Technical Report II

Army National Guard Readiness Center
Arlington, Virginia

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EXECUTIVE SUMMARY

Technical Report II is a Pro-Con structural study, which analyzes and compares alternate floor systems for the Army National Guard Readiness Center Addition as well as the existing system. Research was performed into the existing floor framing system as well as three alternate systems. This report uses the current standards to check the existing design as well as determine possible designs for the alternate systems.

The 8-story joint headquarters building, located in Arlington, Virginia, is a concrete structure that utilizes two-way flat slab floor system with edge beams. Typical interior columns are 22" by 22" and are continuous on each floor with variations in reinforcement. Due to deviations in footprint between the subgrade levels and the tower levels, a 2" expansion joint is located in the 9" floor slabs on the subgrade levels allowing the tower component and plaza component to act separately. Using ACI 318-08 the column strip and middle strip reinforcement was determined and compared to the designed reinforcement using the Direct Design Method.

For this report, the floor systems that were analyzed as possible alternatives were:

- Hollow core precast planks on steel
- Composite steel
- Post-tensioned

Using current standards, designs for each system were determined and then advantages and disadvantages for each system were analyzed. All four systems were then compared to determine which system would most benefit the construction and design of the Army National Guard Readiness Center Addition. From this comparison, it was concluded that post-tensioning was the most advantageous for this building. It reduced the slab thickness, ultimately increasing the floor-to-ceiling height and decreasing the total building weight. Reducing the building weight would benefit the foundation, and the slab would have a smooth ceiling finish and not entail any drop ceiling. This system does not require any additional fireproofing and the forces created by the stressed tendons would balance the live loads and dead loads on the slab allowing for longer spans and controlling deflection and limiting vibrations. When compared to the existing system, post-tensioning was the closest alternative and could easily be substituted since no column layout changes would be necessary and this system could be integrated with the concrete shear wall lateral system. There are disadvantages to this system as well; however, most were associated with the construction and stressing of the tendons. From this technical report, it was determined that post-tensioning could be a viable alternative to the existing system and further investigation will be done to determine if this would be appropriate as a possible proposal topic.

While the hollow core precast planks and the composite steel systems also had several advantages, they were outweighed by the disadvantages of both systems. The main disadvantage was the increase in the floor system which would greatly reduce the floor-to-ceiling height. It was determined from this technical report that neither of these systems would be potential alternatives to the existing two-way flat slab and no further research will be done for either of these floor systems.

INTRODUCTION

The Army National Guard Readiness Center headquarters addition is sited to the south of the existing facility, on the location where previous storm water retention pond was located. Due to the loss of the retention pond, the project also includes the installation of storm water detention tanks. The new building is 82 feet above grade and approximately 251,000 square feet. The contract value was $100 million and is a Design-Bid-Build project with Tompkins Builders, Inc., the general contractor, holding lump sum contracts with all subcontractors. The eight-story facility is comprised of 3 underground levels (Referred to as Levels 3P, 2P and 1P) and a 5 level tower component (Levels referred to as 1T – 5T) as well as a mechanical penthouse. The three underground levels account for the majority of the building's square footage, with a much larger footprint than the above ground floors. The underground levels encompass approximately 150,000 square feet and the five-story tower encompasses 100,000 square feet. This design was developed to increase the amount of green space since a large portion of the underground levels will be topped with an intensive green roof system.

The addition is designed to meet Department of Defense Anti-Terrorism and Force Protection Requirements. This required that physical security measures, such as internal bracing to prevent progressive collapse, blast walls, berms, bollards and heavy landscape, to have been integrated into the design of the building. The facility is also expected to achieve LEED Silver Certification. LEED points are anticipated through the green roof system, offering bicycle storage and changing rooms, low-emitting and fuel efficient vehicles, reduction of water usage, water efficient landscaping, use of low-emitting as well as recycled and regional materials, and creating office space that can be 75% daylight. The building will incorporate open office spaces, general office suites, conference rooms, specialized compartmented information facilities, a fitness center, small library, and an auditorium.

As a result of the location and the existing facilities that are on site, several other features have been incorporated into the project. This includes the installation of the storm water detention tanks, the relocation of an existing radio tower, relocation of existing gate, a one story bridge connecting the new facility with the existing headquarters, construction of a new mailroom, and a construction of a new multi-story parking facility. This report will focus on the new Army National Guard Readiness Center Addition and none of the other project features will be discussed or analyzed.

BACKGROUND

The Army National Guard (ArNG) Readiness Center is located at 111 South George Mason Drive in Arlington County, Virginia. The site is bordered on the east by the U.S. Department of State, National Foreign Affairs Training Center, on the north by Arlington Boulevard, on the west by George Mason Drive, and on the south by a residential community. The fifteen-acre site is comprised of a 248,000 square foot headquarters facility, two 3-story parking garages and several small out buildings.

The Army National Guard Readiness Center houses administrative and resource functions that provide support and liaison to the National Guard in all 50 states and requisite territories and to the Pentagon. Currently there is about 1,300 staff based at this facility. The 2005 Base Realignment and Closure Act (BRAC) actions required the realignment of Jefferson Plaza 1 in Crystal City by relocating National Guard Bureau Headquarters and Air Force Headquarters to the Army National Guard Readiness Center in Arlington and to Andrews Air Force Base, in Maryland. This means the relocation of more than 1,200 National Guard Bureau Joint Staff and Army National Guard Staff to relocate to the Readiness Center. This relocation has created a great need for a Readiness Center Addition. Due to the BRAC Requirements the 1,200 personnel must be relocated before 2011. This makes the construction schedule particularly crucial.

Figure 1: West Perspective

STRUCTURAL SYSTEMS

Foundation

The geotechnical report engineering survey was performed by CH2M Hill on April 21, 2008. In this study, it was found that a relatively high water level of approximately 6 feet to 10 feet below the existing surface was anticipated. As much as 35 feet of excavation was required to reach the building grades. Therefore, drilled in soldier piles with wood lagging and tied-back anchors was recommended for temporary excavation support as well as the installation of dewatering well points. CH2M Hill noted that, with proper ground water management and control, the existing subsurface is suitable for support of the building using a mat foundation system based on evaluation of allowable bearing capacity and anticipated settlement. The recommended allowable bearing capacity for the new building location was 4800 lbs/ft^2 for a mat footing. As a result, a 43-inch concrete mat foundation was designed.

Columns

A reasonably consistent column layout exists throughout the building even with the changes in the shape of the floors between level 3P and 1T. The typical interior gravity column is a 22-inch by 22-inch, reinforced normal weight concrete column. The strength of all columns is 4,000 pounds per square inch. While the size and shape of the column is the same on each floor, there are three changes in reinforcement. For levels 3P to 1P columns are reinforced with sixteen No. 10 vertical bars. These change after the 1P level where the tower component of the building begins. For levels 1T and 2T columns are reinforced with sixteen #8 vertical bars. The reinforcement changes again at the 3T level up to the 5T level; these columns are reinforced with eight #8 vertical bars. #3 ties are located 12 inches on center at every level.

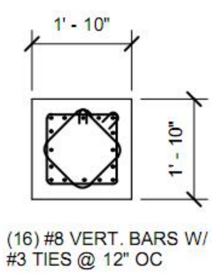


Figure 2: Typical Interior Column

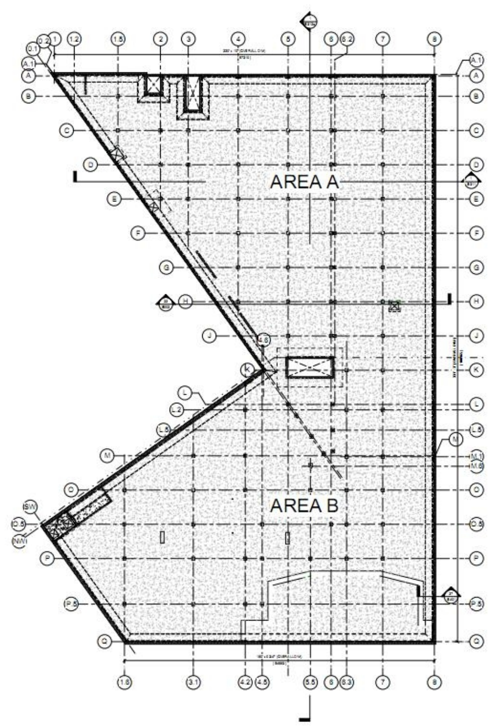


Figure 3: Typical Column Layout for Below Grade Levels

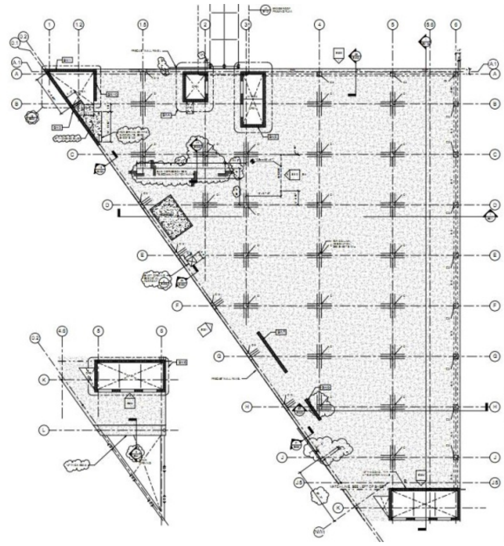


Figure 4: Typical Column Layout for Tower Levels

Floor Systems

The Army National Guard Readiness Center Addition utilizes a reinforced concrete structural system. All of the floors are two-way flat slab with column strips and edge beams along the eastern and northern walls of the Tower component. The typical concrete strength is 4,000 psi. The typical slab thickness is nine inches however; this changes in areas where the access flooring changes and for drainage areas in mechanical and electrical rooms. No. 6 and No. 8 bars are typically used for reinforcement in the floor systems.

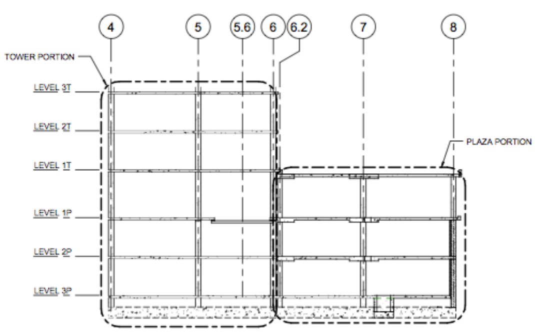


Figure 5: Elevation showing location of expansion joint and relationship between the Plaza portion and Tower Portion

Due to the irregular shape of the building and the change in shape from the underground portion of the building to the tower component, a two-inch expansion joint is located at the 3P to 1T levels along column line 6.2. This expansion joint makes the building act as almost two separate building, the tower portion and the plaza portion. The tower portion extends from level 3P to 5T while the plaza portion is comprised of the subgrade levels and topped of with an intensive green roof. This can be seen in figures 5 and 6.

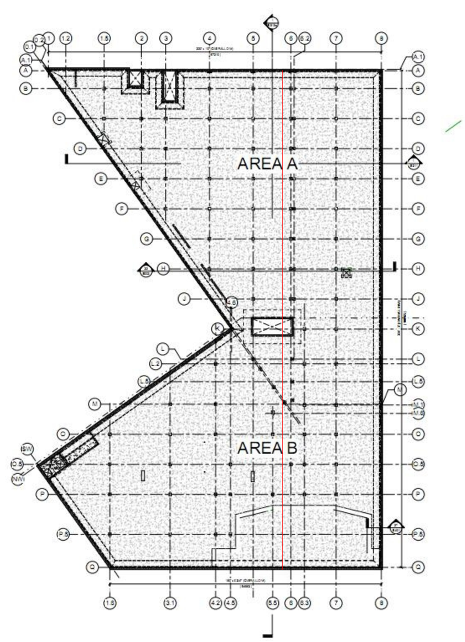


Figure 6: Location of Expansion Joint

Roof Systems

The penthouse roof of the tower is a two-way flat slab. The slab is 10” thick with a concrete strength of 4,000 pounds per square inch. This roof was designed to hold a 30 pounds per square foot snow load and is reinforced with #5 bars at 12 inches on center and 18 inches on center. A large skylight over the northern stairs required steel framing, which consists of beams ranging from W12x14 to W12x26.

The plaza roof is also a two-way slab with drop panels. The slab thickness ranges from eight inches to sixteen inches with a concrete strength of 4,000 pounds per square inch. This roof will act as an intensive green roof and therefore had to be designed to carry a 100-pound per square foot roof garden load. It is reinforced with #6 bars and includes a two-inch expansion joint where the roof abuts the floor of the first tower level (1T), as do the floors below.

Lateral System

The lateral system for the ArNG Readiness Center consists of reinforced concrete shear walls. These walls have a thickness of twelve inches and a concrete strength of 4,500 pounds per square inch. The numbers of shear walls varies between levels due to the building’s change in footprint. Typical shear wall locations can be seen in figures 10 and 11 below. This system resists lateral loads in the north-south and east-west direction depending upon the orientation of the wall.

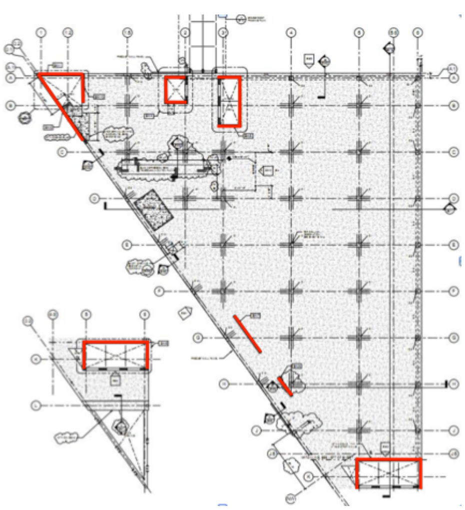


Figure 8: Shear Wall Locations in Tower Levels

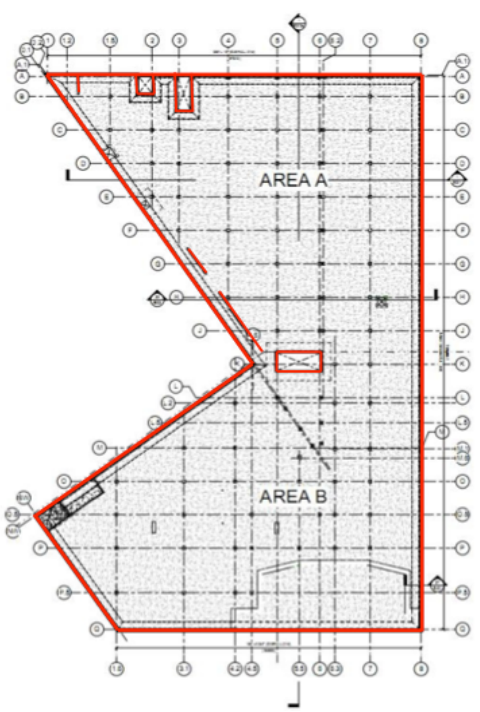


Figure 7: Shear Wall Locations in Levels 3P to 1P

DESIGN & CODE REVIEW

Codes and References

The following documents were either furnished for review or otherwise considered for this report:

- ACI 318-08 *Building Code Requirements for Structural Concrete* published in January 2008 by the American Concrete Institute

- AISC 13th Edition (LRFD) *Steel Construction Manual* Published in December 2005 by the American Institute of Steel Construction, Inc.

- ASCE/SEI 7-05 *Minimum Design Loads for Buildings and Other Structures* published in 2006 by the American Society of Civil Engineers

- IBC 2006 *International Building Code* published in January 2006 by the International Code Council, Inc.

