# Technical Report I

## Army National Guard Readiness Center
Arlington, Virginia

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# EXECUTIVE SUMMARY

Technical Report I is a structural concepts and existing conditions report, which describes the structural system of the Army National Guard Readiness Center Addition. Research was performed into the structural systems of the building and methods used for design. This report uses the current standards to check the design of the structure.

*Figure 1: Site Excavation*

The 8-story joint headquarters building, located in Arlington, Virginia, is a concrete structure utilizing a 43" mat foundation, flat slab concrete floor system with column strips and edge beams, ordinary reinforced concrete shear walls and various reinforced concrete columns. Typical interior columns are 22" by 22", which are monolithic from top to bottom of the building 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.

*Figure 2: Mat Slab Reinforcement*

For this report, seismic and wind loads were calculated using Chapters 12 and 6 of ASCE 7-05 respectively. The analytical method 2 in Section 6.5 was used to calculate the wind loads while seismic loads were calculated using the equivalent lateral force (ELF) procedure. After the loads were found it was determined that seismic base shear controls the lateral forces and wind forces in the East-West direction control the overturning moment.

Spot checks were performed on the three main structural components of the building. The first spot check was an edge beam along the eastern side of the building. A typical interior 22" by 22" column was also checked at three different levels. These levels were where the reinforcement changes. Via the calculations in this report it was determined each component is adequate and seemingly oversized for the forces. As noted in the report, however; only gravity loads were considered for this analysis and it is likely that once that lateral loads are considered, the members will need to carry a greater capacity, hence the larger design. The final spot check was completed to check the thickness of the floor slab using a typical 20' by 25' bay.

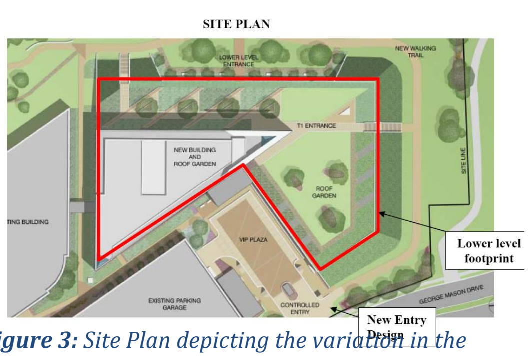



*Figure 3: Site Plan depicting the variation in the building footprint*

## INTRODUCTION

The ArNG headquarters addition is sited to the south of the existing facility, where the storm water retention pond had been 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 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 encompasses 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 entities 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 to 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 entities will be discussed of 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 4:*** West Perspective

## STRUCTURAL SYSTEM

### *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 monolithic 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.

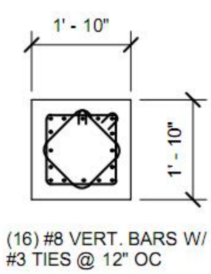



***Figure 5:*** *Typical Interior Column*

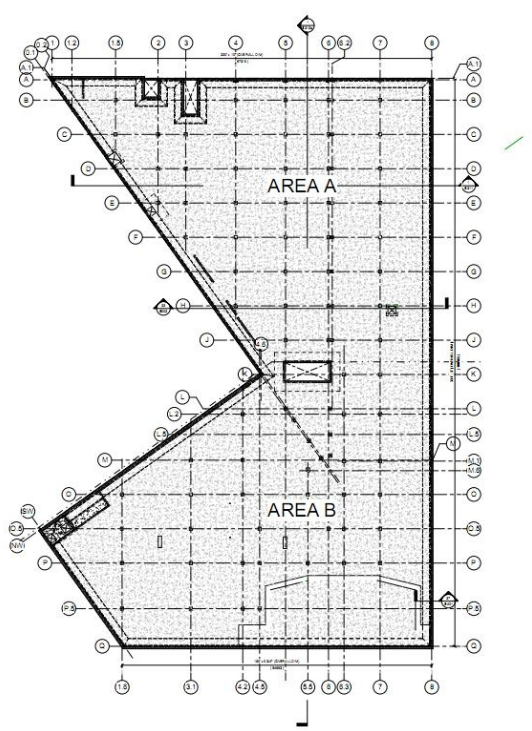



***Figure 6****: Typical Column Layout for Underground Levels*

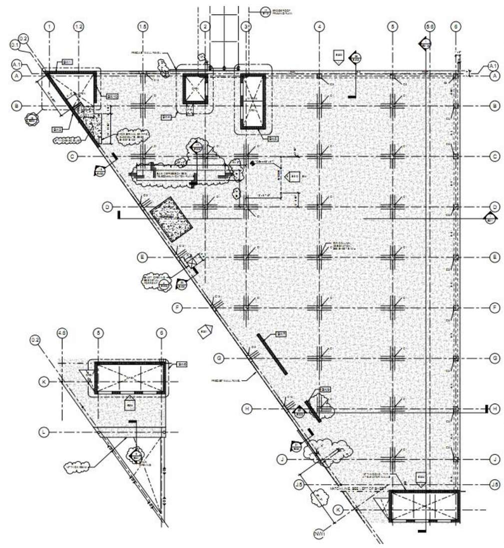


***Figure 7:*** *Typical Column Layout for Tower Levels*

*Floor Systems*

The Army National Guard Readiness Center Addition utilizes a concrete structural system. All of the floors are a 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 pounds per square inch. 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.

