Technical Assignment 2

Piez Hall Extension

Oswego, NY

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Structural Option
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October 12, 2012

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Executive Summary

This technical report will discuss and compare three alternative floor systems to the currently existing floor system in the Piez hall addition. The existing floor system in Piez Hall is a two-way flat slab with drop panels, it is compared to a composite steel deck, a pre-cast hallow core planks on concrete girder, and a one-way post-tensioned slab. The system's cost, weight, and depth are compared among the four types. Other criterions such as impacts on architecture, fire rating, vibration, lead time, constructability are also used to compare the systems. These factors will be used to determine the feasibility of each system.

The existing system of a two-way flat slab with drop panels was believed to be the most feasible system in terms of cost. However it was also the heaviest system. One has a system depth of 20" and a cost of $17 per square foot. I believed that the structural engineer chose this system because of the low cost and ease of construction.

The composite system is a doable alternative. Although it is expensive and has a deeper floor than the original system, its light weight and ease of construction makes it a doable option.

The pre-cast hallow core planks on concrete girders was discovered to be the most expensive and has the greatest depth out of the four systems. These two disadvantages together are enough to rule out the possibility to use the system as an alternative.

The post-tensioned slab was found to be the most favorable alternative. Although it cost more than the original system, it has the smallest depth out of the four systems as well as a lighter weight system compared to the existing systems. As a result, smaller columns and foundations can be used, which may lower the overall cost of the project to compensate for the addition cost per square foot. Hence it is a feasible option.

Building Introduction

The new Piez hall extension at Oswego University located in New York will provide high quality classrooms, teaching and research laboratories, as well as interaction spaces for all kinds of engineering departments. Inside the new facility, there will be a planetarium, meteorology observatory and a greenhouse.

Figure 2: Aerial map from Bing.com showing the location of the site

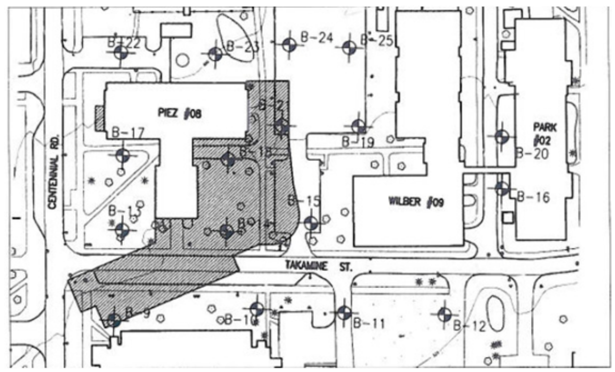


Figure 1: Site map showing existing Piez hall and the new extension (shaded area)

The Piez hall addition will add an expansion of approximately 155,000 square feet to the existing Piez hall. Snygg hall, which is next to the Piez hall, will be demolished as a result of the new addition. In the back of the U shaped Piez hall, there will be a walkway connecting Wilbur hall and the new addition. The construction of Piez hall extension began as early as April 2011. It is anticipated to be complete by April 2013 with an estimated cost of $110 million dollars. The building has 6 stories and it stands 64 feet high. The new 210,000 square feet concrete framed extension was designed by Cannon Design. The building is designed so that its exterior enclosure looks somewhat similar to the existing Piez hall (see Figure 3). The building is decorated with a skin of curtain wall. Brick is used in the south side facade. The second and third levels have spaces cantilevered slightly out to the west.

The Piez hall extension has numerous sustainability features to attain LEED Gold Certification. The building energy efficient curtain wall with a high R value will reduce heat loss. The mechanical system includes a large geothermal heat pump with a design capacity of 800 tons will be implanted to cool and heat the building. Occupied spaces have access to daylight. The roof has photovoltaic array, skylight and wind turbines. These features together will reduce the total energy use of the building to 47% and save 21% of the energy cost each year.

Figure 3: Exterior rendering showing the building enclosure

Structural Overview

Foundation

According to the soil report for Oswego County, the proposed site will be suitable for supporting the renovation and addition with a shallow spread foundation system. The maximum net allowable pressure on soil is 6,000psf for very dense till layers and 4,000 psf for medium dense clay and sand layers. All grade beams, foundation walls and piers will have a concrete strength of 4000psi while all other footings and slabs-on-grade will have a concrete strength of 3000psi.It is estimated that all foundations will undergo a total settlement less than 1 inch. Differential settlement is estimated to be less than 0.5 inch. Details of typical footings are given in Figure 4.

Basement non-yielding walls have granular backfill with drains at locations where surcharge effect from any adjacent live loads may cause problems. These non-yielding walls are designed to resist lateral soil pressure of 65pcf where foundation drains are placed above groundwater level. Any cantilever earth retaining walls are designed based on 45pcf active earth pressure. All retaining wall are designed for a factor of safety equal to or greater than 1.5 against sliding and overturning. The frictional resistance can be estimated by multiplying the normal force acting at the base of the footing by a coefficient of friction of 0.32.

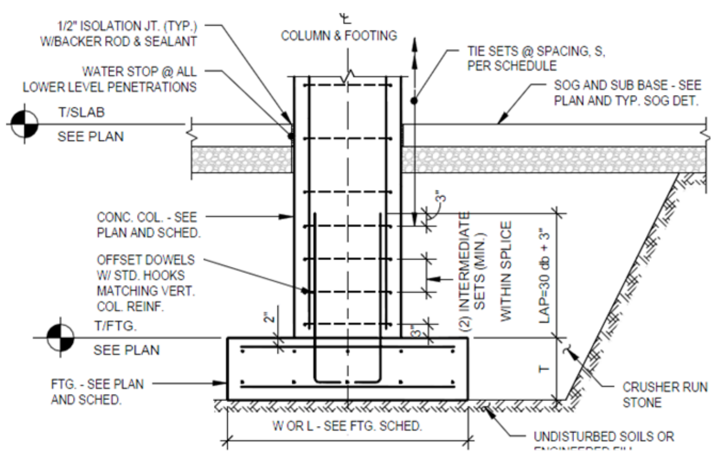


FIGURE 4: TYPICAL COLUMN FOOTING SHOWING REINFORCEMENT PLACEMENT

3. EVENLY SPACE FOOTING REINFORCEMENT. PROVIDE 3-INCHES CLEAR COVER UON.

Floor System

The typical floor structure of Piez Hall addition is a cast-in-place flat slab with drop panels. The slab thickness of the floors is 12" throughout the entire building with primarily #6 @ 9" o.c top and #6 @ 12" o.c bottom bars in 5000 psi strength concrete. 42"x24"concrete beams spans a length of 46.2' with 4 #8 @ top and 6 # 10 @ bottom reinforcement bars are placed in the edge of the floor slab primary located to support the cantilevered portion of the building in the second and third floor. Also, 24"x24" interior concrete beams are placed along the corridor of building to support areas where the slab is discontinuous such as stair and elevator shaft locations. A continuous 50"x10" edge beam each spans a length of 31.5' is placed on the north side of the south wing where the conservatory is connected to the building. The total depth of the floor system is 20". A typical framing plan of the south wing can be found in figure 10 and 11.

A drop panel is placed in almost every column location to increase the slab thickness in order to magnify the moment carrying capacity near the column support as well as resisting punching shear. Typical drop panels are 10.5'x10.5'x8" (see Figure 6)

In the conservatory the structural engineer employed composite steel floor system primary because lateral forces is not a concern due to the fact that the conservatory is embraced by the Piez hall building. Thus expensive moment connections are not necessary.

In addition, reinforcements for temperature change are #6 bars at 18" spacing, which is the maximum required spacing for temperature reinforcement. Typical steel reinforcement placement for the slab is given in figure 5.

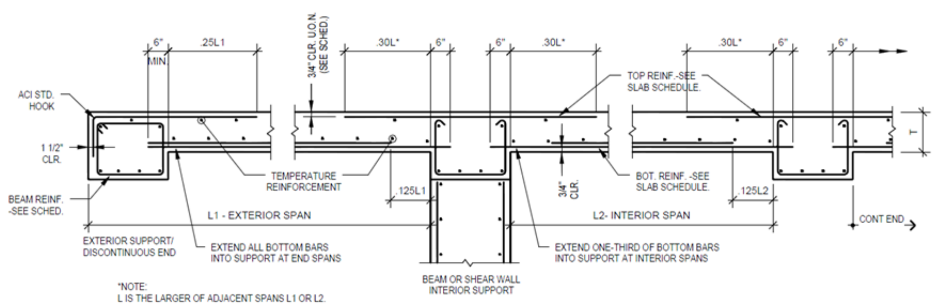


FIGURE 5: TYPICAL ONE WAY SLAB SHOWING REINFORCEMENT PLACEMENTS

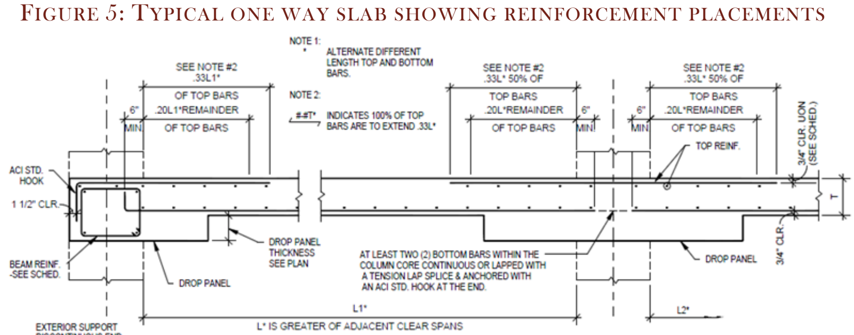


FIGURE 6: TYPICAL COLUMN STRIP DETAIL WITH DROP PANEL AND EDGE BEAM

Framing System

Typical bay in the new south wing of the building are 31.5'x31.5'. Corridor areas have a bay size of 10.3'x31.5'. The 10.3' span is less than two third of its adjacent span of 31.5'. Thus, this limitation suspends the use of direct design method. The equivalent frame method will be used to analyze the slab.

Typical columns are 24"x24" square concrete columns with eight #8 vertical reinforcing bars and #3 ties at 15" spacing. The upper east part of the new addition is supported by circular concrete columns with 30" diameter extending from the foundation to the top of second floor. Typical beams are 24"x24" doubly reinforced concrete beams with #6 top reinforcing bars and #8 bottom reinforcing bars. Because beams are framed into slabs, beams are treated as T-section beams. Typical reinforcement placements for beams are shown in Figure 7.

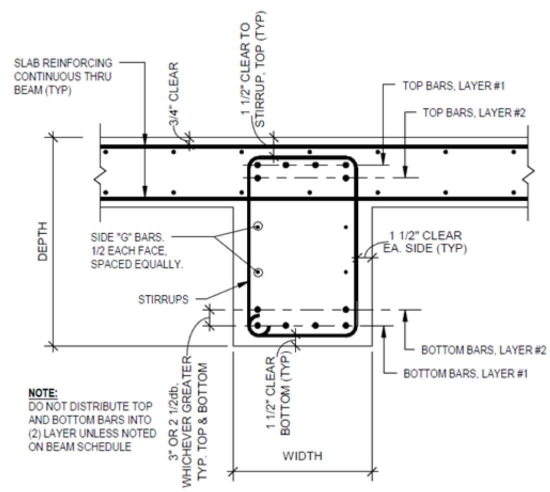


FIGURE 7: TYPICAL BEAM SECTION

The planetarium and conservatory in the middle of the "U" of building is built with structural steel framing. The floor system is a composite steel deck supported by W-shape beams. The sizes of the beams are typically W 14x22, W16x26, and W16x 31. Columns consist of various kinds of hollow structural steel and W10x33. Again, a typical framing plan of the south wing can be found in figure 10.

Lateral System

Shear walls and diagonal bracing are the main lateral force resisting system in the Piez hall new addition. They are evenly distributed and orientated throughout the building to best resist the maximum lateral loads coming from all direction. Typical shear walls are 12" thick and consist of 5000psi concrete. Shear walls extend from the first level to the top of the roof. Loads travel through the walls and are distributed down to the foundation directly. Diagonal bracing are concrete struts that framed into concrete beams. They are located on the second to fourth level and placed on the sides of the cantilevered portion of the building. Since the building is a concrete building, concrete intersection points also serve as moment frames. Together, these elements create a strong lateral force resisting system.