- *Notes on ACI 318-08 Building Code Requirements for Structural Concrete* Published in 2005 by the Portland Cement Association

- PCI 6th Edition *Design Handbook* published in 2003 by the Precast/Presterssed Concrete Institute

- Construction Documents originally dated August 25, 2008 by DMJM H&N, Inc.

Deflection Criteria

Floor Deflection Criteria

Typical Live Load Deflection limited to L/360

Typical Total Deflection limited to L/240

Maximum Deflection limited to ¾”

Lateral Deflection Criteria

Total Allowable Wind Drift limited to H/500

Total Story Wind Drift limited to H/400

Total Allowable Seismic Drift limited to $0.015h_{SX}$

Material Specifications

These materials, their grades, and strengths were the materials that the current Army National Guard Readiness Center Addition is utilizing. All materials were listed on the drawings, general notes, of the specifications. These materials area summarized in table 1.

Table 1: Material Properties

Material	Grade	Strength
Concrete		
Foundation	-	f'c=4,500 psi
Slab on Grade	-	f'c=4,000psi
Columns	-	f'c=4,000psi
Shear Walls	-	f'c=4,500 psi
Floor Slabs	-	f'c=4,000psi
HSS Rectangular	A500 - Gr. B	fy=46,000 psi
HSS Circular	A500 - Gr. B	fy=46,000 psi
Reinforcing Bars	ASTM 615 - Gr. 6	fy=60,000 psi
Steel Deck	ASTM A625 - Gr. 33	fy=33,000 psi
CMU	Type 1 - Gr. N Med Wt	f'm=1,500 psi
Grout	C270 Type S	-

LOADS

Live Loads

The live loads for the Army National Guard Readiness Center were calculated in accordance with IBC 2006, which references ASCE 7-05, Chapter 6. The loads that were determined from these references are noted in Table 2 below.

Table 2: Live Loads		
Occupancy	**Design Load**	**ASCE 7-05 Loads**
Offices	50 psf + 15 for partitions	50 psf
Lobbies	100 psf	100 psf
First Floor Corridors	100 psf	100 psf
Corridors (Above First Floor)	80 psf	80 psf
Fitness Center	100 psf	100 psf
Roof	20 psf	20 psf
Roof Garden	100 psf	100 psf

Dead Loads

The dead loads used for the design of the Army National Guard Readiness Center were noted on the structural drawings for this project. These occupancy types and loading are summarized in Table 3 below.

Table 3: Dead Loads	
Typical Floor Dead Loads	
Occupancy	Design Loads
6" Raised Floor	43 psf
24" Raised Floor	20 psf
Normal Weight Concrete	150 pcf
MEP/Celing	15 psf
CMU Partitions	Actual Weight
Typical Roof Dead Loads	
Occupancy	Design Loads
Normal Weight Concrete	150 pcf
MEP/Celing	15 psf
Roofing Finish	4 psf

Existing Floor System – Two Way Reinforced Flat Slab

Properties

9 inch Slab (NWC)
22”x22” Columns
f'_c = 4,000 psi
f_y = 60,000 psi

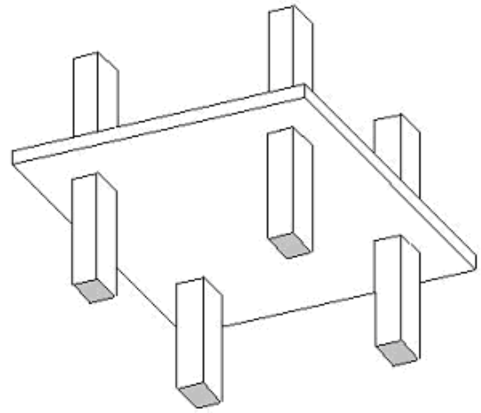


Figure 9: Flat Slab System

Description

The existing floor system designed for the Army National Guard readiness Center is a two-way reinforced flat slab system. This system includes a 9” thick slab that contains #6 reinforcement bars at 18 inches on center. To resist increased moment at supports, additional reinforcement is included. The added bars are #6 and spacing varies depending on the magnitude of the moments.

An analysis of this floor system was completed using typical interior bays at the 3T level using the Direct Design Method and loads determined by ASCE 7-05. The typical bay was split into two frames, Frame A and Frame B. The frames were checked for minimum slab thickness and reinforcement design. Both the calculated minimum thickness and reinforcement requirements were less than the 9” slab thickness and the reinforcement designed. The typical bay that was analyzed was an interior 26’x20’ bay. Calculations reviewing the wide beam (one-way shear) and punching shear (two-way) within the slab were also completed. Neither proved to be an issue and no additional shear reinforcement was required. Deflection calculations were also completed and found to be within the l/480 limits for long-term deflection. All supporting calculations can be referenced in Appendix B.

Advantages

The use of a flat slab system for the Army National Guard Readiness Center Addition was a likely choice because it is both advantageous and economic. The smooth concrete slab allows for aesthetically pleasing exposed ceilings due to the limited amount of beams and drop panels penetrating the area. This smooth slab can also reduce the floor to ceiling heights when compared to a steel frame system. A flat slab system is very common, especially within the Washington D.C. area, and easy to construct. The concrete slab has a 2-hour fire rating and therefore no additional fireproofing is required for this system.

Disadvantages

As with all systems, there are also a few disadvantages to using flat slabs. The number one issue with this type of floor system is shear concerns. In particular, punching shear can be a problem where there is a transfer of moments from the slab to the column. Fortunately, this was not a problem in the Army National Guard Readiness Center due to the 9" slab thickness that was required for the 25' span. Another disadvantage to this system is the constraint on span lengths. Only moderate spans, however, were needed for the Army National Guard Readiness Center therefore this was not an issue. One of the disadvantages of this system during construction is the labor and cost that is required to prepare the formwork, which would not be required for other floor systems.

Alternative Floor System 1 – Hollow Core Precast Panels on Steel

Properties

4'-0" x 6" Panels
2" Normal Weight Concrete Topping
f'_c=5,000 psi
Tendons – 66-S

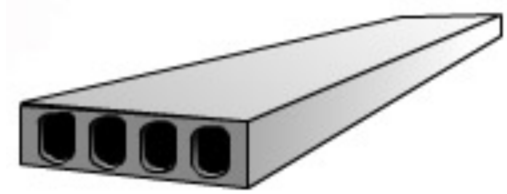


Figure 10: Hollow Core Precast Panel

Description

Due to the precast panels coming in 4'-0" segments it was necessary to adjust the bay sizes for this particular system. It was determined that a 28'-0"x20'-0" would be more logical for this system rather than the existing 26'-0" x 20'-0" bays. For this analysis, an interior bay of 28'-0"x20'-0" was used. At this time columns have not been designed.

The *PCI Design Handbook* was used to select a 6" thick plank with a 2" topping for this bay. 66-S strands were used within the hollow core planks to achieve the required span. The 66-S designation refers to the number of strands (6), the diameter of the strands in sixteenths (6/16), and the strands are to be straight (S).

It was determined using the AISC *Steel Construction Manual* (13th Edition), the beams that precast planks will frame into are sized as W18x55. All supporting calculations can be found in Appendix C.

Advantages

There are numerous benefits to using hollow core precast planks. The system requires low maintenance and is extremely durable. The construction is quick and can allow for early completion and the possibility to fast track the project. Construction can also be completed year round since no curing time is required and the need for expensive formwork is eliminated. A Hollow core precast plank system is also recognized as a LEED rated system and is very efficient for noise attenuation.

Disadvantages

While there are several advantages to using hollow core planks there are also multiple disadvantages. The bay sizes and column layout would need to be drastically rearranged to fit the 4'-0" increments of the typical precast panels. This could result in an unacceptable building size and floor layouts. It is also not appropriate for irregular shaped buildings due to regular shaped panels. A large increase in the floor system depth would also be a problem, as is the case with any steel system. A 25.7" deep floor system would drastically change the floor to ceiling height when compared to the existing system. While the precast panels have a quick lead-time, the time for the steel is longer to account for fabrication, detailing, and transportation. At this time, the vibration associated with this system is unknown. There is also concern with connections to the shear wall that could have unknown impacts on design and cost.

Feasibility

At this time it does not seem that this system would be appropriate for any further investigation. The disadvantages of this system for the Army National Guard Readiness Center far outweigh its benefits.

Alternative Floor System 2 – Composite Steel

Properties

3 ½" Concrete Slab
Metal Deck: 18 Gage 2VLI18
f'_c=3,000 psi
f_y=60,000 psi

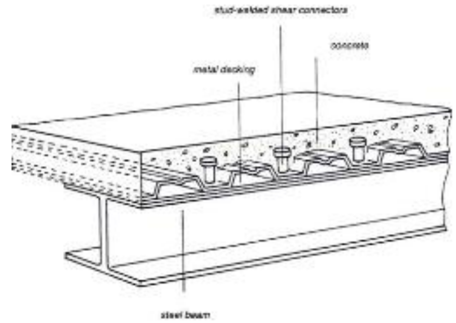


Figure 11: Composite Steel Deck Floor System

Description

The composite deck system was designed using a typical interior panel that was 20'-0"x26'-0". Intermediate beams were placed at 8'-8". Using the Vulcraft website, a Vulcraft 2VLI18 composite deck can span 10'-8" unshored for a 3 span condition, which is greater than the 8'-8" spacing designed for this analysis. A minimum 3¼" concrete topping above the metal deck is required to achieve a 2-hour fire rating according to ANSI/UL 263. A 3½" topping was used for this design and results in a total depth of 5½" slab depth when accompanied with the deck. This design meets the load and deflection requirements.

The AISC *Steel Construction Manual* was used to size the beams and girders as well as the shear studs for this system. It was determined that a W14x26 was a sufficient size for the beams with sixteen ¾" diameter shear studs. While deflection controlled for the design of the girders, it was concluded that a W16x31 with twenty shear studs and a ½" camber would be sufficient. At this preliminary stage columns have not yet been designed. Calculations for the slab and framing can be referenced in Appendix D.

Advantages

There are numerous benefits to using this system including the speed and efficiency of erection. Also, the expensive and time-consuming preparation of formwork is eliminated with this system. With the appropriate concrete thickness, additional fireproofing is not required either. This system also makes it easy for trade coordination and layout of other building systems due to the service plenum that will be created by the drop ceiling.

Disadvantages

As with all steel systems, composite steel decking is a much deeper floor system when compared to concrete systems. If the beam or girder members get too deep they could significantly decrease the floor to ceiling height in the building. In the case that the steel members do get too large it could also create other issues. Deep members would cause the system to become heavier than other systems which would result in problems in seismic regions as well as areas with weak soil. Deep members would also make trade coordination much more complicated.

Feasibility

At this time it does not seem that this system would be appropriate for any further investigation. The disadvantages of this system for the Army National Guard Readiness Center far outweigh its benefits.

Alternative Floor System 3 – Post Tensioned Slab

Properties

7" Normal Weight Concrete Slab
f'_c=5,000 psi
f'_{ci}=3,000 psi
Unbounded Tendons
½" Diameter, 7-wire strands
f_{pu}=270,000 psi

Figure 12: Post-Tension Tendons

Description

The post-tensioned system was designed for a typical 26'-0"x20'-0" panel for the Army National Guard Readiness Center Addition. This system would implement typical 7-wire strand, ½", unbonded tendons. Using a span to depth ratio of L/45, it was determined that a seven inch slab would be adequate for this system. Calculations were completed which resulted in 8 tendons at 26.78 kips/tendon would work for this floor. Design criteria, such as stress limits, were then checked using provisions from ACI 318-08 Chapter 18.

Neither deflection not vibration calculations were completed for the post-tensioned system due to the complexity and unfamiliarity. However, post-tensioned systems are known to perform extremely well under deflection due to the balanced loads from the stress in the tendons. Appendix D contains supporting calculations for the post-tensioned system.

Advantages

Post-tensioned systems allow for longer spans and thinner slabs than other systems. The seven inch post tensioned system is actually thinner than the existing flat slab floor system. Similar to the existing design, there would be a smooth, clean concrete surface at the ceiling as well. Deflection is reduced by the balanced load provided by the tendons and the rigidity and denseness of the post-tensioning limit the effect of vibrations. This system could potentially fit right into the Army National Guard Readiness Center Addition without adjusting the layout and would increase the floor-to-ceiling height.

There are several other benefits to implementing a post-tensioned system. The normal weight concrete slab is 2 hour fire rated and would not require any additional fireproofing. This system also reduces the amount of mild steel reinforcement that is required. A short lead-time is associated with this construction and also allows for easy trade coordination, which could decrease construction time. Finally, the thinner floor slabs could result in an overall lighter system, especially compared to deep steel construction. This could be a benefit in seismic regions and in locations with weak soils.

Disadvantages

While this system seems to be one of the most advantageous systems for the Army National Guard Readiness Center Addition, there are several disadvantages and concerns associated with this system. Most of the negative characteristics are related to the construction process of this system. Not only is the tendon laying process extremely labor intensive and lengthy, the process of jacking the tendons to reach the required strength is dangerous and should only be done by contractors with experience. The jacking must be completed at a consistent rate and if it is not jacked properly or the tendons are placed incorrectly there is a possibility for a tendon to snap and rupture through the concrete. This would not only cause a major delay to the completion of the building to repair the concrete, it can also be extremely dangerous to any people around the tendon when it snaps. Due to the added risk during construction of this system there are several extra safety procedures that would be required on the jobsite.

General construction issues also include the requirement of the formwork and shoring. It would also be very difficult to cut any openings in the floor after the concrete is poured in fear of cutting the stressed tendons located in the slab. There are also issues with shrinkage at corners, which could cause cracking after the tendons are stressed. Construction would also be difficult for underground levels with the post-tensioning.

Feasibility

This floor system seems to be promising when considering alternate framing systems for the Army National Guard Readiness Center Addition. There are several concerns in regards to construction and whether it would be feasible with the three levels below grade. However, there is potential for this system with further investigation and understanding of these issues.

Comparison of System

Comparison of Systems				
	Two-Way Flat Slab	**Hollow Core Precast**	**Composite Steel**	**Post-Tensioned**
Slab Weight	112.5 psf	74 psf	45 psf	100 psf
Slab Depth	9"	8"	3.5"	7"
System Depth	9"	25.7"	22.2"	7"
Vibration Contral	Great	Further Study Needed	Moderate	Good
Fire Rating	2 Hour	1.5-2 Hour	1.5-2 Hour	2 Hour
Additional Fire Proofing	No	No	Spray	No
Construction Difficulty	Medium	Easy	Easy	Medium/Hard
Impact on Column Grid	None	Significant	Little	No
Floor-to-Ceiling Height Impact	None	Decrease	Decrease	Increase
Lead Time	Short	Long	Long	Short
Cost	$19.96/SF	$32.80/SF	$24.50/SF	$21.20/SF
Feasibility	Existing	Least Possible	Possible	Most Possible

When comparing the aforementioned systems as possible alternatives for the Army National Guard Readiness Center Addition, there were several factors that were considered in order to determine the effectiveness in each system. This criterion includes: system depth, foundation changes, fire rating, vibration and deflection control, lead-time, layout changes, constructability, and cost. Each of these factors were considered based on the preliminary analysis for the existing floor system and each of the three alternatives.

Weight

The weight of the system is one of the most crucial factors because it dictates many others, such as vibration, cost, and foundation changes. It can be seen in the table above that the composite system weighs the least while the flat slab existing system weighs the most. Normal weight concrete was used throughout the design of each system in order to conform to the initial design assumptions. It is not surprising that the precast plank system and the post-tensioned system are medium weight since the system thicknesses are average amongst the four.