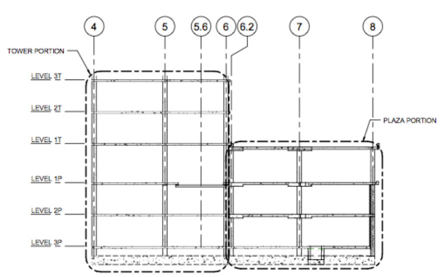



***Figure 9:*** *Elevation showing location of expansion joint and relationship between Plaza portion and Tower portion*

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 8 and 9

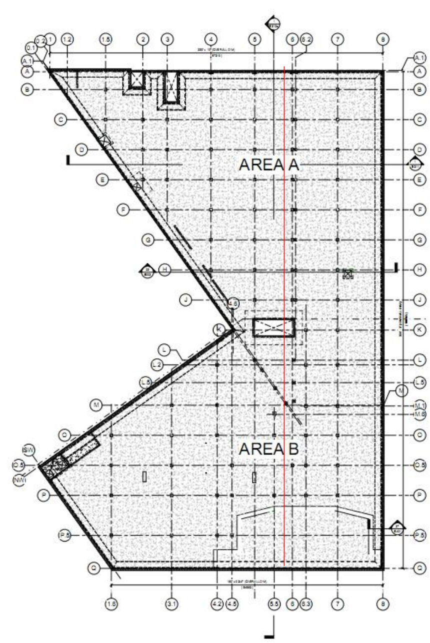



***Figure 8:*** *Location of Expansion Joint*

### *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.

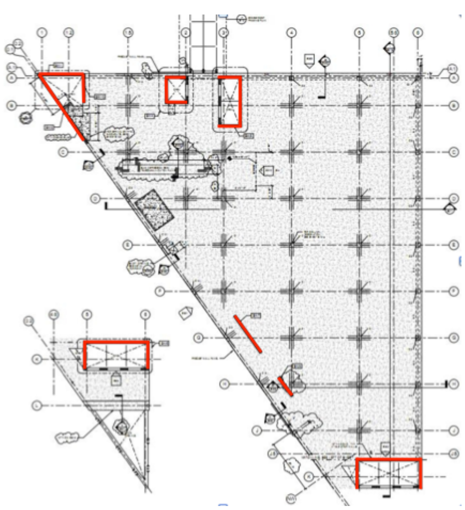


***Figure 10:*** *Typical Shear Wall Locations in Below Grade Levels*

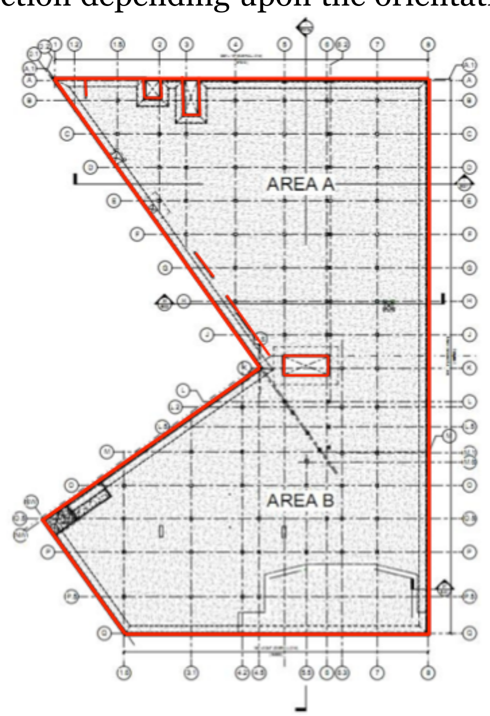



***Figure 11:*** *Typical Shear Wall Locations in Tower Levels*

# 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
- 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 | - |

## GRAVITY & LATERAL 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 |

*Snow Loads*

The flat roof snow load for the Army National Guard Readiness Center was calculated in accordance with Chapter 7 of ASCE 7-05. A summary of the snow load factors that were used can be found in Table 4. Due to the lack of information regarding the heights of mechanical units, the drift snow load was not taken into consideration for this technical report. This load could have a significant impact near mechanical equipment and parapets and will be considered for further analysis.

| Table 4: Snow Load Criteria | |
|---|---|
| Ground Snow Loads | Pg=25 psf |
| Snow Exposure Factor | Ce = 0.9 |
| Snow Importance Factor | I = 1.2 |
| Thermal Factor | Ct = 1.0 |
| Flat Roof Snow Load | pf = 19 psf |
| Minimum Required Load | Pf = 20 psf |

*Wind Loads*

In accordance with IBC 2006, the wind loads on the building were determined by the provisions of ASCE Chapter 6. To examine the lateral wind loads in both the North/South and East/West direction, Method 2, the analytical method, was used. From Figure 6-1 in ASCE 7-05 it was found that the basic wind speed in Arlington, VA was 90 mph. This method does not take into account any apparent shielding afforded by other building to reduce wind velocity.
This could be crucial due to the relative proximity of the new facility with the existing structures that surround the building. For this technical report, a few assumptions were made to simplify the procedure. The main assumption was the ArNG Readiness Center Addition was to be considered a regular-shaped building. A more accurate and detailed analysis of lateral loads will be conducted in a later technical report. Using the commentary within ASCE 7-05 the approximate fundamental frequent of the building was calculated. It was determined from this that the building is flexible in nature and the Gust Factors were calculated accordingly (Refer to Appendix B for calculations). Figures below summarize the story forces and shear in both the north-south and east-west direction. Appendix B contains detailed spreadsheets, calculations, and criteria that were determined to ascertain the wind forces.

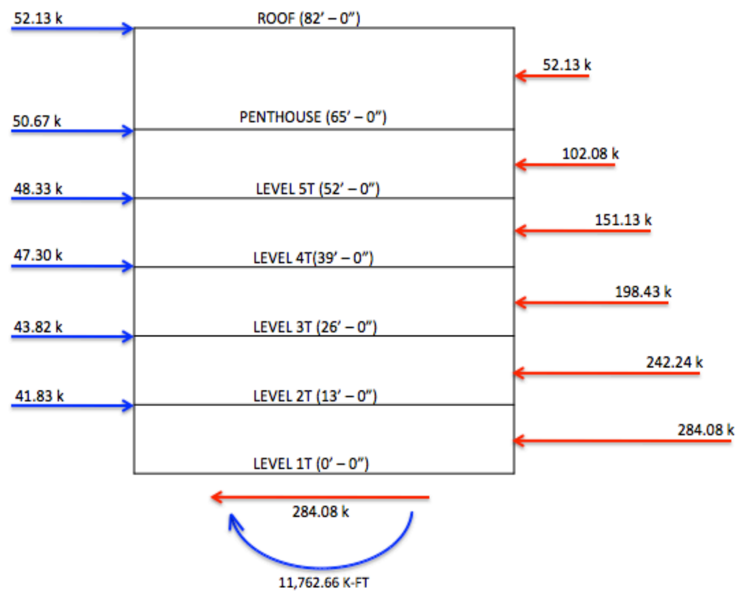



***Figure 12:*** *Story Forces and Shear in the North-South Direction*

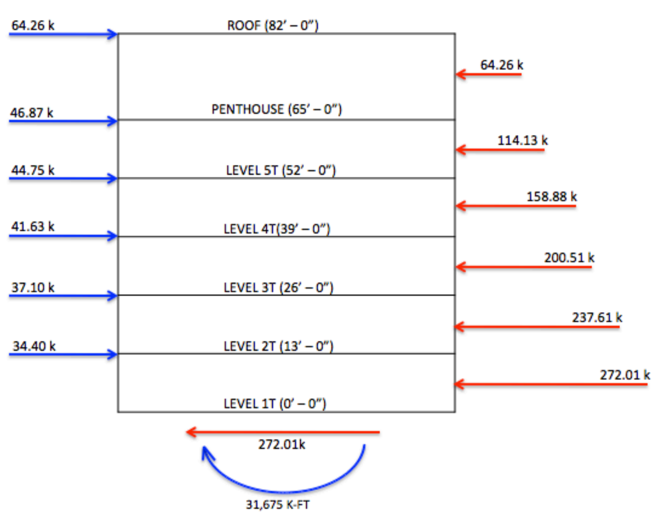



***Figure 13:*** *Story Force and Shear in the East-West Direction*

## Seismic Loads

Chapters 11 and 12 of ASCE 7-05 were referenced in order to calculate the seismic forces on the Army National Guard Readiness Center. It was assumed that the ArNG Readiness Center employed a rigid diaphragm, which allowed for the use of the Equivalent Lateral Force Procedure (ELF) found in section 12.8 of ASCE 7-05 standards. Upon investigation of the geotechnical report provided by CH2M Hill, it was determined that the Army National Guard Readiness Center falls under Site Class D. $S_S$ and $S_1$ were then determined using the United State's Geological Surveying (USGS) website. All design variables and site parameters that were used in determining the seismic loads can be found in Appendix C along with detailed calculations and spreadsheets that were utilized to obtain the building weight, base shear, and overturning moment. Figure 14 is a loading diagram that summarizes the story forces, base shear, and overturning moment acting on the ArNG Readiness Center due to seismic loads.

