAT SLAB | PAGE 6 OF 15

REINFORCEMENT DESIGN AND DISTRIBUTION: ASSUMING #6 BARS

COLUMN STRIP — DESCRIPTION:	INTERIOR SPAN M^-	M^+
1) MOMENT M_u (K-FT)	-379.4	$+163.43$
2) WIDTH OF COLUMN STRIP	$14.5' \times 12" = 174"$	$174"$
3) EFFECTIVE DEPTH	$15.5" - 0.75" - \frac{1}{2}(5/8") = 14.4375"$	8.375
4) $M_n = M_u/\phi$, $\phi = 0.9$	-421.56	$+181.59$
5) $R = \frac{M_n \times 12000}{bd^2}$	139.48	178.55
6) ρ (TABLE A.5a NILSON)	0.00237	0.00306
7) $A_s = \rho bd$	5.9537 in²	4.45 in²
8) $A_{s,min} = 0.0018bt$	2.9754	2.9754
9) $N = \frac{\text{Larger of 7 or 8}}{0.44}$ ↳ Area of #6	$13.53 = 14$	$10.1 = 11$
10) $N_{MIN} = \frac{\text{width of strip}}{2t}$	$9.16 \approx 10$	$9.16 \approx 10$



6 REINFORCEMENT DETAIL

NATHAN McGRAW | (TECH 2) EXISTING-TWO-WAY FLAT SLAB | PAGE 7 OF 15

REINFORCEMENT COMPARISON WITH STRUCTURAL DRAWINGS:

COLUMN STRIP:

$M^- \rightarrow (14)\#6 = (14)(0.44\ in^2) = 6.16\ in^2$

As provided $= (14)\#6 = (14)(0.44\ in^2) = 6.16\ in^2 > 5.95\ in^2$ ✓

* SOME COLUMN STRIPS PROVIDE (16)#6 WHICH IS OK

$M^+ \rightarrow (11)\#6 = (11)(0.44\ in^2) = 4.84\ in^2$

As provided $= \#5 @ 12"$ o.c. $= (14.5')(0.31\ in^2/ft) = 4.495\ in^2 > 4.45\ in^2$ ✓

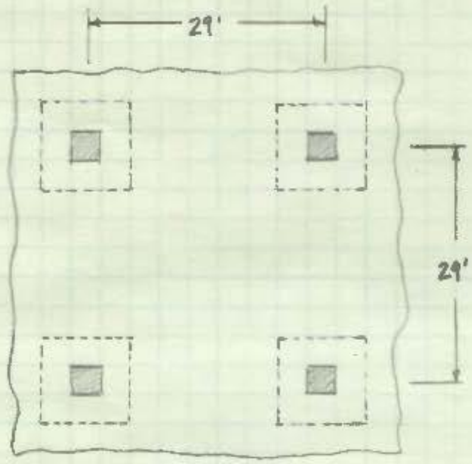
MIDDLE STRIP:

$M^- \rightarrow (10)\#6 = (10)(0.44\ in^2) = 4.4\ in^2$

As provided $= (16)\#5 = (16)(0.31\ in^2) = 4.96\ in^2 > 3.42\ in^2$ ✓

$M^+ \rightarrow (10)\#6 = (10)(0.44\ in^2) = 4.4\ in^2$

As provided $= \#5 @ 12" = (14.5')(0.31\ in^2/ft) = 4.495\ in^2 > 2.9754\ in^2$ ✓



7 TYPICAL INTERIOR PANEL

DEFLECTIONS:

$t = 9.5"$

ASSUMED: (Weighted Average)

- 67.5 % of moment to column strip
- 32.5 % of moment to middle strip

DL = 138.75 psf
LL = 80 psf

IMMEDIATE DEFLECTION DUE TO TOTAL DEAD LOAD:

COLUMN STRIP:

$$W_D = (138.75\ psf)(29')(0.675) = 2.716\ k/ft$$

$$I_g = \frac{(14.5' \times 12")(9.5")^3}{12} = 12431.9375\ in^4$$

$$E_c = 57000\sqrt{4000\ psi} = 3605\ ksi$$

$$\Delta_{D(max)} = \frac{0.0026(2.716)(29')^4(12")^3}{(3605)(12431.9375)} = 0.1926"$$

MIDDLE STRIP:

$$W_D = (138.75\ psf)(29')(0.325) = 1.308\ k/ft$$

$$I_g = \frac{(14.5' \times 12")(9.5")^3}{12} = 12431.9375\ in^4$$

$$\Delta_{D(max)} = \frac{0.0026(1.308)(29')^4(12")^3}{(3605)(12431.9375)} = 0.0927"$$

TOTAL IMMEDIATE Δ DUE TO DL_{TOTAL} = 0.1926" + 0.0927" = 0.285"

NATHAN MGRAW (TECH 2) EXISTING TWO-WAY FLAT SLAB PAGE 9 OF 15

IMMEDIATE DEFLECTION DUE TO TOTAL LIVE LOAD:

COLUMN STRIP:

$W_L = (80\text{psf})(29')(0.675) = 1.566\ ^{k}/_{ft}$

$$\Delta_{L(MAX)} = \frac{0.0048(1.566)(29')^4(12")^3}{(3605)(12431.9375)} = 0.205"$$

MIDDLE STRIP:

$W_L = (80\text{ psf})(29')(0.325) = 0.754\ ^{k}/_{ft}$

$$\Delta_{L(MAX)} = \frac{0.0048(0.754)(29')^4(12")^3}{(3605)(12431.9375)} = 0.0987"$$

TOTAL IMMEDIATE Δ DUE TO LL_{TOTAL} = $0.205" + 0.0987" = 0.304"$

ADDITIONAL DL Δ AFTER A LONG TIME DUE TO DL_{TOTAL}

* ASSUME $\lambda = 3.0$

$\Delta_{D(MAX)} = (3.0)[0.285 + 0.25(0.304)] = 1.083"$

CHECK DEFLECTIONS WITH ACI 318-08: TABLE 9.5 (b)

LIVE LOAD: $l/360$ (Floors not supporting or attached to nonstructural elements likely to be damaged by large deflections)

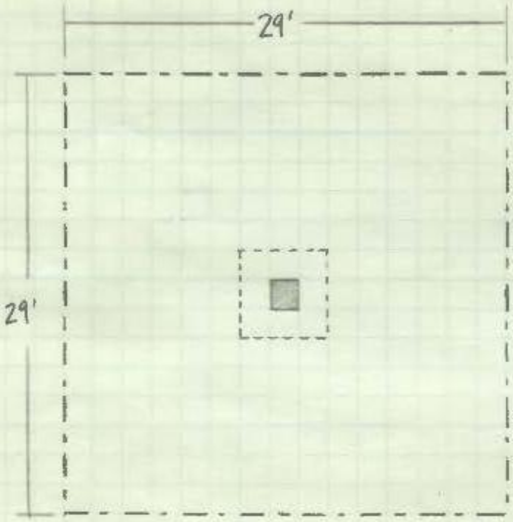
$\frac{l}{360} = \frac{(29' \times 12")}{360} = 0.967" > 0.304"$ ✓

TOTAL DEFLECTION AFTER PARTITIONS:

Δ_{max} for partitions $= 0.1(0.285) + 0.304 + 1.083" = 1.4155"$

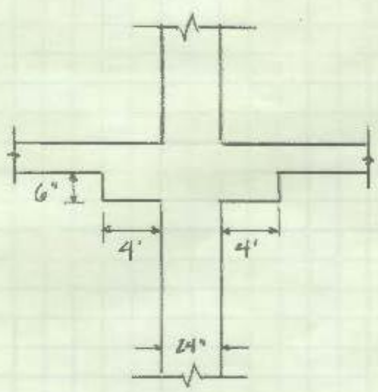
ACI 318-08 → $\frac{l}{240} = \frac{(29' \times 12")}{240} = 1.45" > 1.4155"$ ✓

NATHAN McGRAW | EXISTING TWO-WAY FLAT SLAB (TECH 2) | PAGE 10 OF 15



SHEAR:

8 PLAN VIEW OF TYPICAL INTERIOR COLUMN

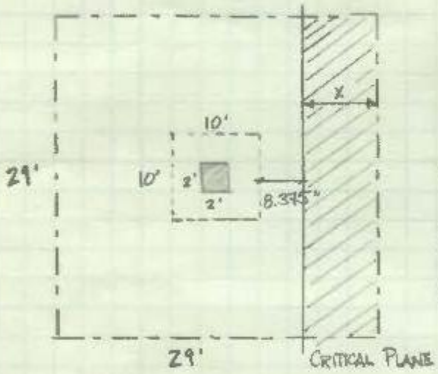


9 TYPICAL COLUMN ELEVATION

(TECH 2)

NATHAN McGRAW | EXISTING-TWO-WAY FLAT SLAB | PAGE 11 OF 15

WIDE BEAM ACTION:



$d = 8.375''$

$x = 29/2 - 5' - (8.375''/12) = 8.8'$

$W_u = 1.2(138.75\ psf) + 1.6(80\ psf)$

$W_u = 294.5\ psf$

10 WIDE BEAM ACTION

$V_n = V_c = 2\sqrt{f'_c}\, b_w d = 2\sqrt{4000}\,(29' \times 12'')(8.375'') = 368.66^k$

$\phi V_n = 368.66^k (0.75) = 276.49^k$

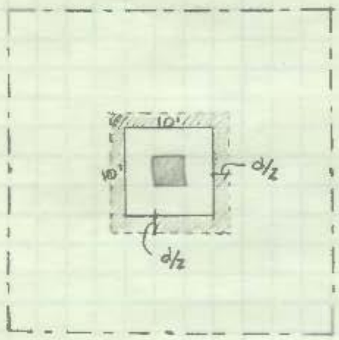
$V_u = W_u \times 8.8' \times 29'$

$V_u = (0.2945)(8.8)(29)$

$V_u = 75.16^k$

$\phi V_n = 276.49^k > V_u = 75.16^k$ ✓

PUNCHING SHEAR:



$d/2 = 8.375/2 = 4.1875"$

$b_o = 2(120" + 8.375 + 120" + 8.375") = 513.5"$

$b_o/d = \frac{513.5"}{8.375"} = 61.31$

$120" + 8.375" = 128.375" = 10.70'$

(11) PUNCHING SHEAR

$V_c = (\alpha_s/b_o/d + 2)\sqrt{f'_c}\, b_o d$ $\alpha_s = 40 \Rightarrow$ interior column

$V_c = (40/61.31 + 2)\sqrt{4000}(513.5)(8.375) = 721.43^k$

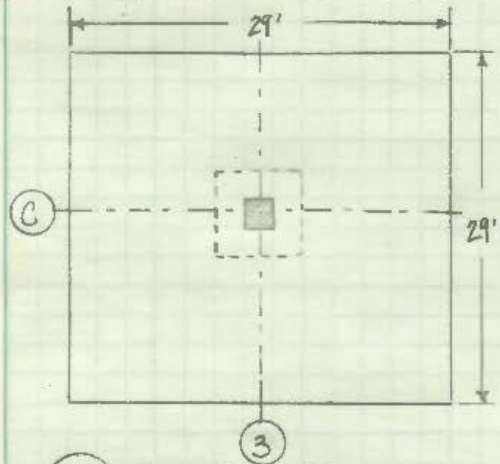
$\phi V_c = 0.75(721.43^k) = 541.1^k$

$V_u = W_u$ AREA

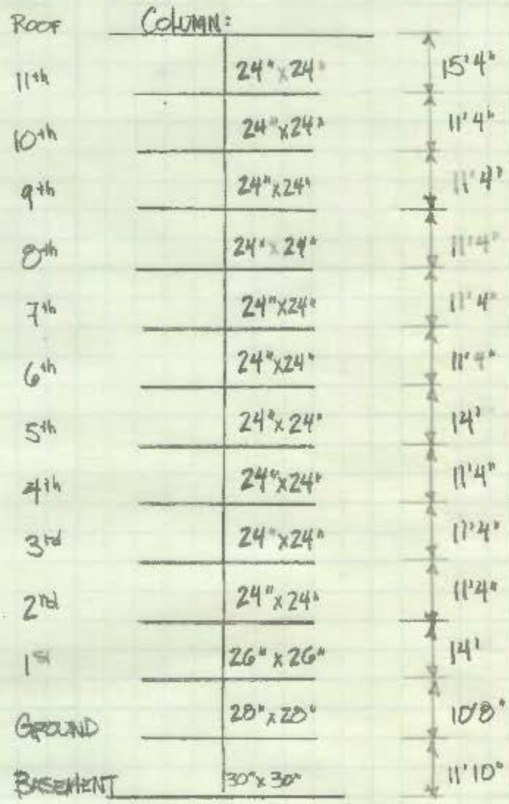
$V_u = (0.2945)(29' \times 29' - 10.70' \times 10.70')$