Depth

The post-tensioned system would allow for a thinner slab than the existing two-way flat slab system at 9". This would increase the floor-to-ceiling heights in the Army National Guard Readiness Center Addition. It would only effectively change the heights a minimal amount and while larger floor-to-ceiling heights may not be as imperative in such a building as in residential buildings, it remains an important consideration because it can ultimately affect several other aspects such as weight, cost, and deflection. While the composite system has the shallowest slab, the beams and girders are much deeper and would require some type of ceiling that would, in the end, increase the ceiling depth thus decreasing the floor-to-ceiling height significantly. The hollow core precast planks are only 6" but require deep steel members for support. Therefore, based on depth, it seems that the post-tensioned system would be the superior system.

Vibration/Deflection

Each of the floor systems, with the exception of the post-tensioned system, was designed to meet the L/320 and L/240 serviceability requirements. Deflection for post-tensioning is typically determined using computer programs. Due to inadequate knowledge with regards to the modeling of post-tensioned systems, deflection calculations were not performed. However, it is likely that this system will perform well for deflection due to the balanced moment.

Given time constraints and limited knowledge, vibration calculations were not performed for this analysis. However, it can be assumed that the more rigid and heavier floor systems would vibrate less since vibration is affected by the mass and stiffness of the beam and slab. Therefore, the concrete system will most likely perform better for vibration than the light steel systems.

Constructability

The steel composite systems would be easy to construct since it is a very common system and does not require formwork, saving both time and money. This system does have a longer lead-time and would require additional fireproofing. The precast hollow-core slab would also be very constructible; however, the Army National Guard Readiness Center site does not allow for a large amount of excess area to store the precast members. While the constructability of the post-tensioned system would not have been especially more difficult, it is much more dangerous and requires many more safety regulations on the jobsite. Lastly, the flat plate system does involve extra labor due to formwork and pouring. However, it can be easily constructed and was constructed rather quickly on the Army National Guard Readiness Center Addition.

Lead Time

For buildings that are fast-tracked, lead-time is especially important. While this was not the case for the Army National Guard Readiness Center Addition, it was important, as is always the case, in lowering labor costs. The existing system was a cast-in-place concrete system therefore; little to no lead-time was involved. The hollow-core precast and the composite steel systems would have had a significantly greater lead-time than the flat slab or post-tensioned systems. While the general lead-times for each system can be estimated, it is unknown at this time which system would ultimately be constructed the fastest, based on the experience of workers within the Arlington, Virginia area.

Layout Changes

The initial column layout of the building is regular and fits a grid-like pattern even with the irregular building shape. While further research into the architecture of the space is necessary, the hollow-core precast system would create significant changes within the column layout. This is due to the typical 4'-0" increments of the precast panels which would also cause a problem with the irregular shape of the building. The composite steel system would require slight changes to the column layout to provide more uniform bays. Post-tensioning would be the most beneficial system when it comes to layout changes since it is cast-in-place and does not require any changes to the existing column layout.

Foundation Changes

The Army National Guard Readiness Center Site contains relatively poor soil; therefore, changes in weight would affect the foundation design. Since the existing foundation was designed for the two-way flat slab system, there would be little affect on the foundation for the post-tensioning system, which is relatively similar to the flat slab system. However, the steel composite and hollow-core precast system could cause potential problems if the steel members were to get deep enough to significantly increase the weight. The heavier systems would likely need more pile caps and deeper foundations, which would prove to be very costly.

Fireproofing

Careful consideration was made in choosing the proper floor systems with appropriate two-hour fire ratings. Normal weight concrete slabs greater than 4 ½" were analyzed because they would not require any additional fireproofing. An additional 2" concrete topping on the hollow-core precast planks provided fireproofing for that system. The composite steel system would be the only system that requires additional, spray on fireproofing to achieve the proper fire rating. This would entail additional cost and labor; it is not significant enough to dictate which system should be chosen for the Army National Guard Readiness Center.

Cost

Using RS Means data, a provisional cost for each of the floor systems was determined. The hollow-core precast system is the most expensive and would only be even more inflated in order to incorporate irregularly shaped planks to fit the irregularly shaped building. The cheapest system was the 2-way flat slab system, which was extremely close to the post-tensioned system. This is not surprising since the tendons for the post-tensioning typically cost two to three times as much as regular steel reinforcement; however, the post-tensioning reduces the amount of steel required by nearly one-third. This was an extremely rough estimate of the cost of each system and more accurate information will be gathered from manufacturers in the Arlington, VA area in the future.

Conclusion

Throughout this technical report, the feasibility of each of the floor systems was discussed. It was important to consider a number of design factors such as cost, weight, possible layout and foundation changes, and the constructability of each system. The Army National Guard Readiness Center Addition utilizes a two-way, flat slab system. The ability to form concrete with the irregular shape of the building is mostly what made this system most appealing to the structural engineers.

After careful evaluation and comparison of the comparison three alternate systems, it seems that the post-tensioned system would be the most feasible alternate floor framing system for this building. There is concern with the post-tensioning with regards to the intense labor and risk associated with this type of system. However, with experienced contractors and proper understanding of how the system works, the construction could be successfully completed, relatively easily.

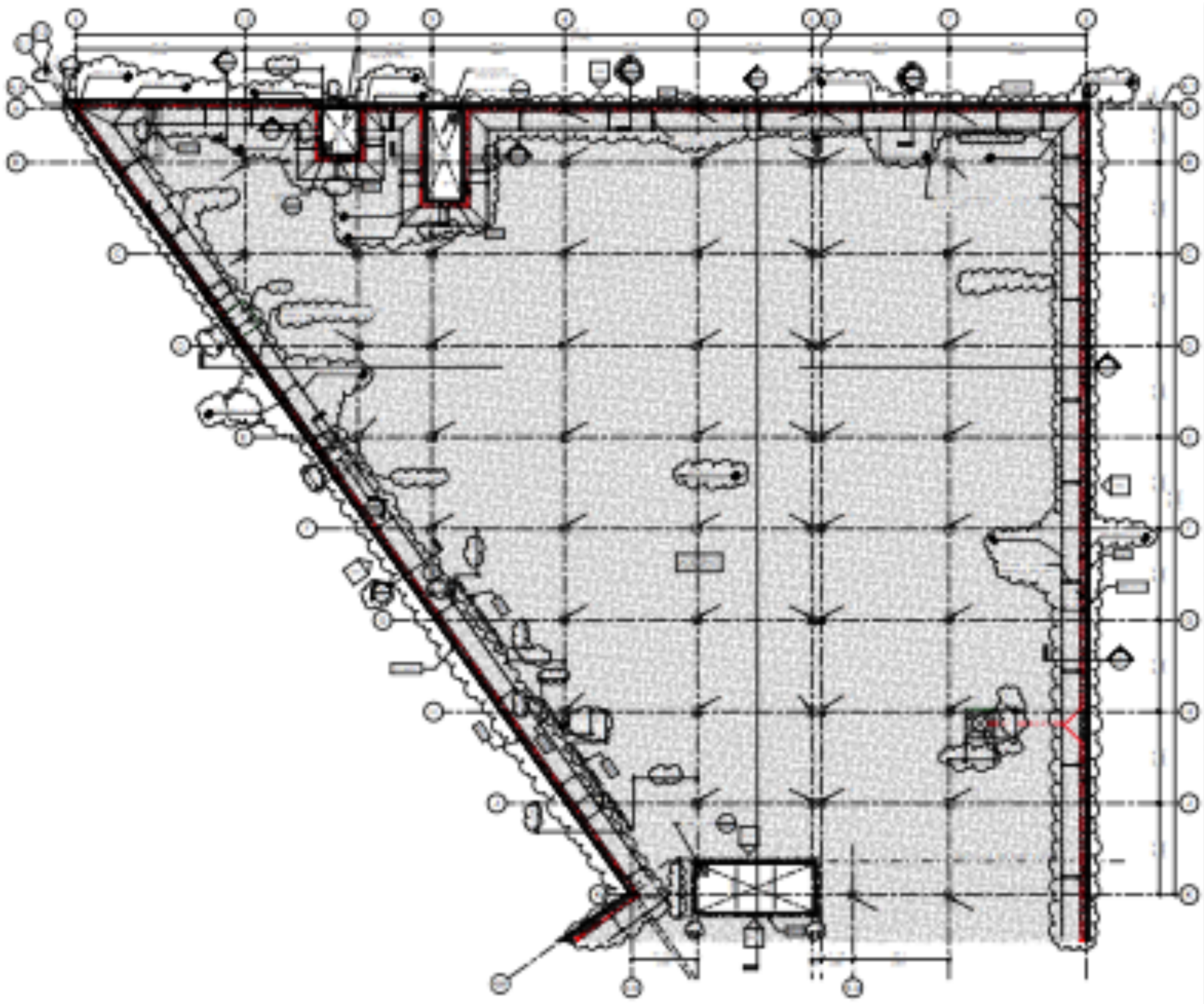
Several benefits could be gained if this system were utilized in the Army National Guard Readiness Center Addition. One of the major advantages to this system is the ability to increase the floor-to-ceiling height with a slab thinner than the existing 9" two-way flat slab. This system is also one of the most economical systems and would not require any additional formwork or fireproofing when compared to the existing flat slab. When compared to the existing floor-framing system it is also the only system that does not require any column layout changes or foundation changes since the building weight would decrease slightly. Finally, the stresses in the tendons balance the loads, which allows the slab to carry more live and dead load, longer spans, reduces vibrations, and limits deflection.

The composite steel and precast hollow-core planks on steel also had several benefits to their implementation into this building. However, these advantages were outweighed by the disadvantages in both cases. The main issue with both of these systems is the increase in the floor-to-ceiling height. Both systems are more than twice as deep as the existing flat slab and then would require a drop ceiling. The precast hollow-core planks would require significant changes to the column layout to fit the typically 4'-0" increments and would make the irregular shape of the building more difficult and more expensive. While the composite steel system would only slightly change the column layout, it could still cause issues within the building layout. Both systems also have the potential to add a significant amount of the weight to the building if deep steel members are required. This would produce problems with the foundation and the need to design deeper piles and change the existing foundation. The steel composite system is also known for having deflection and vibration issues and it is unknown at this time how the precast hollow-core planks would deflect or vibrate. Lastly, there are also potential issues with these systems being integrated with the existing concrete shear walls as the lateral system.

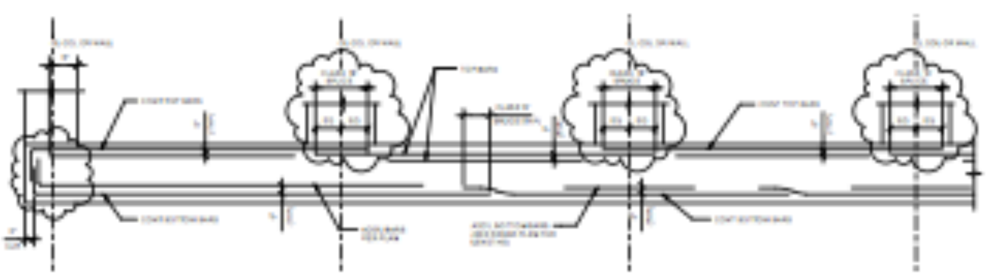
Throughout this technical report it has been concluded that further investigation into the post-tensioning system is required. The other two systems, steel composite and precast hollow-core planks, are not feasible in the Army National Guard Readiness Center Addition due to the considerable disadvantages associated with these systems.

APPENDIX: A: BUILDING LAYOUTS

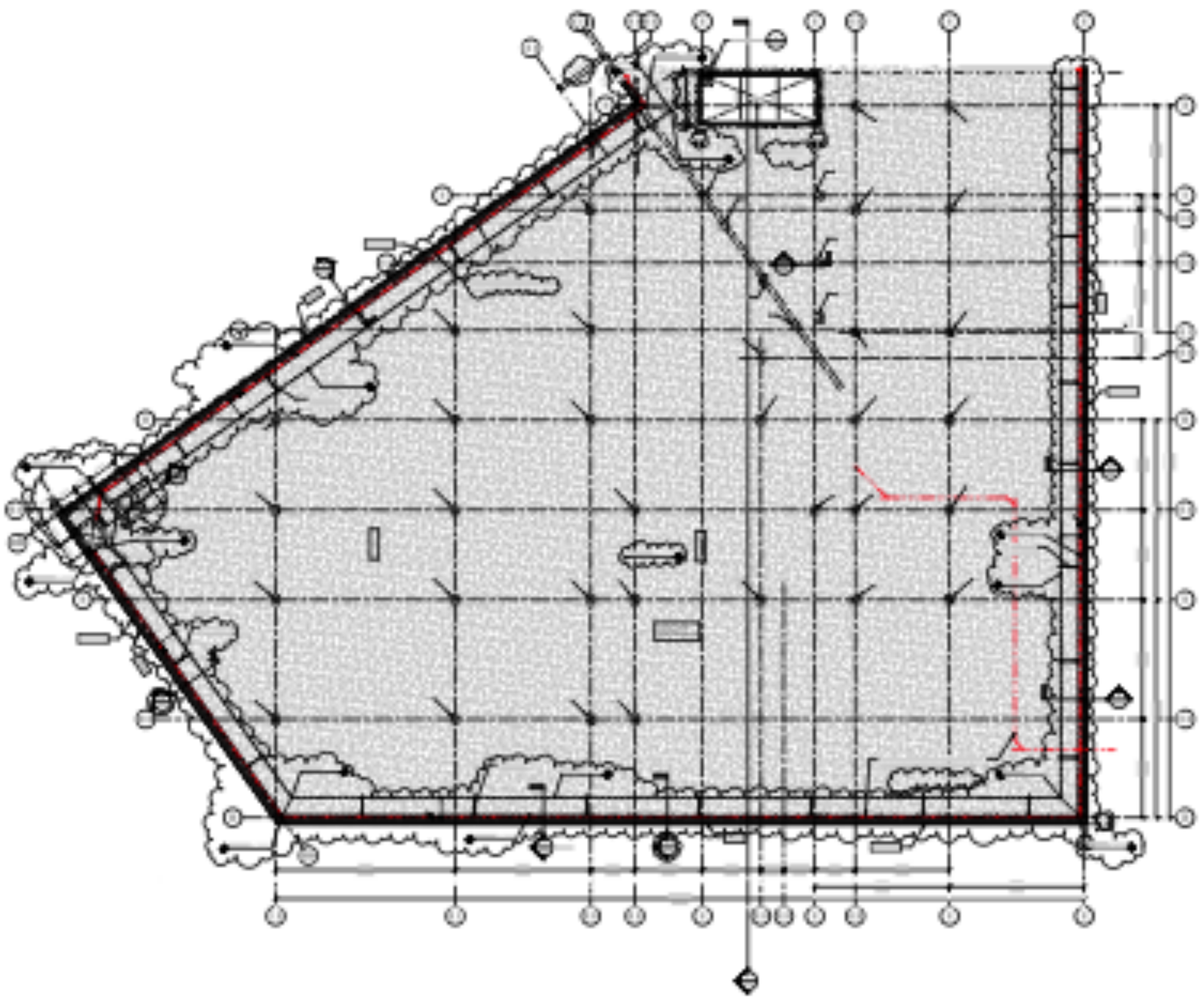
Presented in this appendix are some of the main drawings and details that were referenced during the investigation and research to complete this technical report.