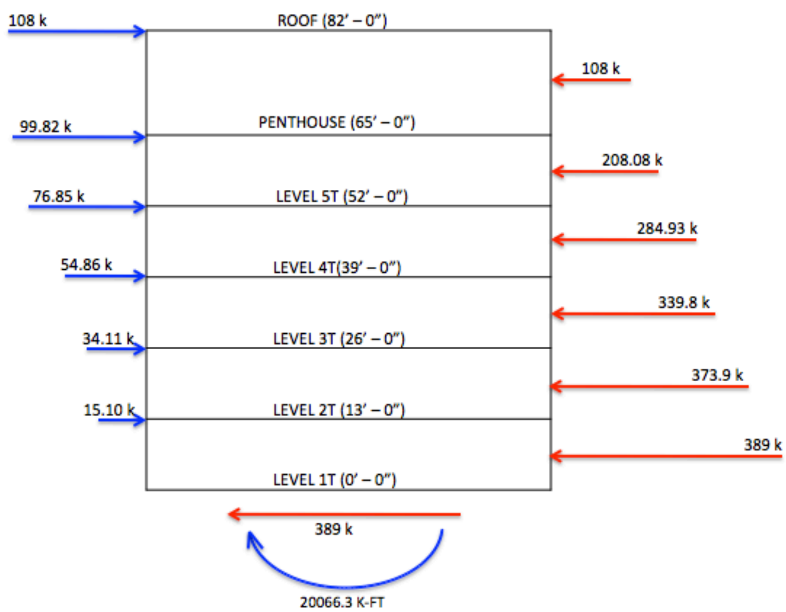



***Figure 14:*** *Story Forces and Shear*

## Other Loads

There are a number of other important loads that require further research but will need to be considered in future analysis. For instance, the subgrade walls are subjected to a lateral soil pressure that needs to be taken into consideration. Also, the foundation should be checked to determine if it is sufficient to overcome any hydraulic lift forces that would be present. Since the Army National Guard Readiness Center is a government building, there are Department of Defense building regulations that are required, such as blast and progressive collapse criteria. If these requirements create a worse case scenario it could result in the need for larger members and more reinforcement than compared to an ordinarily designed structure.

# SPOT CHECKS

## *Gravity Beam Spot Check*

For this technical report, three types of spot checks were completed, one edge beam spot check and 3 column spot checks. For the first spot check, a typical edge beam on the east side of level 3T (See figure 15 below for exact location) was chosen to be analyzed to see if it was capable of carrying the determined loads. The results were determined using methods from ACI 318-05. The middle of the beam was reinforced differently than the left and right side of the beam therefore; the two most crucial sections on the beam were analyzed. The beam was 18 inches by 24 inches with #9 reinforcing bars and a #3, closed stirrup spaced at six inches on center. To get the moments and shears in the beam, ACI moment coefficients were used. Calculations in Appendix D show the beam is adequate to carry the determined forces.

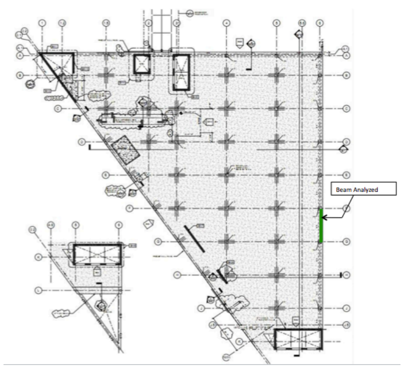


***Figure 15:** Location of Edge Beam Analyzed*

## *Gravity Column Spot Check*

For the column spot checks, three capacity checks at the existing column line 5E were performed at each level where the reinforcement changes, level 3P, 1T, and 3T (See figure below for exact location). For each check three points were chosen to check the interaction diagram. Due to the simplifying assumption that the columns were not part of the lateral system, only the compressive forces were actually required. PCA Column was used to plot the interaction diagrams and the load of the column; these results can be seen in Appendix D. From the results of the calculations and interaction diagrams, it is clear that the column is adequately designed for the determined loading. After the analysis, it seems as if the columns are oversized, however this is most likely due to the fact that the structural material is concrete and therefore the columns will see moment and take some of the lateral load. The lateral system will be analyzed in depth in a later technical report. Detailed calculations can be referenced in Appendix D.

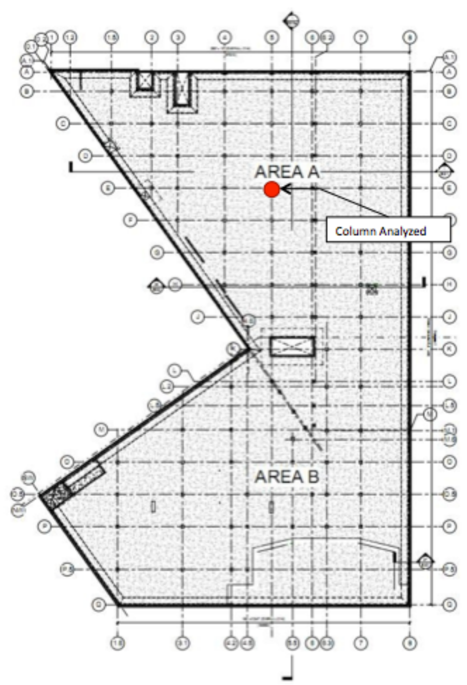



***Figure 16:** Location of Column Analyzed*

### *Slab Spot Check*

While the columns follow a typical gridline pattern, it was difficult finding an area of the building with typical adjacent bays that included edge beams and could be analyzed due to the irregular shape of the building. The area that was chosen for this slab check was in the northeast corner of the tower portion of the building. Here the building forms a 90 degree angle and follows a typical bay layout (Reference Figure 17). The spot check was used to analyze the structural engineer's use of a 9" slab thickness. ACI's Direct Design Analysis was used to determine the alpha values for each of the edge beams. Following the ACI Chapter 16 guidelines it was determined that a 7.5" slab thickness is sufficient as the minimum therefore the 9" slab was adequate. Detailed calculations are provided in Appendix D.