$V_u = 213.97^k < \phi V_c = 541.1^k$ ✓

NATHAN MCGRAW | (TECH 2) EXISTING - TWO-WAY FLAT SLAB | PAGE 13 OF 15



12 PARTIAL PLAN OF COLUMN C3



13 ELEVATION

TRIBUTARY AREA = 29' x 29' = 841 ft²
INFLUENCE AREA = 4 x 841 ft² = 3364 ft²
LOADS:

ROOF:

LL = 100 psf (Unreducible)
DL = 152.9375 psf
S = 21 psf
SDL = 20 psf
P_{DL} DROP PANELS = 7.5^K

TYPICAL FLOOR LOAD:

LL = | 80 psf corridors
min | 60 + 20 psf patient floors = 80 psf

LL MECHANICAL FLOOR = 150 psf (Unreducible)
SDL = 20 psf
DL = (9.5"/12")(150 lb/ft³) = 118.75 psf
P_{DL} DROP PANELS = (50 ft³)(150 lb/ft³) = 7.5^K
DL MECHANICAL FLOOR = (10.5/12)(150) = 131.25
LL ULTRASOUND = 150 psf (Unreducible)
$K_{LL} A_T$ > 400 ft² → USE LIVE LOAD REDUCTION

(TECH 2)

NATHAN MGRAW | EXISTING - TWO-WAY FLAT SLAB | PAGE 14 OF 15

COLUMN LOADS:

ROOF: $P_D = (841)(152.9375+20)/1000 = 145.4^k + 7.5^k = 152.9^k$
$P_L = (841)(100)/1000 = 84.1^k$

11th: $P_D = [(841)(20+118.75) + (24\times24/144)(150)(15.333)]/1000 + 7.5 = 133.4^k$
$LL_r = 0.25 + \frac{15}{\sqrt{3364}} = 0.509$

$P_L = 0.509(841)(80)/1000 = 34.22^k$

10th: $P_D = [(841)(20+118.75) + (24\times24/144)(150)(11.333)]/1000 + 7.5 = 131.0^k$
$LL_r = 0.25 + \frac{15}{\sqrt{2\times3364}} = 0.433$

$P_L = 0.433(841)(80)/1000 = 29.12^k$

9th: $P_D = 131.0^k$
$LL_r = 0.25 + \frac{15}{\sqrt{3\times3364}} = 0.399 < 0.4 \Rightarrow$ USE 0.4

$P_L = 0.4(841)(80)/1000 = 26.9^k$

8th: $P_D = 131.0^k$
$P_L = 26.9^k$

7th: $P_D = 131.0^k$
$P_L = 26.9^k$

6th: $P_D = 131.0^k$
$P_L = 26.9^k$

5th: $P_D = [(841)(20+131.25) + (24\times24/144)(150)(14')]/1000 + 7.5 = 143.1^k$
$P_L = (150)(841)/1000 = 126.15^k$

4th: $P_D = 131.0^k$
$P_L = 26.9^k$

3rd: $P_D = 131.0^k$
$P_L = 26.9^k$

2nd: $P_D = 131.0^k$
$P_L = 26.9^k$

1st: $P_D = [(841)(20+131.25) + (26\times26/144)(150)(14')]/1000 + 7.5 = 144.6^k$
$P_L = 26.9^k$

NATHAN McGRAW | (TECH 2) EXISTING - TWO-WAY FLAT SLAB | PAGE 15 OF 15

GROUND: $P_D = [(84)(20+113.25) + (28\times28/144)(150)(10.667')]/1000 + 7.5 = 128.3^k$

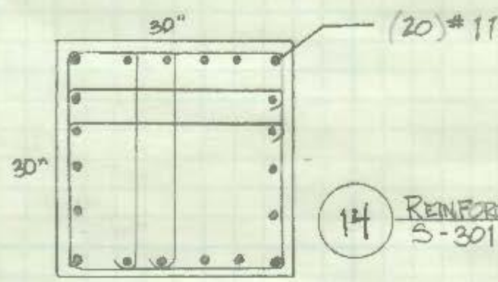
$P_L = 126.15^k$

$P_{D\,TOTAL} = 1750.3^k$
$P_{L\,TOTAL} = 530.84^k$
$P_{ROOF\,LIVE} = 84.1^k$

$P_U = 1.2(1750.3^k) + 1.6(530.84^k) + 0.5(84.1^k)$

$P_U = 2991.75^k$

CHECK COLUMN REINFORCING:



(14) REINFORCEMENT DETAIL S-301

$A_s = (20)(1.56\,in^2) = 31.2\,in^2$

* COLUMN IS CHECKED FOR PURE COMPRESSION BECAUSE IT IS AN INTERIOR COLUMN

$\phi P_o = \phi(0.85 f'_c A_c + A_s f_y)$

$\phi P_o = 0.65[(0.85)(7)(30"\times30" - 31.2) + (31.2)(60)]$

$\phi P_o = 4576.9^k$

PURE COMPRESSION LIMITED BY α

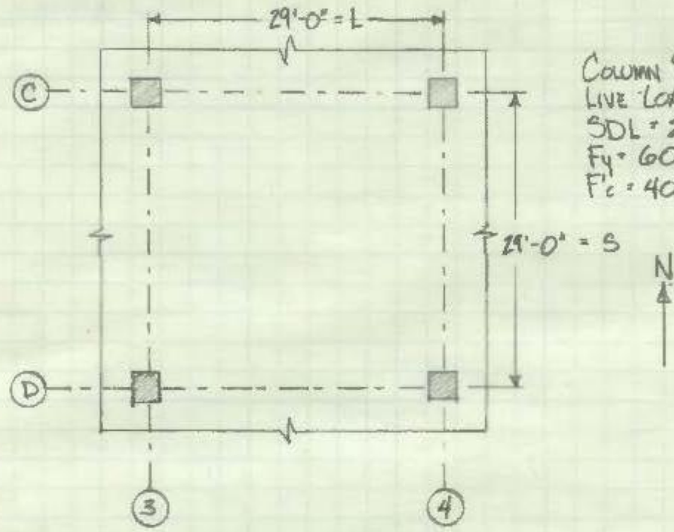
$\phi P_n = \alpha \phi P_o = 0.8(4576.9^k) = 3661.5^k > 2991.75^k = P_u$ ✓

$\rho = \frac{31.2}{30"\times30"} = 0.035 > 0.01$ ✓ (MINIMUM REINFORCEMENT ACI318-08)

Appendix B: Post-Tensioned Concrete Calculations

(TECH 2)

NATHAN M^cGRAW | TWO-WAY POST-TENSIONED | PAGE 1 OF 7



COLUMN SIZES: 24" x 24"
LIVE LOAD = 80 psf
SDL = 20 psf
Fy = 60 ksi
F'c = 4000 psi

1 TYPICAL INTERIOR PANEL

$f'_c = 4000$ psi
$f'_{ci} = 3000$ psi
$f_{pu} = 270{,}000$ psi
$f_{py} = 240{,}000$ psi
$f_{pe} = 159{,}000$ psi
$E_{ps} = 29000$ ksi
$f_y = 60{,}000$ psi
$E_s = 29000$ ksi

MAXIMUM f'_c due to combined stresses $= 0.45 f'_c = 0.45(4000) = 1800$ psi
E-W MAXIMUM f_c due to net prestress after losses ≤ 200 psi
N-S MAXIMUM f_c due to net pressures after losses ≤ 350 psi

TRIAL SIZE FOR SLAB THICKNESS:

$$\frac{l}{h} \cong 45$$

$$h \cong \frac{l}{45}$$

$$h = \frac{\left[\frac{(29'+29')}{2}\right] \times 12"}{45} = 7.73"$$

$\boxed{\text{*USE 8.0" SLAB THICKNESS}}$

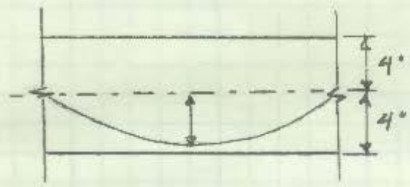
(TECH 2)

NATHAN MGRAW | TWO-WAY POST-TENSIONED | PAGE 2 OF 7

BALANCING LOAD:

$W_D = SDL + DL = 20\,psf + (8/12)(150\,pcf) = 120\,psf$

$W_{BAL} = W_D$ FOR ZERO DEFLECTION OR CAMBER



$e_s = (8/2) - 1" = 3"$

EFFECTIVE PRESTRESS:

NORTH-SOUTH DIRECTION

$P_L = 200(8)(12) = 19200\ lb/strip$

$W_{BAL(L)} = \frac{8 P_L e_L}{L_L^2} = \frac{8(19200)(3)}{(29')^2 \times 12} = 45.66\ psf$

$W_{BAL(S)} = W_D - W_{BAL(L)} = 120 - 45.66 = 74.34\ psf$

$P_S = \frac{W_{BAL(S)} L_S^2}{8 e_S} = \frac{(74.34)(29')^2(12)}{8(3")} = 31,259.97\ ^K$

$f_c = \frac{31259.97}{12" \times 8"} = 325.6\ psi < 350\ psi$ ∴ OK

* USING ½" ϕ 7-wire strand ($A = 0.153\ in^2$) 270 K tendons

$P_e = 159,000\,(0.153) = 24,327\ lb$

REQUIRED SPACING:

REQUIRED SPACING IN THE NORTH-SOUTH DIRECTION:

$S_S = \frac{24327}{31259.97} = 0.78' = 9.34" \Rightarrow 9"$

REQUIRED SPACING IN THE EAST-WEST DIRECTION:

$S_L = \frac{24327}{19200} = 1.27' = 15.2" \Rightarrow 15"$

(TECH 2)

NATHAN MCGRAW | TWO-WAY POST-TENSIONED | PAGE 3 OF 7

RECOMMENDED SPACING → 3h → 5h

$3h = 3(8) = 24"$
$5h = 5(8) = 40"$

*BOTH ARE UNDER RECOMMENDED ∴ OK

SERVICE LOAD STRESSES:

$W_L = 80$ psf

$K = \frac{L_L}{L_S} = \frac{29'}{29'} = 1.0$

$\alpha_{N-S} = 0.05$ (USED FIGURE 9.10 TO OBTAIN α VALUES)
$\alpha_{E-W} = 0.05$

$L_S = 29' - (24"/12) = 27'$
$L_L = 29' - (24"/12) = 27'$

LIVE LOAD MOMENTS:

$M_S = 0.05(80)(27')^2(12) = 34992$ in-lb/ft

$M_L = 0.05(80)(27')^2(12) = 34992$ in-lb/ft

MOMENT OF INERTIAL:

$I_s = \frac{12(8)^3}{12} = 512 \text{ in}^4$

CONCRETE STRESSES DUE TO LIVE LOAD: (BOTH DIRECTIONS EQUAL)

$f = \frac{Mc}{I} = \frac{34992\,(4")}{512} = 273.375$ psi $= 273$ psi

$f_{top} = f_t = -\frac{P_s}{bh} - \frac{M_s c}{I_s}$

$f_{bottom} = f_b = \frac{-P_s}{bh} + \frac{M_s c}{I_s}$

NORTH-SOUTH DIRECTION:

$f_t = -325.6 - 273 = -598.6$ psi (C)

$f_b = -325.6 + 273 = -52.6$ psi (C) (VERY SMALL → NEGLIGIBLE)

NATHAN MCGRAW (TECH 2) TWO-WAY POST-TENSIONED PAGE 4 OF 7

EAST-WEST DIRECTION:

$f^t = -325.6 - 273 = -598.6$ psi (C)

$f_b = -325.6 + 273 = -52.6$ psi (C) (VERY SMALL → NEGLIGIBLE)

ALLOWABLE COMPRESSIVE STRESS $= f_c = 0.45(4000) = 1800$ psi $> f^t$ ∴ OK
$> f_b$ ∴ OK

DEFLECTION CHECK:

$\Delta_{LL} = \dfrac{5 M L^2}{48 E_c I_s}$

$I_s = 512 \text{ in}^4$
$E_c = 57000\sqrt{4000} = 3605000 = 3.605 \times 10^6$ psi

$\Delta_{E\text{-}W} = \Delta_{N\text{-}S} = \dfrac{5(34992)(29')^2(144)}{48(3.605 \times 10^6)(512)} = 0.239''$

AVERAGE MIDSPAN DEFLECTION $= \Delta = 0.239''$

$\Delta_{LL\ ALLOWABLE} = \dfrac{L}{360} = \dfrac{29' \times 12''}{360} = 0.967'' \gg 0.239''$ ∴ OK

NOMINAL MOMENT STRENGTH:

$W_U = 1.2(120) + 1.6(80) = 272$ psf

$L_{S\,EFF} = 27'$
$L_{L\,EFF} = 27'$

$\alpha_{N\text{-}S} = 0.055$
$\alpha_{E\text{-}W} = 0.055$

FACTORED $M_U = 0.055(272)(27')^2(12) = 130870$ in-lb/ft

REQUIRED $M_n = \dfrac{M_U}{\phi} = \dfrac{130870}{0.9} = 145{,}411$ in-lb/ft

NORTH-SOUTH DIRECTION:

$A_{ps} = 0.153 \text{ in}^2$ @ $0.78'$

$\dfrac{A_{ps}}{ft} = \dfrac{0.153}{0.78} = 0.196 \text{ in}^2/\text{ft}$

$\rho_{N\text{-}S} = \dfrac{0.196}{12'' \times 8''} = 0.002$

NATHAN MCGRAW | (TECH 2) TWO-WAY POST-TENSIONED | PAGE 5 OF 7

SLAB SPAN TO DEPTH RATIO $= \frac{29' \times 12"}{8} = 43.5 > 35$

$f_{ps} = f_{pe} + 10{,}000 + \frac{f'_c}{300 \rho_p} \leq f_{py} \leq f_{pe} + 30000$

$f_{ps} = 159000 + 10000 + \frac{4000}{300(0.002)} \leq 240000 \leq 159000 + 30000$

$f_{ps} = 175666.7$ psi $\leq 240000 \leq 189000$

$f_{ps} = 175{,}667$ psi

$a = \frac{A_{ps} f_{ps}}{0.85 f'_c b} = \frac{0.196(175667)}{0.85(4000)(12")} = 0.844"$

$M_n = A_{ps} f_{ps} (d - a/2)$ $\quad d = 8" - (0.5/2 + 3/4") = 7"$

$M_n = (0.196)(175667)(7" - 0.844/2)$

$M_n = 226{,}485$ in-lb $> 145{,}411$ lb-lb $= M_n$ required $\therefore$ OK

EAST-WEST DIRECTION:

$A_{ps} = 0.153 \text{ in}^2$ @ 1.27'

$\frac{A_{ps}}{ft} = \frac{0.153}{1.27} = 0.121 \text{ in}^2/ft$

$\rho_{E\text{-}W} = \frac{0.121}{12" \times 8"} = 0.00126$

SLAB SPAN TO DEPTH RATIO $= 43.5 > 35$

$f_{ps} = 159000 + 10000 + \frac{4000}{300(0.00126)} \leq 240000 \leq 189000$

$f_{ps} = 179600$ psi

$a = \frac{(0.121)(179600)}{0.85(4000)(12")} = 0.533$

$M_n = (0.121)(179600)(7" - 0.533/2)$

$M_n = 146{,}330$ in-lb $> 145{,}411$ in-lb $= M_n$ required $\therefore$ OK

(TECH 2)
NATHAN McGRAW | TWO-WAY POST-TENSIONED | PAGE 6 OF 7

MINIMUM REINFORCEMENT: (PER ACI 318-08 Chapter 18)

- In positive moment areas where computed tensile stresses at service load exceeds $2\sqrt{f'_c}$

  ∴ NO AREAS WHERE TENSILE STRESSES EXCEED $2\sqrt{f'_c}$
  → No bonded reinforcement needed in positive moment areas.

- In negative moment areas at column supports: (IN EACH DIRECTION)

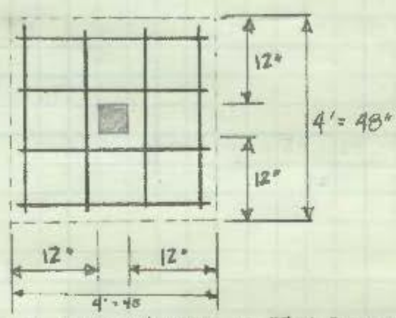
$A_s = 0.00075 A_{cf}$

$A_{cf} = (29' \times 12'')(8'') = 2784$

$A_{s\,min} = 0.00075(2784) = 2.088$

DISTRIBUTED BETWEEN 1.5h FROM COLUMN OUT

$1.5h = 1.5(8'') = 12''$



- NEED AT LEAST 4 BARS IN EACH DIRECTION WITH SPACING ≤ 12"

*USE (4)#7 IN EACH DIRECTION → $A_s = (4)(0.60) = 2.4\,in^2 > 2.088$ ∴ OK

(TECH 2)
NATHAN M^cGRAW | TWO-WAY POST-TENSIONED | PAGE 7 OF 7

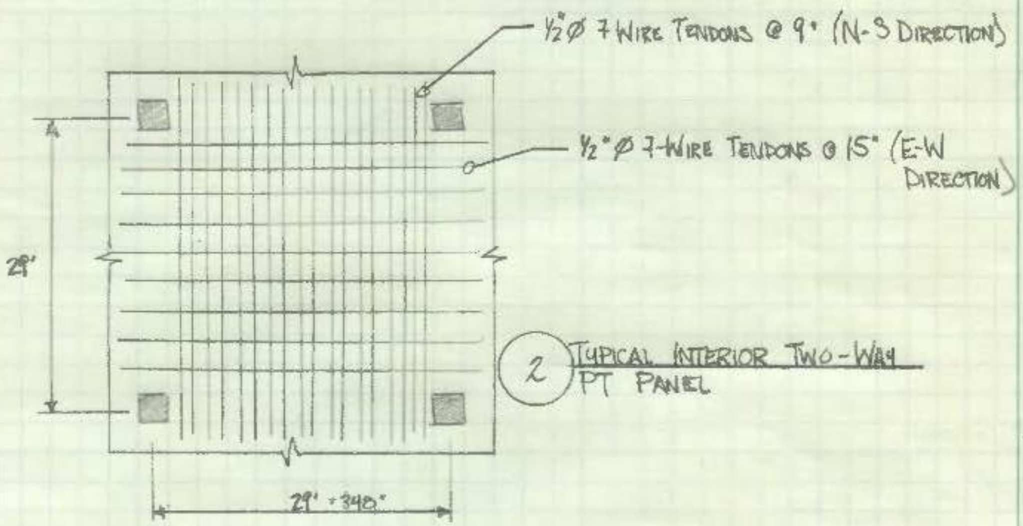
SHEAR STRENGTH: (WIDE BEAM ACTION)

$V_u = 1/3\, w_u L_s = 1/3(272)(27') = 2448$ lb/ft (BOTH DIRECTIONS)

$V_c = 2\sqrt{f'_c}\, b_w d_p = 2\sqrt{4000}\,(12'')(7'') = 10{,}625$ lb/ft

$\phi V_c = 0.75(10{,}625) = 7969$ lb/ft $\gg 2448$ lb/ft $= V_u$ ∴ OK

SUMMARY:
8" SLAB
$d_p = 7''$
1/2 ∅ 7-wire 270-K tendons
Spaced @ 9" in N-S DIRECTION
Spaced @ 15" in E-W DIRECTION



2 TYPICAL INTERIOR TWO-WAY PT PANEL

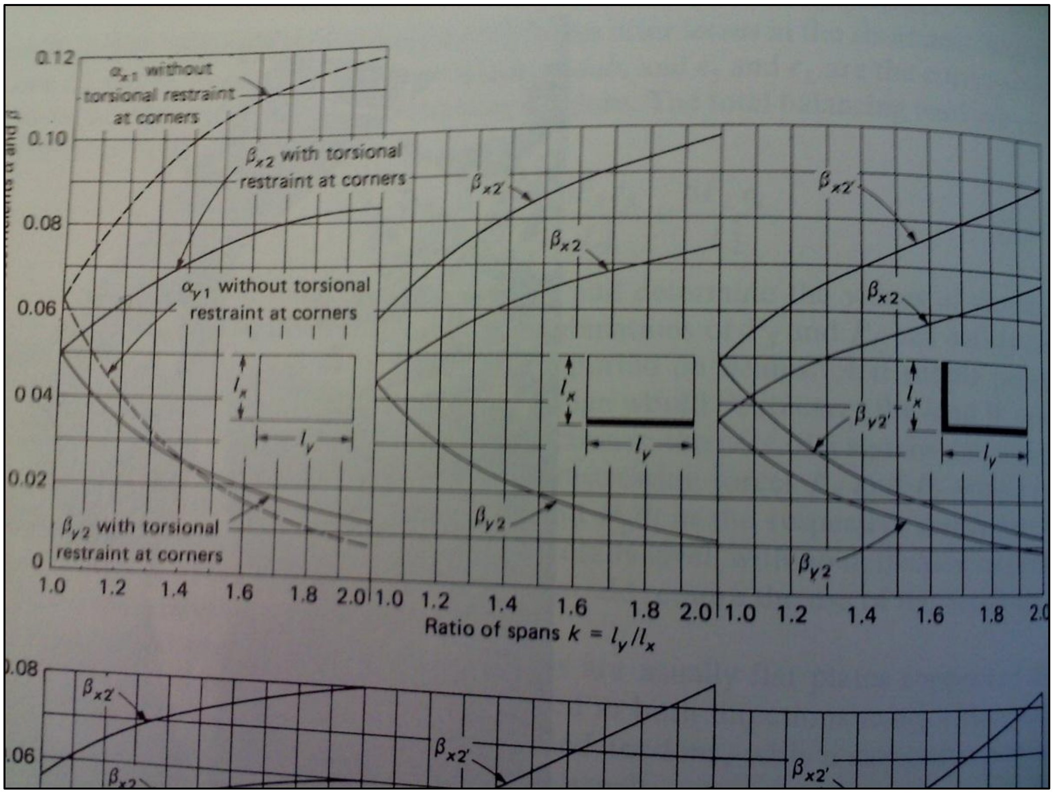
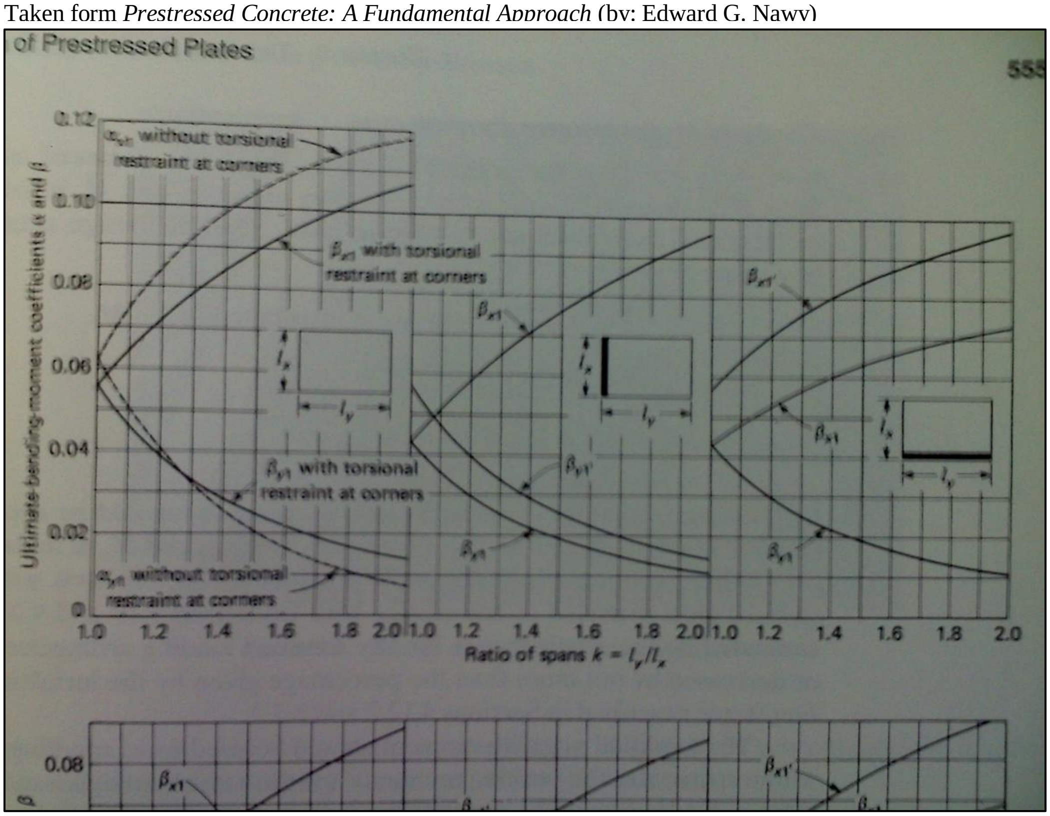


Figure 9.10:
Taken form *Prestressed Concrete: A Fundamental Approach* (by: Edward G. Nawy)

Figure 9.11:
Taken form *Prestressed Concrete: A Fundamental Approach* (by: Edward G. Nawy)

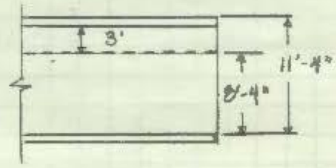


Appendix C: Composite Steel Calculations

NATHAN McGRAW | COMPOSITE STEEL (TECH 2) | PAGE 1 OF 8

HEIGHT CONSIDERATIONS:

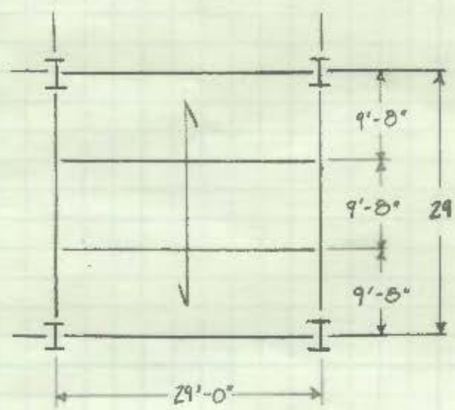
TYP. FLOOR - FLOOR HEIGHT = 11'-4"
TYP. CEILING HEIGHT LEVEL = 8'-4"
MAX DEPTH = 3' = 36"



1 FL - FL DIMENSIONS

DECK: → USED VULCRAFT MANUAL (2008 EDITION)

- USE 1.5" OR 2" DECK → Sizes being used on project already
- 2 HOUR FIRE RATING → MUST USE 4½" NW CONCRETE TOPPING OR SPRAY ON FIRE PROOFING
  ∴ Since spans are relatively long, choose spray on fire proofing to reduce weight
- DESIGN FOR 3-SPAN MINIMUM CONDITION



$b_{eff\ beam} = \min \left| \begin{array}{l} 29' \times 12/8 \\ (\tfrac{1}{2})(9'\text{-}0")(12) + \min \left| \begin{array}{l} 29' \times 12/8 \\ (\tfrac{1}{2})(9'\text{-}8")(12) \end{array}\right. \end{array}\right.$

$b_{eff\ beam} = 87"$

$b_{eff\ girder} = \min \left| \begin{array}{l} 43.5 \\ (\tfrac{1}{2})(29')(12) \end{array}\right. + \min \left| \begin{array}{l} 43.5 \\ (\tfrac{1}{2})(29')(12) \end{array}\right.$

$b_{eff\ girder} = 87"$

2 TYPICAL BAY

- MAXIMUM SPAN = 9'-8"
- LOADS:
  SUPERIMPOSED DEAD LOAD = 20 psf
  LIVE LOAD = 80 psf
  TOTAL = 100 psf
- TOPPING → USE 3.5" TOPPING NW CONCRETE

⇒ 1.5VL: 5" TOTAL THICKNESS (t = 3.5")

1.5VL 18
SDI MAX SPAN = 9'-8" = 9'-8" ∴ OK
216 psf @ 10'-0" > 100 psf ∴ OK
DECK WEIGHT = 2.32 psf

NATHAN McGRAW | COMPOSITE STEEL (TECH 2) | PAGE 2 OF 8

⇒ 1.5VLR : DOES NOT MEET SPAN REQUIREMENTS

⇒ 2VLI : 5.5" TOTAL THICKNESS (t = 3.5")

2VLI20

SDI MAX UNSHORED SPAN = 9'-9" > 9'-8" ∴ OK

LOAD @ 10'-0" = 143 psf > 100 psf ∴ OK

DECK WEIGHT = 1.97 psf

*SINCE 2VLI20 DECK IS LIGHTER/CHEAPER → CHOOSE

$\boxed{\text{USE 2VLI20 with 3.5" NW topping}}$ SELF WEIGHT = 57 PSF

DESIGN OF INTERIOR BEAM:

- MAXIMUM DEPTH = 36" − 5.5" = 30.5" → No REAL ISSUES (Choose smallest member to help fit mechanical equipment)
- ASSUME SIMPLY SUPPORTED
- ADD 5 PSF FOR BEAM SELF-WEIGHT

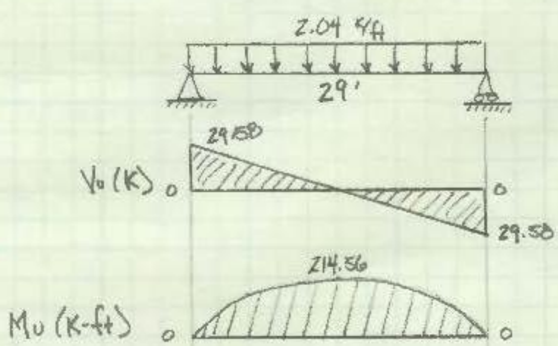
$w_D = (20 + 57 + 5)(9.667') = 792.67 \text{ lb/ft} = 0.793 \text{ k/ft}$

$LL_r = 0.25 + \dfrac{15}{\sqrt{(2)(29')(9.667')}} = 0.883$

$= 561 \# > 400 \#$ ✓

$w_L = 0.883 (80 \text{ psf})(9.667') = 683.2 \text{ lb/ft} = 0.683 \text{ k/ft}$

$w_U = 1.2(0.793) + 1.6(0.683) = 2.04 \text{ k/ft}$



$V_u = 2.04\,(29/2) = 29.58^K$

$M_u = \dfrac{wl^2}{8} = \dfrac{(2.04)(29')^2}{8} = 214.46^{K\text{-}ft}$

Q_n:

- DECK $\perp$ TO BEAM
- WEAK POSITION
- 1 STUD/RIB
- 3/4" ϕ STUDS
- $f'_c = 4000$ psi
- NW CONCRETE ($w_c = 145$ pcf)

$Q_n = 17.2^k$ (PER AISC TABLE 3-21)

DEFLECTIONS:

$$\Delta_{LL\,ALLOWABLE} = \frac{l}{360} = \frac{29'(12")}{360} = 0.967"$$

$$\Delta_{LL} = \frac{5 w_L l^4 (1728)}{384 E I_{LB}} \Rightarrow I_{LB} = \frac{5 w_L l^4 (1728)}{384 E \Delta_{LL}}$$

$$I_{LB} = \frac{5(0.683)(29')^4(1728)}{384(29000)(0.967)} = 387.72 \text{ in}^4 \quad \text{(minimum)}$$

ASSUME $a = 1.0" \Rightarrow y_2 = 5.5" - (1"/2) = 5.0"$

TRIAL SIZES USING AISC TABLES 3-19 AND 3-20:

- W10x22 w/ $\Sigma Q_n = 273^k$ $\phi M_n = 226'^k$ $I_{LB} = 420$ in^4
  \# of studs required = 273/17.2 = 15.87 $\Rightarrow$ 32 TOTAL STUDS
  ECONOMY = 22(29') + 32(10) = 958 lbs of steel

- W10x26 w/ $\Sigma Q_n = 190^k$ $\phi M_n = 216'^k$ $I_{LB} = 405$ in^4
  \# of studs required = 190/17.2 = 11.04 $\Rightarrow$ 24 TOTAL STUDS
  ECONOMY = 26(29') + 24(10) = 994 lbs of steel

- W12x19 w/ $\Sigma Q_n = 243^k$ $\phi M_n = 219'^k$ $I_{LB} = 450$ in^4
  \# of studs required = 243/17.2 = 14.12 $\Rightarrow$ 30 TOTAL STUDS
  ECONOMY = 19(29') + 30(10) = 851 lbs of steel

- W12x22 w/ $\Sigma Q_n = 196^k$ $\phi M_n = 221'^k$ $I_{LB} = 460$ in^4
  \# of studs required = 196/17.2 = 11.4 $\Rightarrow$ 24 TOTAL STUDS
  ECONOMY = 22(29') + 24(10) = 878 lbs of steel

TRY W12x19

DEFLECTIONS:

$$\Delta_{LL} = \frac{5(0.683)(29')^4(1728)}{384(29000)(450)} = 0.833" < 0.967" \quad \therefore \underline{OK}$$

NATHAN MCGRAW | COMPOSITE STEEL (TECH 2) | PAGE 4 OF 8

$\Delta_{TL\ ALLOWABLE} = \frac{l}{240} = \frac{(29')(12")}{240} = 1.45"$

$\Delta_{TL} = \frac{5(0.793+0.683)(29')^4(1728)}{384(29000)(450)} = 1.80" > 1.45"$ ∴ No Good

*USE 1/2" CAMBER

UNSHORED STRENGTH:

W12x19 $\phi_b M_p = 92.6'^K$

$W_U = 1.4(57\text{ psf})(9.667') + 1.4(19) = 798\text{ plf} = 0.798\text{ klf}$
OR $W_U = 1.2(57)(9.667') + 1.2(19) + 1.6(20)(9.667') = 993.3\text{ plf} = 0.993\text{ klf}$

$M_U = \frac{(0.993)(29')^2}{8} = 104.4'^K > 92.6'^K$ ∴ No Good

TRY W12x22 : $I = 156\text{ in}^4$

$\Delta_{LL} = \frac{5(0.683)(29')^4(1728)}{384(29000)(460)} = 0.815" < 0.967"$ ∴ OK

$\Delta_{TL} = \frac{5(0.793+0.683)(29')^4(1728)}{384(29000)(450)} = 1.8" > 1.45"$ ∴ No Good

*USE 1/2" CAMBER

$W_U = 1.2(57)(9.667') + 1.2(22) + 1.6(20)(9.667') = 0.997\text{ klf}$

$M_U = \frac{(0.997)(29')^2}{8} = 104.8'^K < \phi_b M_p = 110'^K$ ∴ OK FOR NO SHORING

WET CONCRETE DEFLECTION:

$W_{WC} = 57(9.667') + 22 = 0.573\text{ klf}$

$\Delta_{WC} = \frac{5(0.573)(29')^4(1728)}{384(29000)(156)} = 2.02" > \Delta_{WC\ ALLOWABLE} = 1.45"$
∴ No Good

*USE 3/4" CAMBER

SELF-WEIGHT $= \frac{22\text{ plf}}{9.667'} = 2.27\text{ psf} < 5\text{ psf}$ ASSUMED ∴ OK

$a = 196/0.85(4)(87") = 0.663 < 1.0$ ASSUMED ∴ OK

$\boxed{\text{COMPOSITE BEAM: W12x22 w/ 24 STUDS AND 3/4" CAMBER}}$

*NO NEED TO CHECK LATERAL TORSIONAL BUCKLING DUE TO SHEAR STUDS

NATHAN McGRAW | COMPOSITE STEEL (TECH 2) | PAGE 5 OF 8

DESIGN OF GIRDER:

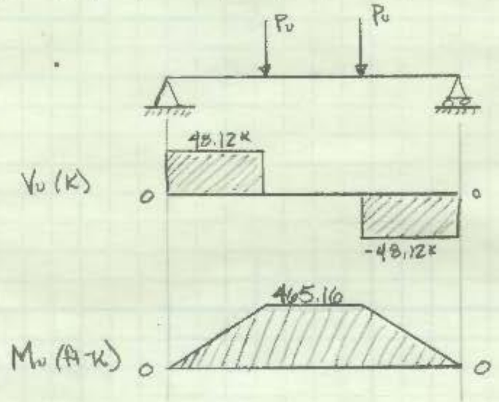
- ASSUME SIMPLY SUPPORTED
- ADD 1^K TO EACH POINT LOAD FOR GIRDER SELF-WEIGHT

$P_D = 0.793(29') = 23^K$

$LL_r = 0.25 + \frac{15}{\sqrt{(2)(29')(29')}} = 0.616$

$P_L = 0.616(0.683)(29') = 12.2^K$

$P_u = 1.2(23) + 1.6(12.2) + 1^K = 48.12^K$



$V_u = \frac{(2)(48.12)}{2} = 48.12^K$

$M_u = (9.667')(48.12) = 465.16'^K$

Q_n:

- DECK // TO GIRDER
- WEAK POSITION
- 1 STUD/RIB
- $\frac{w_r}{h_r} = \frac{5}{2} = 2.5 > 1.5$
- 3/4" ϕ STUD
- $f'_c = 3000$ PSI
- NW CONCRETE

$Q_n = 21.5^K$ (PER AISC TABLE 3-21)

DEFLECTIONS:

$\Delta_{LL\ ALLOWABLE} = 0.967" + 3/4"$ CAMBER $= 1.717"$

$\Delta_{LL} = \frac{P\ell^3}{28EI}$ FOR $a = \ell/3$ $\therefore$ OK

NATHAN M'GRAW | COMPOSITE STEEL (TECH 2) | PAGE 6 OF 8

$$I_{LB} = \frac{P\ell^3}{28EA_u} = \frac{(12.2^k)(29')^3(1728)}{28(29000)(1.717")} = 368.8 \text{ in}^4$$

TRIAL SIZES USING AISC TABLES 3-19 AND 3-20:

ASSUME $a = \frac{1}{2}"$ $\Rightarrow Y_2 = 5.5 - 0.5" = 5"$

- W14x38 w/ $\Sigma Q_n = 473^k$ $\phi M_n = 473'^k$ $I_{LB} = 1130 \text{ in}^4$
  \# of studs required $= 473/21.5 = 22 \Rightarrow 44$ TOTAL STUDS
  ECONOMY $= 38(29) + 44(10) = 1542$ lbs of steel