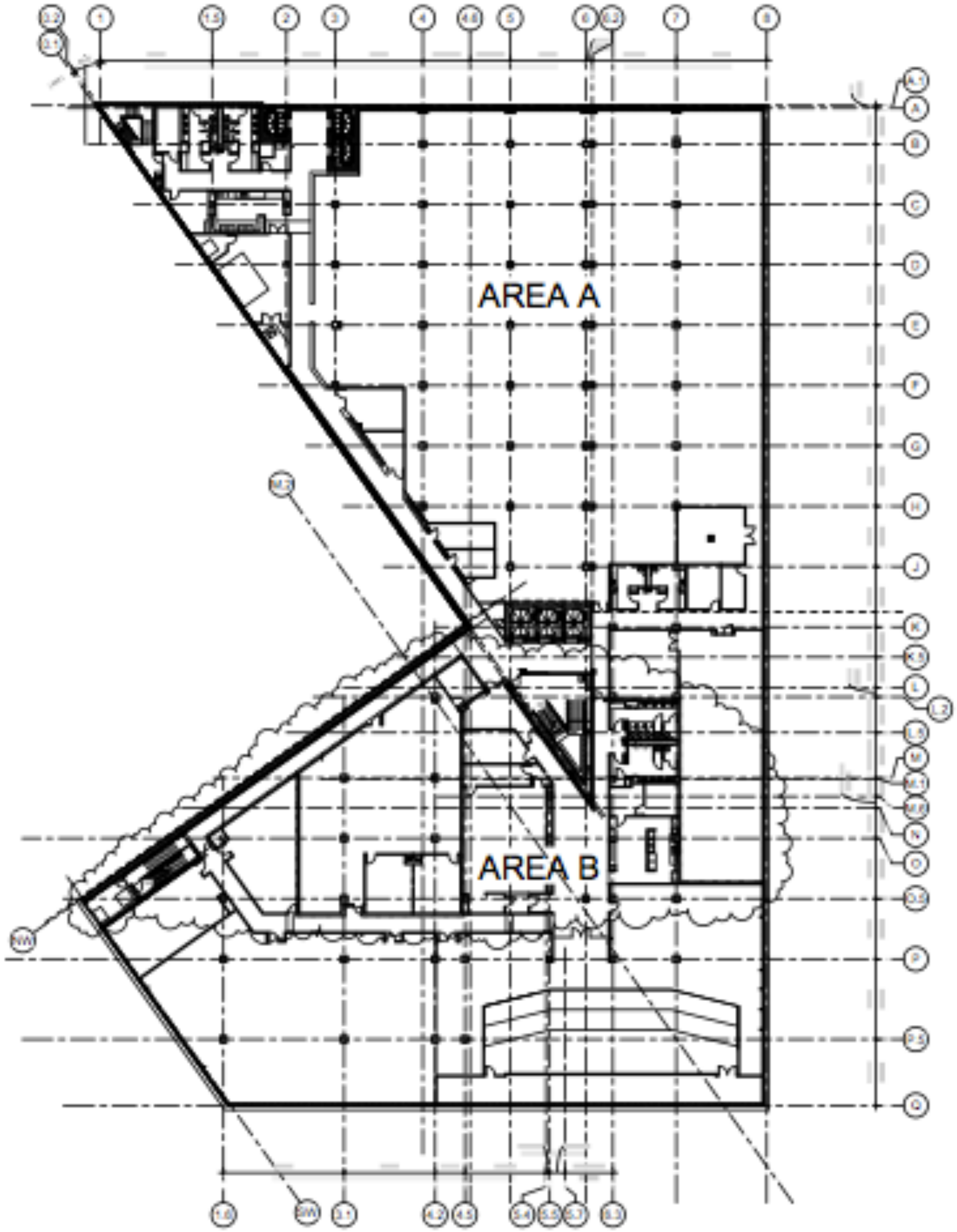
Partial Mat Foundation Plan



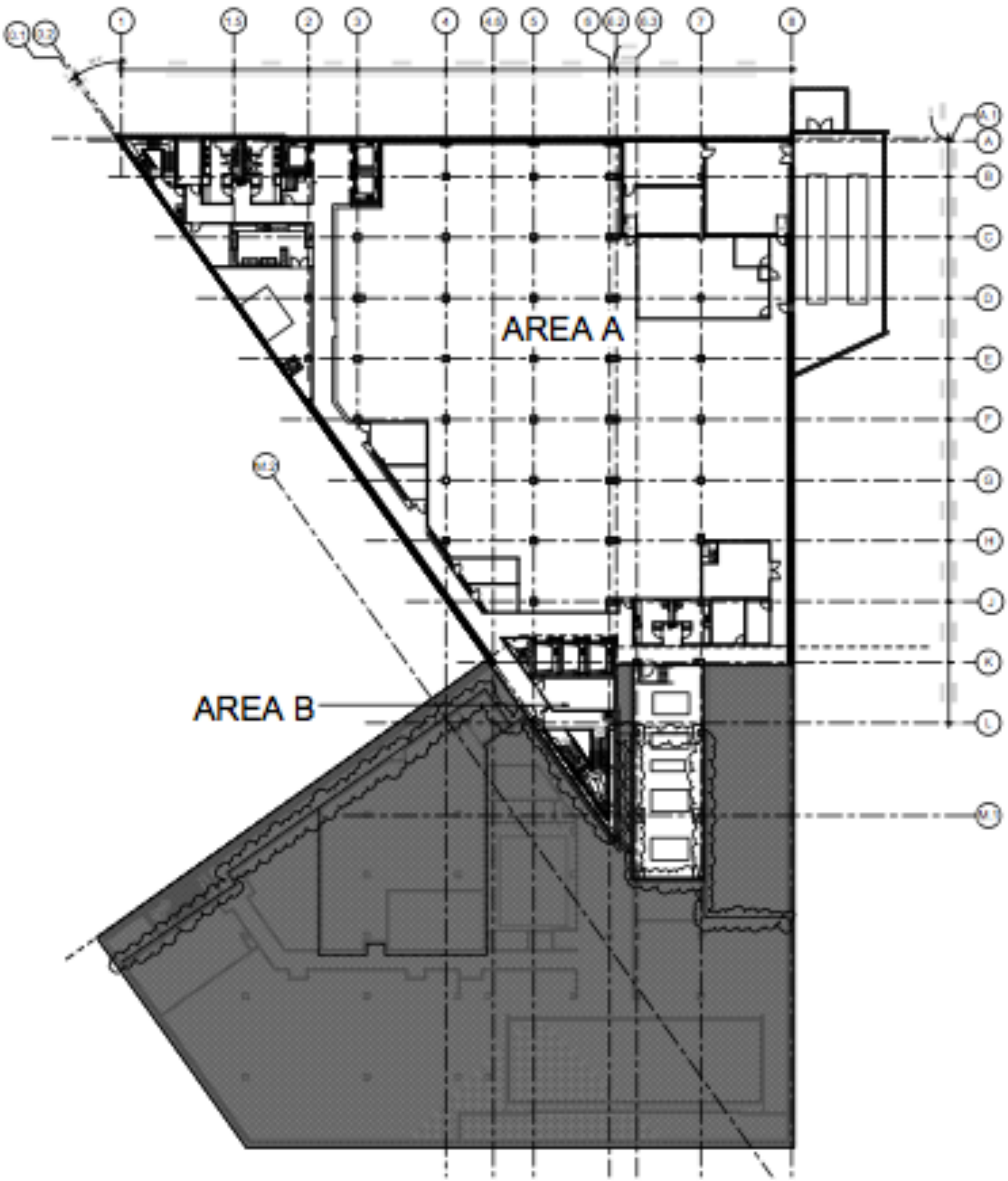
Typical Mat Foundation Detail



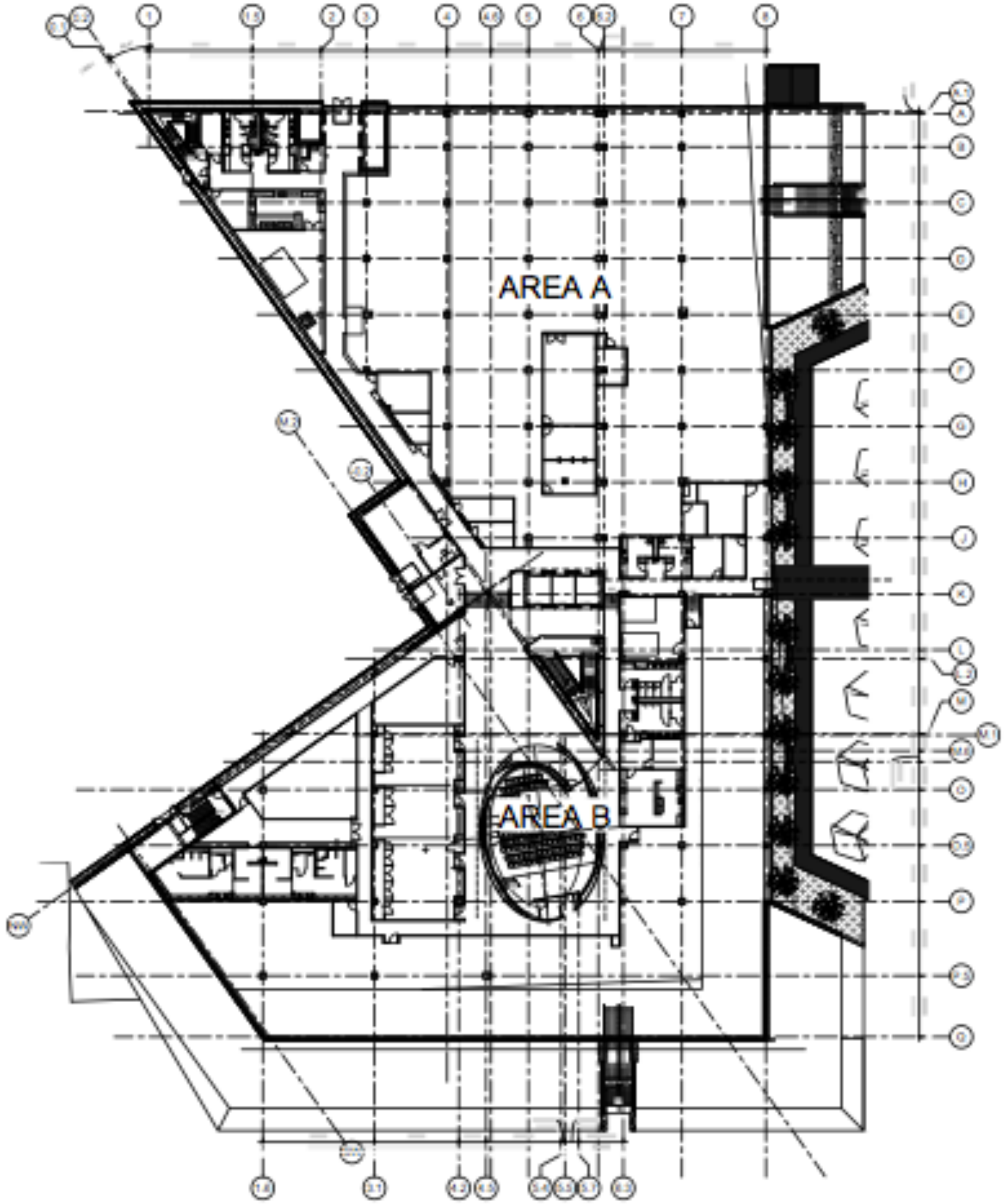
Partial Mat Foundation Plan



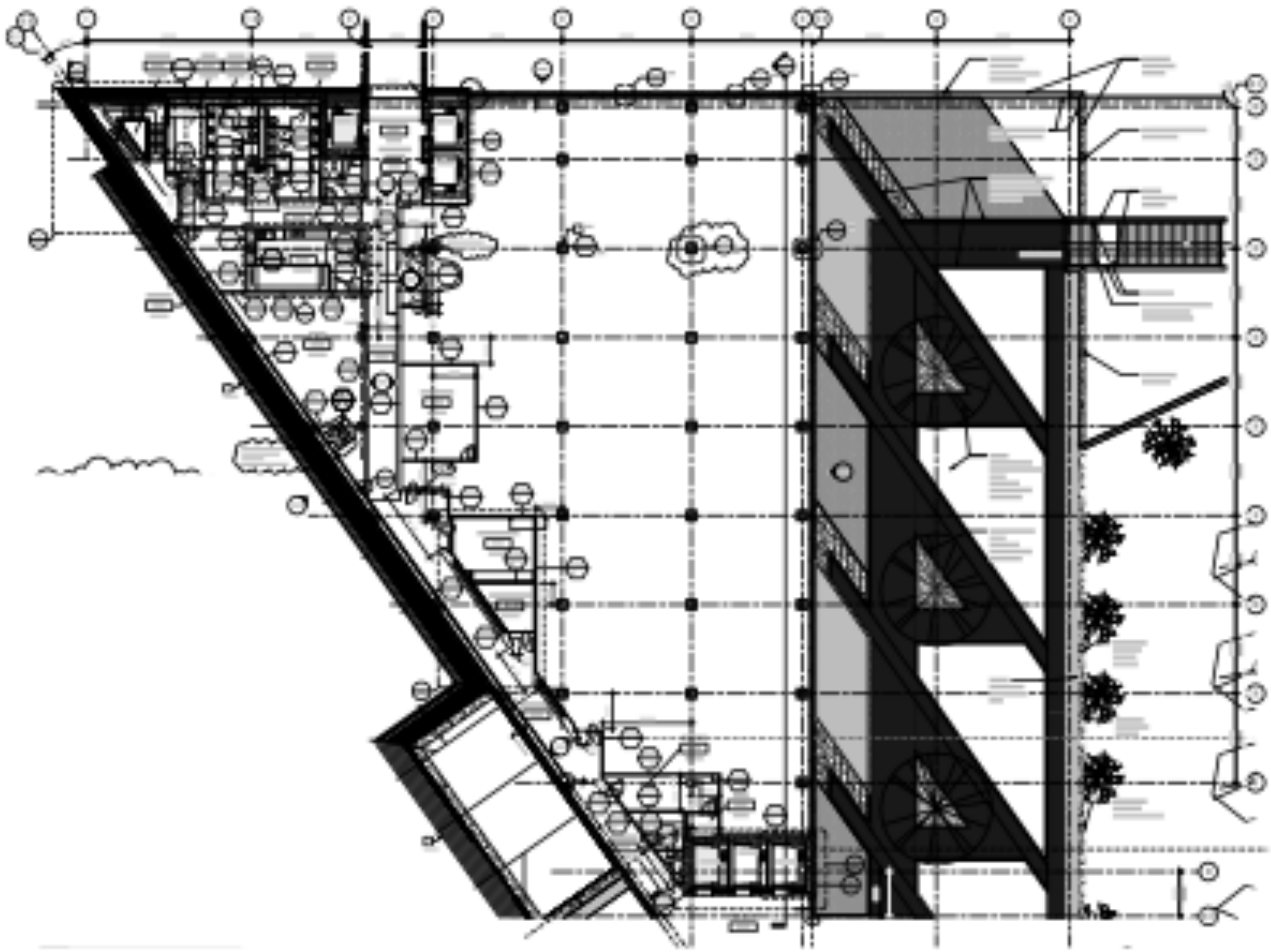
Level 3P Floor Plan



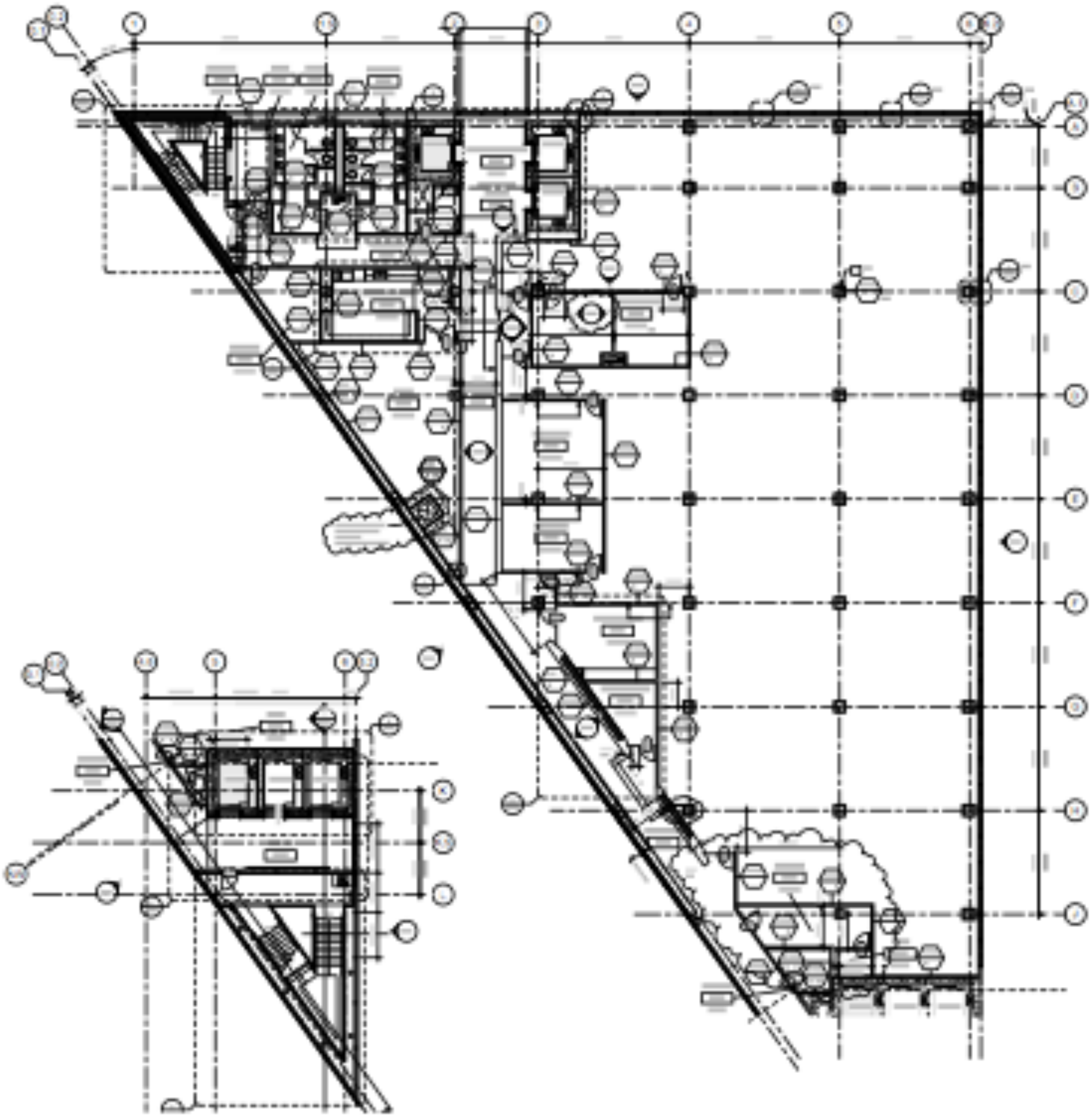
Level 2P Floor Plan



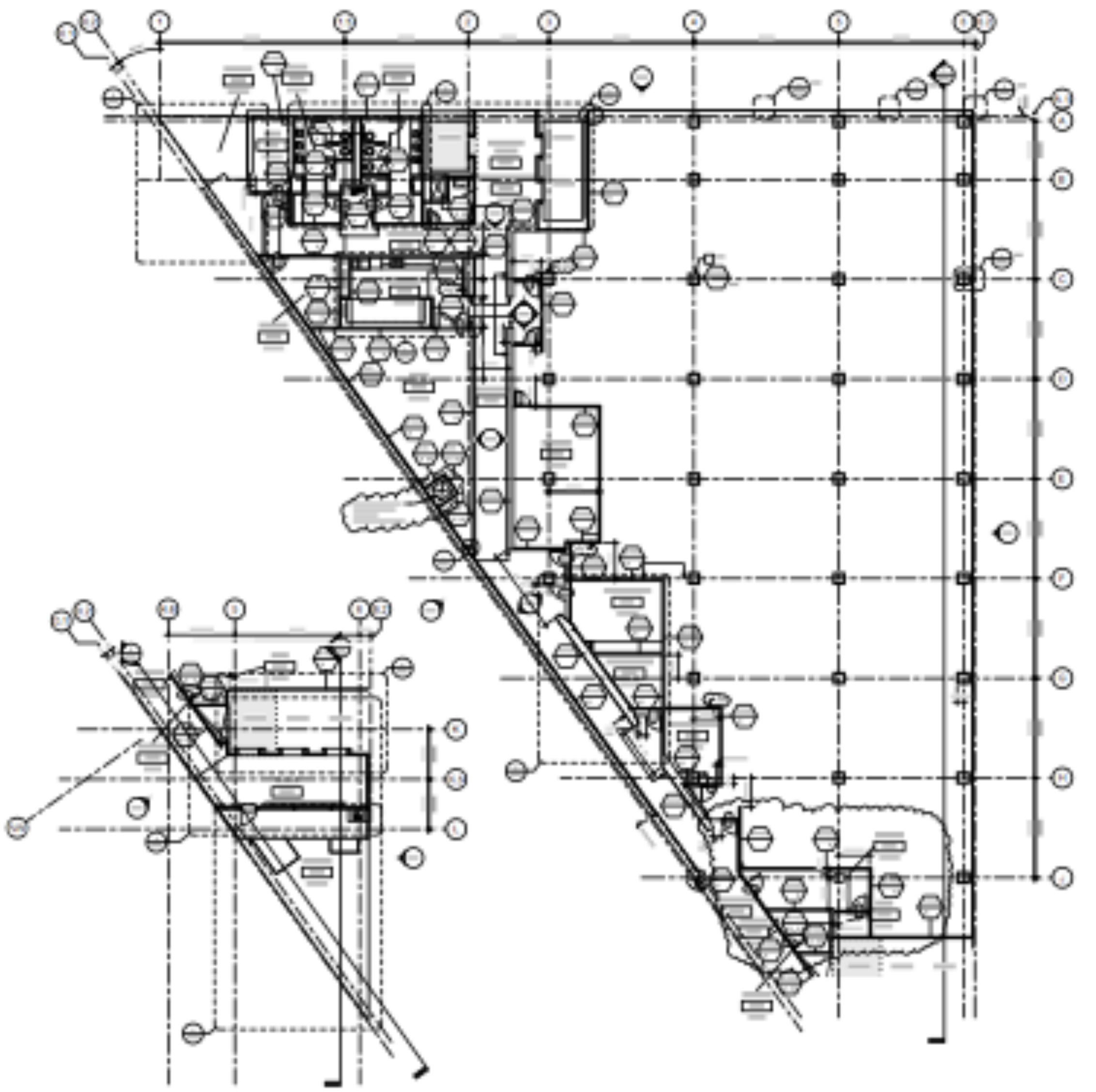
Level 1P Floor Plan



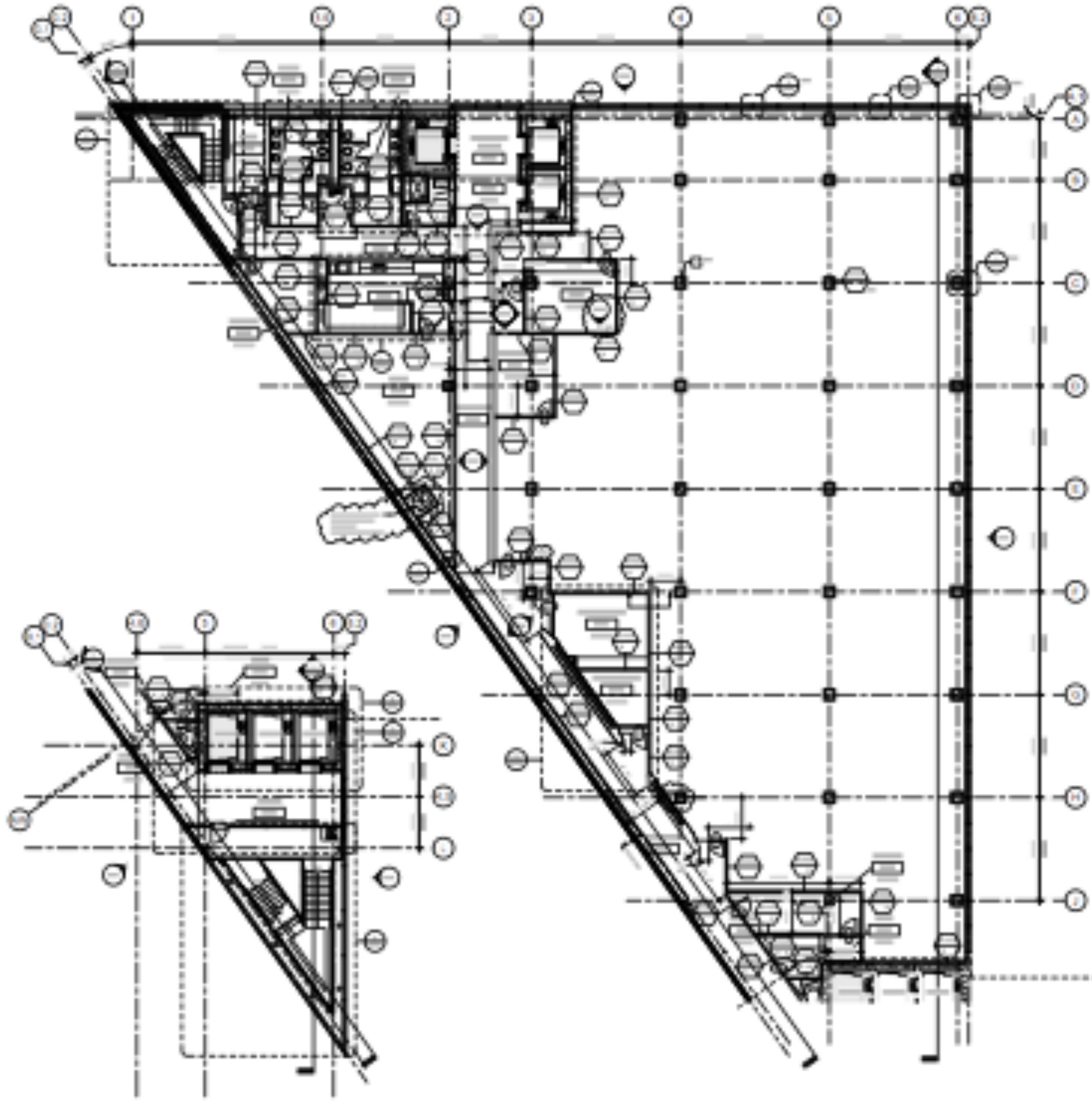
Partial Level 1T Floor Plan



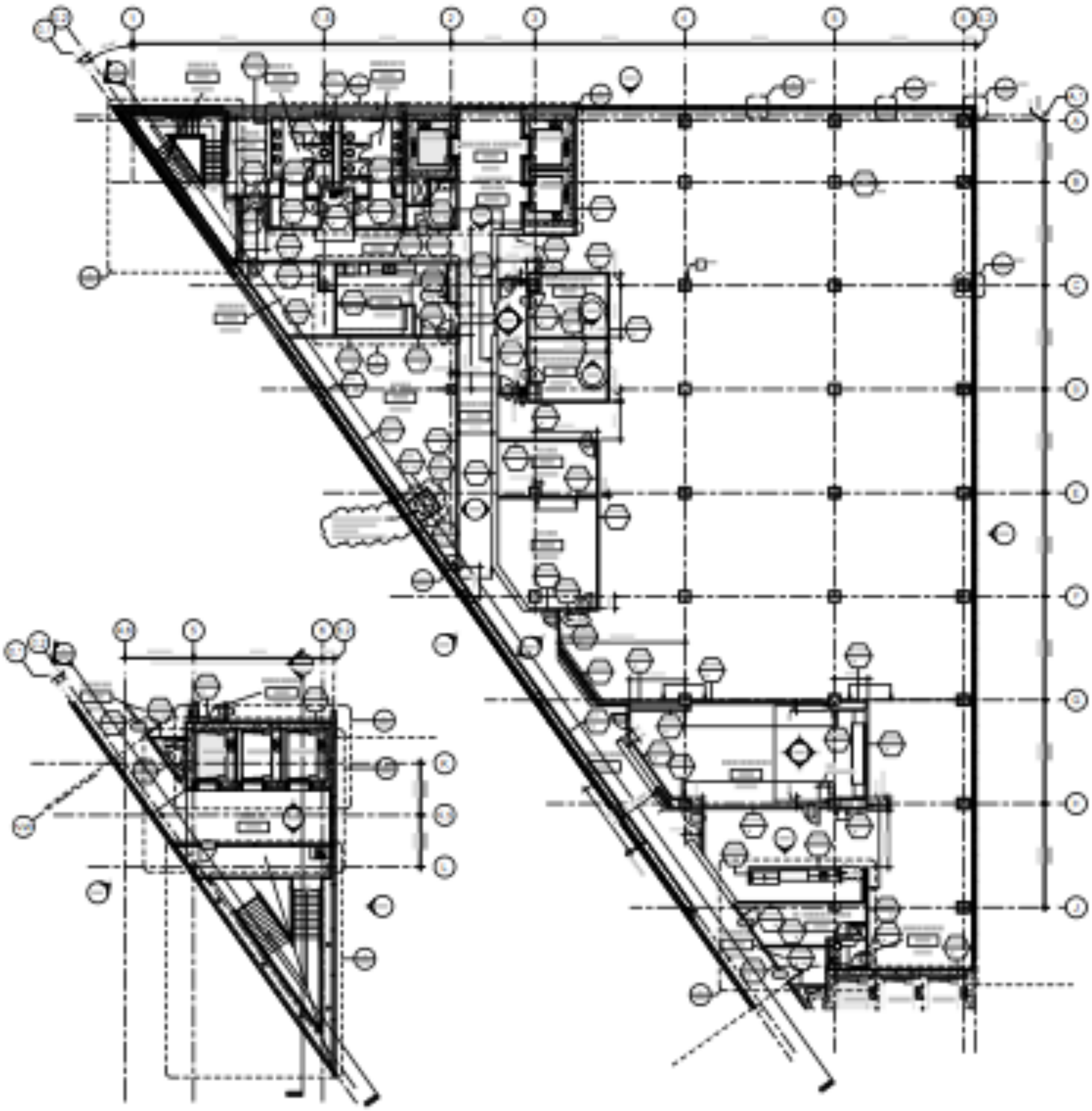
Level 2T Floor Plan



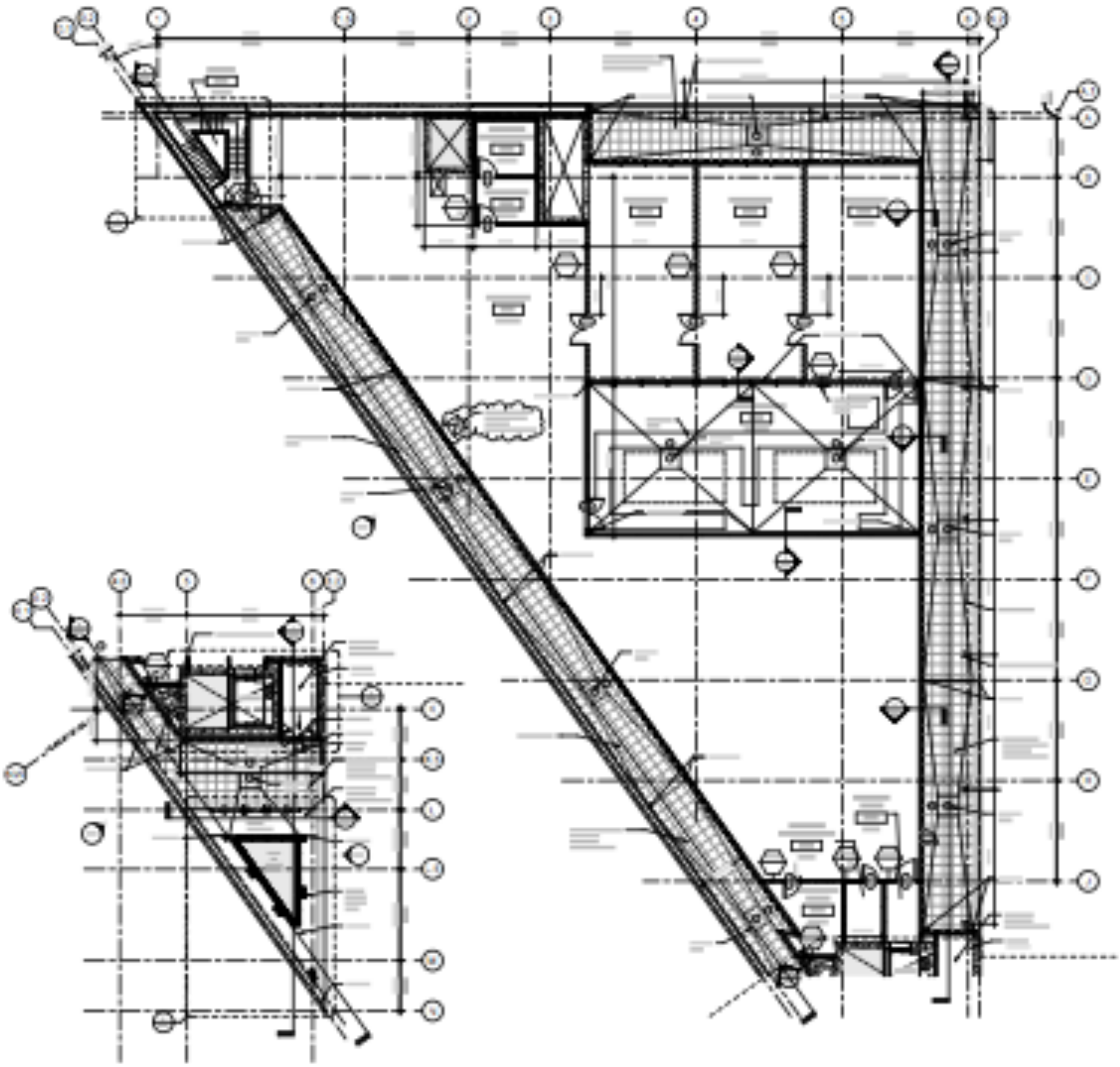
Level 3T Floor Plan



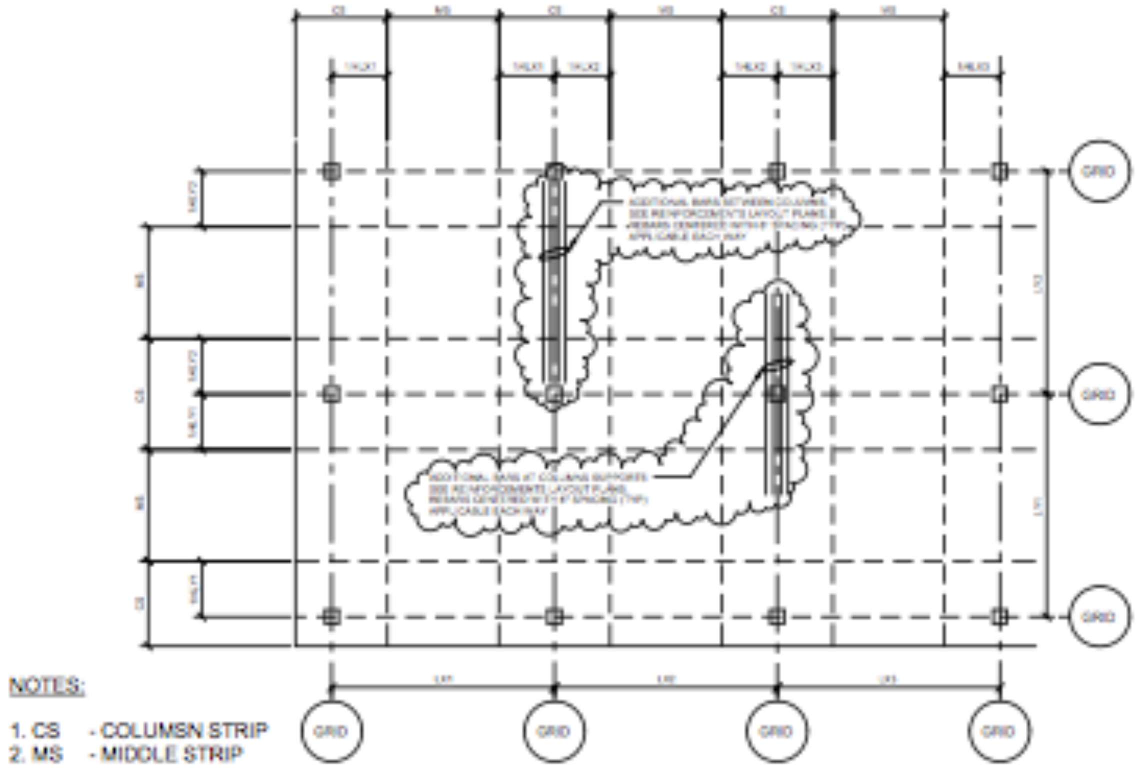
Level 4T Floor Plan



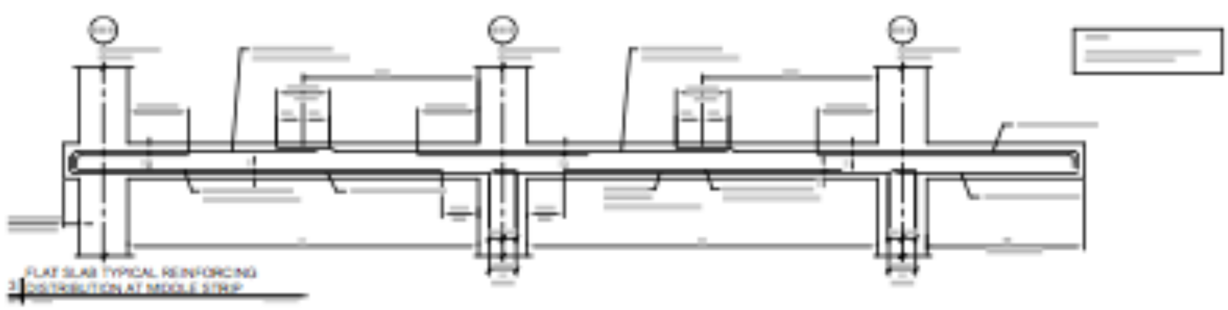
Level 5T Floor Plan



Level PH Floor Plan



Typical Column Strip and Middle Strip Detail



Typical Flat Slab Detail

APPENDIX B: EXISTING TWO-WAY SLAB CALCULATIONS

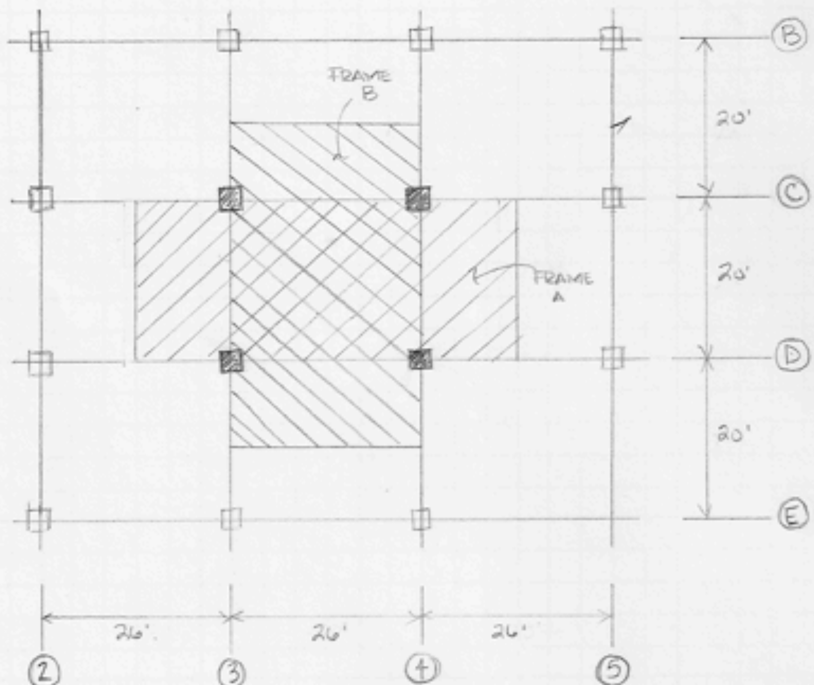
Presented in this appendix are the hand calculations that were performed to analyze the existing two-way flat slab floor system employed in the Army National Guard Readiness Center Addition.

EXISTING CONDITIONS:

2-Way Flat Slab w/ edge beams

9" Normal Weight Concrete Slab
$f'_c = 4000$ psi
$f_y = 60\,000$ psi
#6 Bars @ 18" o.c. & Additional as noted
22" by 22" Columns

Typical 26'x20' Interior Bay



$l_n = 26 - \frac{22}{12} = 24.17'$

ANALYSIS - USING DIRECT DESIGN METHOD (Ref. ACI 318-08 Ch. 13)

LOADS
DL = Selfweight + Superimposed Dead Load
$= (9/12)(150\,pcf) + 15\,psf = 127.5\,psf$

LL ⇒ Office Space + Partitions
$= 50\,psf + 15\,psf = 65\,psf$

Factored loads = 1.2 DL + 1.6 LL