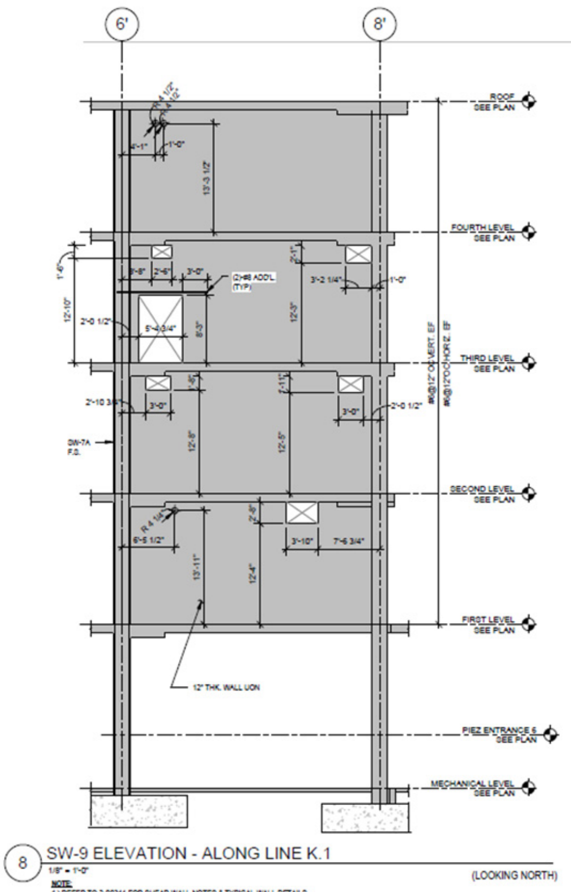


Figure 8: Typical concrete shear wall

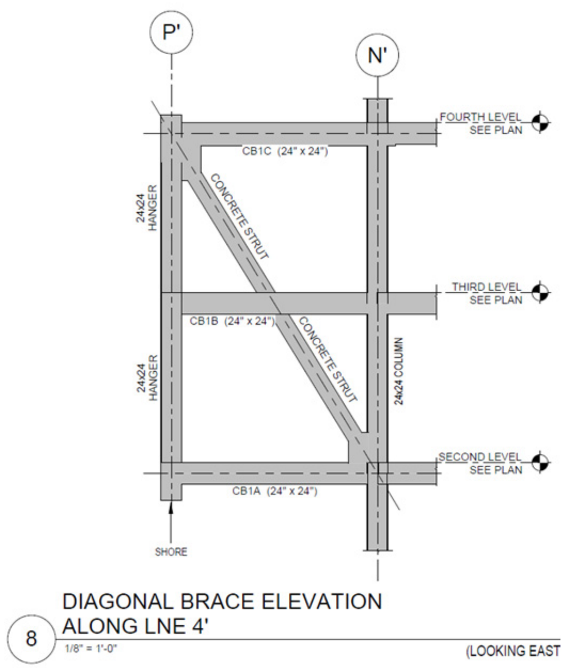


Figure 9: Typical concrete diagonal braces

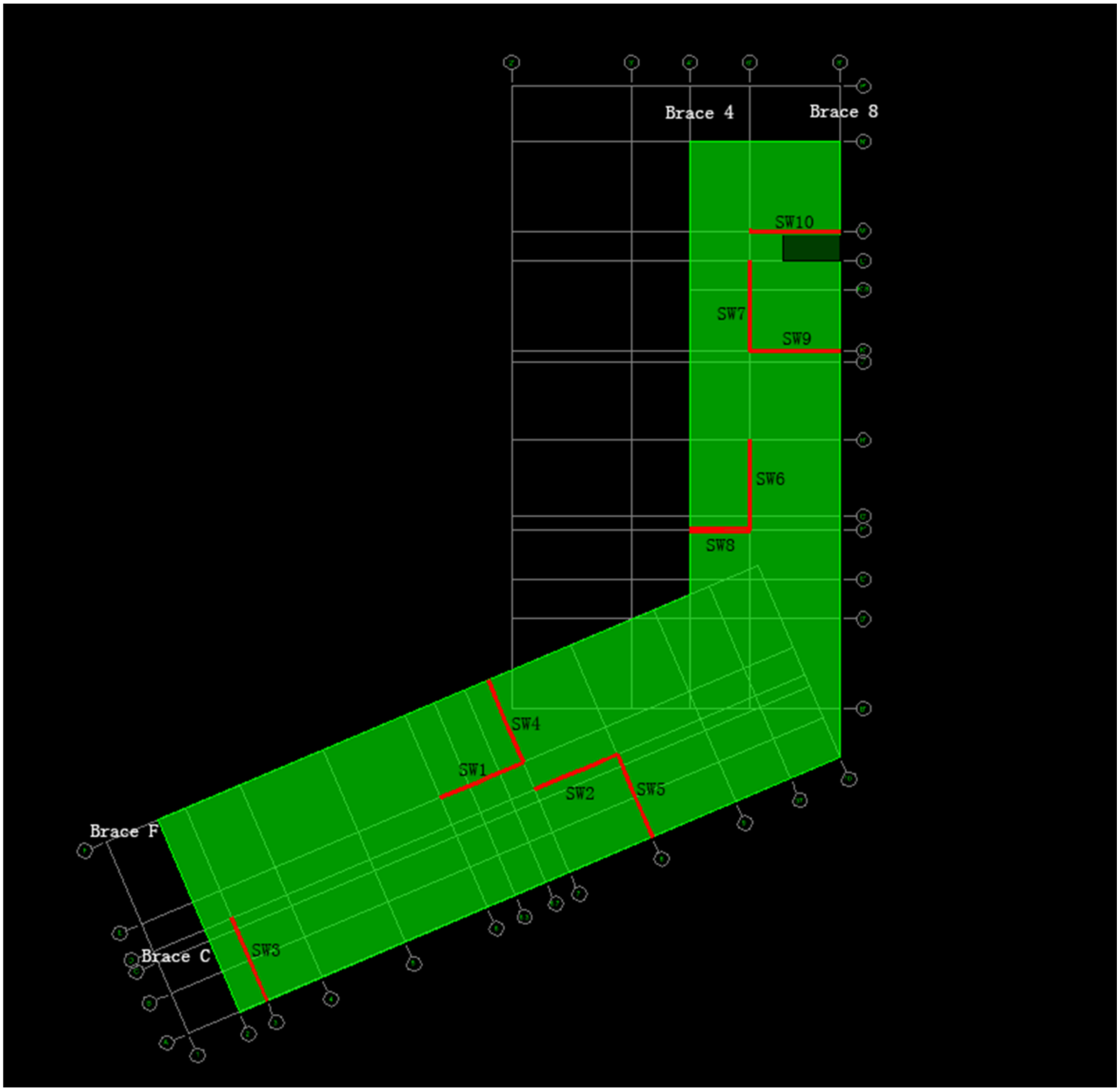


FIGURE 10: SHEAR WALL LOCATIONS OF A TYPICAL FLOOR

Roof System

There are three different kinds of roof system for the Piez hall extension. Steel decks and steel beams are used to support the roof for the planetarium. The roof for the cantilever part of the third level is designed to let people walk on top of them. Therefore, a fairly thick roof of 10" concrete is required. All other roof for the fourth level uses 6.5" thick concrete because they are not intended for excessive live load. On top of the roof, there are photovoltaic array, skylights, wind turbine and mechanical equipment that contribute to LEED.

Design Codes

- Building Code Requirements for Structural Concrete (ACI 318-05)
- Specifications for Masonry Structures (ACI 530.1)
- Building Code Requirements for Masonry Structures (ACI 530)
- Masonry Structure Building Code Commentary (ACI)
- AISC Specifications and Code (AISC)
- Structural Welding Code – Steel (AWS D1.1 2002)
- Structural Welding Code – Sheet Steel
- Building Code of New York State 2007
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-02)

Design Codes used for Thesis

- Minimum Design Loads for Buildings and Other Structures (ASCE 7-10)
- International Building Code (2009 Edition)
- Building Code Requirement for Reinforced Concrete (ACI 318-11)
- Steel Construction Manual (AISC 14th Edition)

Materials Used

Concrete		
Usage	**Strength (psi)**	**Weight (pcf)**
Footings	3000	Normal
Grade Beams	4000	Normal
Foundation Walls and Piers	4000	Normal
Columns and Shear Walls	5000	Normal
Framed Slabs and Beams	5000	Normal
Slabs-on-Grade	3000	Normal
Slabs-on-Steel-Deck	3000	Normal
All Other Concrete	4000	Normal

TABLE 1: SUMMARY OF MATERIAL USED WITH STRENGTH AND DESIGN STANDARD

Steel		
Type	**Standard**	**Grade**
Typical Bars	ASTM A-615	60
Welded Bars	ASTM A-706	60
Steel Fibers	ASTM A-820 Type 1	N/A
Wide Flange Shapes, WT's	ASTM A992	50
Channels and Angles	ASTM A36	N/A
Pipe	ASTM A53	B
Hollow Structural Sections (Rectangular & Round)	ASTM A500	B
High Strength Bolts, Nuts and Washers	ASTM A325 or ASTM A-490	N/A
Anchor Rods	ASTM F1554	36
Welding Electrode	AWS A5.1 or A5.5	E70XX
All Other Steel Members	ASTM A36 UON	N/A

TABLE 2: SUMMARY OF MATERIAL USED WITH STRENGTH AND DESIGN STANDARD

Gravity Loads

Dead, live and snow loads are computed and compared to the loads listed on the structural drawings. After determining the loads using ASCE 7-10, spot checks on members of the structural system were checked to verify their adequacy to carry gravity loads.

Dead and Live Loads

Although the Structural engineer has given a superimposed dead load of 15psf for all levels, but a more conservative and general superimposed dead load of 20psf were used in the calculation. Façade, column, shear wall and slab were all taken into account to obtain the overall dead load in each level. The exterior wall consists of curtain wall, CMU, precast concrete panels in different location. Thus to simplify the calculation, a uniform 30psf were taken as the load of the façade in all sides of the building. The overall weight of the building is found to be 29577 kips. This total weight is needed to compute the base shear for seismic calculation later on.

Weight Per Level		
Level	Weight (kips)	Weight (psf)
1	5293.10	197.67
2	6449.73	221.54
3	6246.66	222.84
4	6246.66	222.84
Roof	3265.58	121.95
Total Weight	29577.02	

Table 3: Distribution of weight per level and total weight

Live Loads shown in the middle column of Table 4 are given by the structural engineer. The structural engineer is rather conservative to use all design live load to be 100psf when an 80psf can typically be used for educational occupancy. Since this is a University building, typical floor is likely to be classrooms which have live load of 50psf as defined by ASCE 7-10. Similarly, public spaces can be interpreted as corridor above the first floor which has a live load of 80psf.

Live Load		
Space	**Design Live Load (psf)**	**ASCE 7-10 Live Load (psf)**
Typical Floors	100	50
Public Spaces	100	80
Exit Corridors	100	100
Stairs	100	100
Lobbies	100	100

TABLE 4: COMPARISON OF LIVE LOADS

Snow Loads

Following the procedure outlined in ASCE 7-10, the result of snow loads were obtained. The resulting snow loads were found to be 46psf. This is close to what the structural engineer had calculated.

Alternative Floor Systems

In this technical report, three alternative floor systems were compared to the existing floor system. Factors such as system weight, depth, cost, construability, impact on architecture, impact on lateral system, and impact on foundation between the systems will be compared among each other. The result of the comparisons can be found in table 5 at the end.

It is also worth mentioning that the cost estimate for the four floor systems were based on 2013 RS means assemblies cost data. A bay size of 31.5'x31.5' was used in the calculation, but the cost estimate was based on a slightly different bay size. This is done because RS Means does not provide an assemblies estimate for the bay size used in this analysis. Therefore, all four floor systems used a bay size of 30'x30' for cost estimate. Moreover, the particular assembly accounted for 3000psi concrete, but 5000psi concrete was used in the design. This should not be a problem because all other floor systems also accounted for 3000psi concrete in the assembly. Hence, an approximate difference in cost between each system can be used in evaluation.

The following floor systems were compared and discussed

- Existing two-way flat slab with drop panels
- Composite steel
- Pre-cast hallow core
- Post-tension concrete

Figure 13 shows a typical bay used to design and analyze the four floor systems.

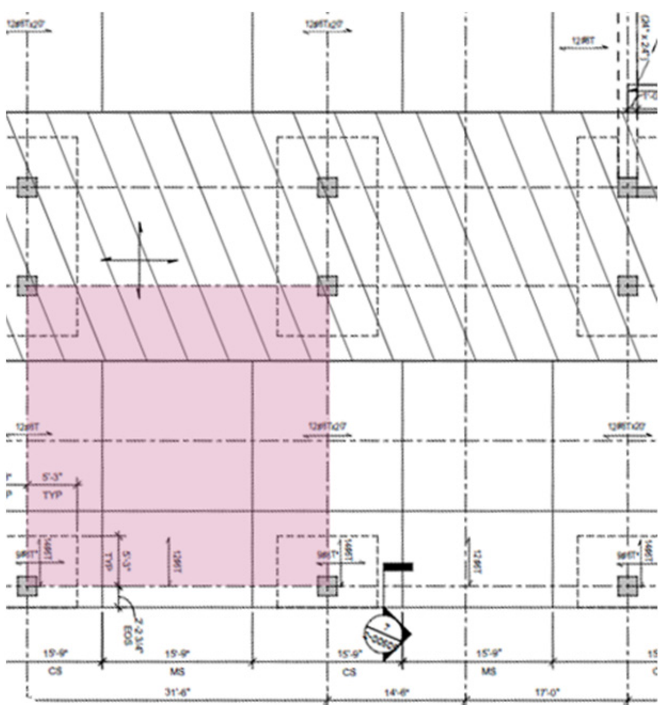


FIGURE 11: TYPICAL BAY USED FOR DESIGN

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

The existing floor system of Piez Hall addition is a cast-in-place two-way flat slab with drop panels. The system was analyzed to provide a comparative base against other alternative systems. A series of spot check on a typical bay, beams, girders and columns were found to be able to carry the loads. A typical bay of the existing system can be found in figure 14.

Advantages:

Flat plate floor system is known to be highly buildable. One of the advantages of flat plate system is flexibility in room layout. This will allow architects to introduce partition walls anywhere as well as given the choice to change the size of a room. Another advantage of flat plate system is the ease of construction, which will shorten the total construction time needed to complete the entire building. Thus this will lower the total project cost since labor cost will be cheaper due to the lowered overall hours workers work in the project. Also, any mechanical and electrical services can be mounted directly on the underside of the slab instead of bending them to avoid beams.

Disadvantages:

There are only a few disadvantages of a flat plate system. First, it has a relatively high deflection compared to other systems, which will require a thicker slab. The weight of the system is also heavy, thus bigger columns and foundation is needed.

Comment:

The two-way flat plate system used in the Piez hall addition had an overall depth of 20" (including drop panels). This system would cost about $17 per square foot. Out of the four systems compared, it is by far the most inexpensive system. Given the many advantages and the low cost of the system, a two-way flat slab with drop panels was a wise choice for the Piez hall extension.

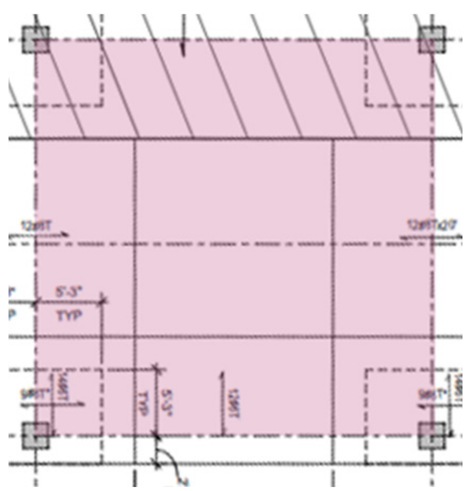


FIGURE 12: EXISTING TWO-WAY FLAT SLAB OF A 31.5'X31.5' BAY

Composite Steel

A composite system was chosen as the first alternative to the existing floor system. In a typical 31.5'x31.5' bay size, the results revealed that a 1.5VL18 composite deck with 2.5" concrete topping and spray fiber coating was required to adequately carry the loads and achieve a 2 hour fire rating. The beams needed for this composite system were W16x26 with 40 studs per beam (2 studs per rib) at 10.5' on center. The girders required were W24x55 with 24 studs per girder (1 stud per rib) spanning a length of 31.5'

Advantage:

A composite system could have been chosen for many reasons. A composite system is the light, which makes it an advantageous choice. By allowing the concrete to act in compression in the top and the steel to act in tension in the bottom, each material is utilized effectively to carry the load. Thus members can be sized smaller and lighter. Moreover, it is quick to erect and construct, making it perfect if scheduling is tight. As with any other steel framing system, bay sizes are able to be increased.

Disadvantage:

Despite the advantages of a composite system, there are several flaws as well. Fire-proofing is required to be added to the deck, steel beams, and girders to meet the fire rating needed for the system. In addition, shear studs must be welded to the beams, which adds additional labor and material costs. Also, girders with holes in the web are often necessary to allow mechanical and electrical services to go through. However, a hole in the web will lower the load bearing capacity of the member. Therefore, a deeper section is needed to compensate for the loss in strength, which adds additional depth to the system.

Comment:

A composite system analyzed for the Piez hall extension had a 43.2psf total weight and a total deck thickness of 4". However, the system had an overall depth of 28". This is about 8" deeper than the original system, which will result in an increase in total building height of about 32". This does not seen to be a significant disadvantage of the system because an overall height increase of just 32" will not add much lateral loads to the building to cause the foundation to fail. Nor it will violate height restrictions of the current area.

The total deflection of the system was found to be about 1.55", but it was considered to be acceptable since it was less than the maximum allowed L/240. It was found that cambering were not necessary to minimize deflection. Another concern for this system is that vibration will be expected to be greater than the other alternatives, but still at an acceptable range. Further calculation is needed to evaluate the need of deeper beam or thicker slab to control vibration. Also a composite system will allow the use of wind-bracing system, which is lighter than the shear walls in the current system.

The cost of a composite system is about $22.75 per square foot. Through analysis, it was found that this system is feasible for the Piez Hall extension. Although the cost for a composite system is higher than that of the existing system, the lighter weight system and faster construction schedule makes up for these disadvantages.

It is believed that the structural engineer did not choose a composite system due to higher deflections and possibly higher vibration. Also, the deeper floor system and higher cost may have be the reason for not selecting this system.

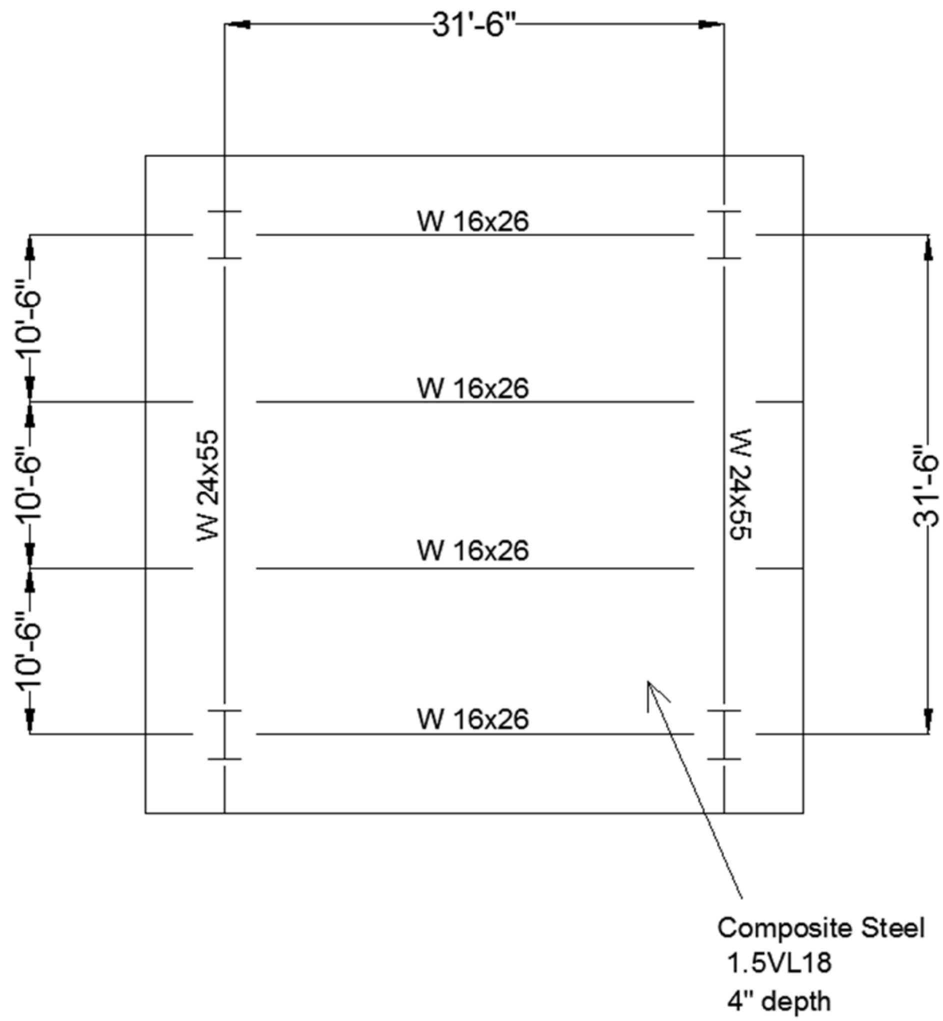


Figure 13: Designed composite steel system for a 31.5'x31.5' bay

Pre-cast Hallow Core Slab

The second alternative floor system that was evaluated was a precast hallow core planks on concrete girders. This system was designed by referring to the tables in the Nitterhouse products catalog. Keeping the original 31.5'x31.5' bay size, it was found that a 10"x48" hallow core planks with 2" concrete topping was needed for the slab. A 20"x20" concrete girder with both 8 #9 top and bottom reinforcement bars would be required over a 31.5' span. The beams parallel to the planks were not sized because the load they carry is minimal.