- W16x36 w/ $\Sigma Q_n = 455^k$ $\phi M_n = 486'^k$ $I_{LB} = 1270 \text{ in}^4$
  \# of studs required $= 455/21.5 = 21.2 \Rightarrow 44$ TOTAL STUDS
  ECONOMY $= 36(29) + 44(10) = 1484$ lbs of steel

- W16x40 w/ $\Sigma Q_n = 325^k$ $\phi M_n = 472'^k$ $I_{LB} = 1230 \text{ in}^4$
  \# of studs required $= 325/21.5 = 15.1 \Rightarrow 32$ TOTAL STUDS
  ECONOMY $= 40(29) + 32(10) = 1480$ lbs of steel

- W18x35 w/ $\Sigma Q_n = 388$ $\phi M_n = 486'^k$ $I_{LB} = 1360 \text{ in}^4$
  \# of studs required $= 388/21.5 = 18.05 \Rightarrow 38$ TOTAL STUDS
  ECONOMY $= 35(29) + 38(10) = 1395$ lbs of steel

UNSHORED STRENGTH ISSUE:

$$P_u = [1.2(57 \times 9.667' + W) + 1.6(20 \times 9.667')]\, 29'$$

$$= [661.2 + 1.2W + 309.333]\, 29'$$

$$= \frac{28145.47 + 34.8W}{1000 \text{ lb/k}}$$

$$M_u = (28.14547 + 0.0348W)(9.667')$$

W14x38 $\Rightarrow M_u = 285'^k > \phi_b M_p$ $\therefore$ NO GOOD
W16x36 $\Rightarrow M_u = 284'^k > \phi_b M_p$ $\therefore$ NO GOOD
W16x40 $\Rightarrow M_u = 286'^k > \phi_b M_p$ $\therefore$ NO GOOD
W18x35 $\Rightarrow M_u = 284'^k > \phi_b M_p$ $\therefore$ NO GOOD

TRY:
W16x45 $\Rightarrow M_u = 287'^k < \phi_b M_p = 309'^k$ $\therefore$ OK
W18x46 $\Rightarrow M_u = 288'^k < \phi_b M_p = 340'^k$ $\therefore$ OK

- W16x45 w/ $\Sigma Q_n = 217$ $\phi M_n = 470'^k$ $I_{LB} = 1140 \text{ in}^4$
  \# of studs required $= 217/21.5 = 10.1 \Rightarrow 22$ TOTAL STUDS
  ECONOMY $= 45(29') + 22(10) = 1525$ lbs of steel

- W18x46 w/ $\Sigma Q_n = 169$ $\phi M_n = 488'^k$ $I_{LB} = 1280 \text{ in}^4$
  \# of studs required $= 169/21.5 = 7.9 \Rightarrow 16$ TOTAL STUDS
  ECONOMY $= 46(29') + (16)(10) = 1494$ lbs of steel

NATHAN McGRAW | COMPOSITE STEEL (TECH 2) | PAGE 7 OF 8

CONTINUE WITH W18x46

$$\Delta_{LL} = \frac{(12.2^k)(29')^3(1728)}{28(29000)(1280)} = 0.5'' < \Delta_{LL\ ALLOWABLE} \quad \therefore \underline{OK}$$

WITHOUT CAMBER

$$\Delta_{TL\ ALLOWABLE} = \frac{(29')(12'')}{240} = 1.45''$$

$$\Delta_{TL} = \frac{(23^k + 1^k + 12.2^k)(29')^3(1728)}{28(29000)(1280)} = 1.468'' < 1.45''$$

* USE 1/4" CAMBER

CHECKS:

$a = 169/0.85(4)(87'') = 0.57 < 1.0$ ASSUMED

Self-Weight = 46 plf x 9.667' = $0.44^k < 1^k$

BOTH OK

UNSHORED STRENGTH:

W18x46 $\phi_b M_p = 340'^k > M_u = 288'^k \quad \therefore \underline{OK}$ (M_u CALCULATED ON PREVIOUS PAGE)

COMPOSITE GIRDER: W18x46 w/ 16 STUDS AND 1/4" CAMBER

DESIGN INTERIOR COLUMN:

$P_u = 2(48.12^k) + 2(29.58^k) = 155.4^k$ @ 11'-4" UNBRACED LENGTH

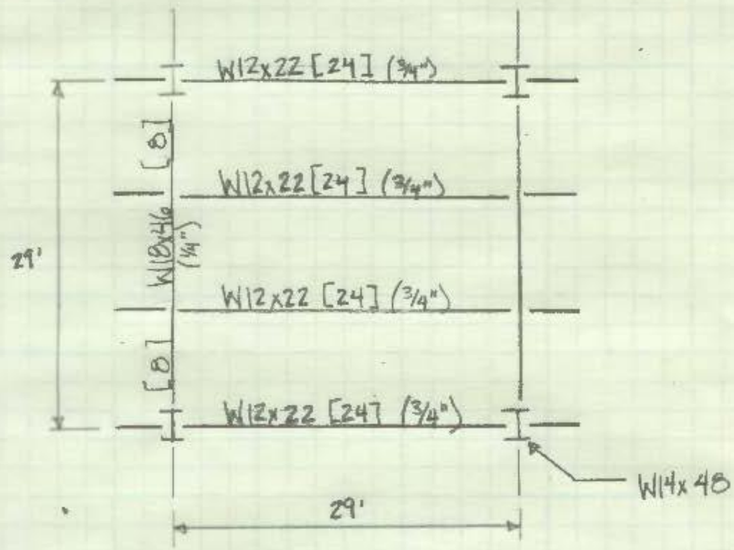
* ASSUME K = 2.0 (CONSERVATIVE APPROACH)

KL = 22.667'

FROM AISC TABLE 4-1:

INTERIOR COLUMN: W14x48 w/ $\phi P_n = 158.0^k > 155.4^k \quad \therefore \underline{OK}$

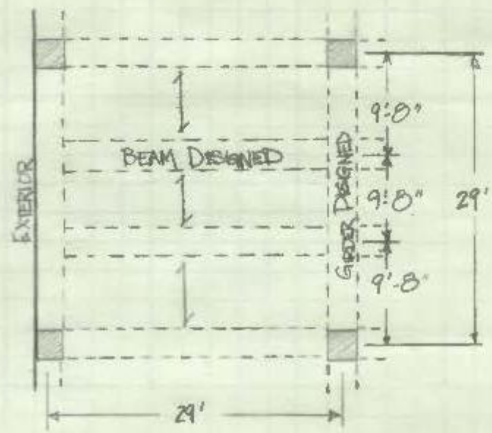
NATHAN M^cGRAW — COMPOSITE STEEL (TECH 2) — PAGE 8 OF 8



3 TYPICAL COMPOSITE INTERIOR PANEL

Appendix D: One-Way Slab and Beam Calculations

NATHAN MCGRAW | ONE-WAY SLAB (TECH 2) | PAGE 1 OF 11



COLUMN SIZES = 24" x 24"

F'c = 4000 PSI

Fy = 60 KSI

(1) TYPICAL ONE WAY SLAB

MINIMUM SLAB THICKNESS: PER TABLE 9.5(a) ACI318-08

EXTERIOR BAY = $\frac{l}{24} = \frac{(9.667' \times 12")}{24} = 4.8"$

INTERIOR BAY = $\frac{l}{28} = \frac{(9.667' \times 12")}{28} = 4.14"$

$\boxed{\text{* USE A SLAB THICKNESS (t) = 5.0"}}$

* ASSUME #4 BARS
$d = h - \text{CLR COVER} - d_b/2 = 5" - 3/4" - 0.5/2 = 4"$

$W_D = (5/12")(150 \text{ PCF}) = 62.5 \text{ PSF} + 20 \text{ PSF SUPERIMPOSED} = 82.5 \text{ PSF}$
$W_L = 60 \text{ PSF} + 20 \text{ PSF PARTITION} = 80 \text{ PSF}$ (CANNOT BE REDUCED)

$W_U = 1.2(82.5) + 1.6(80) = 227 \text{ PSF}$

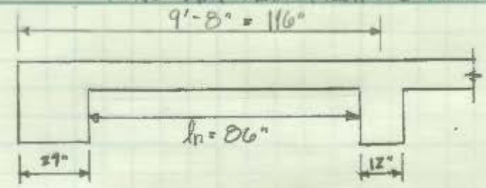
* ASSUME TENSION-CONTROLLED SECTION → $\phi = 0.9$

* SINCE $W_L < 3W_D$ → CAN USE ACI MOMENT COEFFICIENTS

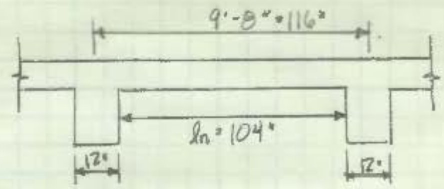
* ASSUME A BEAM WIDTH OF 12"

NATHAN McGRAW | ONE-WAY SLAB (TECH 2) | PAGE 2 OF 11

EXTERIOR BAY:



INTERIOR BAY



2 l_n CALCULATIONS

$$l_{n(AVG)} = \frac{(86+104)}{2 \times 12} = 7.92'$$

FIRST INTERIOR SUPPORT:

$$M_u = -\frac{w_u l_n^2}{10} = -\frac{(227)(7.92')^2}{10}(1' \text{ width}) = 1424 \text{ lb-ft/ft width} = 1.424 \text{ k-ft/ft}$$

SECOND INTERIOR SUPPORT:

$$M_u = -\frac{w_u l_n^2}{11} = -\frac{(227)(104/12)^2}{11}(1') = 1550 \text{ lb-ft/ft width} = 1.55 \text{ k-ft/ft}$$

MAXIMUM NEGATIVE DESIGN MOMENT = $M_u = 1.55$ k-ft/ft

REINFORCEMENT: *ASSUME $jd = 0.95d$

$$A_s \geq \frac{M_u}{\phi f_y (d - a/2)} \cong \frac{M_u}{\phi f_y (jd)} = \frac{1.55 \text{ k-ft/ft} \times 12"}{(0.9)(60)(0.95 \times 4")} = 0.0906 \text{ in}^2/\text{ft}$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.0906)(60)}{0.85(4)(12")} = 0.133" \rightarrow \phi = 0.9$$

$$A_s \geq \frac{1.55 \text{ k-ft/ft} \times 12"}{(0.9)(60)(4 - 0.133/2)} = 0.0876 \text{ in}^2/\text{ft}$$

$$\rho = \frac{A_s/\text{ft}}{bd} = \frac{0.0876}{12" \times 4"} = 0.00182$$

SHEAR CHECK:

SHEAR AT EXTERIOR FACE OF THE FIRST INTERIOR SUPPORT

$$V_u = \frac{1.15 w_u \ell_n}{2} = \frac{1.15(227)(89/12)}{2} = 935 \text{ lb/ft width of slab}$$

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

$$\phi V_c = 0.75(2)(1.0)\sqrt{4000}(12")(4") = 4554 \text{ lb/ft-width of slab} \gg V_u$$

$\therefore$ OK

DESIGN OF REINFORCEMENT:

	EXTERNAL SUPPORT	EXTERIOR MIDSPAN	1ST INTERIOR SUPPORT	INTERIOR MIDSPAN	SECOND INTERIOR SUPPORT
1. ℓ_n (ft)	7.17'	7.17'	7.92'	8.67'	8.67'
2. $W_u \ell_n^2$ (K-ft)	11.7	11.7	14.2	17.1	17.1
3. M COEFFICIENT	-1/24	1/14	-1/10	1/16	-1/11
4. M_u (K-ft/ft)	0.4875	0.8357	1.42	1.07	1.555
5. A_s req'd (in²/ft)	0.03	0.052	0.0876	0.066	0.096
6. A_s min (in²/ft)	0.108	0.108	0.108	0.108	0.108
7. BARS	No. 4 @ 12"	→	→	→	→
8. FINAL A_s (in²/ft)	0.20	0.20	0.20	0.20	0.20

SPACING:

$$S = 15(40000/f_s) - 2.5C_c \le 12(40000/f_s) \qquad f_s = 2/3 f_y = 40000$$

$$S = 15(40000/40000) - 2.5(0.75) \le 12(40000/40000)$$

$$S = 13.125 \le 12$$

$S = 13.125" > 12"$ SPACING USED $\therefore$ OK

TRANSVERSE DIRECTION:

A_s (Shrinkage + Temperature) $= 0.0018(12")(5") = 0.108"$ → USE #4 BARS
→ MOST COMMON

MAXIMUM SPACING $\le 5h = 5(5) = 25"$
≤ 18 → CONTROLS

> * SLAB DETAILS:
> 5" SLAB
> No. 4 BARS @ 12" FOR TOP AND BOTTOM FLEXURAL STEEL
> No. 4 BARS @ 18" FOR TRANSVERSE REINFORCEMENT