$= 1.2(127.5) + 1.6(65) = \underline{0.257\,ksf}$

PANEL A:

Column Strip Width (§13.2.1)

$$\text{Center line} = \min \begin{cases} 0.25\ell_1 = 0.25(20') = 5' \\ 0.25\ell_2 = 0.25(26') = 6.5' \end{cases}$$

Column Strip = 10'

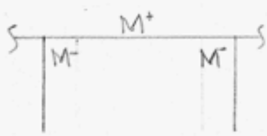
$$M_o = \frac{1}{8} w_u \ell_2 \ell_n^2 = \frac{1}{8}(0.257)(20')(24.17)^2 = 375.34'^k$$

No Interior Beams $\Rightarrow I_b = 0$
$\alpha = 0$

From Table 9.5(c) → Minimum Thickness for Slab w/o beams
Interior Panel w/o Drop Panels → $\ell_n/33$

$$\frac{(24.17 \times 12)}{33} = 8.8'' < 9'' \quad \therefore \text{ Slab Thickness is adequate}$$

Frame A Moment Distribution



$\ell_2/\ell_1 = 20/26 = 0.77$
$\alpha\, \ell_2/\ell_1 = 0$ (No Beams)

Static Moment Distribution Factors (§13.6.3.2)
Total Moment Distribution for Interior Span:

$M^- = 0.65\, M_o = 243.97'^k$
$M^+ = 0.35\, M_o = 131.37'^k$

According to §13.6.4.1 (Negative Moment Distribution)

ℓ_2/ℓ_1	0.5	0.77	1.0
$\alpha \ell_2/\ell_1 = 0$	75%	$\boxed{75\%}$	75%

(75%) NEGATIVE MOMENT TO CS = $182.97'^k$
(25%) NEGATIVE MOMENT TO MS = $61'^k$

According to §13.6.4.4 (Positive Moment Distribution)

ℓ_2/ℓ_1	0.5	0.77	1.0
$\alpha \ell_2/\ell_1 = 0$	60%	$\boxed{60\%}$	60%

(60%) POSITIVE MOMENT TO CS = $78.82'^k$
(40%) POSITIVE MOMENT TO MS = $52.55'^k$

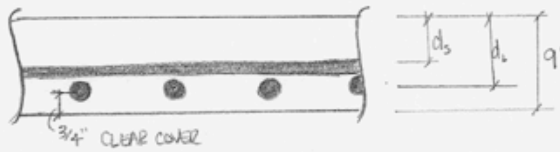
Summary:

	M^-	M^+	M^-
Total	-243.97	131.37	-243.97
CS	-182.97	78.82	-182.97
MS	-61	52.55	-61

TECH REPORT II	EXISTING FLOOR SYSTEM	ArNG Readiness Center	3

Panel A
Total Width = 20'
Column Strip = 10' = 120"
Middle Strip = 5' on each side → 120"

DESIGN OF SLAB REINFORCEMENT