Advantage:

Similar to a composite system, a precast hallow core system is quick to erect and construct. It uses high strength concrete that is very easy to install, which will speed up the construction process. It is capable of carrying large loads and achieves a 2 hour fire rating with a very thin slab. It is also known to be light weight and durable.

Disadvantage:

The major disadvantage of a hallow core system is the expensive cost. It has the highest material and total cost compared to the other three alternative systems. This kind of floor system also has the highest total system depth, which could possibly bring a concern to the zoning requirement for total building height. In addition, pre-cast hallow core planks comes in 48 increments, which means that the column layout of the building will need to be rearranged. In another words, a typical 31.5'x31.5' bay in the Piez hall extension will need to be adjusted to become 31.5'x32' in order to use a pre-cast hallow core system.

Comments:

The total system depth of the hallow core system was 32", making it the deepest of all. It is about 12" deeper than the current system. To maintain the existing floor-to ceiling heights, an overall increase in building height of about 48" is required. It is preferred not to increase building height to a great extent because this would increase mass and surface area of the building, and thus influences the seismic and wind forces.

The estimated cost of this system is about $26 per square foot. Of the four systems, the pre-cast hallows core system cost the most. Compared to the current system, this is a $9 per square foot increase in cost.

The investigation showed that the pre-cast hallow core system is not a feasible solution to the Piez hall extension due to its high cost. The deep floor system and the need to rearrange column layouts are also very unfavorable. Therefore, it will not be investigated further in future technical reports.

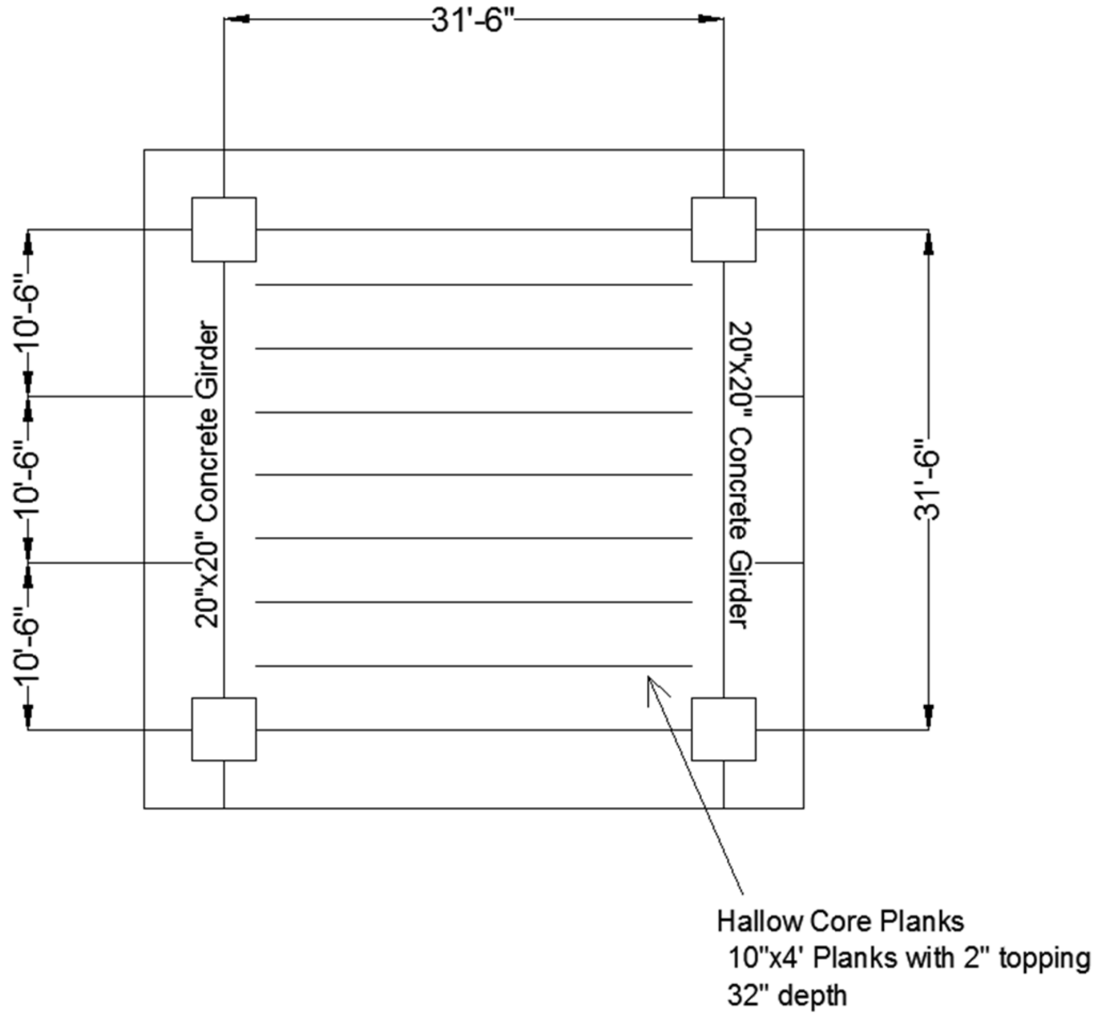


FIGURE 14: DESIGNED PRE-CAST HALLOW CORE SYSTEM FOR A 31.5'X31.5' BAY

Post-Tension

A one-way post tension system was chosen as the third alternative floor system. Three continuous bay having two 31.5'x31.5' exterior bay and a 10.5'x31.5' interior bay were chosen to analyze. It is found that an overall slab thickness of only 9" is required to carry the load. Bonded reinforcements were chosen to be #5 at 12" o.c for interior spans and #8 at 12" o.c for exterior spans. 9 #5 top bars were placed in top of the slab near the columns where negative moment is critical. Normal slab reinforcement is required in a post-tensioned slab because the tendons are unbounded to the concrete.

(32) ½θ, 7-wire strands with a jacking force of 266kips were distributed evenly in the slab of a width of 31.5'. Tendons are placed according to the locations of positive and negative moments in the slab. Post-tension tendons need to be in the tension face of the concrete to impose compression and cracking control. The strands in the 10.5'x31.5' interior bay are placed above the neutral axis because the shorter span in between two long spans causes a negative moment to exist above the neutral axis.

Advantages:

A post-tension system usually allow for longer span length and thinner slabs that already has a 2 hour fire rating. Additionally, it also allows greater crack and deflection control. Moreover, only very simple concrete formwork is necessary to construct a flat plate system. Since the system is a flat plate, it will result in a uniform flat ceiling that is convenient for mechanical and electrical services as well as maintaining most of the advantages given by the existing two-way flat slab system.

Disadvantages:

There are few disadvantages to use a post-tension system. For one, anchoring devices and grouting equipment are required to tighten the post-tension tendons. This will add to the cost and time of the project. Additionally, punching shear and future slab cutting must be thoroughly addressed since it is one of the most critical failures for flat plate post-tension system.

Comments:

The post-tension system was estimated to have a cost of about $18 per square foot. Although this is slightly more than the current system, it is still consider within a feasible cost range. However, the thin and light weight slab of this system makes it an attractive alternative, which will potentially decrease the overall building height and column size.

New consideration and design principles will be introduced in the future technical report. A decision will need to be made between using a two-way post-tensioned flat slab with drop panels and a one-way slab using post-tensioning girders. The two-way post-tensioned flat slab with drop panels seems to be a reasonable alternative to the current system due to the current column layout of the Piez hall addition.

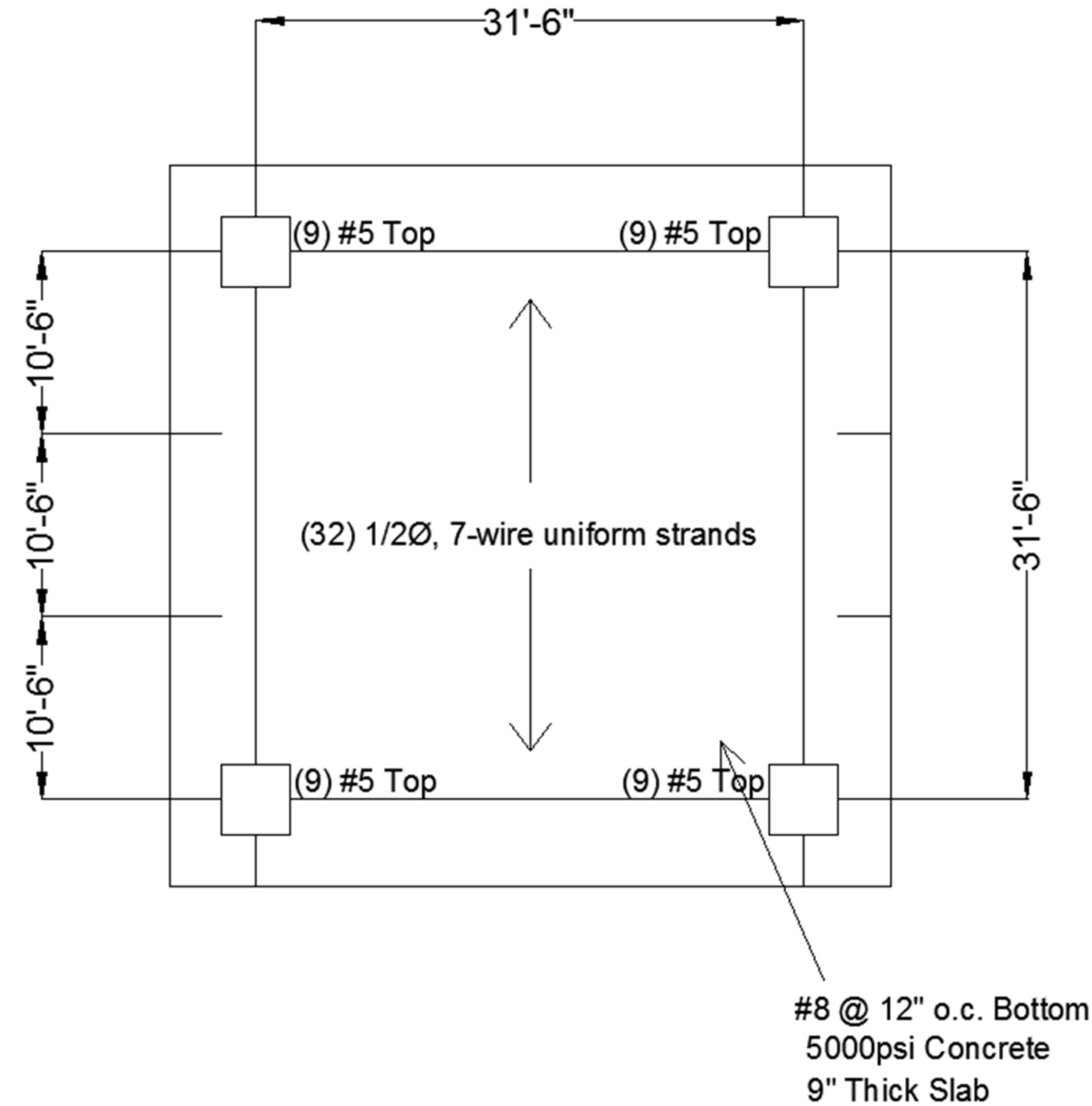


FIGURE 15: DESIGNED POST-TENSIONED SLAB FOR A 31.5'X31.5' BAY

Criterion	Floor Systems			
	Two-Way Flat Slab with Drop Panels	Composite Steel	Pre-Cast Hallow Core	Post-Tension
Cost (USD/SF)	$17	$22.75	$26	$18
Weight (psf)	170	43.2	68	137
Slab Thickness (inches)	12	4	12	9
Floor Depth (inches)	20	28	32	17
Architectural Impacts	No impacts	Increased Depth may cause problem	May need to rearrange column layouts	May reduce depth
Fire Protection	2 Hour	2 Hour – Spray Fiber	2 Hour	2 Hour
Foundation Impact	No impacts	May Reduce foundation size	May slightly reduce size	May slightly reduce size
Lateral System Impact	Shear walls/diagonal braces	Steel braces/moment frames	Shear walls/diagonal braces	Shear walls/diagonal braces
Deflection (inches)	1.20	1.55	1.25	Minimal
Vibration	Average	Poor	Average	Average due to thin slab
Constructability	Easy	Medium	Easy	Medium
Lead Time	Short	Long	Short	Medium
Feasibility	Yes	Yes	No	Yes

TABLE 5: SUMMARY OF THE FOUR SYSTEMS

Conclusion

This technical report is prepared to provide several possible alternative floor systems that can be used in the Piez hall extension. The existing system, a two-way flat slab with drop panels, was compared to a composite steel deck, a pre-cast hallow core planks on concrete girder, and a one-way post-tensioned slab. The criterion for this comparison included cost per square foot, system depth, weight, deflection, vibration, impact on lateral system, impact on foundation, impact on architecture, constructability, fire protection, and lead time. It is desirable to minimize the cost, weight, and the overall height of the building.

The existing two-way flat slab was the least costly system, but also the heaviest system. The floor system contained many advantage included flexibility in room layout, ease of construction, and good coordination of trade. It is verified to be a good choice for the Piez hall addition.

Composite steel is the lightest system of them all. It is also easy to erect and construct, which may drastically reduce the project schedule. However, the additional cost is relatively high and the system depth was also the largest compared. The composite steel shall be considered a feasible option, but it is not as good as the current system or the post-tension system.

Through comparison, the result showed that a pre-cast hallow core planks on concrete girder will be an uneconomic and inefficient alternative system to the Piez hall addition. The post-tension and the existing two-way flat slab are the most attractive systems found through evaluation.

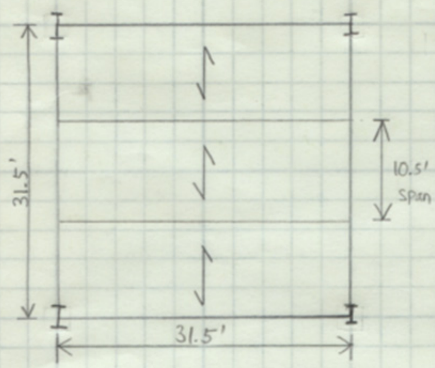
Out of all the alternatives, the post-tension concrete system is the most comparable to the original system. With this system, the building weight will decrease as well as the total depth. Additionally, it will maintain most advantages given by the existing system. One major disadvantage with the post-tension system is the construction difficulty associated with the post-tensioning process and the lack of adaptability to future change as well as the additional cost per square foot. However, the advantages for this system compensate for the drawbacks, and hence it is considered to be a viable option.

In future technical reports, the author will investigate further into the existing flat slab and the post-tension system. A decision will be made between a two-way post-tensioned flat slab with drop panel and a one-way slab on post-tensioning girder. These systems will be examined for their impact on the overall system of the Piez hall addition.

Appendices

Appendix A: Composite Steel Calculations

Composite slab | MinGao Li | Pg 1/5



Normal Weight Concrete.

$f'_c = 5000$ psi

50 ksi Psi

$L_L = 100$ Psf

$SID = 15$ Psf

Unshored Construction, 3-span

For 2 Hr unprotected deck, $4\frac{1}{2}$" thick NWC is required } Pg 71

" " sprayed fiber deck, $2\frac{1}{2}$" thick topping is required. } Vulcraft

Assume sprayed fiber will be use to reduce slab weight.