NATHAN MCGRAW | ONE-WAY SLAB (TECH 2) | PAGE 4 OF 11

BEAM DESIGN: (TYPICAL INTERIOR BEAM)

* START WITH $b = 12"$ (ASSUMED IN SLAB DETERMINATION)

$h \approx l/12$ TO $l/18$ (GOOD APPROXIMATION)

$h = \frac{l}{12} = \frac{29' \times 12}{12} = 29"$

$h = \frac{l}{18} = \frac{29' \times 12}{18} = 19.33"$

* USE $h = 24"$ (BETWEEN TWO VALUES)

$w_{BEAM} = \frac{(24" - 5")(12")}{144}(150 \text{ PCF}) = 237.5 = 0.2375 \text{ K/ft}$

$w_{SLAB+SI} = (62.5 \text{ psf} + 20 \text{ psf})(9.667') = 0.798 \text{ K/ft}$

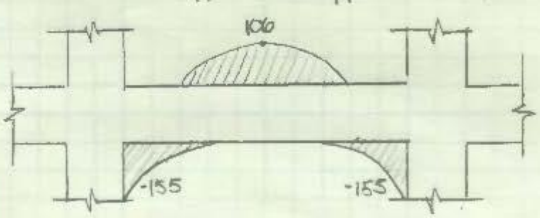
$LL_r = .25 + \frac{15}{\sqrt{(29')(9.667')(2)}} = 0.883$ > 400 ∴ OK

$w_L = (0.883)(80 \text{ PSF})(9.667') = 0.683$

$w_u = 1.2(0.2375 + 0.798) + 1.6(0.683) = 2.34 \text{ K/ft}$

$M_u^+ = \frac{wl^2}{16} = \frac{(2.34)(27')^2}{16} = 106 \text{ 'K}$

$M_u^- = -\frac{wl^2}{11} = -\frac{(2.34)(27')^2}{11} = -155 \text{ 'K}$



AT MIDSPAN: $M_u = 106 \text{ 'K}$

$A_s = \frac{M_u}{4d} = \frac{106}{4(21")} = 1.26 \text{ in}^2$

TRY (2) #8
$A_s = (2)(0.79) = 1.58 \text{ in}^2$

$\rho \leq 0.0125$ → $\rho = 1.58/(12 \times 21) = 0.00627$ ∴ OK
$f'_c = 4000$ PSI ∴ OK

NATHAN McGRAW | ONE-WAY SLAB (TECH 2) | PAGE 5 OF 11

$M_n = A_s f_y (d - a/2)$

$d = 24" - 1.5" - 0.5" - (1"/2) = 21.5"$

#4 STIRRUP

$$b_{eff} = \min \begin{cases} \text{1/4 SPAN LENGTH} = 1/4(29')(12) = 87" \Rightarrow \text{CONTROLS} \\ b_w + 16h_f = 12 + 16(5) = 92" \\ b_w + (2)(\text{1/2 CLR DISTANCE}) = 12 + (2)(1/2(9.667' - 1'))\times 12 = 116" \end{cases}$$

$$M_{u,T\text{-}BM} = \phi 0.85 f'_c b_{eff} h_f (d - h_f/2)$$
$$= [0.9(0.85)(4)(87)(5)(21.5 - 5/2)]/12$$
$$= 2107.6\ 'K > M_u \Rightarrow \text{TREAT AS RECTANGULAR BEAM}$$

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

$$c = \frac{a}{\beta_1} = \frac{2.32}{0.85} = 2.73$$

$$\varepsilon_s = \frac{(d-c)\varepsilon_u}{c} = \frac{(21.5 - 2.73)(0.003)}{2.73} = 0.021 > 0.005 \therefore \phi = 0.9$$

$$\phi M_n = [(0.9)(1.58)(60)(21.5 - 2.32/2)]/12$$

$$\phi M_n = 144.6\ 'K > 106\ 'K = M_u \therefore \underline{OK}$$

$$A_{s,min} = \max \begin{cases} \dfrac{3\sqrt{f'_c}}{f_y} bd = \dfrac{3\sqrt{4000}(12)(21.5)}{60000} = 0.816\ in^2 \\ \dfrac{200bd}{f_y} = \dfrac{200(12)(21.5)}{60000} = 0.86\ in^2 < A_s \text{ provided} \therefore \underline{OK} \end{cases}$$

SPACING MEETS ACI 318-08 REQUIREMENTS

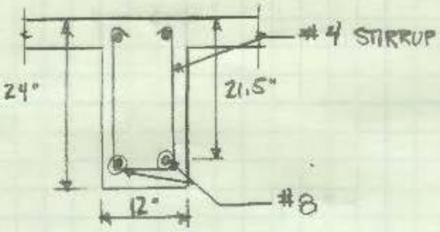
VERTICAL SHEAR: (BEAM IS REINFORCED W/ #4 STIRRUPS @ 12" O.C.)

$$V_u = \frac{w_u l_n}{2} = \frac{2.34(29' - 2')}{2} = 31.59^K$$

$$\phi V_n = \phi(V_c + V_s) = 0.75[2\sqrt{4000}(12")(21.5) + (2)(0.2)(60000)(21.5/12)]$$

$$\phi V_n = 56.7^K > 31.59^K = V_u \therefore \underline{OK}$$

BEAM DESIGN:
(MIDSPAN)



AT SUPPORTS: $M_u = -155'^K$

$$A_s = \frac{M_u}{4d} = \frac{155}{4(21")} = 1.85 \text{ in}^2$$

TRY (2)#9

$$A_s = (2)(1.0) = 2.0 \text{ in}^2$$

$$\rho = \frac{2}{12 \times 21} = 0.008 < 0.0125 \therefore \underline{OK}$$

* TREAT AS RECTANGULAR BEAM

$$a = \frac{(2.0)(60)}{0.85(4)(12)} = 2.94"$$

$$c = \frac{2.94}{0.85} = 3.46$$

$$d = 24" - 1.5" - 0.5" - (1.128/2) = 21.436"$$

$$\varepsilon_s = \frac{(21.436 - 3.46)(0.003)}{3.46} = 0.0156 > 0.005 \quad \therefore \phi = 0.9$$

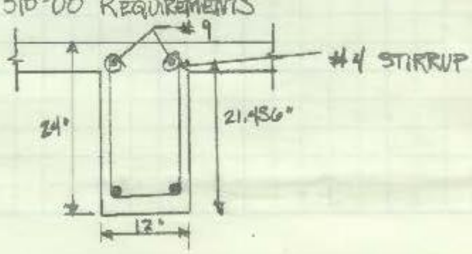
$$\phi M_n = [(0.9)(2.0)(60)(21.436 - 2.94/2)]/12$$

$$\phi M_n = 179.7'^K > 155'^K = M_u \quad \therefore \underline{OK}$$

$$A_{s\,min} = 0.86 \text{ in}^2 < A_s \text{ provided} \quad \therefore \underline{OK}$$

SHEAR ⇒ OK BY INSPECTION (SAME VALUES AS BEFORE)
SPACING MEETS ACI 318-08 REQUIREMENTS

BEAM DESIGN:
(AT SUPPORTS)



NATHAN McGRAW | ONE-WAY SLAB (TECH 2) | PAGE 7 OF 11

DEFLECTION: (ASSUME SIMPLY SUPPORTED)

$$\Delta_{LL} = \frac{5(683)(29')^4(1728)}{384(57000\sqrt{4000})(\frac{1}{12}(12)(24)^3)} = 0.218''$$

$$\Delta_{LL\,MAX} = \frac{29' \times 12}{480} = 0.725'' > 0.218'' \quad \therefore \underline{OK}$$

$$\Delta_{TL} = \frac{5(1036+683)(29')^4(1728)}{384(57000\sqrt{4000})(\frac{1}{12}(12)(24)^3)} = 0.549''$$

$$\Delta_{TL\,MAX} = \frac{29' \times 12}{240} = 1.45'' > 0.549'' \quad \therefore \underline{OK}$$

LONG TERM DEFLECTION:

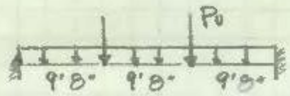
$$w_{TL} = 3(0.25\,w_L + w_D) = 3(0.25(683) + 1036) = 3620.25$$

$$\Delta_{TL} = \frac{5(3620.25)(29')^4(1728)}{384(57000\sqrt{4000})(\frac{1}{12}(12)(24)^3)} = 1.15'' < \Delta_{TL\,MAX} \quad \therefore \underline{OK}$$

NATHAN MCGRAW | ONE-WAY SLAB (TECH 2) | PAGE 8 OF 11

GIRDER DESIGN:

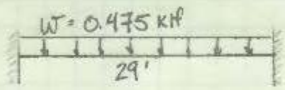
b = 24" (TO MATCH COLUMN DIMENSIONS)
TRY h = 24" (TO MATCH BEAMS)



$LL_r = .25 + \dfrac{15}{\sqrt{(2)(29')(29')}} = 0.657$

$= 1682 > 400$ # OK

① =



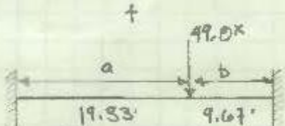
$W_U = 1.2(82.5\text{ psf}) + 1.6(0.657)(80)$
$W_U = 0.178$ klf

$P_U = 0.178$ klf $(9.667')(29') = 49.8^K$

$W_{self\text{-}weight} = \dfrac{(24-5)(24")}{144}(150\text{ pcf}) = 475$ #/ft

+

②



* NEED TO DO ② TWICE

FOR:

① $M^- = \dfrac{wl^2}{12} = \dfrac{(0.475)(29')^2}{12} = 33.3'^K$

$M_x^+ = \dfrac{w}{12}(6lx - l^2 - 6x^2) = \dfrac{(0.475)}{12}\left[6(29')(19.33') - (29')^2 - (6)(19.33')^2\right]$

$= 11.1'^K$

$V = \dfrac{wl}{2} = \dfrac{(0.475)(29')}{2} = 6.89^K$

② $M^- = \dfrac{Pa^2b}{l^2} = \dfrac{(49.8)(19.33)^2(9.67)}{(29')^2} = 214'^K$ (RIGHT SIDE)

$M^+ = \dfrac{2Pa^2b^2}{l^3} = \dfrac{2(49.8^K)(19.33')^2(9.67')^2}{(29')^3} = 142.69'^K$ (@ POINT LOAD)

$V = \dfrac{Pa^2}{l^3}(a+3b) = \dfrac{(49.8)(19.33')^2}{(29')^3}(19.33 + 3(9.67')) = 36.9^K$ (RIGHT SIDE)

② $a = 9.67'$
$b = 19.33'$

$M^- = \dfrac{Pa^2b}{l^2} = \dfrac{(49.8)(9.67')^2(19.33)}{(29')^2} = 107.0'^K$

$M_x^+ = R_1x - \dfrac{Pab^2}{l^2} = (9.5)(9.67') - \dfrac{(49.8)(19.33)(9.67')^2}{(29')^2} = -15.17'^K$

$V = \dfrac{Pa^2}{l^3}(a+3b) = \dfrac{(49.8)(9.67)^2}{(29)^3}(9.67 + (3)(19.33)) = 9.5^K$

NATHAN MCGRAW | ONE-WAY SLAB (TECH 2) | PAGE 9 OF 11

$V_{MAX} = 9.5^K + 36.9^K + 6.89^K = \underline{53.29^K}$

$M^+_{MAX} = 11.1'^K + 142.69'^K - 15.17'^K = 138.62'^K$

$M^-_{MAX} = 33.3'^K + 214'^K + 107'^K = 354.3'^K$

AT MIDSPAN: $M_U = 138.62'^K$

$A_s = \frac{M_U}{4d} = \frac{138.62'^K}{4(21")} = 1.65 \text{ in}^2$

$d = 24 - 1.5" - 0.5" - 0.875/2" = 21.5625$

TRY (3)#7

$A_s = (3)(0.6) = 1.8 \text{ in}^2$

$\rho = \frac{1.8}{(24)(21)} = 0.00357 < 0.0125 \quad \therefore \underline{OK}$

$b_{eff} = \min \begin{cases} 1/4(29')(12) = 87" \rightarrow \text{CONTROLS} \\ 24 + 16(5) = 104" \\ 24 + 2(1/2(29' - 2'))x12 = 348" \end{cases}$

$M_{U,T\text{-}BM} = [0.9(0.85)(4)(87)(5)(21.5625 - 5/2)]/12$
$= 2114.5'^K > M_U \rightarrow$ TREAT AS RECTANGULAR BEAM

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(1.65)(60)}{0.85(4)(24)} = 1.213"$