Max Spacing = 2t = 18"
$A_{s\,min}$ = 0.0018 bc
(For f_y = 60000 ksi)

EFFECTIVE DEPTHS (USING #6 BARS)

$d_s = 9" - 3/4" - \frac{0.750"}{2} = 7.88"$
$d_t = 7.88" - 0.750" = 7.13"$

REINFORCEMENT DESIGN IN CS

ITEM	DESCRIPTION	INTERIOR SPAN		
		LEFT	MID	RIGHT
1	M_u ('k)	-182.97	78.82	-182.97
2	CS Width (b)	120"	120"	120"
3	d	7.13"	7.13"	7.13"
4	$M_n = M_u/0.9$	-203.3	87.58	-203.3

CHECK $d_{min} = \sqrt{\frac{M_n \times 12000}{\rho f_y b (1 - 0.59 \rho f_y/f'_c)}}$

FOR f_y = 60 000 psi
f'_c = 4 000 psi
ρ = 0.0206

$d_{min} = \sqrt{\frac{203.3 \times 12000}{(0.0206)(60000)(120)(1-0.59(0.0206)(60/4)}} = 4.49" < 7.13"$ ∴ OK

5	$R = \frac{M_n \times 12000}{bd^2}$	400	172.3	400
6	ρ (From Graph)	0.0104	0.004	0.0104
7	$A_s = \rho bd$	8.90	3.42	8.90
8	$A_{s\,min}$ = 0.0018 bc	1.94	1.94	1.94
9	N = A_s/0.44	20.23	7.78	20.23
10	$N_{min} = \frac{\text{span width}}{2t}$	6.66	6.66	6.66

CALCULATED:
M^-_{left} ⇒ 21 #6
M^+_{mid} ⇒ 8 #6
M^-_{right} ⇒ 21 #6

DESIGNED:
M^-_{left} = 22 #6
M^+_{mid} = 7 #6
M^-_{right} = 20 #6

REINFORCEMENT DESIGN IN MS

ITEM	DESCRIPTION	INTERIOR SPAN	
		M^-	M^+
1	M_u ('k)	-61	52.55
2	MS Width (b)	120"	120"
3	d	7.13"	7.13"
4	$M_n = M_u/0.9$	-67.8	58.4

CHECK $d_{min} = \sqrt{\dfrac{67.8 \times 12000}{(0.0206)(60000)(120)[1-0.59(0.0206)(60/4)]}} = 2.58"$

$d_{min} = 2.58" < d_{used} = 7.13"$ ∴ OK

5	$R = \dfrac{M_n \times 12000}{bd^2}$	133.4	114.9
6	ρ (From Graph)	0.0031	0.0024
7	$A_s = \rho bd$	2.72	2.05
8	$A_{s\,min} = 0.0018bh$	1.94	1.94
9	$N = A_s/0.44$	6.18	4.67
10	$N_m = b/2t$	6.66	6.66

CALCULATED → 7#6
DESIGNED → 7#6

PANEL B:

Column Strip Width (§13.2.1)