Use 1.5 VL 18 (for $t = 2.5$)

SDI Max span = 10'7" > 10.5' ⇒ ok

156 psf @ 10.5' > (100 Psf + 15 Psf) ⇒ ok

Deck weight = 39 Psf ; Total thickness = 4"

$\boxed{\text{Beam Design:}}$

Assume simply supported

Assume 5psf for Beam self-weight

$$LL_r = 0.25 + \frac{15}{\sqrt{\underbrace{2 \times 10.5' \times 31.5'}_{=661.5 > 400 \Rightarrow ok}}} = 0.833$$

Composite slab MinGaoLi Pg 2/5

$$1.2D + 1.6L = 1.2(15+39+5)\,psf + 1.6(100 \times 0.833) = 205\,psf$$

$$M = \frac{w\ell^2}{8} \quad \text{for simply support}$$

$$M_U = \frac{205\,psf\,(10.5')(31.5')^2}{8(1000)} = 265.8\ k\cdot ft$$

$$\Delta_{LL} \le \frac{L}{360} : \quad \frac{31.5' \times 12}{360} = 1.05''$$

$$\Delta_{LL} = \frac{5 w_L \ell^4}{384EI} = 1.05'' \Rightarrow \frac{5(0.833 \times 100\,psf)\left(\frac{10.5'\,o.c}{1000}\right)(31.5)^4}{384(29000)\,I_{req.}} \times 1728 = 1.05''$$

$I_{req'd} = 636.3\ in^4$ for Δ_{LL}

$$\Delta_{tot} \le \frac{L}{240} : \quad \frac{31.5' \times 12}{240} = 1.575''$$

$$1.575'' = \frac{5(0.833 \times 100 + 15 + 39 + 5)\left(\frac{10.5}{1000}\right)(31.5)^4}{384(29000)\,I_{req'd}} \times 1728$$

$I_{req'd} = 724.7\ in^4$ for Δ_{tot}

$\Rightarrow \Delta_{tot}$ controls over Δ_{LL}

Assume $a = 1''$ $\therefore$ $y_2 = 4 - \frac{a}{2} = 3.5''$

$$b_{eff} = \min \begin{cases} \frac{31.5 \times 12}{8} \times 2 = \underline{94.5''} \\ \frac{10.5 \times 12}{2} \times 2 = 126'' \end{cases}$$

$$a = \frac{\Sigma Q_{n,max}}{0.85 f'_c b_{eff}} \Rightarrow 1.0 = \frac{\Sigma Q_{n,max}}{0.85(5)(94.5)} \Rightarrow Q_{n,max} \le 401.6^k$$

Try W16x31 $\Rightarrow \Sigma Q_n = 213^k$; $I_{LB} = 756\ in^4$; $\phi M_n = 324^{k\text{-}ft}$

\# of studs required: $(213/17.2) \times 2 = 25$ studs/Beam

economy: $25(10) + 31(31.5) = 1226.5$ lbs of steel

Try W16x26 $\Rightarrow \Sigma Q_n = 289^k$; $I_{LB} = 726 in^4$; $\phi M_n = 301^{k\text{-}ft}$

\# of studs required: $(289/17.2) \times 2 = 34$ studs/beam $> 31.5' \Rightarrow$ N.G

Try 2 weak studs per rib

$(289/14.6) \times 2 = 40$ studs/beam $< 63' \Rightarrow$ ok

economy: $40(10) + 26(31.5) = 1219^{lbs}$ of steel

Check unshored Strength

$\phi M_n = 166^{k\text{-}ft}$ for W16x26 (table 3-2)

$$W_u = 1.2(39 \times 10.5 + \underset{\text{Self wt}}{26}) + 1.6(\underset{\text{Construction LL}}{20} \times 10.5) = 0.8586^{k/ft}$$

$$M_u = \frac{(0.8586)(31.5)^2}{8} = 106.5^{k\text{-}ft} < \phi M_n \Rightarrow$$ ok for no shoring.

Check wet Concrete deflection

$$W_{wc} = 39(10.5) + 26 = 0.4355^{k/ft}$$

$$\Delta_{wc} = \frac{5(0.4355)(31.5)^4(1728)}{384(29000)(\underset{I_x}{301})} = 1.1'' < \frac{l}{240} = 1.58'' \Rightarrow \text{ok}$$

↳ No Camber needed.

$\boxed{\text{*Use W16x26 With 40 studs/beam for beam}}$

$26^{lbs/ft} < 5psf \times 10.5^{ft} = 52.5^{lbs/ft}$

5psf self-wt allowance is okay

Composite slab | MinGao Li | Pg 4/5

$\boxed{\text{Girder Design:}}$

Assume 3psf girder self-wt allowance.

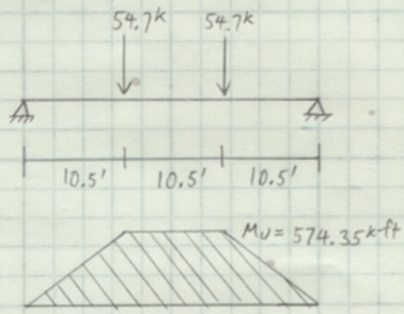
$$L_{Lr} = 0.25 + \frac{15}{\sqrt{2 \times 31.5^2}} = 0.587$$

$$L_L = 100(0.587) = 58.7 \text{ psf}$$

$$D_L = 15 + 39 + 26/10.5 + 3 = 59.5 \text{ psf}$$

$$W_U = 1.2(59.5) + 1.6(58.7) = 165.3 \text{ psf}$$

$$P_U = 165.3 \text{ psf} (10.5' \text{ beam O.C}) (31.5' \text{ O.C}) = 54.7^{\text{ kip}}$$



$$\Delta_{tot} \le \frac{L}{240} : \quad \Delta = \frac{P l^3}{28EI}$$

$$\frac{(58.7 + 59.5)\left(\frac{12}{1000}\right)(31.5')(31.5)^3}{28(29000)\, I_{req'd}} \times 1728 = 1.575''$$

$$I_{req'd} = 1887 \text{ in}^4$$

Try W24 x 55 : $\Sigma Q_n = 203^k$; $\phi M_n = 697^{k'}$; $I_{lb} = 2110 \text{ in}^4$

\# of studs required: $(203/17.2) \times 2 = 24$ studs/beam.

economy: $24(10) + 55(31.5) = 1972.5^{\text{ lbs}}$ of steel

Composite slab | MinGao Li | Pg 5/5

Check unshored strength for W24 x 55

$P_u = [1.2(39 \times 10.5 + 55) + 1.6(20 \times 10.5)] \times 31.5' \text{ O.C} = 28.2^{kip}$

$M_u = 28.2 \times 10.5' = 295.5^{k\text{-}ft} < \phi M_n = 503^{k\text{-}ft} \Rightarrow$ ok for no shoring

Check wet Concrete deflection

$P_{wc} = 39(10.5)(31.5) + 55(31.5) = 14.6^{k}$

$\Delta_{wc} = \dfrac{(14.6)(31.5)^3(1728)}{28(29000)(1350)} = 0.72'' < \dfrac{l}{240} = 1.575'' \Rightarrow$ ok

↳ no Camber required.

$\boxed{\text{* Use W24 x 55 with 24 studs/girder for girder}}$

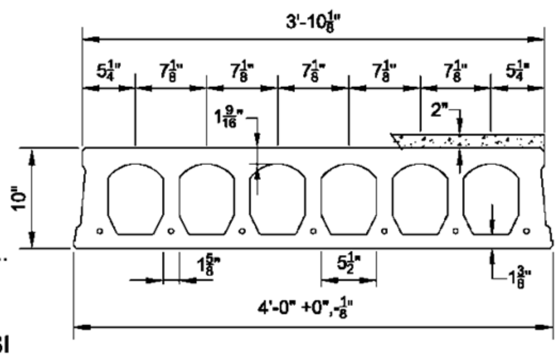
$\dfrac{26}{10.5} + \dfrac{55}{31.5} = 4.2 \text{ psf} \le (3+5) \text{ psf}$ allowance $\Rightarrow$ ok

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

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 327 in.^{2}	Precast b_w = 13.13 in.
I_c = 5102 in.^{4}	Precast S_{bcp} = 824 in.^{3}
Y_{bcp} = 6.19 in.	Topping S_{tct} = 1242 in.^{3}
Y_{tcp} = 3.81 in.	Precast S_{tcp} = 1340 in.^{3}
Y_{tcp} = 5.81 in.	Precast Wt. = 272 PLF
	Precast Wt. = 68.00 PSF



DESIGN DATA

1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø and 0.6"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   6-1/2"Ø, 270K = 168.1 k-ft at 60% jacking force
   7-1/2"Ø, 270K = 191.7 k-ft at 60% jacking force
7. Maximum bottom tensile stress is $10\sqrt{f'c}$ = 775 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)

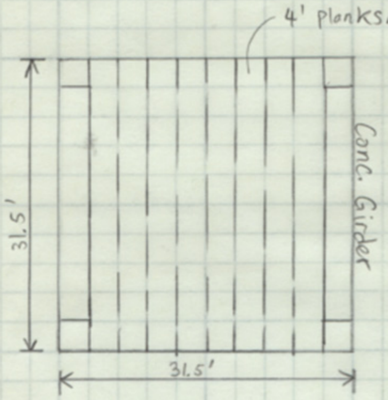
Strand Pattern		SPAN (FEET)																		
		26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44
6 - 1/2"ø	LOAD (PSF)	202	181	161	144	128	114	101	90	79	69	60	52	45	38					
7 - 1/2"ø	LOAD (PSF)	246	222	200	180	162	146	131	118	105	94	84	74	66	58					

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

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

11/03/08

10F2.0T



Plan View

Loads:

$L_L = 100$ psf

LLr is not used here to be conservative.

SID: 15 psf

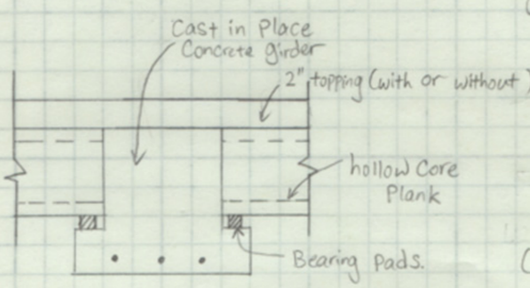
Use Nitterhouse hallow Core Planks.
with or without 2" toppings.

Service Load: $100 + 15 = 115$ psf

Choose 10"x4'-0"
hallow Core Plank
2" topping (wt = 25 psf)

With Strand Pattern
7 - ½"Ø
(self-wt = 68 psf)

(7) ½Ø Strand pattern @ 32'
= 131 psf > 115 psf
=> ok



Section A-A

Check Stresses:

4' Plank acts as simply supported beam.

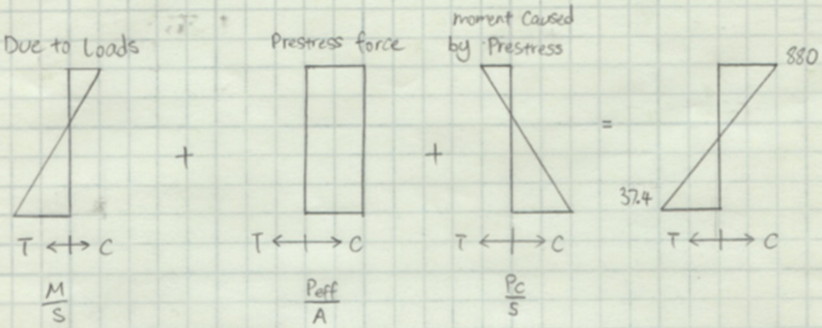
$$M_U = \frac{W_U l^2}{8} = \frac{(100+15+68+25)(4')(31.5')^2}{8(1000)} = 103.2 \text{ k-ft}$$

$$= 1238.4 \text{ k-in}$$

$A_{strand} = 0.153 in^2$ (determined from experiment)

$P_{eff} = 0.6\,(270\text{ ksi})\,A_{strand} = 0.6\,(270)(0.153 in^2) = 24.786^k/\text{strand}.$

Precast Hollow Core Conc | MinGao Li | Pg 2/3



$S_{top} = 1340 in^3$ (Given) ; $S_{bot} = 824 in^3$ (Given) ; $A_c = 327 in^2$ (Given)

$e = \underbrace{6.19 - 1.75}_{\text{Given.}} = 4.44''$

$$f_t = -\frac{1238.4}{1340} - \frac{(7)(24.786)}{327 in^2} + \frac{(7)(24.786)(4.44)}{1340}$$

$f_t = -0.88\ ksi = 880\ psi$ Compression $< 0.45f'c = 2700\ psi$

$$f_b = \frac{1238.4}{824} + \frac{(7)(24.786)}{327} - \frac{(7)(24.786)(4.44)}{824}$$

$f_b = 0.0374\ ksi = 37.4\ psi$ tension $< 10\sqrt{f'c} = 775\ psi \Rightarrow ok$

Check Deflection:

$$\Delta_{LL} = \frac{5(100)(\frac{4}{1000})(31.5)^4}{384(29000)(5102)} \times 1728 = 0.6\ in < \frac{l}{360} = 1.05'' \Rightarrow ok$$

$$\Delta_{tot} = \frac{5(100+68+25+15)(\frac{4}{100})(31.5)^4}{384(29000)(5102)} \times 1728 = 1.25'' < \frac{l}{240} = 1.575'' \Rightarrow ok$$

*Use 10" Hollow Core Conc. With 2" topping & (7) ½"Ø Strands

Precast Hallow Core Conc. | Min Gao Li | Pg3/3

Design Girder for Hollow Core Conc. Slab:

$W_u = [1.2(15+25+68) + 1.6(100 \times \overset{L_r}{0.587})]\, 31.5 = 7.05$ kip/ft

$M_u = \frac{W_u l^2}{8} = \frac{(7.05)(31.5)^2}{8} = 874.4$ k-ft

Estimate size: Try $b = \frac{2}{3}d$

$bd^2 = 20M_u = \frac{2}{3}d^3 = 20(874.4)$

$d = 30$ in

$h = 30 + 2.5 = 32.5''$

Try: $20'' \times 33''$ → self-wt $= \frac{20 \times 33}{144} \times 150 = 687.5$ lbs/ft
$\times 1.2$
$.825$ k/ft

$M_u = \frac{(7.05 + 0.825)(31.5)^2}{8} = 976.75$ k-ft

$A_s = \frac{M_u}{4d} = \frac{976.75}{4(30.5)} = 8 \text{ in}^2$ ⇒ Provide (8) #9 rebars. $A_s = 8 \text{ in}^2$

Assume Doubly reinforced.

Try $20'' \times 20''$ Beam

$\rho_{max} = 0.85^2\left(\frac{5}{60}\right)\left(\frac{0.003}{0.008}\right) = .0226$

$A_{s1} = 0.0226(17.5)(20) = 7.91 \text{ in}^2$

$a = \frac{(7.91)(60)}{0.85(20)(5)} = 4.75 \rightarrow c = 5.6''$

$M_{n1} = 0.85(5)(4.75)(20)\left(17.5 - \frac{4.75}{2}\right) = 509$ k-ft

$M_{n2} = 976.75/0.9 - 509 = 576.3$

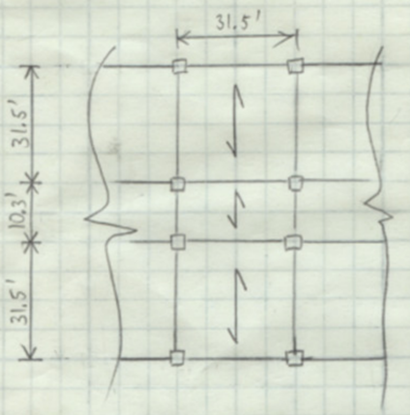
$A_{s2} = \frac{576.3 \times 12}{60(17.5 - 2.5)} = 7.7 \text{ in}^2 \rightarrow A_s = 7.7 + 7.91 = 15.61 \text{ in}^2$

Provide (8)#9 for top & bottom reinforcement ($A_s = A'_s = 8 \text{ in}^2$)

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

Appendix C: Post-Tensioned Slab Calculations

Post - Tension | | | Min Gao Li | Pg 1/10



Typical floor Plan

Loads: Framing Dead Load = self wt

SID : 15 psf

LL = 100 psf

2 Hr fire rating

Material:

Concrete wt = 150 pcf

$f'_c = 5000$ psf

$f'_{ci} = 3000$ psi (assumed)

Rebar = 60 ksi

Unbonded tendons

½" Ø, 7- wire strands, $A = 0.153 in^2$

$f_{pu} = 270$ ksi

Estimated prestress losses = 15ksi (ACI 18.6)

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

$P_{eff} = A \times f_{se} = (0.153)(174ksi) = 26.6$ k/tendon

Determine Preliminary slab thickness:

$$L/h = 45 \Rightarrow \frac{(31.5 \times 12)}{45} = 8.4 \text{ in}$$

Try 9 in slab. (self-wt = $9/12 \times 150 = 112.5$ psf)

Calculate Section Property:

$$S = bh^2/6 = \frac{(31.5 \times 12)(9)^2}{6} = 5103 \text{ in}^3$$

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

Set design parameter:

@ time of Jacking (ACI 18.4.1)

$f'_{ci} = 3000$ psi

Compression $= 0.6 f'_{ci} = 1{,}800$ psi

Tension $= 3\sqrt{f'_{ci}} = 164$ psi

@ Service load: (ACI 18.4.2(a) and 18.3.3)

$f'_c = 5000$ psi

Compression $= 0.45 f'_c = 0.45(5{,}000 \text{ psi}) = 2{,}250$ psi

Tension $= 6\sqrt{f'_c} = 424$ psi

Average Precompression limits: (per ACI 18.12.4)

P/A = 125 psi min.
= 300 psi max

Target load balance.

60% ~ 80% of DL (Self weight)

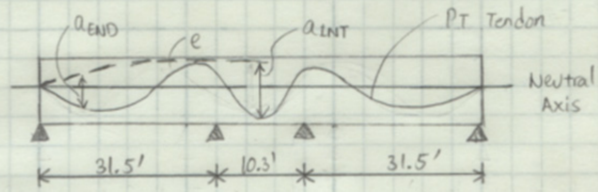
0.75(112.5) = 84.4 psf

Cover Requirement: (2-Hr rating, assume Carbonate aggregate)

Restrained slabs = 3/4" bottom

Unrestrained slabs = 1½" bottom
= 3/4" TOP.