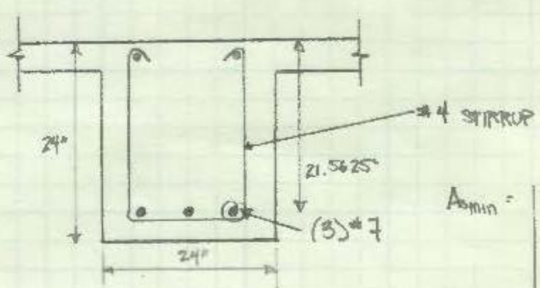
$c = \frac{a}{\beta_1} = \frac{1.213}{0.85} = 1.43"$

$\varepsilon_s = \frac{(21.5625 - 1.43)(0.003)}{1.43} = 0.04 > 0.005 \quad \therefore \phi = 0.9$

$\phi M_n = [(0.9)(1.65)(60)(21.5625 - 1.213/2)]/12$

$\phi M_n = 155.6'^K > 138.62'^K = M_U \quad \therefore \underline{OK}$

GIRDER DESIGN:
(AT MIDSPAN)



$A_{s,min} = \max \begin{cases} \frac{3\sqrt{4000}(24)(21.5625)}{60000} \\ \frac{200(24)(21.5625)}{60000} \end{cases}$

$A_{s,min} = 1.725 < A_s$ provided $\therefore \underline{OK}$

NATHAN M^cGRAW | ONE-WAY SLAB (TECH 2) | PAGE 10 OF 11

AT SUPPORT: $M_u = 354.3'^{K}$

$$A_s = \frac{M_u}{4d} = \frac{354.3}{(4)(21)} = 4.22 \text{ in}^2$$

TRY (6)#8

$$A_s = (6)(0.79) = 4.74 \text{ in}^2$$

$$\rho = \frac{4.74}{24(21)} = 0.009 < 0.0125 \quad \therefore \underline{OK}$$

$$d = 24" - 1.5" - 0.5" - 1^{1}/_{2} = 21.5"$$

*TREAT AS RECTANGULAR BEAM

$$a = \frac{(4.74)(60)}{0.85(4)(24)} = 3.49"$$

$$c = \frac{3.94}{0.85} = 4.64"$$

$$\varepsilon_s = \frac{(21.5 - 4.64)(0.003)}{4.64} = 0.011 > 0.005 \quad \therefore \phi = 0.9$$

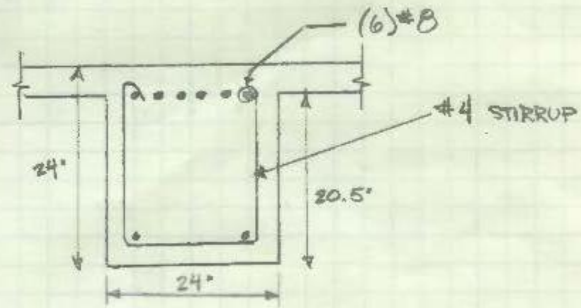
$$\phi M_n = [(0.9)(4.74)(60)(21.5 - 3.49/2)]/12$$

$$\phi M_n = 421'^{K} > 354.3'^{K} = M_u \quad \therefore \underline{OK}$$

$$A_{s\,min} = \max \left| \begin{array}{l} \frac{3\sqrt{4000}(24)(21.5)}{60000} = 1.63 \text{ in}^2 \\ \frac{200(24)(21.5)}{60000} = 1.72 \rightarrow \text{CONTROLS} < A_{s\,provided} \quad \therefore \underline{OK} \end{array} \right.$$

SPACING MEETS ACI318-08 REQUIREMENTS

GIRDER DESIGN:
(AT SUPPORTS)



NATHAN MCGRAW | ONE-WAY SLAB (TECH 2) | PAGE 11 OF 11

VERTICAL SHEAR: (GIRDER IS REINFORCED WITH #4 STIRRUPS @ 12" O.C)

$V_u = 53.29^K$

$\phi V_n = 0.75\left[2\sqrt{4000}(24")(20.5") + (2)(0.2)(60000)(^{20.5}/_{12})\right]$

$\phi V_n = 77.4^K > 53.29^K = V_u \;\therefore \underline{OK}$

DEFLECTION: ASSUME SIMPLY SUPPORTED

$\Delta_{LL} = \dfrac{5wl^4}{384EI} + \dfrac{Pa}{24EI}(3l^2 - 4a^2)$

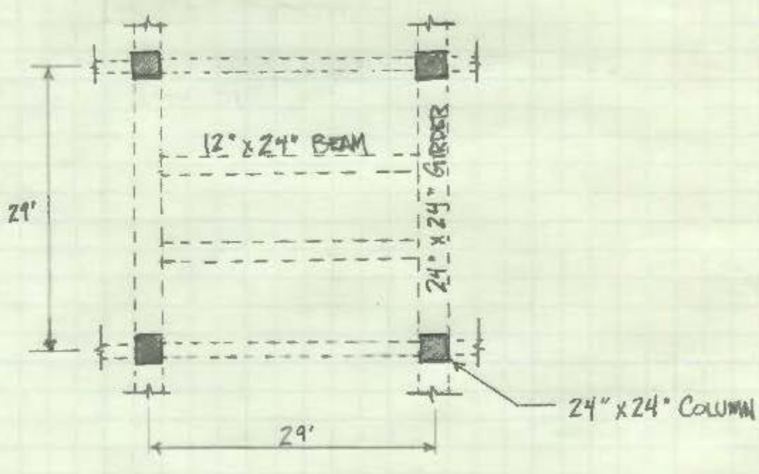
$\Delta_{LL} = \dfrac{(13000^{\#})(9.67')}{24(57000\sqrt{4000})(^1/_{12}(24)(24)^3)}(3(29')^2 - 4(9.67')^2)(1728) = 0.207"$

$\Delta_{LL\,MAX} = 0.725" > 0.207" \;\therefore \underline{OK}$

$\Delta_{TL} = \dfrac{5(475)(29')^4(1728)}{384(57000\sqrt{4000})(^1/_{12}(24)(24)^3)} + \dfrac{(36937)(9.67')(3(29')^2 - 4(9.67')^2)(1728)}{24(57000\sqrt{4000})(^1/_{12}(24)(24)^3)}$

$\Delta_{TL} = 0.0758" + 0.554" = 0.630"$

$\Delta_{TL\,MAX} = 1.45" > 0.630" \;\therefore \underline{OK}$



3 TYPICAL ONE-WAY INTERIOR PANEL

Appendix E: Floor System Cost Breakdowns

Two-Way Flat Slab With Drop Panels					
Flat slab, concrete, with drop panels, 10.5" slab/7.5" panel, 14" column, 30'x30' bay, 40 PSF superimposed load, 182 PSF total load					
Description	**Quantity**	**Unit**	**Material**	**Installation**	**Total**
C.I.P. concrete forms, beams and girders, exterior spandrel, plywood, 12" wide, 4 use, includes shoring, erecting, bracing, stripping and cleaning	0.035	SFCA	0.03	0.35	0.38
C.I.P. concrete forms, elevated slab, flat slab with drop panels, to 15' high, 4 use, includes shoring, erecting, bracing, stripping and cleaning	0.998	S.F.	1.28	5.69	6.97
Reinforcing Steel, in place, elevated slabs, #4 to #7, A615, grade 60, incl labor for accessories, excl material for accessories	3.194	Lb.	1.63	1.37	3
Structural concrete, ready mix, normal weight, 4000 psi, includes local aggregate, sand, Portland cement and water, delivered, excludes all additives and treatments	0.944	C.F.	3.8	0	3.8
Structural concrete, placing, elevated slab, pumped, 6" to 10" thick, includes strike off & consolidation, excludes material	0.944	C.F.	0	1.2	1.2
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 and 4, to achieve a Composite Overall Floor Flatness & Levelness value up to F35/F25, bull float, machine float & steel trowel (walk-behind), excludes placing, striking	1	S.F.	0	0.82	0.82
Concrete surface treatment, curing, sprayed membrane compound	0.01	C.S.F.	0.06	0.09	0.15
Total			**$6.80**	**$9.52**	**$16.32**

Post-Tensioned Flat Slab Concrete					
Flat slab, concrete, 9.5" slab, 20" column, 25'x25' bay, 75 PSF superimposed load, 194 PSF total load					
Description	**Quantity**	**Unit**	**Material**	**Installation**	**Total**
C.I.P. concrete forms, elevated slab, flat plate, plywood, to 15' high, 4 use, includes shoring, erecting, bracing, stripping and cleaning	0.986	S.F.	1.11	5.42	6.54
C.I.P. concrete forms, elevated slab, edge forms, alternate pricing, to 6" high, 1 use, includes shoring, erecting, bracing, stripping and cleaning	0.031	SFCA	0.02	0.19	0.21
Reinforcing Steel, in place, elevated slabs, #4 to #7, A615, grade 60, incl labor for accessories, excl material for accessories	3.028	Lb.	1.54	1.3	2.85
Structural concrete, ready mix, normal weight, 4000 psi, includes local aggregate, sand, Portland cement and water, delivered, excludes all additives and treatments	0.791	C.F.	3.19	0	3.19
Structural concrete, placing, elevated slab, pumped, 6" to 10" thick, includes strike off & consolidation, excludes material	0.791	C.F.	0	1	1
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 and 4, to achieve a Composite Overall Floor Flatness & Levelness value up to F35/F25, bull float, machine float & steel trowel (walk-behind), excludes placing, striking	1	S.F.	0	0.82	0.82
Concrete surface treatment, curing, sprayed membrane compound	0.01	C.S.F.	0.06	0.09	0.15
Pre-Stressing Tendons	0.87	Lb.	2	1	3
Total			**$6.90**	**$9.92**	**$16.82**

Composite Steel					
Floor, composite metal deck, shear connectors, 5.5" slab, 30'x30' bay, 29.5" total depth, 125 PSF superimposed load, 168 PSF total load					
Description	**Quantity**	**Unit**	**Material**	**Installation**	**Total**
Welded wire fabric, sheets, 6 x 6 - W1.4 x W1.4 (10 x 10) 121 lb. per C.S.F., A185	0.01	C.S.F.	0.14	0.36	0.49
Structural concrete, placing, elevated slab, pumped, less than 6" thick, includes strike off & consolidation, excludes material	0.333	C.F.	0	0.5	0.5
Structural concrete, ready mix, normal weight, 140 #/C.F., 4000 psi, includes local aggregate, sand, portland cement and water, excludes all additives and treatments	0.333	C.F.	2.41	0	2.41
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 and 4, to achieve a Composite Overall Floor Flatness & Levelness value up to F35/F25, bull float, machine float & steel trowel (walk-behind), excludes placing, striking	1	S.F.	0	0.82	0.82
Concrete surface treatment, curing, sprayed membrane compound	0.01	C.S.F.	0.06	0.09	0.15
Weld shear connector, 3/4" dia x 4-7/8" L	0.163	Ea.	0.12	0.31	0.43
Structural steel project, apartment, nursing home, etc, 100-ton project, 3 to 6 stories, A992 steel, shop fabricated, incl shop primer, bolted connections	6.806	Lb.	8.58	2.86	11.43
Metal floor decking, steel, non-cellular, composite, galvanized, 3" D, 20 gauge	1.05	S.F.	1.89	1.01	2.9
Metal decking, steel edge closure form, galvanized, with 2 bends, 12" wide, 18 gauge	0.033	L.F.	0.11	0.08	0.18
Sprayed cementitious fireproofing, sprayed mineral fiber or cementitious for fireproofing, beams, 2 hour rated, 1-3/8" thick, excl. tamping or canvas protection	0.667	S.F.	0.39	0.64	1.03
Total			**$13.70**	**$6.67**	**$20.37**

One-Way Slab with Beams					
Cast-in-place concrete beam and slab, 7.5" slab, one way, 18" column, 30'x30' bay, 75 PSF superimposed load, 191 PSF total load					
Description	**Quantity**	**Unit**	**Material**	**Installation**	**Total**
C.I.P. concrete forms, beams and girders, exterior spandrel, plywood, 12" wide, 4 use, includes shoring, erecting, bracing, stripping and cleaning	0.122	SFCA	0.11	1.21	1.32
C.I.P. concrete forms, beams and girders, interior, plywood, 12" wide, 4 use, includes shoring, erecting, bracing, stripping and cleaning	0.303	SFCA	0.33	2.48	2.81
C.I.P. concrete forms, elevated slab, flat plate, plywood, to 15' high, 4 use, includes shoring, erecting, bracing, stripping and cleaning	0.866	S.F.	0.98	4.76	5.74
Reinforcing Steel, in place, elevated slabs, #4 to #7, A615, grade 60, incl labor for accessories, excl material for accessories	3.804	Lb.	1.94	1.64	3.58
Structural concrete, ready mix, normal weight, 4000 psi, includes local aggregate, sand, Portland cement and water, delivered, excludes all additives and treatments	0.772	C.F.	3.11	0	3.11
Structural concrete, placing, elevated slab, pumped, 6" to 10" thick, includes strike off & consolidation, excludes material	0.772	C.F.	0	0.98	0.98
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 and 4, to achieve a Composite Overall Floor Flatness & Levelness value up to F35/F25, bull float, machine float & steel trowel (walk-behind), excludes placing, striking	1	S.F.	0	0.82	0.82
Concrete surface treatment, curing, sprayed membrane compound	0.01	C.S.F.	0.06	0.09	0.15
Total			**\$6.55**	**\$11.98**	**\$18.53**