CS = 120"
MS = 16' = 192"

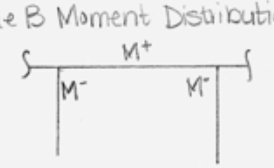
$M_o = \frac{1}{8} w \ell_2 \ell_n^2 = \frac{1}{8}(0.257)(26)(20-\frac{22}{12})^2 = 275.66'^k$

No Interior Beams ⇒ $I_b = 0$
$\alpha = 0$

From Table 9.5(c) → Minimum Thickness for Slab w/o Beams
Interior Panel w/o Drop Panels → $\ell_n/33$

$\dfrac{(20-\frac{22}{12})}{33} = 6.61" < 9"$ ∴ Slab Thickness is adequate

Frame B Moment Distribution



$\ell_2/\ell_1 = 26/20 = 1.3$
$\alpha\,\ell_2/\ell_1 = 0$ (No Beams)

TECH REPORT II	EXISTING FLOOR SYSTEM	ArNG Readiness Center	5

Static Moment Distribution Factors (§13.6.3.2)
Total Moment Distribution for Interior Span:

$M^- = 0.65 M_o = -179.18^{'k}$
$M^+ = 0.35 M_o = 96.48^{'k}$

According to §13.6.4.1 (Negative Moment Distribution)

ℓ_2/ℓ_1	1.0	1.3	2.0
$\alpha \ell_2/\ell_1 = 0$	75%	$\boxed{75\%}$	75%

(75%) NEGATIVE MOMENT TO CS = 134.38'k
(25%) NEGATIVE MOMENT TO MS = 44.80'k

According to §13.6.4.4 (Positive Moment Distribution)

ℓ_2/ℓ_1	1.0	1.3	2.0
$\alpha \ell_2/\ell_1 = 0$	60%	$\boxed{60\%}$	60%

(60%) POSITIVE MOMENT TO CS = 57.89'k
(40%) POSITIVE MOMENT TO MS = 38.59'k

Summary:

	M^-	M^+	M^-
TOTAL	-179.18	96.48	-179.18
CS	-134.38	57.89	-134.38
MS	-44.80	38.59	-44.80

REINFORCEMENT DESIGN IN CS

ITEM	DESCRIPTION	INTERIOR SPAN	
		M^-	M^+
1	M_u ('k)	-134.38	57.89
2	CS Width (b)	120"	120"
3	d	7.88"	7.88"
4	$M_n = M_u/0.9$	-149.3	64.3

$$\text{CHECK } d_{min} = \sqrt{\frac{149.3 \times 12000}{(0.0206)(60000)(120)[1-0.59(0.0206)(60/4)]}} = 3.84''$$

$d_{min} = 3.84'' < d_{used} = 7.88''$ ∴ OK

5	$R = \frac{M_n \times 12000}{bd^2}$	240.4	103.6
6	ρ (From Graph)	0.0044	0.0012
7	$A_s = \rho bd$	6.68	1.82
8	$A_{s\,min} = 0.0018bh$	1.94	1.94
9	$N = A_s/0.44$	15.18	4.41
10	$N_{min} = b/2t$	6.67	6.67

TECH REPORT II	EXISTING FLOOR SYSTEM	ArNG Readiness Center	6

CALCULATED:
$M^- = 16\#6$
$M^+ = 7\#6$

DESIGNED:
$M^- = 16\#6$
$M^+ = 7\#6$

REINFORCEMENT DESIGN IN MS

ITEM	DESCRIPTION	INTERIOR SPAN	
		M^-	M^+
1	M_u ('k)	-44.80	38.59
2	MS Width (b)	192"	192"
3	d	7.88"	7.88"
4	$M_n = M_u/\phi$	-49.8	42.9

CHECK $d_{min} = \sqrt{\dfrac{49.8 \times 12000}{(0.9206)(60000)(192)[1-0.59(0.0206)(60/4)]}} = 3.08"$

$d_{min} = 3.08" < 7.88" = d_{used}$ ∴ OK

5	$R = \frac{M_n \times 12000}{bd^2}$	50.13	43.2
6	ρ (FROM GRAPH)	0.0008	0.0007
7	$A_s = \rho bd$	1.21	1.06
8	$A_{s\,min} = 0.0018B_t$	3.11	3.11
9	$N = A_s/0.44$	7.08	7.08
10	$N_{min} = b/2t$	10.67	10.67

CALCULATED = $11\#6$
DESIGNED = $11\#6$

SHEAR CHECKS

- Wide Beam Action

CRITICAL SECTION WIDTH $= 26/2 - (\frac{7.875}{12}) = 12.43'$

$w_u = 0.257$ ksf

$V_u = w_u A = (0.257)(20' \times 12.43') = 63.81^k$

$V_n = 2\sqrt{f'_c}\, bd = 2\sqrt{4000}\,(20')(7.88 \times 12) = 239^k$

$\phi V_n = 0.75\,(239) = 179.25^k$

$\phi V_n > V_u$ $\therefore$ GOOD

- Punching Shear

$V_c = 4\sqrt{f'_c}\, b_o d$

$b_o/4 = 2(d/2) + 22'' \Rightarrow b_o = 119.5''$

$= 4\sqrt{4000}\,(119.5)(7.88) = 238^k$

MUST BE $\leq V_c = (\frac{\alpha_s d}{b_o} + 2)\sqrt{f'_c}\, b_o d$

$\alpha_s = 40$ for interior columns

$= (\frac{40 \times 7.88}{119.5} + 2)(\sqrt{4000})(119.5)(7.88) = 275.9^k$

MUST BE $\leq V_c = (2 + 4/\beta_c)\sqrt{f'_c}\, b_o d$

$\beta_c = 1.0$ for sq. columns

$= (2 + \frac{4}{1.5})\sqrt{4000}\,(119.5)(7.88) = 357.2^k$

$V_u = (0.257)(650\text{ sf} - (\frac{22}{12})^2) = 166.18'^k$

$\phi V_c = 0.75\,(238) = 178.6'^k$

$\phi V_c > V_u$ $\therefore$ GOOD

NO ADDITIONAL SHEAR REINFORCEMENT REQUIRED

DEFLECTION

• Elastic Δ Due to Self Weight

$$E = 57000\sqrt{f'_c} = 57000\sqrt{4000} = 3604996.5 \text{ psi}$$

$$I_{20} = \frac{(26\times12)(9'')^3}{12} = 18954 \text{ in}^4$$

$$I_{26} = \frac{(20\times12)(9'')^3}{12} = 14580 \text{ in}^4$$

$$w_{self\,wt} = (9/12)(150\text{pcf}) = 112.5 \text{ psf}$$

$$\Delta = \frac{wl^4}{384EI}$$

SHORT SPAN: $\Delta_{20} = \frac{112.5(26)(20)^4\times1728}{384(3604996.5)(18954)} = 0.0308''$

LONG SPAN: $\Delta_{26} = \frac{112.5(20)(26)^4(1728)}{384(3604996.5)(14580)} = 0.088''$

Frame A:

$$I_{cs} = \frac{120''(9'')^3}{12} = 7290 \text{ in}^4 = I_{ms}$$

Frame B:

$$I_{cs} = \frac{120''(9'')^3}{12} = 7290 \text{ in}^4$$

$$I_{ms} = \frac{192''(9'')^3}{12} = 11664 \text{ in}^4$$

68% TO CS (M^+ & M^- Average)
32% TO MS (M^+ & M^- Average)

Frame A (Long Span)

$$\Delta_{col} = 0.088(0.68)\left(\frac{14580}{7290}\right) = 0.119''$$

$$\Delta_{mid} = 0.088(0.32)\left(\frac{14580}{7290}\right) = 0.056''$$

Frame B (Short Span)

$$\Delta_{col} = 0.0308(0.68)\left(\frac{18954}{7290}\right) = 0.0545''$$

$$\Delta_{mid} = 0.0308(0.32)\left(\frac{18954}{7290}\right) = 0.016''$$

LONG TERM DEFLECTION ⇒ $3.0\Delta_a$

Frame A

$$\Delta_{col} = 0.119\times3 = 0.357''$$

$$\Delta_{mid} = 0.056\times3 = 0.168''$$

Frame B

$$\Delta_{col} = 0.0545\times3 = 0.1635''$$

$$\Delta_{mid} = 0.016\times3 = 0.048''$$

By Direct Proportion:

LIVE LOAD = $\Delta\left(40/112.5\right)$

Frame A
$\Delta_{col} = 0.0423''$
$\Delta_{mid} = 0.0199''$

Frame B
$\Delta_{col} = 0.0194''$
$\Delta_{mid} = 0.0057''$

DEFLECTION LIMIT (ACCORDING TO ACI 318-08)

$$\Delta_{max} < \ell/480 = \Delta_{LL} + 3\Delta_{DL}$$

$$\Delta_{max} = \frac{240}{480} = 0.5''$$

Frame A
$\Delta_{f,col} = 0.357 + 0.0423 = \underline{0.399'' < 0.5''}$

$\Delta_{f,mid} = 0.168 + 0.0199 = \underline{0.1879'' < 0.5''}$

Frame B
$\Delta_{f,col} = 0.0194'' + 0.1635'' = \underline{0.1829'' < 0.5''}$

$\Delta_{f,mid} = 0.0048'' + 0.0057'' = \underline{0.0105'' < 0.5''}$

<u>GOOD</u>

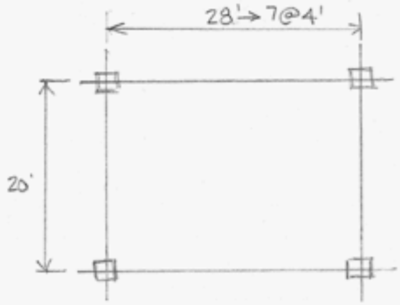
*All Deflections are within ACI 318-08 Limits

APPENDIX C: PRECAST HOLLOW-CORE PANELS ON STEEL CALCULATIONS

Presented in this appendix are the hand calculations that were performed to analyze the precast hollow-core planks on steel as an alternate floor framing system for the Army National Guard Readiness Center.

HOLLOW CORE SLAB ON METAL

Adjusted Grid Layout to fit typical 4' increments of hollow core planks



Reference → PCI Handbook

SLAB

LOAD:

LL = 65 psf (50 + 15)
SDL = 15 psf
DL = 15 psf (From PCI Handbook)

Superimposed Service Load = 95 psf

4'-0" × 6" Normal Weight Concrete Panel
2" Normal Weight Topping
66-S Carrying ____ psf → 6 Strands @ 6/16" ϕ - Straight

Self weight = 74 psf
Estimated Camber @ Erection = 0.2 in
Estimated Longtime Camber = -0.1 in

GIRDERS

LOAD:

LL = 65 psf
DL = 15 psf + 74 psf = 89 psf

Factored loads = 1.2(89) + 1.6(65) = 210.8 psf

$$M_u = \frac{(210.8)(20)(28)^2}{8} = 413.2'^k$$

FROM AISC Steel Manual (13th Ed.)

W 18 × 55

$\phi M_n = 420'^k$

TECH REPORT II	ALTERNATE SYSTEM 1	ArNG Readiness Center	2

(Hollow Core Slab on Metal)

LIVE LOAD DEFLECTION

$\Delta_{LL} = l/360 = \frac{28 \times 12}{360} = 0.933''$

$0.933 = \frac{5(65)(28)(20)^4(1728)}{384(29000)(I_x)(1000)} \Rightarrow I_x = 242.2 \text{ in}^4$

$I_x = 890 \text{ in}^4 > 242.2 \text{ in}^4$ OKAY

TOTAL LOAD DEFLECTION

$\Delta_{TL} = \frac{5(65+15+74)(28)(20^4)(1728)}{384(29000)(890)(1000)} = 0.664$

$\Delta = l/240 = 1.4'' > 0.664''$ ∴ OKAY

4'-0" × 6" Normal Weight Concrete
2" Normal Weight Concrete Topping
4HC6+2, 66-S on W18×55 Girders

Strand Pattern Designation
76-S

S = straight
Diameter of strand in 16ths
No. of Strand (7)

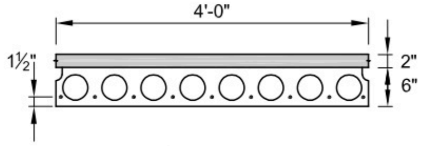
Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Capacity of sections of other configurations are similar. For precise values, see local hollow-core manufacturer.

Key
444 – Safe superimposed service load, psf
0.1 – Estimated camber at erection, in.
0.2 – Estimated long-time camber, in.

HOLLOW-CORE

4′-0″ x 6″
Normal Weight Concrete



f'_c = 5,000 psi
f_{pu} = 270,000 psi

Section Properties

	Untopped		Topped	
A =	187	in.^{2}	283	in.^{2}
I =	763	in.^{4}	1,640	in.^{4}
y_b =	3.00	in.	4.14	in.
y_t =	3.00	in.	3.86	in.
S_b =	254	in.^{3}	396	in.^{3}
S_t =	254	in.^{3}	425	in.^{3}
wt =	195	plf	295	plf
DL =	49	psf	74	psf
V/S =	1.73	in.		

4HC6

Table of safe superimposed service load (psf) and cambers (in.) — **No Topping**

Strand Designation Code	Span, ft																				
	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	444	382	333	282	238	203	175	151	131	114	100	88	77	68	59	52	46	40	33	28	
	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.5	–0.7		
	0.2	0.2	0.2	0.2	0.3	0.3	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.3	–0.5	–0.7	–0.9	–1.2	–1.5	–1.9	
76-S		445	388	328	278	238	205	178	155	136	120	105	93	82	73	65	57	49	42	36	31
		0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.1	0.0	–0.1	–0.3	–0.4	–0.6
		0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.7	–0.9	–1.2	–1.6	–2.0
96-S		466	421	386	338	292	263	229	201	177	157	139	124	110	99	88	78	68	60	53	46
		0.3	0.3	0.3	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.1	0.0	–0.1
		0.3	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.6	0.5	0.5	0.4	0.3	0.2	0.1	–0.1	–0.3	–0.6	–0.9	–1.3
87-S		478	433	398	362	322	290	264	240	212	188	167	149	134	119	107	95	85	76	68	60
		0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.7	0.8	0.8	0.7	0.7	0.7	0.6	0.5	0.4	0.3
		0.4	0.5	0.5	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.3	0.2	0.0	–0.3	–0.6
97-S		490	445	407	374	346	311	276	242	220	203	186	166	148	133	119	107	96	86	78	70
		0.4	0.4	0.5	0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	1.0	0.9	0.9	0.9	0.8	0.7	0.6
		0.5	0.6	0.6	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.0	0.9	0.9	0.8	0.7	0.5	0.3	0.1	–0.2

4HC6 + 2

Table of safe superimposed service load (psf) and cambers (in.) — **2 in. Normal Weight Topping**

Strand Designation Code	Span, ft																		
	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	470	396	335	285	244	210	182	158	136	113	93	75	59	46	34				
	0.2	0.2	0.2	0.2	0.2	02	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2				
	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.5	–0.7	–0.9	–1.2				
76-S		461	391	334	287	248	216	188	163	137	115	95	78	63	50	38	27		
		0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.1	–0.0	–0.1	–0.3		
		0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.2	–0.3	–0.5	–0.7	–0.9	–1.2	–1.5		
96-S			473	424	367	319	279	245	216	186	160	137	116	98	82	68	55	43	33
			0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.1	0.0	–0.1
			0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	–0.1	–0.3	–0.5	–0.7	–1.0	–1.4	–1.7
87-S			485	446	415	377	331	292	258	224	195	169	147	127	109	94	80	67	55
			0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.7	0.8	0.8	0.7	0.7	0.7	0.6	0.5	0.4	0.3
			0.5	0.5	0.5	0.6	0.6	0.6	0.5	0.5	0.4	0.4	0.2	0.1	–0.1	–0.3	–0.5	–0.8	–1.2
97-S			494	455	421	394	357	327	288	251	219	192	168	146	127	110	95	82	70
			0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	1.0	0.9	0.9	0.9	0.8	0.7	0.6
			0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.4	0.2	0.0	–0.2	–0.5	–0.8

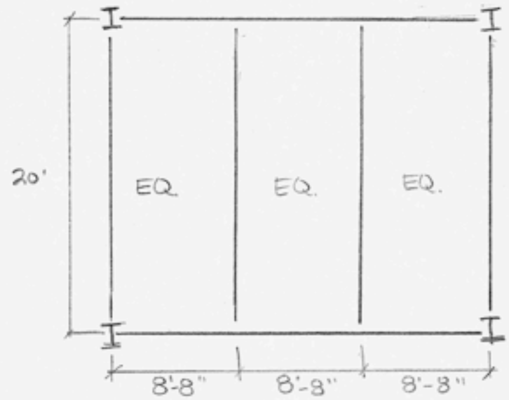
Strength is based on strain compatibility; bottom tension is limited to $7.5\sqrt{f'_c}$; see pages 2–7 through 2–10 for explanation.