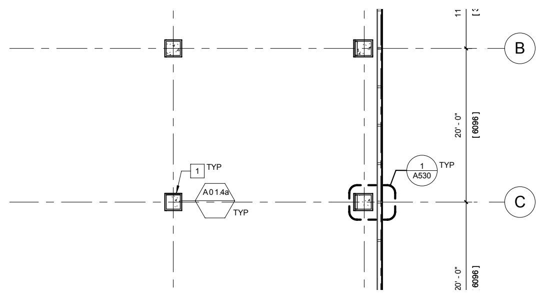



***Figure 17:*** *Typical Bay used to Analyze Slab*

## CONCLUSION

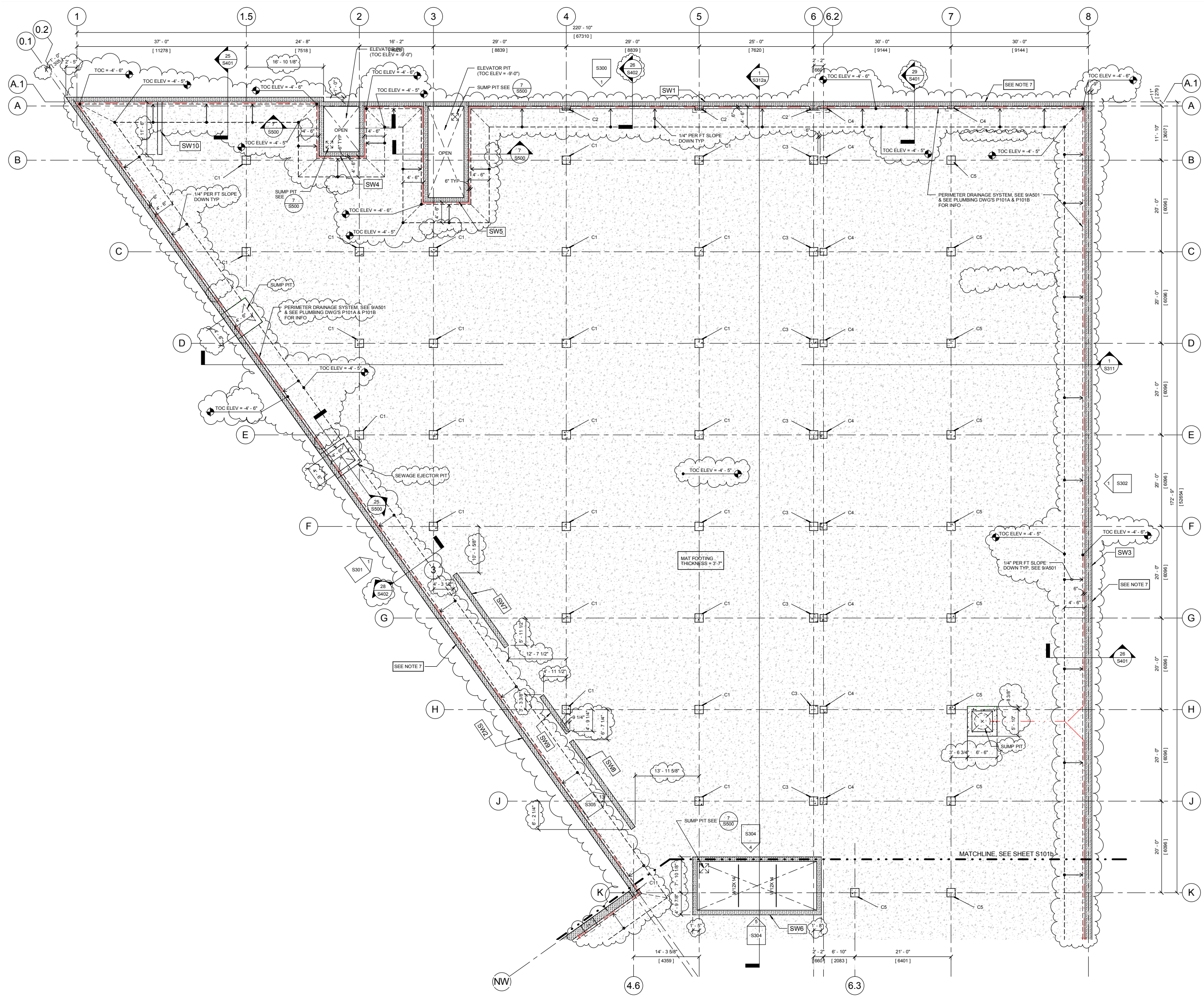
Technical Report I is an investigation into the structural concepts and existing conditions of the Army National Guard Readiness Center in an attempt to better understand the design decision made at the onset of the project. This report includes a detailed discussion of the structural system and floor framing plans for a more comprehensive look into how the structure works.

Calculations were performed on individual members in order to verify the engineer's design. These spot checks concluded that some of the members may be oversized however, as stated in the report, these checks were completed with the simplifying assumption that they do not act as part of the lateral system and therefore do not take any moment forces. This will be taken into consideration in a more in-depth analysis of the lateral system in a future technical report.

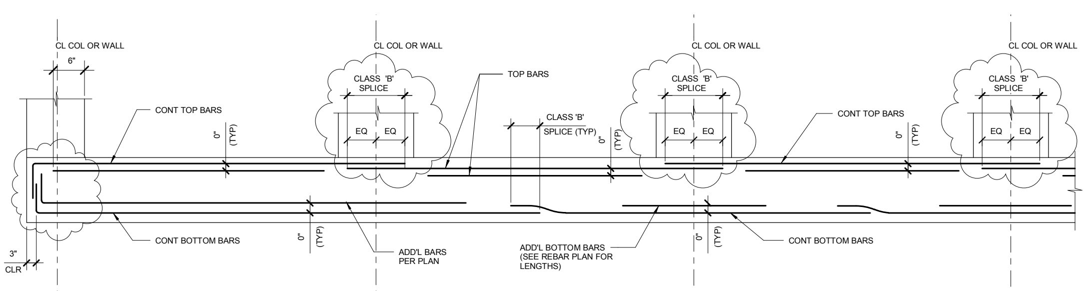
This Technical Report also includes calculations into the lateral design forces that act on the Army National Guard Readiness Center. From the calculations of the wind loads and seismic loads it was determined that the seismic loads control the base shear on the building while the wind forces in the east-west direction control the overturning moment.