Appendix F: Typical Plans

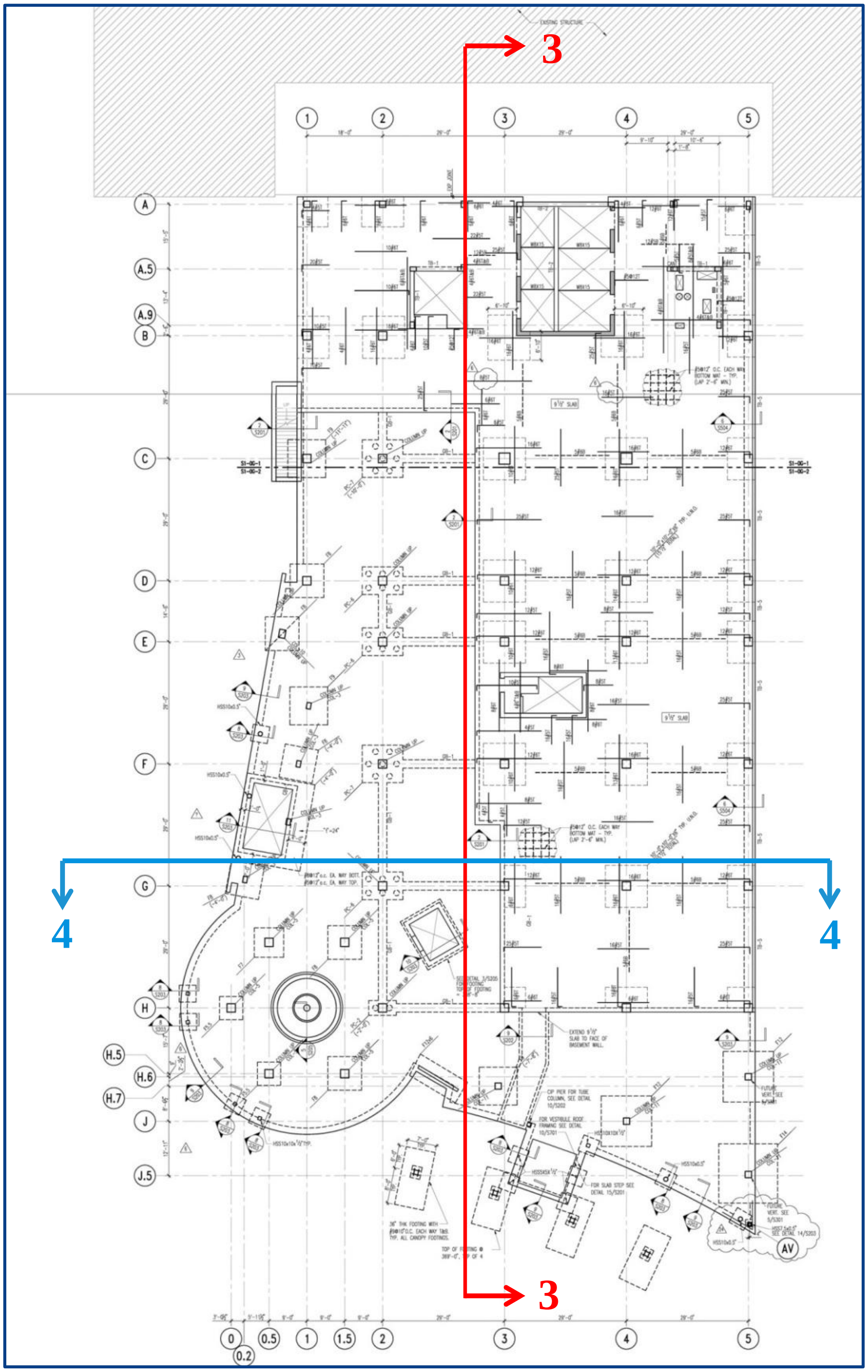


Figure 1:
Ground floor plan (See following figures for sections indicated on the plan)

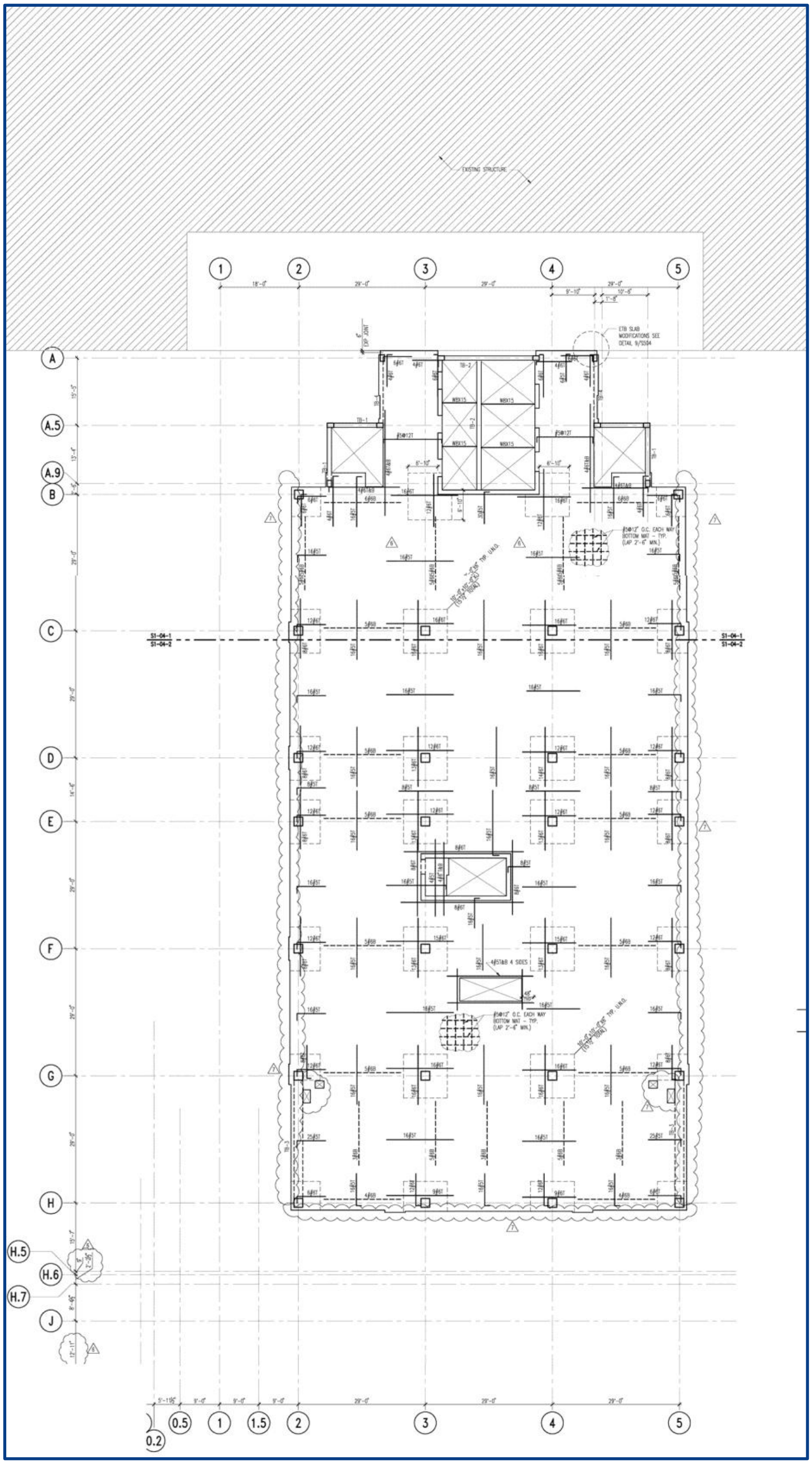


Figure 2:
Typical floor plan (6th – 11th)

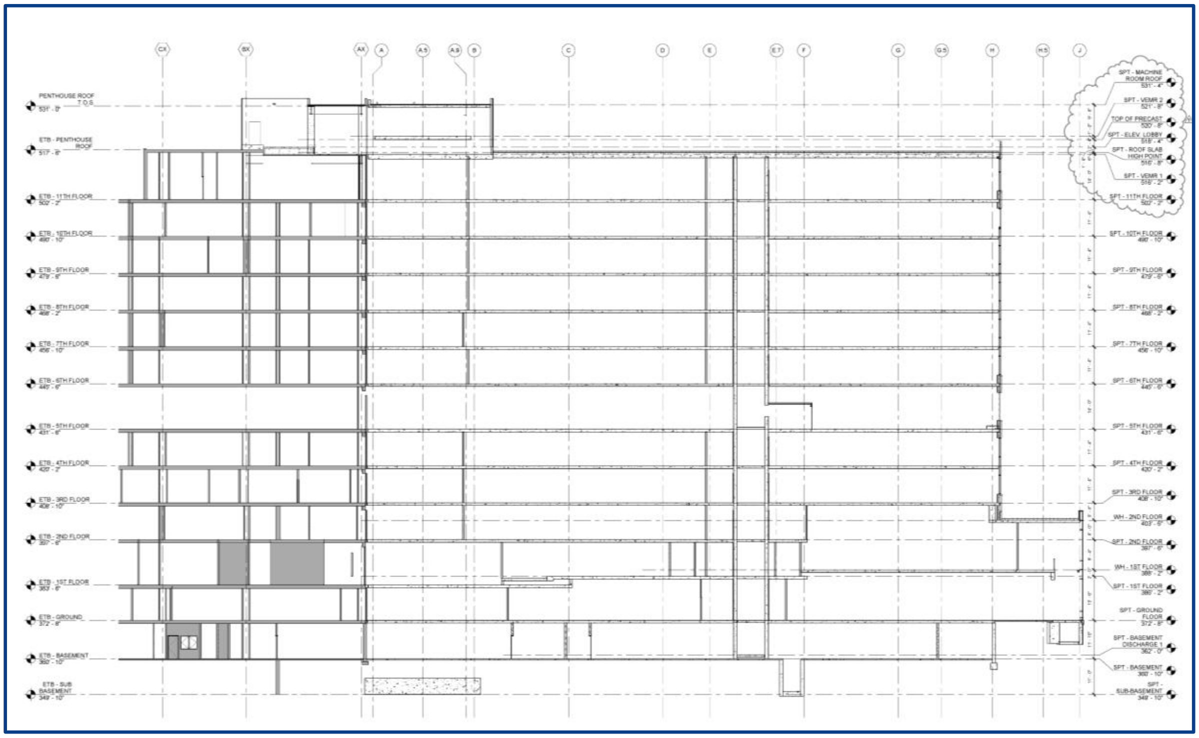


Figure 3:
North – South section cut

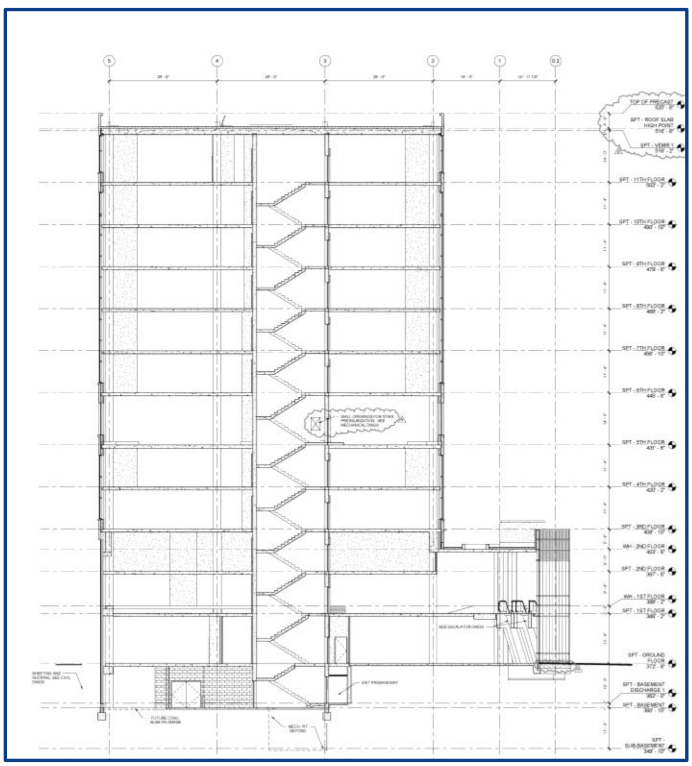


Figure 4:
East – West section cut