Continuous Post-Tension Beam



Prestress Force Required to balance 75% of Self-wt DL

$W_b = 0.75\, W_{DL} = 0.75\,(112.5)(31.5\,ft) = 2.66\ k/ft$

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

$$P = \frac{W_b L^2}{8 a_{end}} = \frac{(2.66\,k/ft)(31.5\,ft)^2}{8(4.75''/12)} = 834^k$$

$a_{end} = (4.5 + 8.5)/2 - 1.75 = 4.75''$

Determine # tendons required:

\# tendons = 834/26.6(k/tendons) = 31.35

use 32 tendons ($\frac{1}{2}''$ diameter strands @ 12'' spacing)

Actual Force for banded tendons.

P_{actual} = (32 tendons)(26.6) = 851^k

adjusted balance Load

$W_b = (851/834) \times (2.66\,k/ft) = 2.71\,^k/$tendon

Determine actual Pre Compression Stress

$P_{actual}/A = 851/(9'' \times 31.5' \times 12''/') = 250\,psi > 125\,psi$ min. Ok

$< 300\,psi$ max Ok

Interior bay Check:

$a_{INT} = 8.5'' - 6.5'' = 2''$

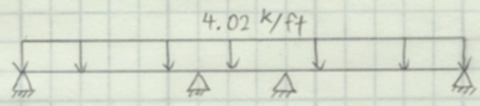
$$P = \frac{w_b L^2}{8 a_{INT}} = \frac{(2.66)(10.3)^2}{8(2.0/12)} = 212^k << 851^k$$

Much less force is needed in the interior bay

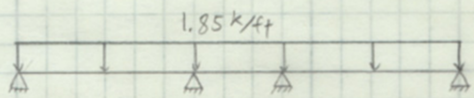
<u>effective prestress force = 851^k</u>

Tendon Ordinate	Tendon Center of gravity Location
Exterior support - anchor	4.5"
Interior support - Top	8.5"
Interior span - bottom	6.5"
End span - bottom	1.75"

Dead Load moments.



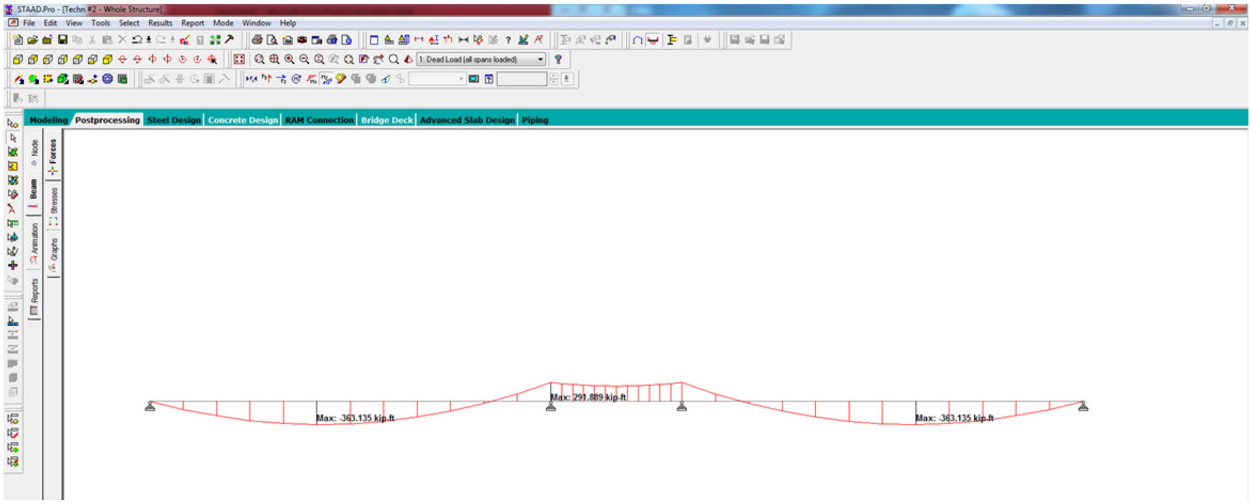
$W_{DL} = (112.5 + 15)(31.5')/1000 = 4.02$ k/ft

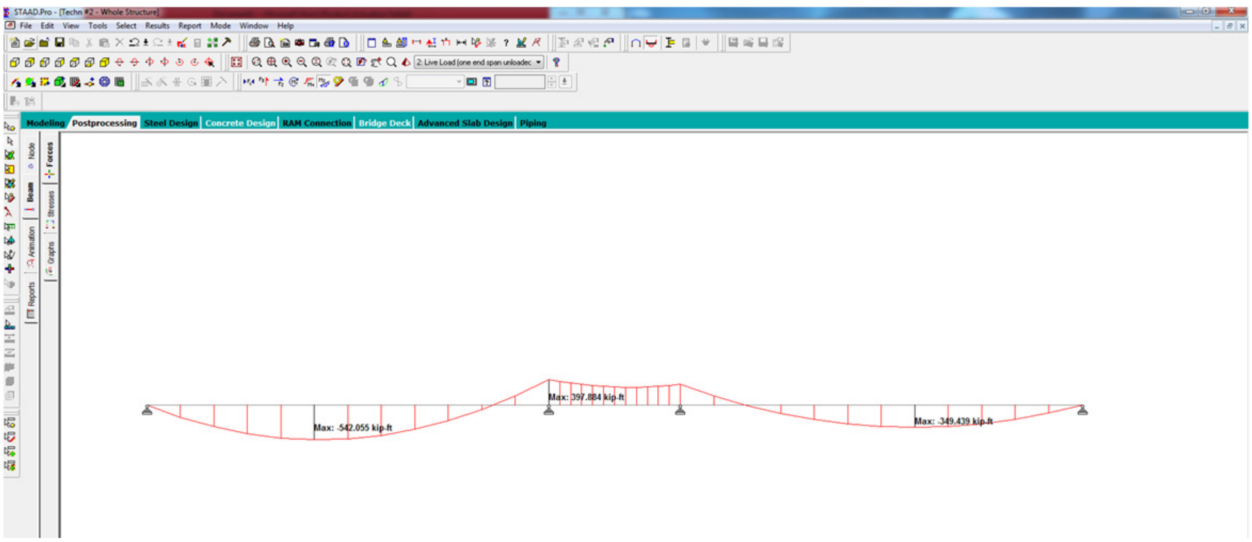


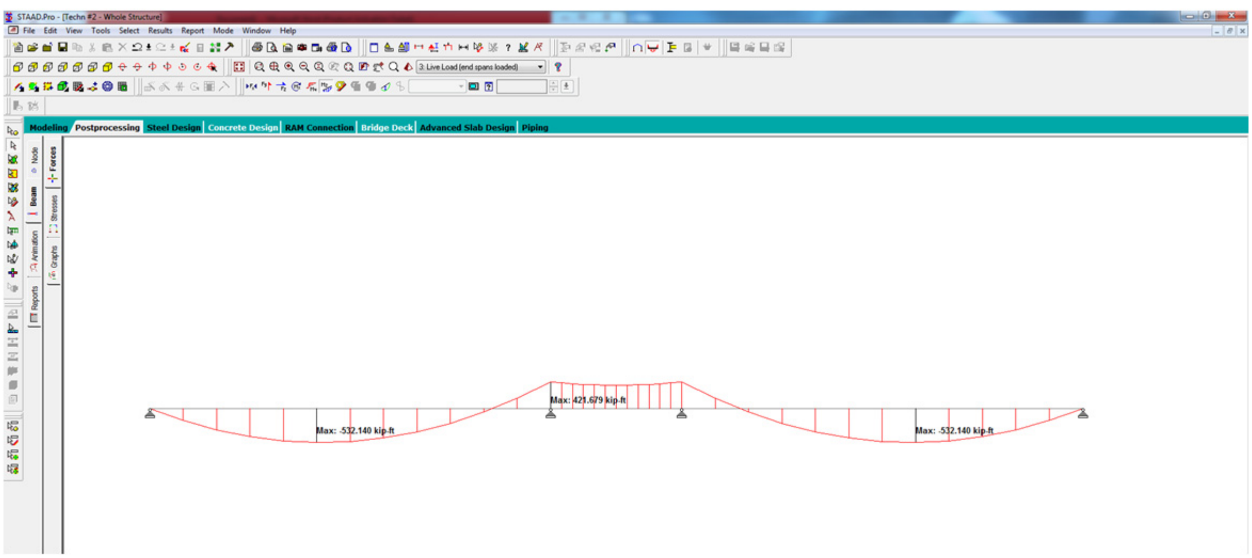
$W_{LL} = (.587 \times 100\,psf)(31.5')/1000 = 1.85$ k/ft

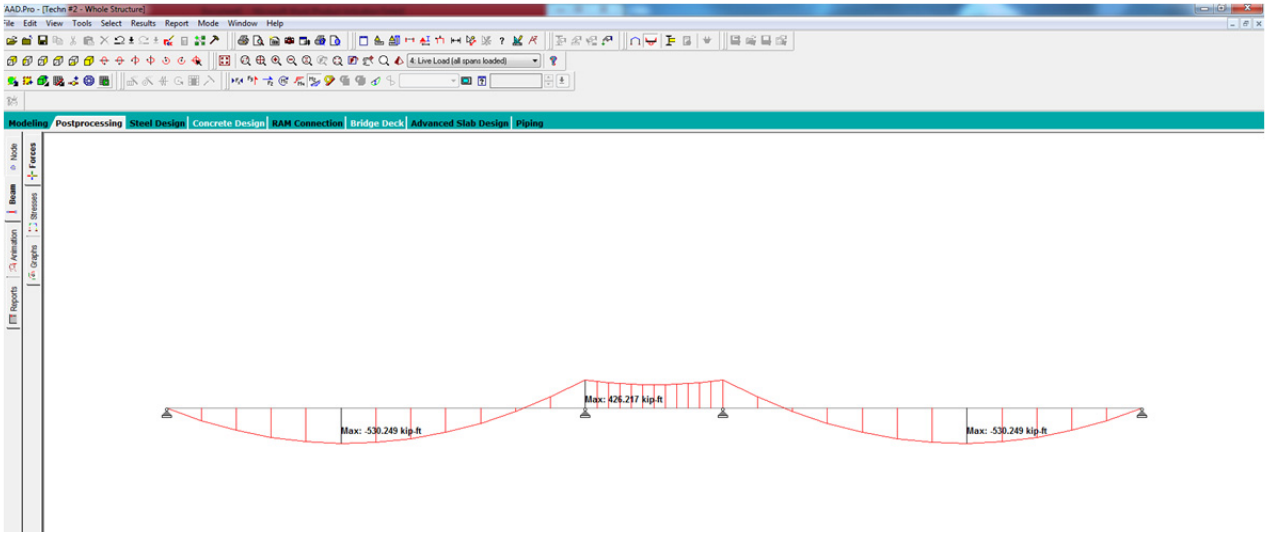
Total unbalancing moment $= W_b = 3.00$ k/ft

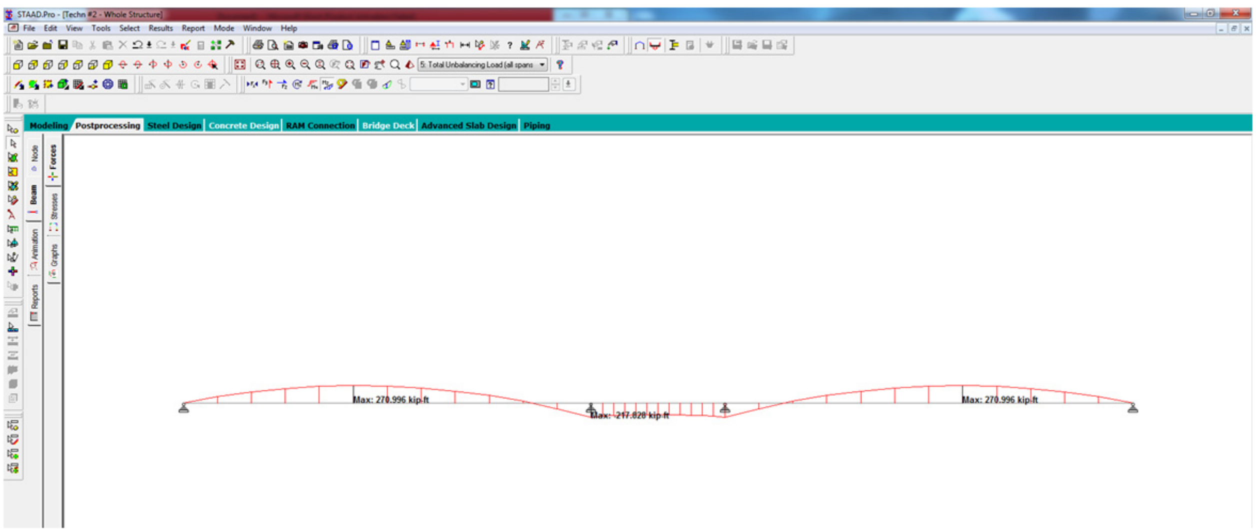
See STAAD results for different load cases.


STAAD.Pro - [Techn #2 - Whole Structure]
1: Dead Load (all spans loaded)
Max: -363.135 kip-ft
Max: 291.889 kip-ft
Max: -363.135 kip-ft


STAAD.Pro - [Techn #2 - Whole Structure]
Max: -542.055 kip-ft
Max: 397.884 kip-ft
Max: -349.439 kip-ft


STAAD.Pro - [Techn #2 - Whole Structure]
3: Live Load (end spans loaded)
Max: -532.140 kip-ft
Max: 421.679 kip-ft
Max: -532.140 kip-ft


Max: 426.217 kip-ft
Max: -530.249 kip-ft
Max: -530.249 kip-ft


Max: 270.996 kip-ft
Max: 270.996 kip-ft
Max: -217.028 kip-ft

Stage 1: Stresses immediately after Jacking ($DL + P_T$) (ACI 18.4.1)

Mid span stresses.

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

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

Interior Span

$f_{top} = [(-239 + 178)(12)(1000)/(5103 in^3)] - 250 psi$

$= -393$ psi (Compression) $< 0.6f'ci = 1800psi \Rightarrow ok$

$f_{bot} = [(239 - 178)(12)(1000)/(5103 in^3)] - 250 psi$

$= -107$ psi (Compression) $< 0.6f'ci = 1800 psi \Rightarrow ok$

Support Stresses:

$f_{top} = [(-292 + 218)(12)(1000)/5103] - 250 psi$

$= -424$ psi (Compression) $< 0.6f'ci = 1800 psi \Rightarrow ok$

$f_{bot} = [(+292 - 218)(12)(1000)/5103] - 250 psi$

$= -76$ psi (Compression) $< 0.6f'ci \Rightarrow ok$

Stage 2: Stresses @ service load ($DL + LL + P_T$) (18.3.3 and 18.4.2)

Mid Span stresses.

Interior Span

$f_{top} = [(-239 - 110 + 178)(1000)(12)/5103] - 250 psi$

$= -652$ (Compression) $< 0.45f'c = 2250 psi \Rightarrow ok$

$f_{bot} = [(239 + 110 - 178)(1000)(12)/5103] - 250 psi$

$= 152$ tension $< 6\sqrt{f'c} = 424 psi \Rightarrow ok$

End span

$f_{top} = [(-363 - 167 + 271)(12)(1000)/5103] - 250\ psi$

$= -860$ (compression) $< 0.45 f'c \Rightarrow ok$

$f_{bot} = [(363 + 167 - 271)(12)(1000)/5103] - 250\ psi$

$= 359\ psi$ (tension) $< 6\sqrt{f'c} \Rightarrow ok$

Support stresses

$f_{top} = [(+292 + 134 - 218)(12)(1000)/5103] - 250\ psi$

$= 239$ (tension) $< 6\sqrt{f'c} \Rightarrow ok$

$f_{bot} = [(-292 - 134 + 218)(12)(1000)/5103] - 250\ psi$

$= -739$ (compression) $< 0.45 f'c \Rightarrow ok$

* All stresses are within the permissible code limits.

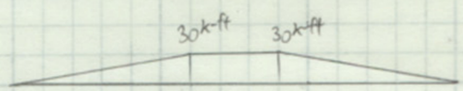
Ultimate strength

Assume eccentricity $e = 3.5\ in$ @ interior support
$e = 0$ @ exterior support

$M_1 = (851^k)(3.5)/12 = 248\ k\text{-}ft$

$M_{sec} = M_{bal} - M_1$

$= 218 - 248 = -30^{k\text{-}ft}$ @ interior support



Typical load Combination for Ultimate Strength design:

$M_U = 1.2\,M_{DL} + 1.6\,M_{LL} + 1.0\,M_{sec}$

At midspan:

$M_U = 1.2(363) + 1.6(167) + 1.0(-15) = 687.8^{k\text{-}ft}$

At support

$M_U = 1.2(-292) + 1.6(-134) + 1.0(-30) = -595^{k\text{-}ft}$

Determine minimum bonded reinforcement

Positive moment region

Interior span: $f_t = 152\,psi > 2\sqrt{f'c} = 141\,psi$

Exterior span: $f_t = 359\,psi > 2\sqrt{f'c} = 141\,psi$

} Reinforcement required.