# APPENDIX A: BUILDING LAYOUT

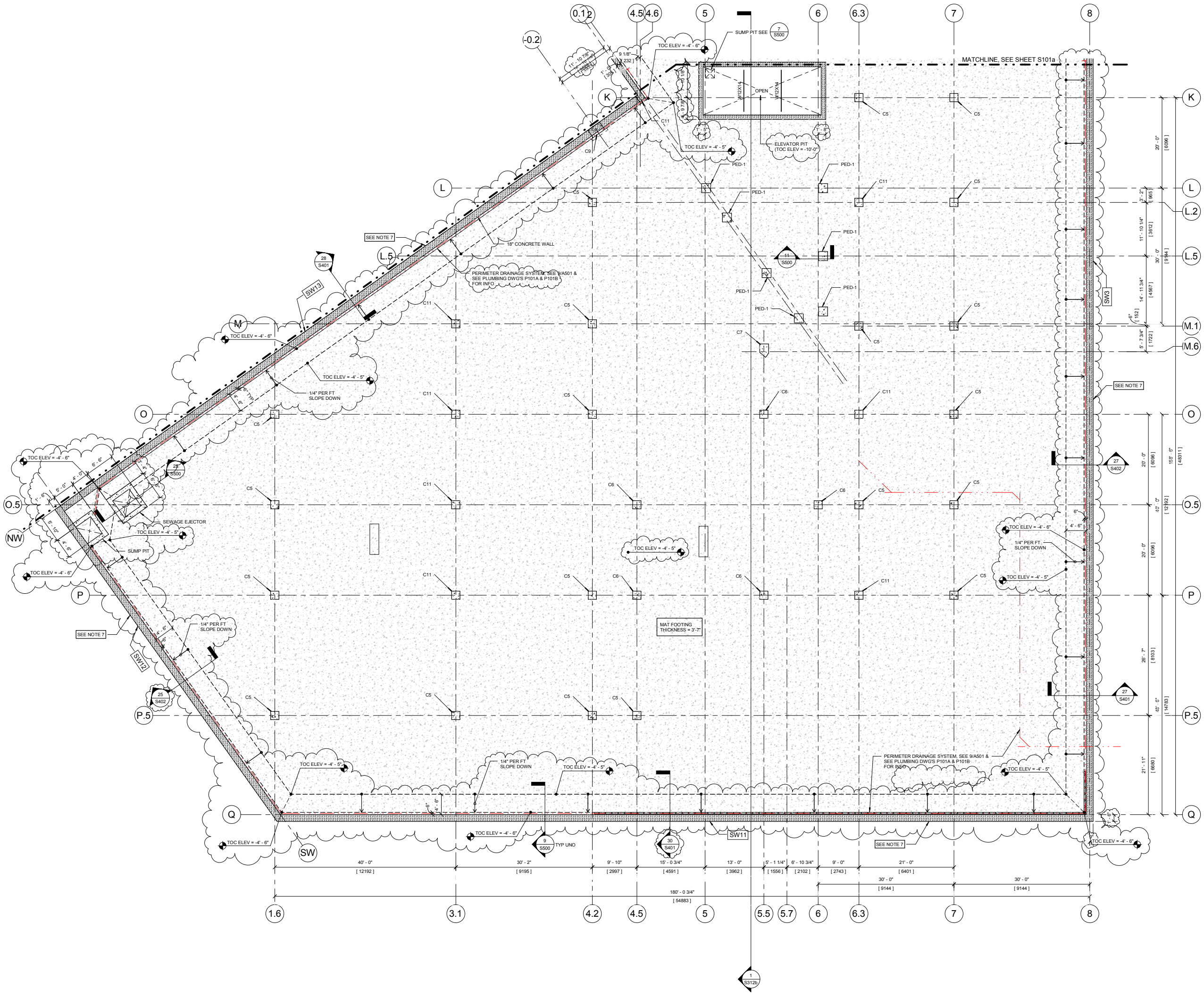
Presented in this appendix are some of the main drawings and details that were referenced during the investigation and research to complete this technical report.