APPENDIX D: COMPOSITE STEEL CALCULATIONS

Presented in this appendix are the hand calculations that were performed to analyze composite steel decking as an alternate floor framing system for the Army National Guard Readiness Center.

TECH REPORT II	ALTERNATE SYSTEM 2	ArNG Readiness Center	1

COMPOSITE STEEL FLOOR SYSTEM



USING VULCRAFT STEEL DECK PRODUCTS (WEB)
USING AISC STEEL MANUAL (13th Ed)

METAL DECK DESIGN

- TO ACHIEVE A 2 HR FIRE-RATING (ACCORDING TO ANSI/UL 263) A MINIMUM 3¼" TOPPING OF LIGHTWEIGHT CONCRETE MUST BE USED ABOVE METAL DECK [110 pcf]

- LOADS
  - Dead Load = 15 psf
  - Live Load = 65 psf

  $f'_c = 3000$ psi
  $f_y = 60\,000$ psi

  $w_u = 1.2(15) + 1.6(65) = 122$ psf

- TRY 5½" SLAB DEPTH → 18 GAGE, 2" DECK [2VLI18]

  Allowable Load = 219 psf > 122 psf ∴ OKAY

  Max Unshored Span = 10'-8" > 8'-8" ∴ OKAY

$\boxed{\text{USE 5½" SLAB DEPTH WITH 18 GAGE 2VLI18}}$

TECH REPORT II	ALTERNATE SYSTEM 2	ArNG Readiness Center	2

BEAM DESIGN

$$M_u = \frac{122(8.67')(20')^2}{8} = 52.87'^k$$

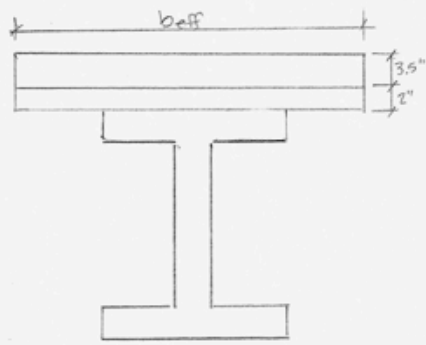
Assume $a = 1.5"$
$Y_2 = 5.5" - 1.5" = 4"$

FROM TABLE 3-19 (AISC Steel Manual)

W14x26 ⇒ $235'^k$ [P.N.A. @ #6]

$235 > M_u = 52.87'^k$ ∴ OKAY

$\Sigma Q_n = 135^k$



$$b_{eff} \le \begin{cases} \frac{SPAN}{2} = \frac{20 \times 12}{2} = 180" \\ \frac{1}{4}(SPAN) = \frac{1}{4}(20 \times 12) = \underline{90" = b_{eff}} \end{cases}$$

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{135}{0.85(3)(90)} = 0.59" < 1.5"$$

∴ CONSERVATIVE ASSUMPTION WAS MADE

NUMBER OF SHEAR STUDS: $\Sigma Q_n / Q_n$

ASSUMPTION: 3/4" Shear Studs ⊥ to Deck
1 Weak Stud per Rib

ACCORDING TO TABLE 3-21 (AISC Steel Manual)
$Q_n = 17.1\ ^k/stud$

$\#\ STUDS = \frac{135}{17.1} = 7.89 \Rightarrow 2 \times 8 =$ 16 SHEAR STUDS

CHECK BEAM DEFLECTIONS
W14x26 → Bare Steel → $\phi M_n = 151'^k$

$w_{conc+deck} = 42psf$
$w_{beam} = 0.026\ ^k/ft$
$w_{LL} = 65psf$
$w_{DL} = (0.042 \times 8.67') + 0.026 = 0.39\ ^k/ft$
$w_{LL} = (0.065 \times 8.67') = 0.56\ ^k/ft$

TECH REPORT II | ALTERNATE SYSTEM 2 | ArNG Readiness Center | 3

$$w_u = 1.2DL + 1.6LL = 1.2(0.39) + 1.6(0.56) = 1.36 \text{ k/ft}$$

$$M_u = \frac{w_u l^2}{8} = \frac{(1.36)(20)^2}{8} = 68'k$$

$\phi M_n > M_u \therefore$ OKAY

DEFLECTION DURING CONSTRUCTION
- Only Dead Load is Considered
- Δ Limit $= l/360 = 0.67"$

$$0.67 = \Delta = \frac{5 w_D l_n^4 \times 1728}{384 E I_{req}} \Rightarrow \frac{5(0.39)(20)^4 \times 1728}{384(29000) I_{req}} = 0.67$$

$I_{req} = 72.26 \text{ in}^4 \quad I_{W14\times26} = 245 \text{ in}^4 \quad \therefore$ OKAY

LIVE LOAD DEFLECTION
- Δ Limit $= l/360 = 0.67"$

$w_{LL} = 65\text{psf} \times 8.67' = 0.563 \text{ k/ft}$

From Table 3-20 (AISC Steel Manual)

$I_{LB} = 485 \text{ in}^4$ FOR W14x26

$$\Delta = \frac{5 w_L l_n^4 \times 1728}{384 E I_{LB}} = \frac{5(0.563)(20)^4 \times 1728}{384(29000)(485)} = 0.144" < 0.67" \therefore \text{OKAY}$$

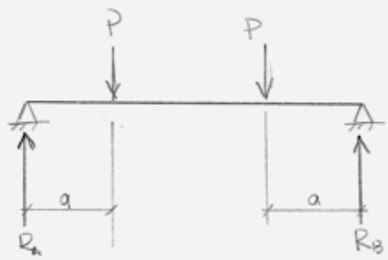
TOTAL LOAD DEFLECTION
- Δ Limit $= l/240 = 1.0"$

$$\Delta = \frac{5 w_n l_n^4 \times 1728}{384 E I_{LB}} = \frac{5(1.36)(20)^4 \times 1728}{384(29000)(485)} = 0.015" < 0.67" \therefore \text{OKAY}$$

$\boxed{\text{USE A W14x26 <16> COMPOSITE BEAM}}$

TECH REPORT II	ALTERNATE SYSTEM 2	ArNG Readiness Center	4

GIRDER DESIGN



$$P = \left[\frac{122\,psf\,(8.67')(20')}{2 \times 1000}\right] \times 2 = 21.15^k$$

FROM TABLE 3-23 (AISC Steel Manual)

$R_A = R_B = P$

$M_{max} = M_u = Pa = 21.15 \times 8'\text{-}8'' = 183.3^{'k}$

Assume $a = 1.5''$
$Y_2 = 4.0''$

FROM TABLE 3-19 (AISC Steel Manual)

$W16 \times 31 \Rightarrow \phi M_n = 313^{'k}$

$\phi M_n > M_u \therefore$ OKAY

$\Sigma Q_n = 164^k$

$$b_{eff} \le \begin{cases} \text{SPACING}/2 = \frac{20 \times 12}{2} = 120'' \\ \frac{\text{SPAN}}{4} = \frac{26 \times 12}{4} = \underline{78''} \leftarrow \text{CONTROLS} \end{cases}$$

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{164}{0.85(3)(78)} = 0.83'' < 1.5'' \therefore \text{ASSUMPTION MADE WAS CONSERVATIVE}$$

ASSUMPTION: 3/4" studs, // to slab
$w_r/n_r < 1.5$

$Q_n = 17.1\ ^k/\text{STUD}$

$$\#\text{SHEAR STUDS} = \frac{\Sigma Q_n}{Q_n} = \frac{164}{17.1} = 9.59 \Rightarrow 2 \times 10 \quad \underline{20 \text{ STUDS}}$$

TECH REPORT II | ALTERNATE SYSTEM 2 | ArNG Readiness Center | 5

CHECK GIRDER DEFLECTIONS

W16x31 ⇒ Bare Steel = $\phi M_n = 203'^k$

$w_{CONTDECK} = 42psf$

$w_{GIRDER} = 0.031\ ^{k}/ft$

$w_{DL} = (0.042 \times 20') + 0.031 = 0.87\ ^{k}/ft$

$w_{CONST} = 20psf$

$w_{L} = (0.020 \times 20') = 0.40\ ^{k}/ft$

$w_u = 1.2(0.87) + 1.6(0.40) = 1.68\ ^{k}/ft$

$M_u = \frac{w_u l_n^2}{8} = \frac{(1.68)(26)^2}{8} = 142'^k < \phi M_n \quad \therefore$ OKAY

DEFLECTION DURING CONSTRUCTION

- Δ Limit = $l/360$ = 0.867"

$\Delta = \frac{5 w_0 l_n^4 \times 1728}{384 EI} = \frac{5(0.87)(26)^4 \times 1728}{384(29000)(375)} = 0.83'' < 0.867'' \quad \therefore$ OKAY

LIVE LOAD DEFLECTION

- Δ Limit = $l/360$ = 0.867"

$w_{LL} = 65psf \times 20' = 1.3\ ^{k}/ft$

$\Delta = \frac{5(1.3)(26)^4 \times 1728}{384(29000)(375)} = 1.23''$ ← SHOULD NOT CONTROL
INCLUDE 1/2" CAMBER
IN MEMBER

$\Delta = 1.23'' - \frac{1}{2}'' = 0.73'' < 0.867'' \quad \therefore$ OKAY

USE A W16x31 <20> GIRDER WITH 1/2" CAMBER

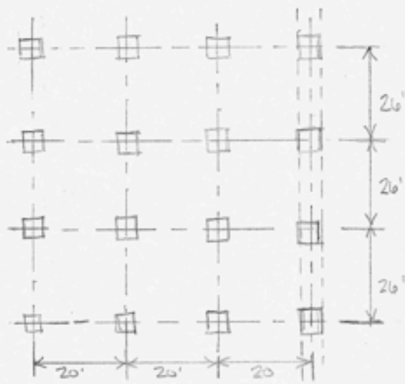
APPENDIX E: POST-TENSIONING CALCULATIONS

Presented in this appendix are the hand calculations that were performed to analyze post-tensioning as an alternate floor framing system for the Army National Guard Readiness Center.

TECH REPORT II	ALTERNATE SYSTEM 3	ArNG Readiness Center	1

POST TENSION SYSTEMS

Typical Interior Panels



Columns ⇒ 22"x22"

Loads

Dead Load = Self Weight

Superimposed Dead Load = 15 psf

Live Load = 65 psf (office Space + Partitions)

Material Assumptions:

Normal Weight Concrete = 145 pcf

$f'_{ci} = 3000$ psi

$f'_c = 5000$ psi

Rebar

$f_y = 60{,}000$ psi

PT → Unbounded Tendons

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

$f_{pu} = 270$ psi

ACCORDING TO ACI 318-08 (18.5.1)

$f_{se} = 0.7 f_{pu} = 0.7(270) - 14 psi = 175 psi$

$P_{eff} = A(f_{se}) = 0.153 \times 175 = 26.78$ k/tendon

TECH REPORT II	ALTERNATE SYSTEM 3	ARNG Readiness Center	2

DETERMINE SLAB THICKNESS

$\text{SPAN}/\text{DEPTH} = \frac{L}{45}$

$\frac{(26)(12''/ft)}{45} = 6.93'' \rightarrow \boxed{\text{USE 7'' SLAB}}$

EAST-WEST INTERIOR FRAME

Loading:

Self Weight $= \frac{7in(145pcf)}{12^{in/ft}} = 84.6$ psf

SDL = 15 psf

LL = 65 psf

$A = bh = (20' \times 12''/ft)(7in) = 1680 \text{ in}^2$

$S = \frac{bh^2}{6} = \frac{(20 \times 12)(7)^2}{6} = 1960 \text{in}^3$

AT TIME OF JACKING: $f'_{ci} = 3000$ psi

Compression $= 0.6f'_{ci} = 0.6(3000) = 1800$ psi

Tension $= 3\sqrt{f'_{ci}} = 3\sqrt{3000} = 164$ psi

AT SERVICE LOADS: $f'_c = 5000$ psi

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

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

ACCORDING TO ACI 318-08 (18.12.4)

$P/A > 125$ psi

TARGET LOAD BALANCE $= 0.75 w_{DL}$

$0.75w_{DL} = 0.75\left[\frac{7in(145psf)}{12}\right] = 84.6$ pcf

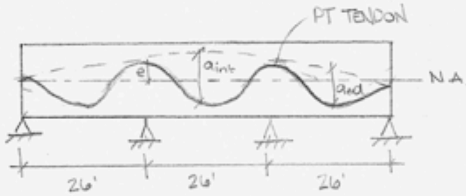
USE CARBONATE AGGREGATE TO ACHEIVE 2HR FIRE RATING

RESTRAINED SLABS = 3/4" Bottom

UNRESTRAINED SLABS = 1½" Bottom
3/4" Top

TECH REPORT II	ALTERNATE SYSTEM 3	ArNG Readiness Center	3

TENDON PROFILE :



TENDON ORDINATE	TENDON (CG) LOCATION
EXT. SUPPORT - ANCHOR	4.0"
INT. SUPPORT - TOP	7.0"
INT. SPAN - BOTTOM	1.0"
END SPAN BOTTOM	1.75"

$a_{int} = 7.0 - 1.0 = 6.0"$

$a_{end} = \left[\frac{4+7}{2}\right] - 1.75 = 3.75"$

$W_b = 0.75 W_{DL} = 0.75(84.6 \times 20') = 1.30\ ^{k/ft}$

$P = \frac{w_b l^2}{8(a_{end})} = \frac{(1.30)(26)^2}{8\ (3.75)} = 206.78^k$

$\text{No TENDONS} = \frac{206.78^k}{26.78\ ^k/\text{tendon}} = 7.72 \Rightarrow$ $\underline{8 \text{ TENDONS}}$

$P_{ACTUAL} = 8 \text{ TENDONS} \left(26.78\ ^{k/tendon}\right) = 214.14^k$

$w_b = \left(\frac{214.14}{206.78}\right)(1.30) = 1.35\ ^{k/ft}$

$P_{ACTUAL}/A = \frac{214.14}{1680} = 128 \text{ psi} > 125 \text{ psi} \quad \therefore \underline{\text{OKAY}}$

CHECK INTERIOR SPAN FORCE

$$P = \frac{1.30(26)^2}{8(10/12)} = 130^k$$

$130^k < 206^k$ $\therefore$ LESS FORCE IS REQUIRED FOR INTERIOR SPAN (DO NOT NEED TO RECHECK)

$\boxed{P_{eff} = 206.73^k}$