Minimum positive moment required: (ACI 18.9.3.2)

$y = f_t / (f_t + f_c)\,h$

$= 359 / (359 + 860)(9) = 2.65\,in$

$N_c = M_{DL+LL} / S \times \frac{1}{2} \times y \times l_2$

$= [363 + 167](12) / 5013 \times \frac{1}{2} \times 2.65 \times 31.5' \times 12$

$= 635.4^{k}$

$A_{s,min} = N_c / 0.5 f_y$

$= (635.4) / (\frac{1}{2} \times 60\,ksi) = 21.18\,in^2$

$A_{s,min} = (21.18\,in^2) / 31.5' = 0.67\,in^2/ft$

Provide #8 bars @ 12in O.C Bottom ($0.79\,in^2/ft$)

Minimum length shall be 1/3 clear span & centered Positive moment reigion

$y = 152 / (152+652)(9) = 1.7$

$N_c = (239+110)(12) / 5013 \times \frac{1}{2} \times 1.7 \times 31.5 \times 12 = 268.4$

$A_{s,min} = 268.4 / (0.5 \times 60) = 9 in^2$

$A_{s,min} = 9 in^2 / 31.5' = 0.28 in^2/ft$

Provide #5 bars @ 12" O.C Top ($.31 in^2/ft$)

Negative moment region:

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

Interior support

$$A_{cf} = \max \left| \begin{array}{l} (9")(10.5'+31.5)/2 \times 12 \\ (9")(31.5) \times 12 = 3402 \end{array} \right.$$

$A_{s,min} = 0.00075(3402) = 2.55 in^2$

$= (9) \#5$ Top ($2.79 in^2$)

Exterior supports:

$$A_{cf} = \max \left| \begin{array}{l} 9" \times 31.5'/2 \times 12 \\ 9" \times 31.5'/2 \times 12 = 3402 \end{array} \right.$$

$= (9) \#5$ Top ($2.79 in^2$)

Must span a minimum of 1/6 the clear span on each side of support (ACI 18.9.4.2)

At least 4 bars required in each direction (ACI 18.9.3.3)

Place top bars within 1.5h away from face of support on each side (ACI 18.9.3.3) $\Rightarrow 1.5 \times 9 = 13.5"$ away.

Maximum bar spacing = 12" (ACI 18.9.3.3)

Post - Tension Min Goo Li Pg10/10

Check minimum reinforcement if it is sufficient for ultimate strength.

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

d = effective depth

$A_{ps} = 0.153 in^2$ (# of tendons)

$= 0.153 in^2 (32) = 4.9 in^2$

$f_{ps} = f_{se} + 10,000 + (f'c\, b\, d)/(300 A_{ps})$ for slabs with $l/h > 35$ (ACI 18.7.2)

$= 174,000 psi + 10000 + [5000 psi (31.5) \times 12 d]/(300 \times 4.9 in^2)$

$= 184000 + 1286 d$

$a = (A_s f_y + A_{ps} f_{ps})/(0.85 f'c\, b)$

At support:

$d = 9'' - 3/4'' - 1/4'' = 8''$

$f_{ps} = 184000 + 1286(8) = 194288 psi$

$a = [(2.79)(60) + (4.9)(194.3 ksi)]/(0.85 \times 5 \times 31.5 \times 12) = 0.7$

$\phi M_n = 0.9[(2.79 in^2)(60 ksi) + (4.9)(194.3)](8'' - 0.7/2)/12$

$= 642^{k\text{-}ft} > M_u = 595^{k\text{-}ft}$ => Ultimate strength does not govern

$A_{s, required} = 2.79 in^2$

At mid span: (end span)

$d = 9'' - 1\frac{1}{2}'' - \frac{1}{4}'' = 7.25''$

$f_{ps} = 184000 + 1286(7.25) = 193324 Psi$

$a = [(21.18)(60) + (4.9)(193 ksi)]/[0.85(5)(31.5 \times 12)] = 1.38''$

$\phi M_n = 0.9[(21.18)(60) + (4.9)(193 ksi)][7.25 - (1.38/2)]/12]$

$= 1090^{k\text{-}ft} >> 687.8^{k\text{-}ft}$ => minimum reinforcement is ok

At support:
use (9) #5 @ both Top interior & exterior supports

At mid-span:
use #8 @ 12" O.C Bottom @ end Span
#5 @ 12" O.C bottom @ interior span

Appendix D: Cost Analysis

MinGao Li

Cost Analysis: (RS means, Assemblies Cost Data 2013)

(Pg 75) Hallow Core

Using RS Means for Precast Beams & Plank With 2" Topping.

31.5' × 31.5' bay ≈ 30' × 30' for a total Load of 181 psf

Cost = $26 / sq. ft (material + labor)

(Pg 63) Cost in Place Flat slab With Drop Panels.

30' × 30' bay for a total Load of 182 psf

Cost = $17 / sq. ft (Material + Labor)

(P94) Composite beams, Deck & slab

30' × 30' bay for a total Load of 168 psf

Cost = $22.75 / sq. ft (Material + Labor)

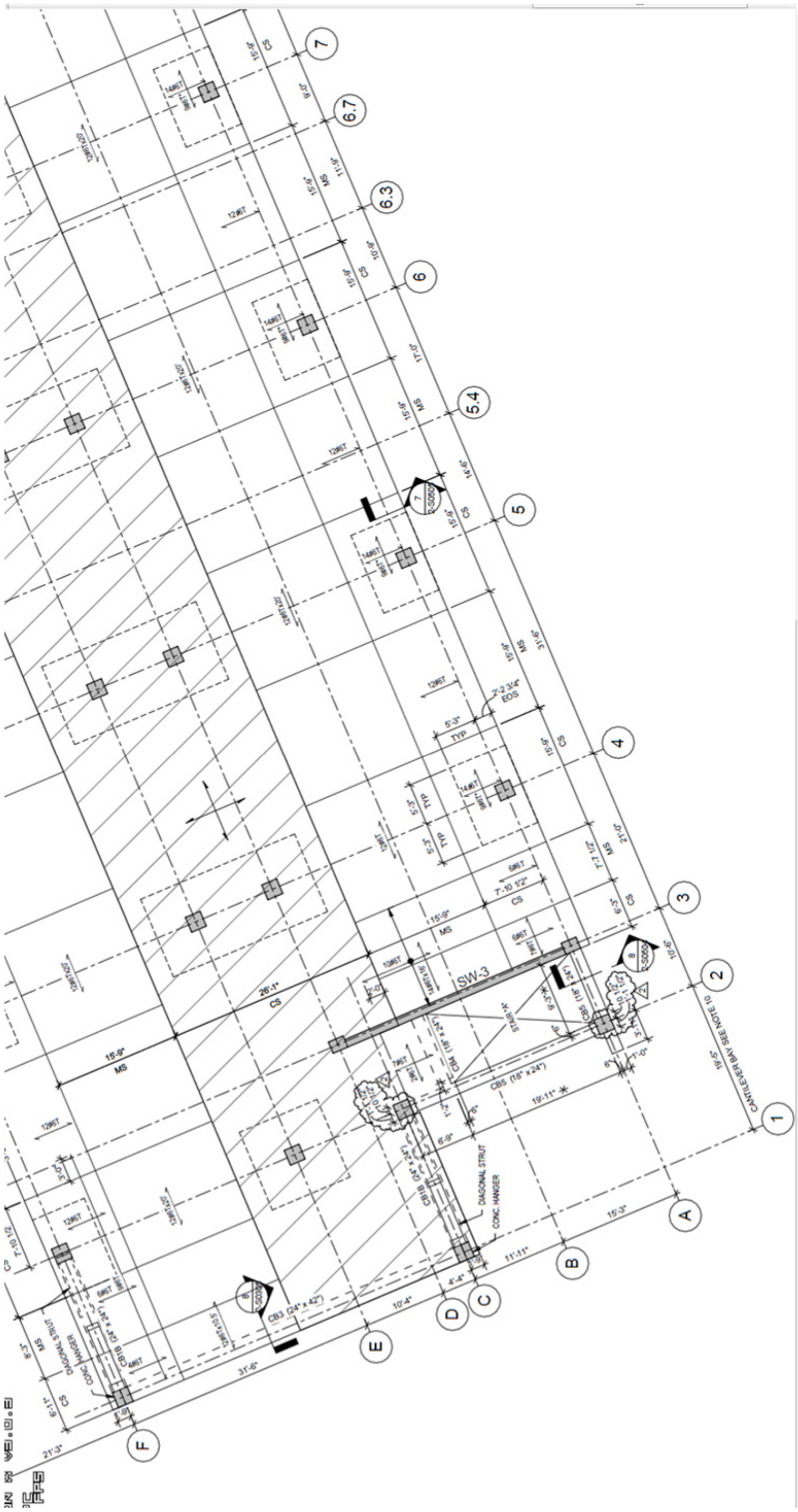
Post - Tension :

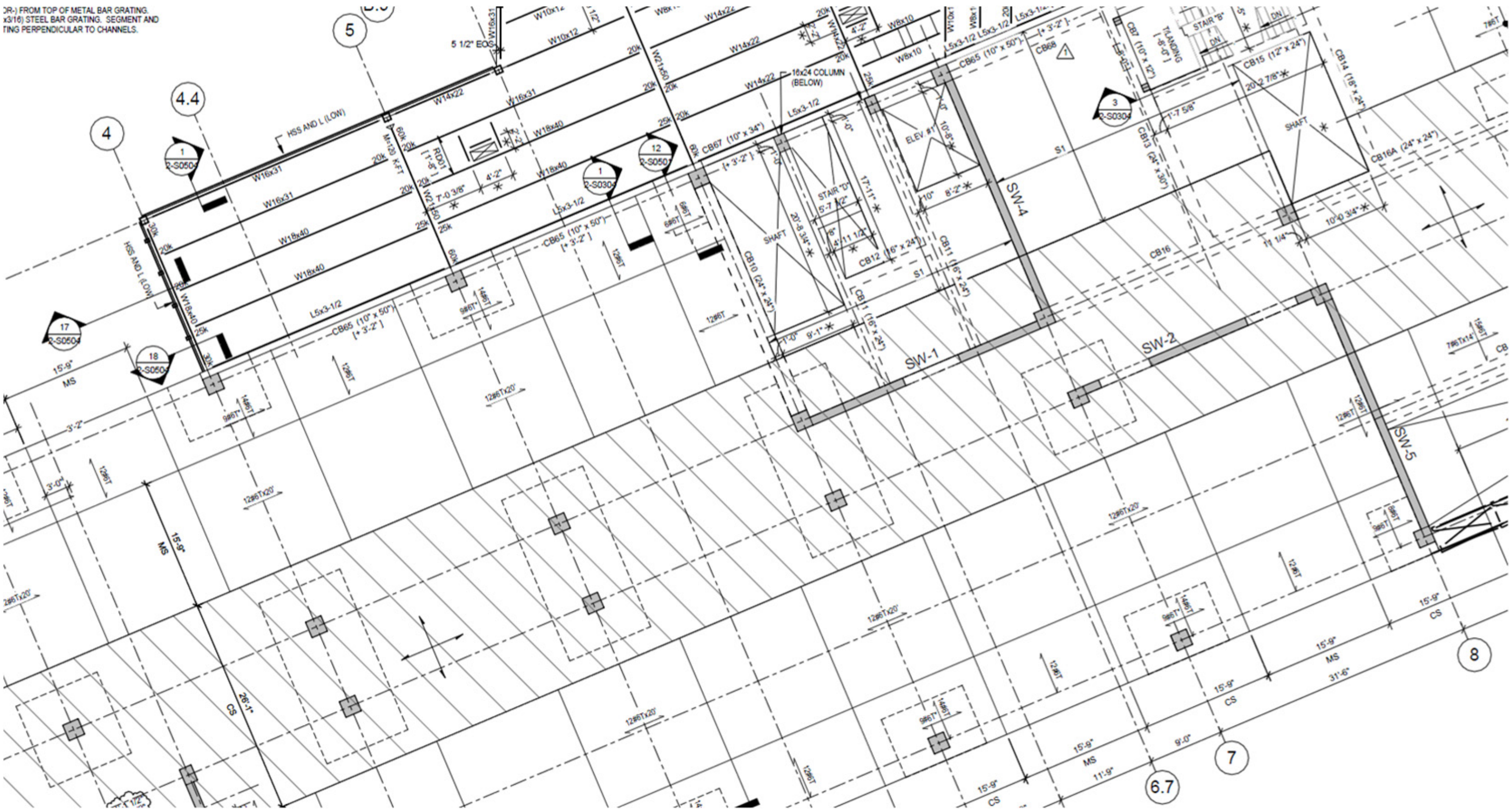
30' × 30' bay for a total Load of 194 psf

Cost ≈ $18 / square ft. (material + Labor)