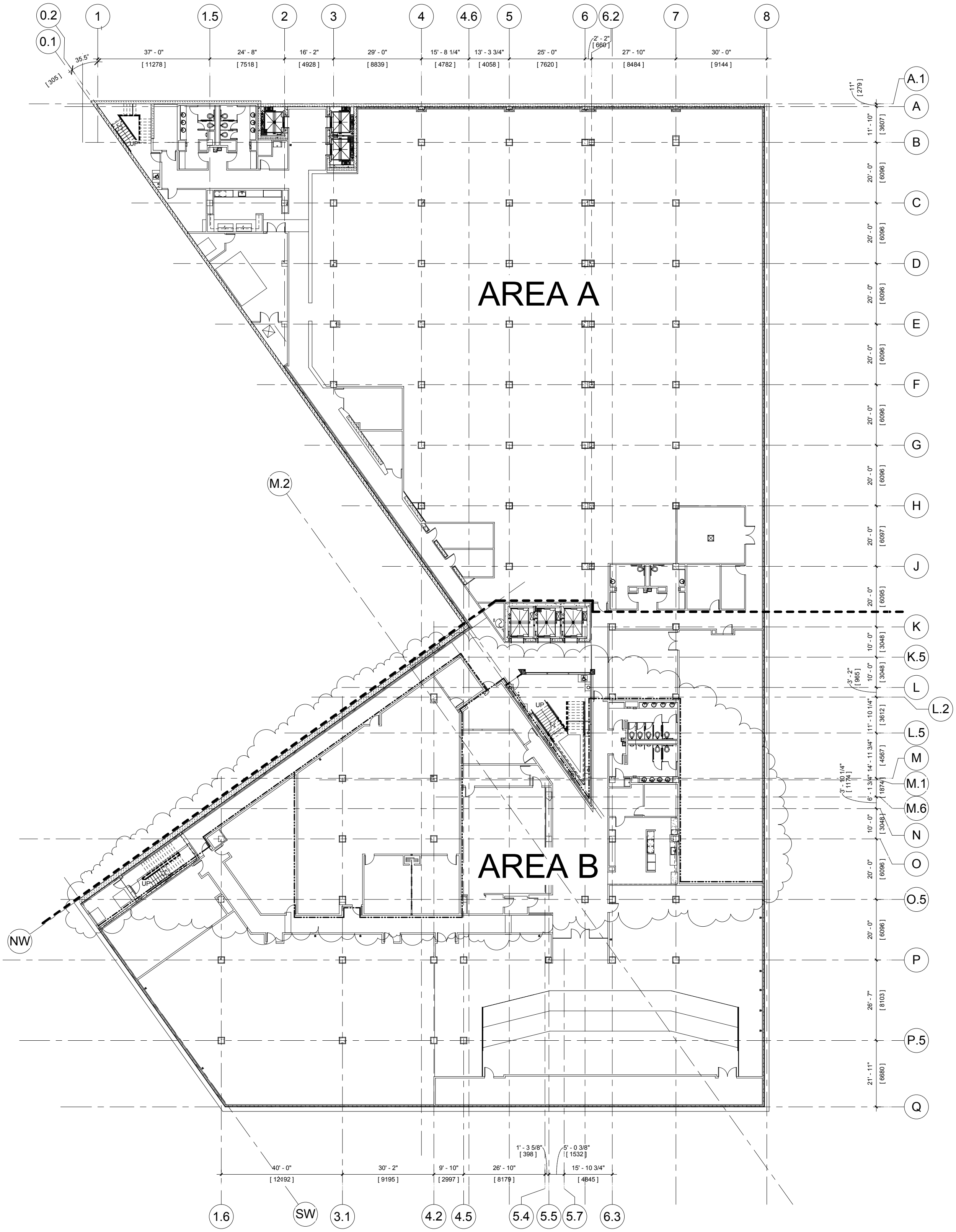
Partial Mat Foundation Plan



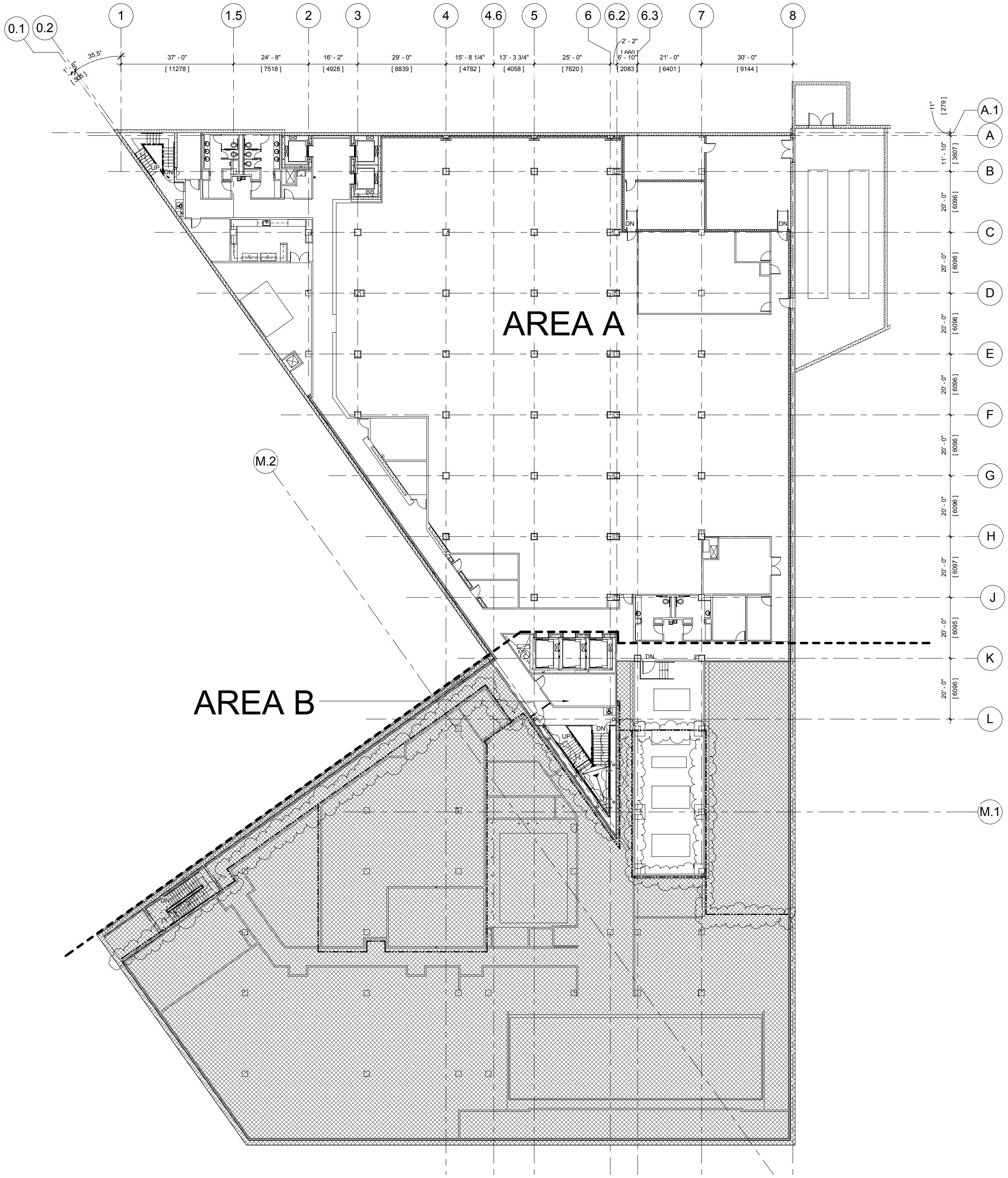

Typical Mat Foundation Detail