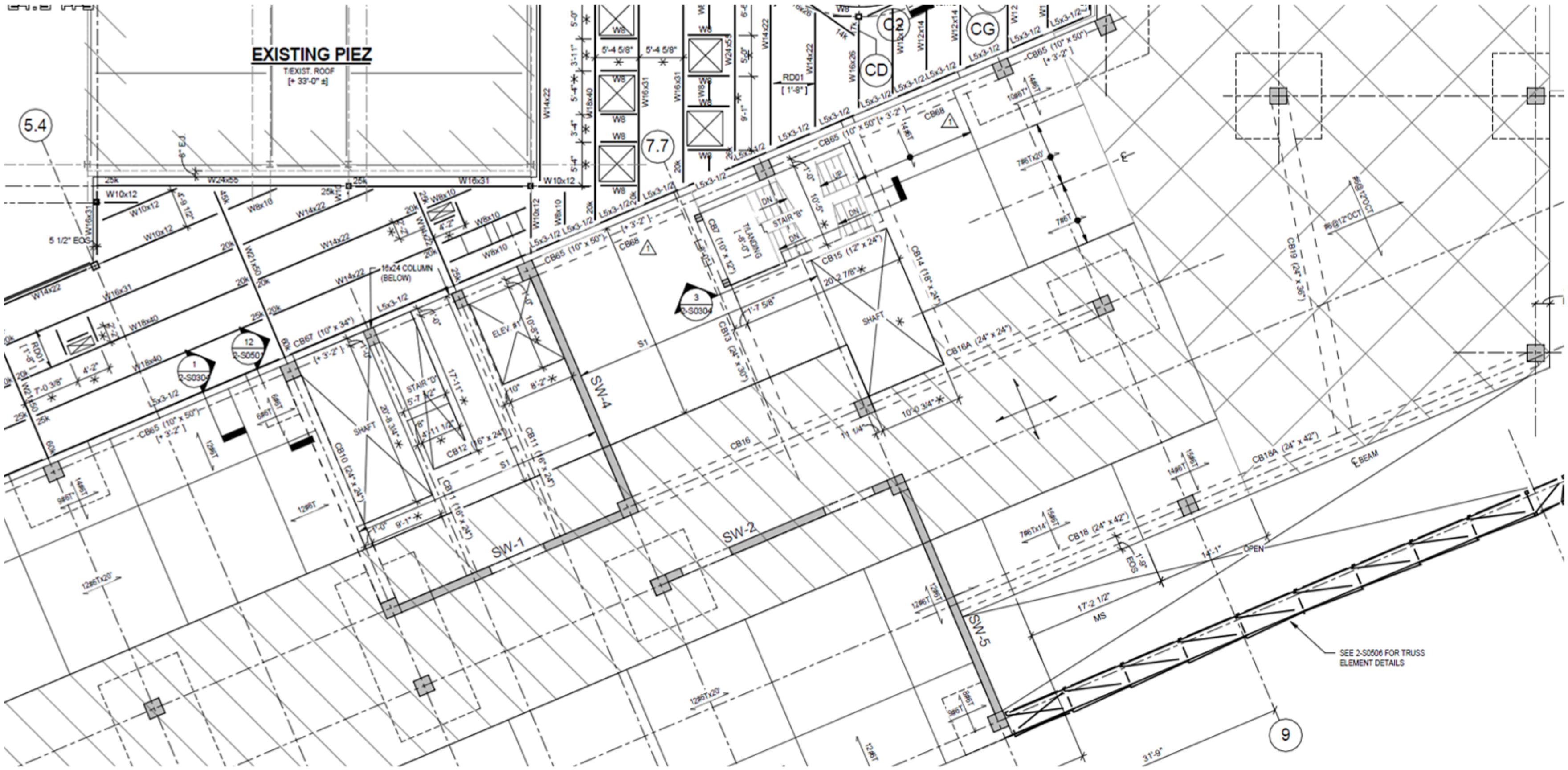
Appendix E: Typical Plans

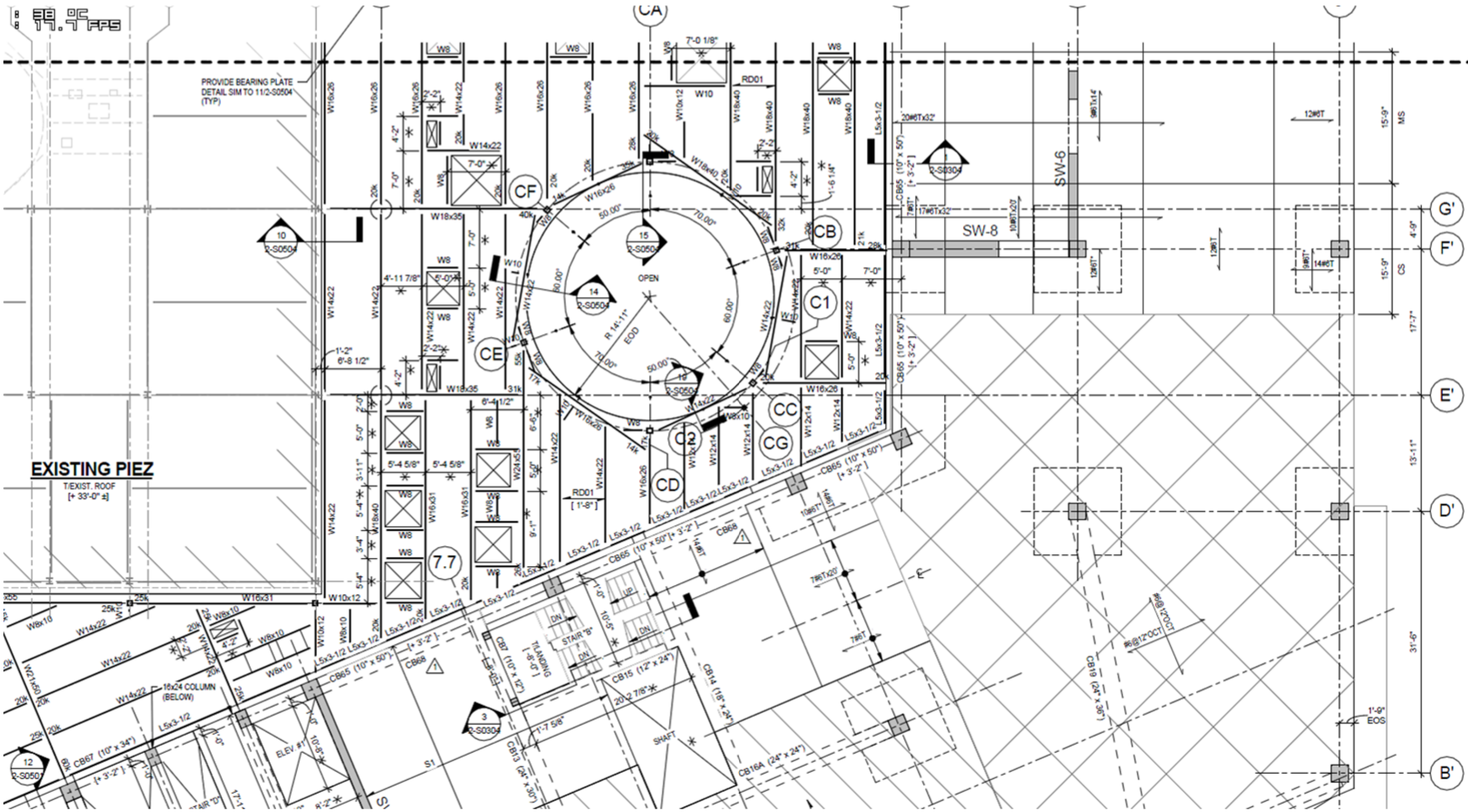
← N

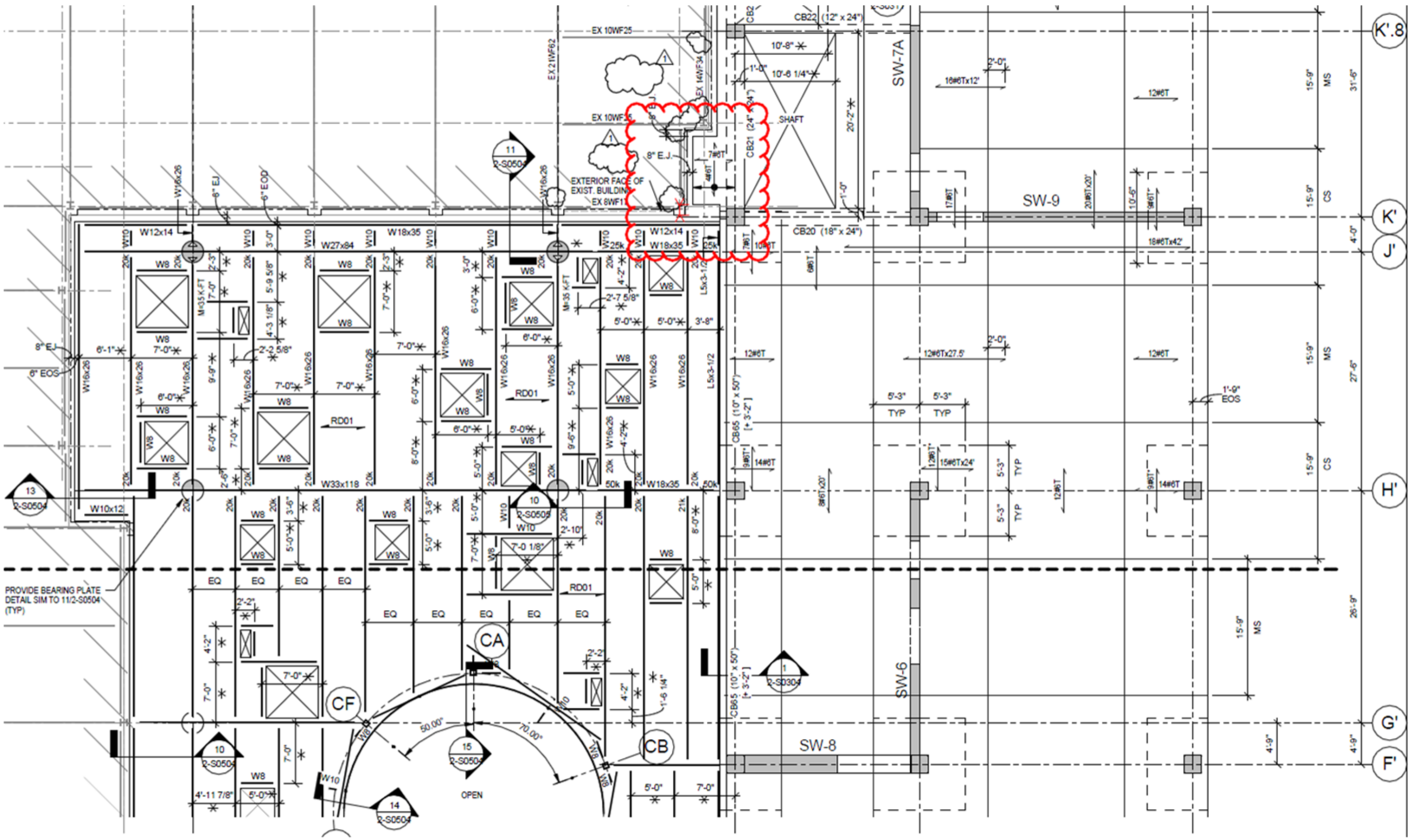


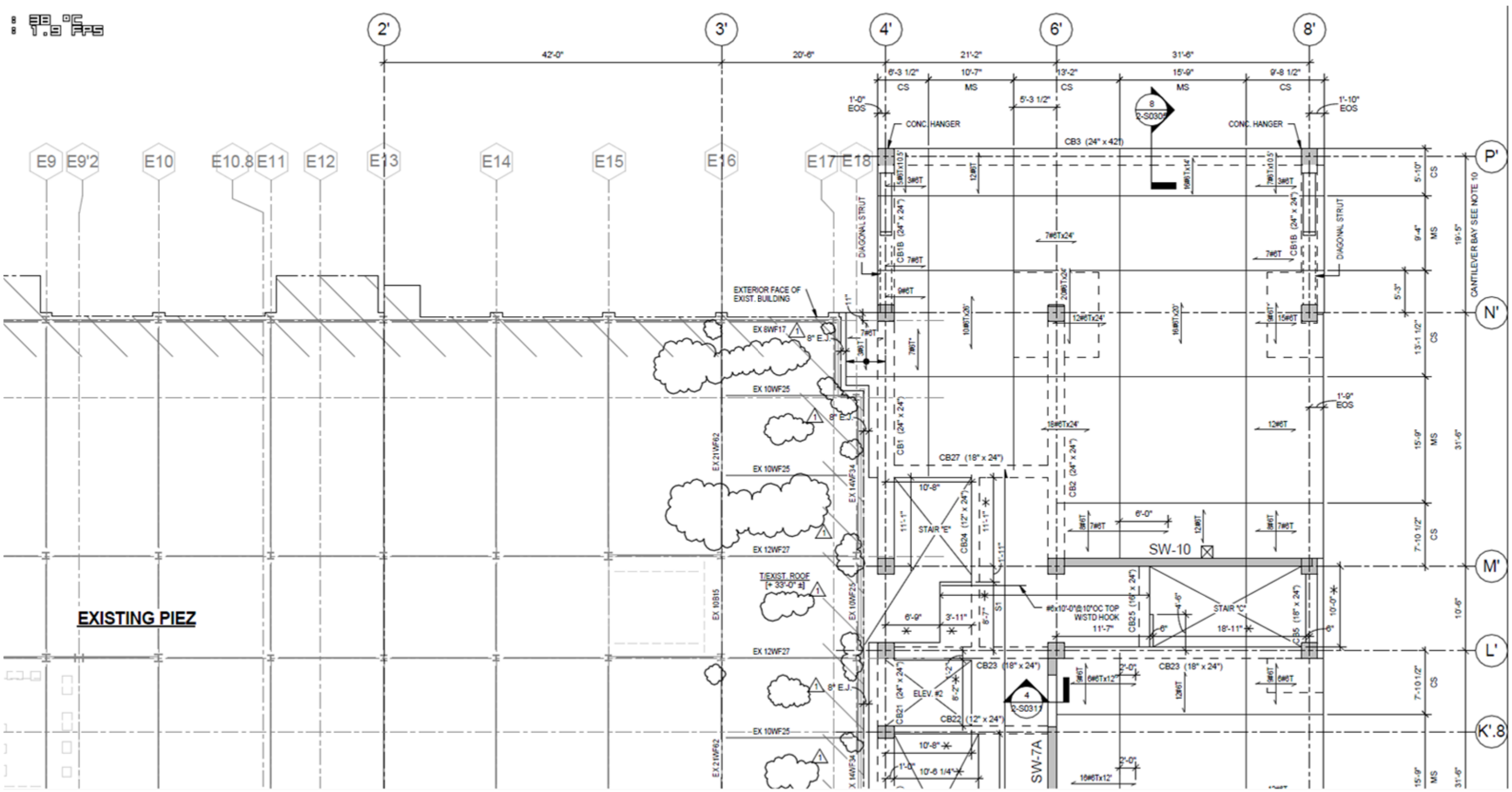


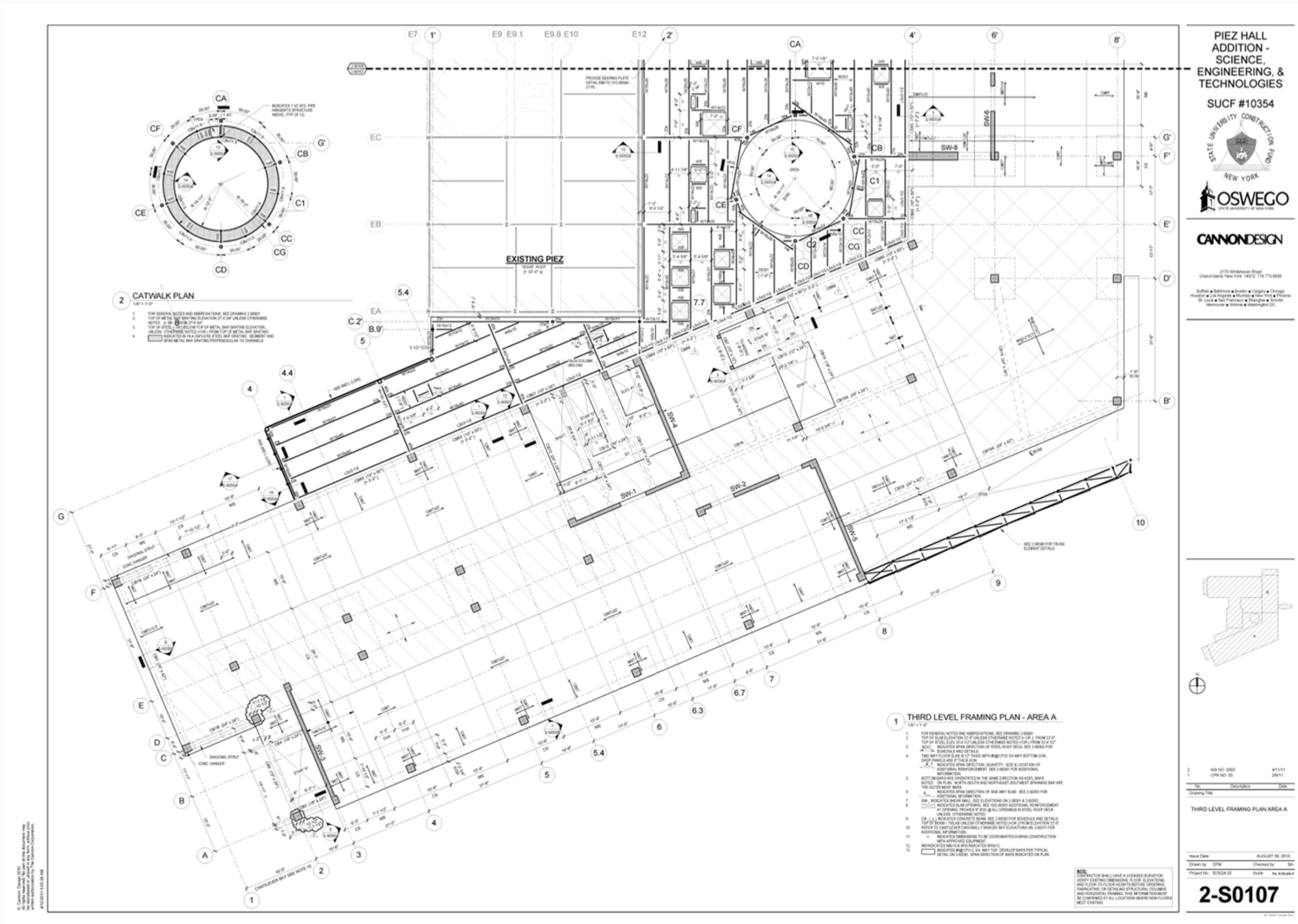
SW-1
SW-2
SW-4
SW-5
4
4.4
5
6.7
7
8

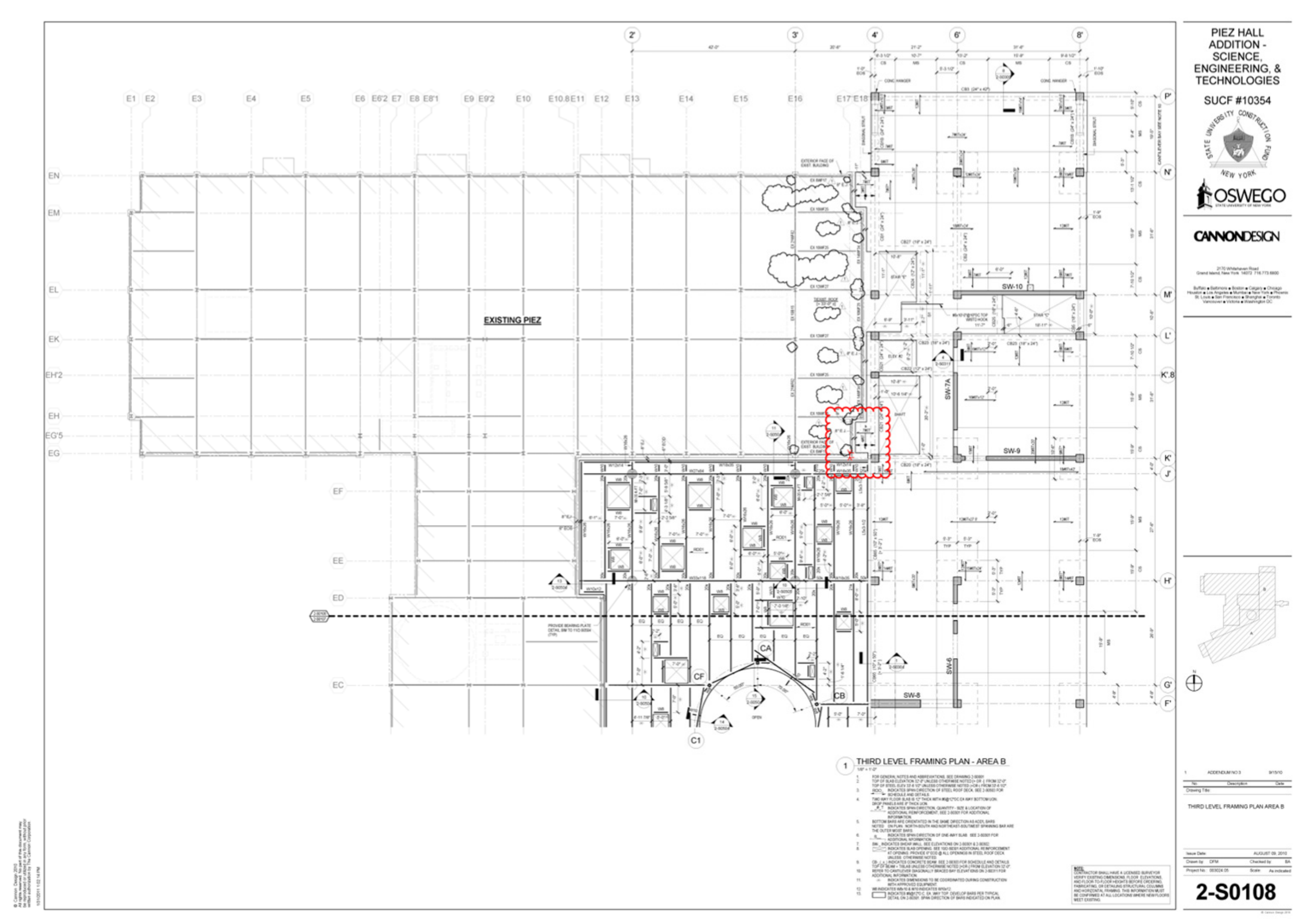

EXISTING PIEZ
SW-1
SW-2
SW-4
SW-5
SEE 2-S0506 FOR TRUSS ELEMENT DETAILS

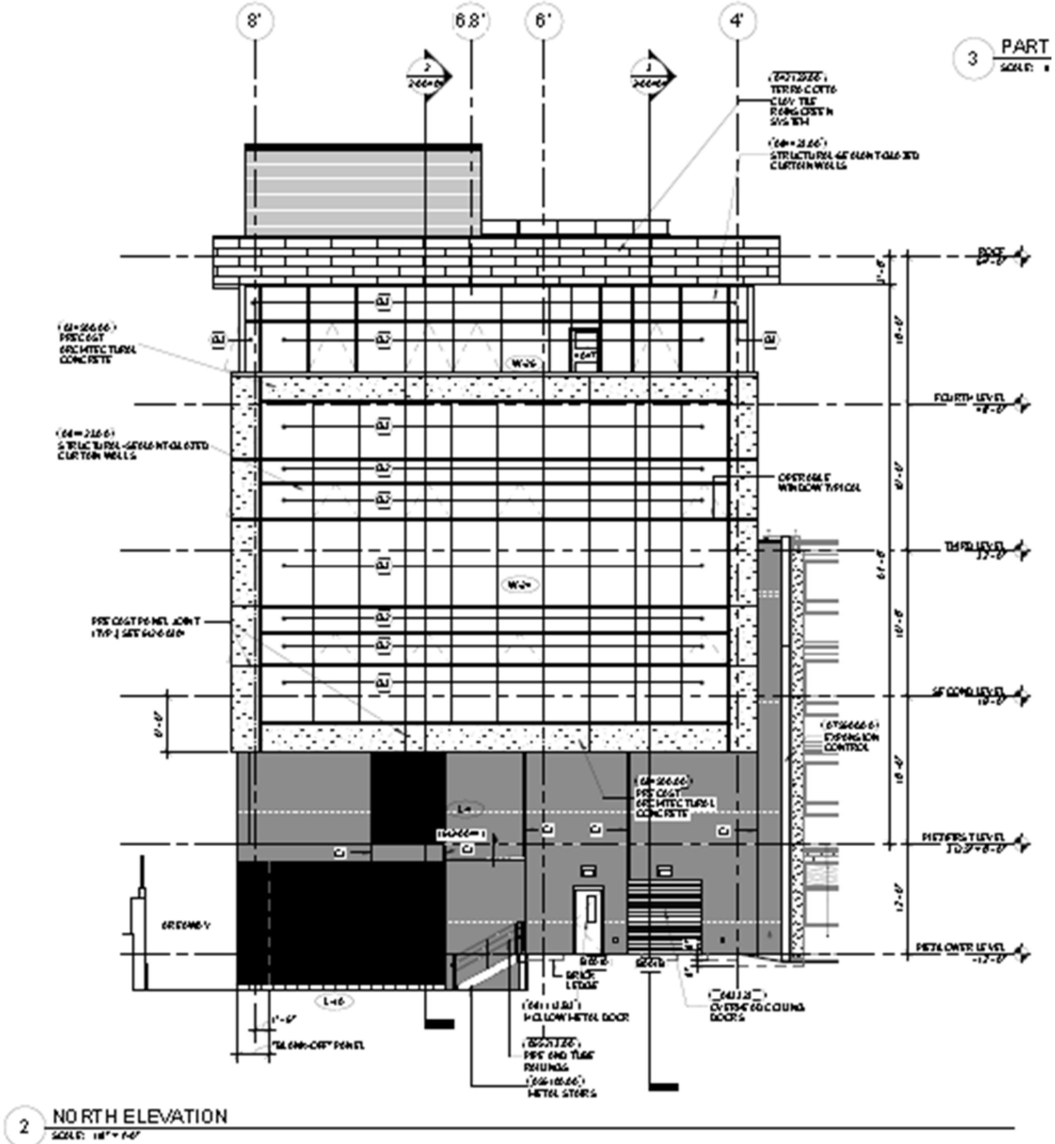

EXISTING PIEZ
T/EXIST. ROOF
[+ 33'-0" ±]
PROVIDE BEARING PLATE
DETAIL SIM TO 11/2-S0504
(TYP)
SW-6
SW-8
OPEN
R 14'-11"
EOD
CA
CB
CC
CD
CE
CF
CG
C1
C2
7.7
G'
F'
E'
D'
B'
CB19 (24" x 36")
CB14 (18" x 24")
CB16A (24" x 24")
CB15 (12" x 24")
CB13 (24" x 30")
CB7 (10" x 12")
CB67 (10" x 34")
SHAFT
ELEV #1
16x24 COLUMN
(BELOW)

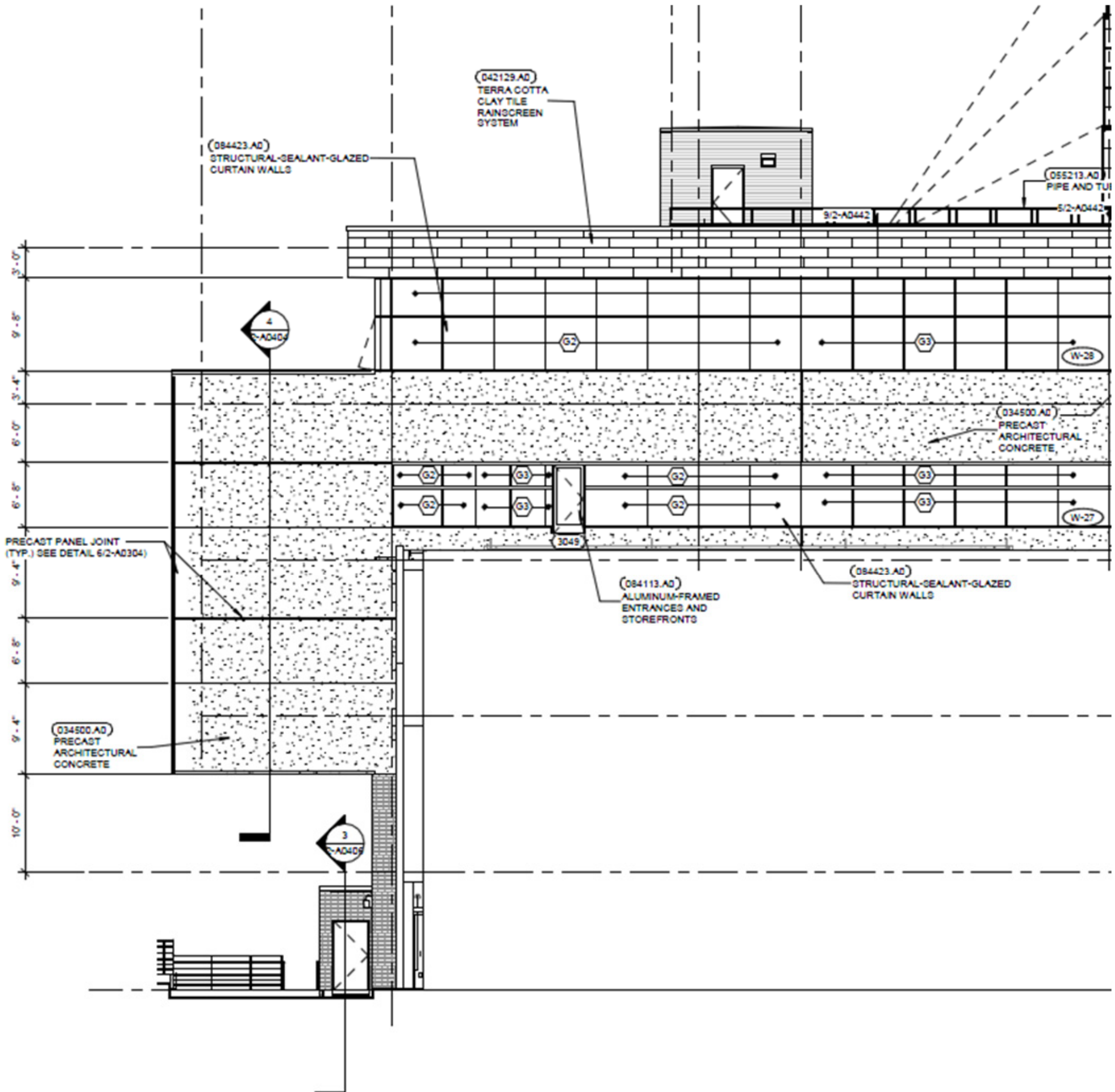



2'
3'
4'
6'
8'
P'
N'
M'
L'
K'.8
E9
E9'2
E10
E10.8
E11
E12
E13
E14
E15
E16
E17
E18
EXISTING PIEZ
EXTERIOR FACE OF EXIST. BUILDING
CONC. HANGER
DIAGONAL STRUT
CANTILEVER BAY SEE NOTE 10
CB3 (24" x 42")
CB1B (24" x 24")
CB1 (24" x 24")
CB27 (18" x 24")
CB2 (24" x 24")
CB24 (12" x 24")
CB25 (18" x 24")