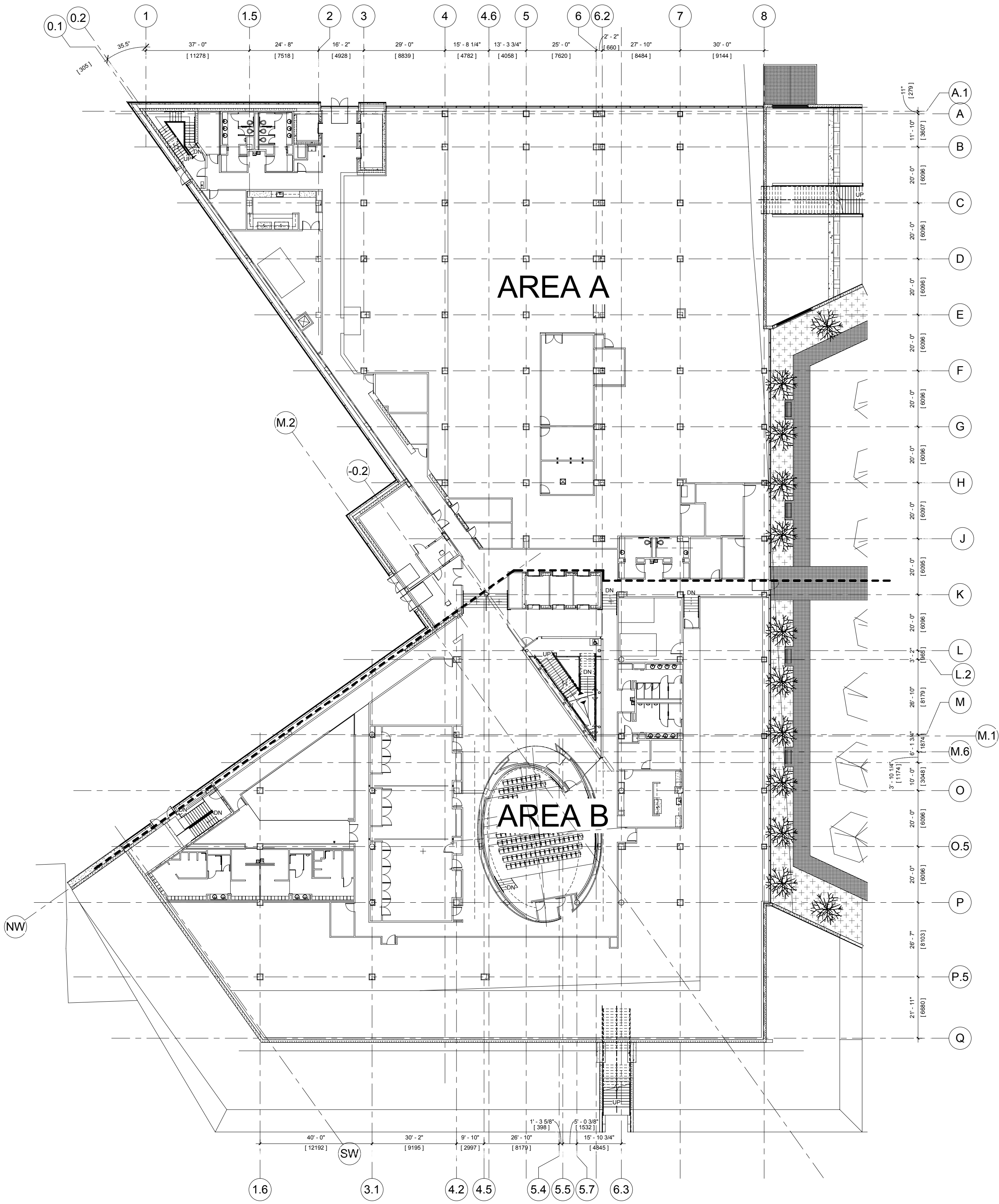
Partial Mat Foundation Plan



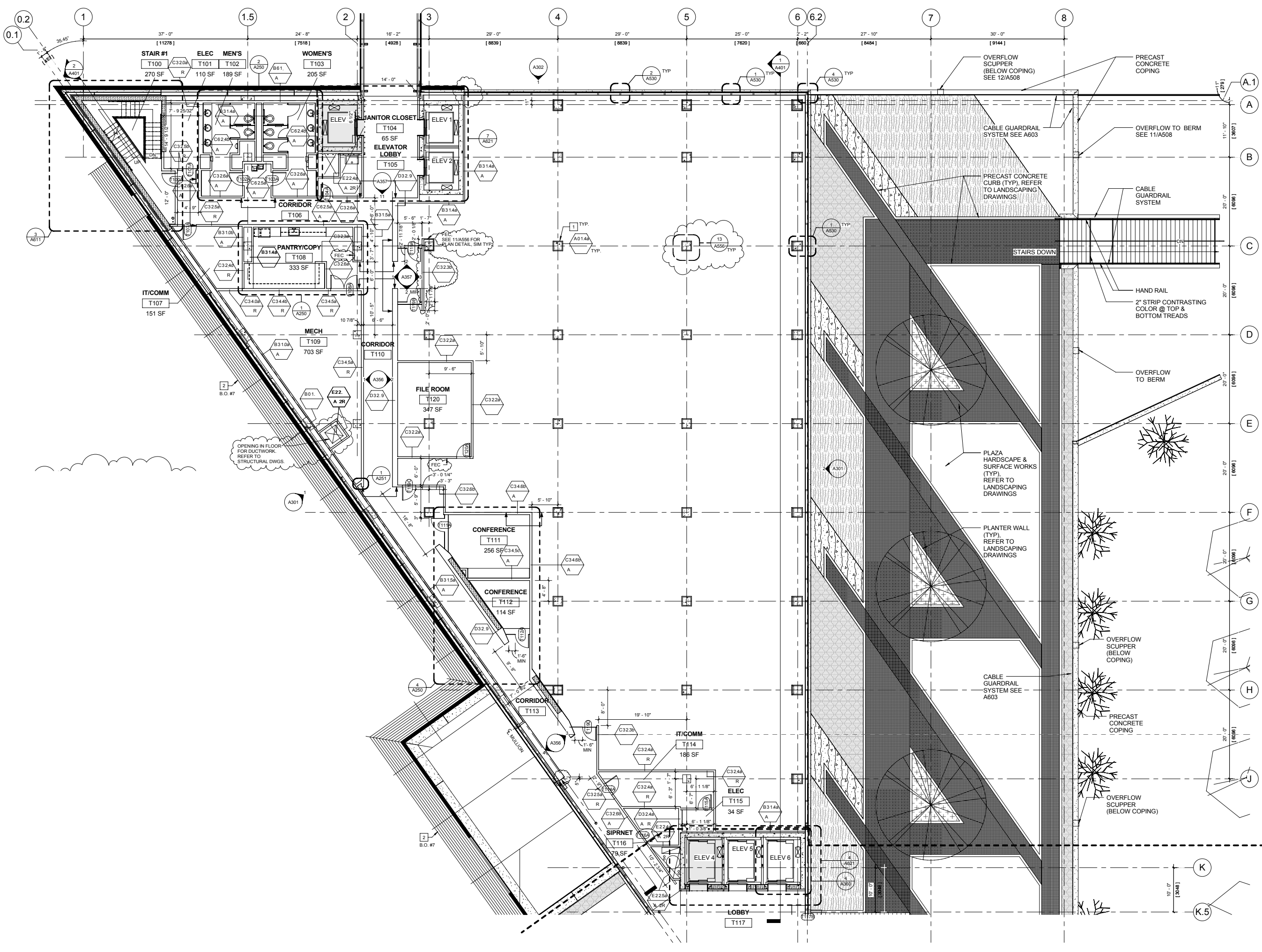

Level 3P Floor Plan