CB23 (18" x 24")
CB21 (24" x 24")
CB22 (12" x 24")
CB5 (18" x 24")
STAIR "E"
STAIR "C"
ELEV. #2
SW-10
SW-7A
T/EXIST. ROOF
EX 21WF62
EX 10B15
EX 14WF34
EX 10WF25
EX 12WF27
EX 8WF17
8" E.J.
42'-0"
20'-6"
21'-2"
31'-6"
CS
MS


PIEZ HALL ADDITION - SCIENCE, ENGINEERING, & TECHNOLOGIES
SUCF #10354
STATE UNIVERSITY CONSTRUCTION FUND
NEW YORK
OSWEGO
CANNONDESIGN
EXISTING PIEZ
2 CATWALK PLAN
1 THIRD LEVEL FRAMING PLAN - AREA A
THIRD LEVEL FRAMING PLAN AREA A
2-S0107

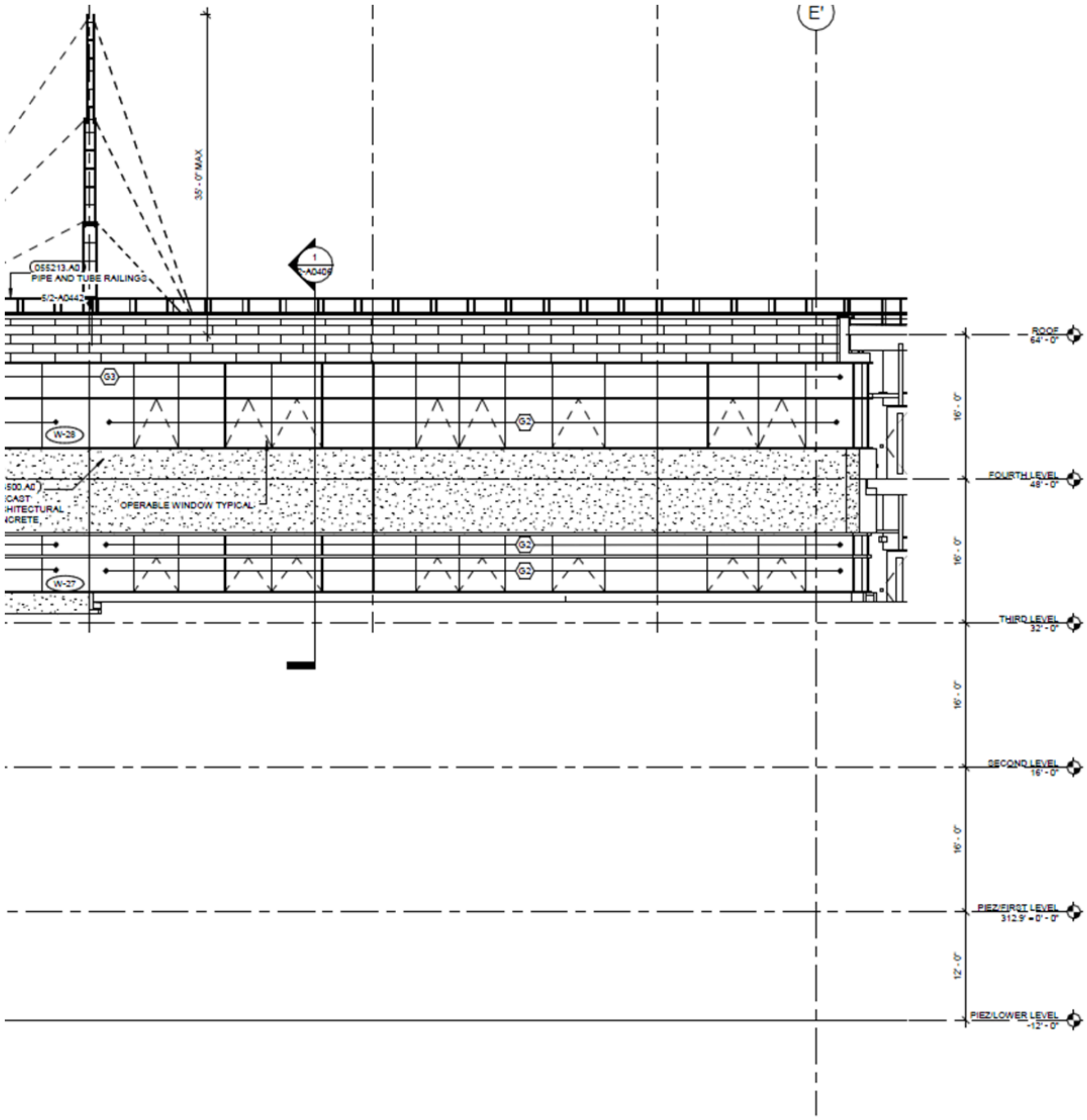

PIEZ HALL ADDITION - SCIENCE, ENGINEERING, & TECHNOLOGIES
SUCF #10354
STATE UNIVERSITY CONSTRUCTION FUND NEW YORK
OSWEGO
CANNONDESIGN
EXISTING PIEZ
SW-10
SW-7A
SW-9
SW-6
SW-8
CA
CB
CF
C1
1 THIRD LEVEL FRAMING PLAN - AREA B
THIRD LEVEL FRAMING PLAN AREA B
2-S0108


8'
6.8'
6'
4'
3
PART
2
NORTH ELEVATION



1 PARTIAL WEST ELEVATION

SCALE: 1/8" = 1'-0"


E'
PIPE AND TUBE RAILINGS
35' - 0" MAX
G3
W-28
G2
OPERABLE WINDOW TYPICAL
W-27
ROOF
64' - 0"
FOURTH LEVEL
48' - 0"
THIRD LEVEL
32' - 0"
SECOND LEVEL
16' - 0"
PIEZ/FIRST LEVEL
312.9' = 0' - 0"
PIEZ/LOWER LEVEL
-12' - 0"
16' - 0"
12' - 0"