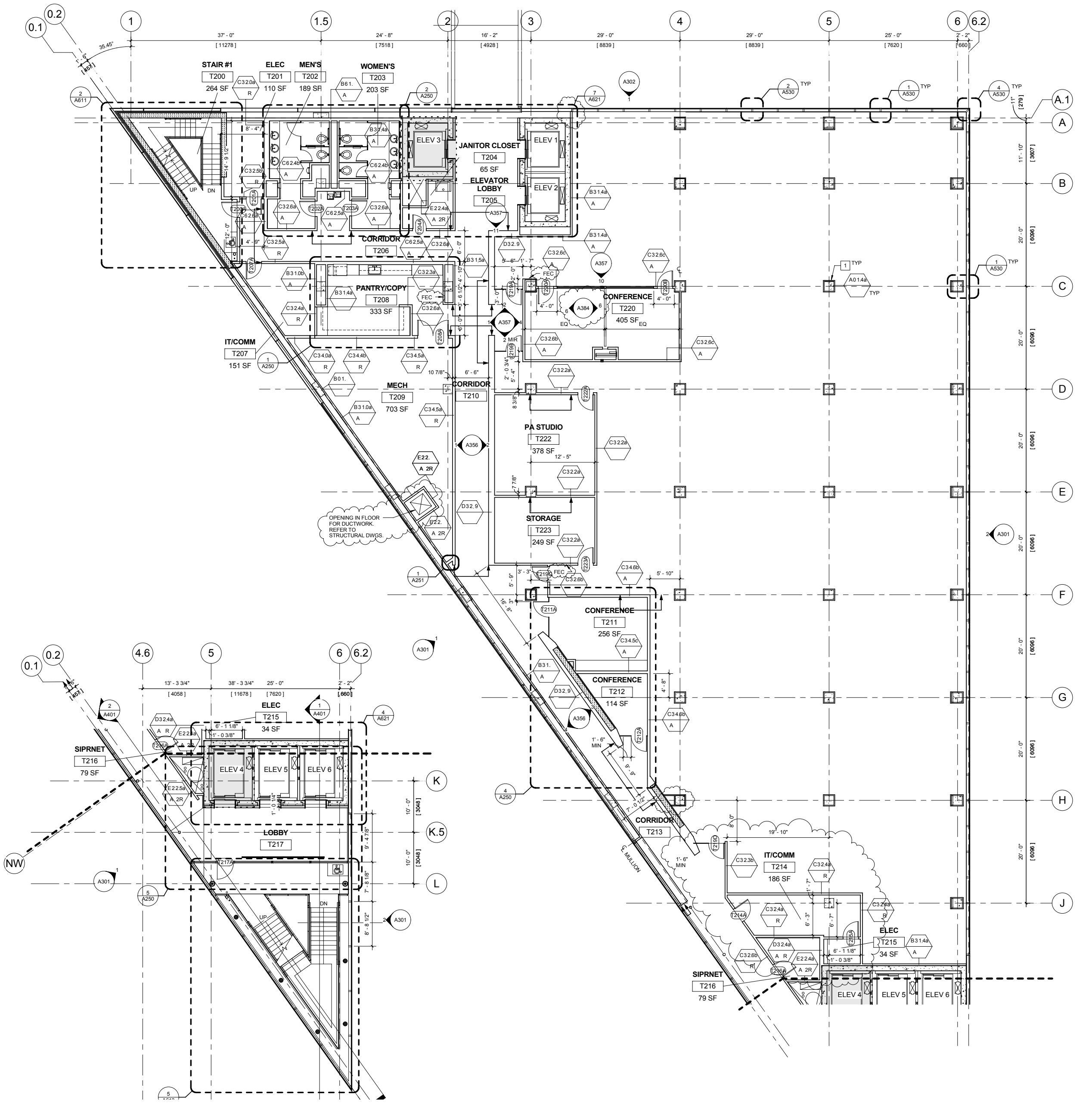

Level 2P Floor Plan



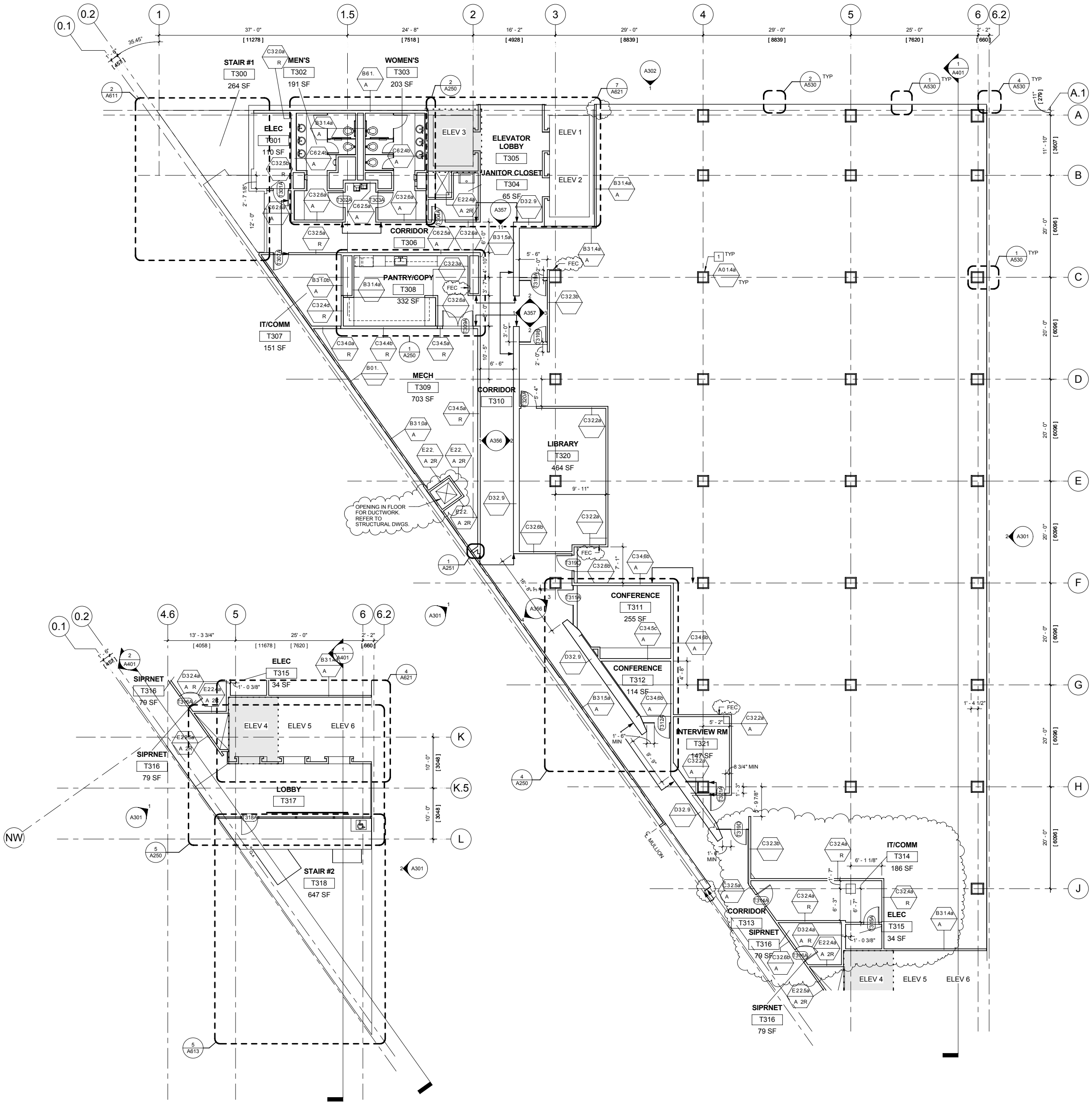

Level 1P Floor Plan



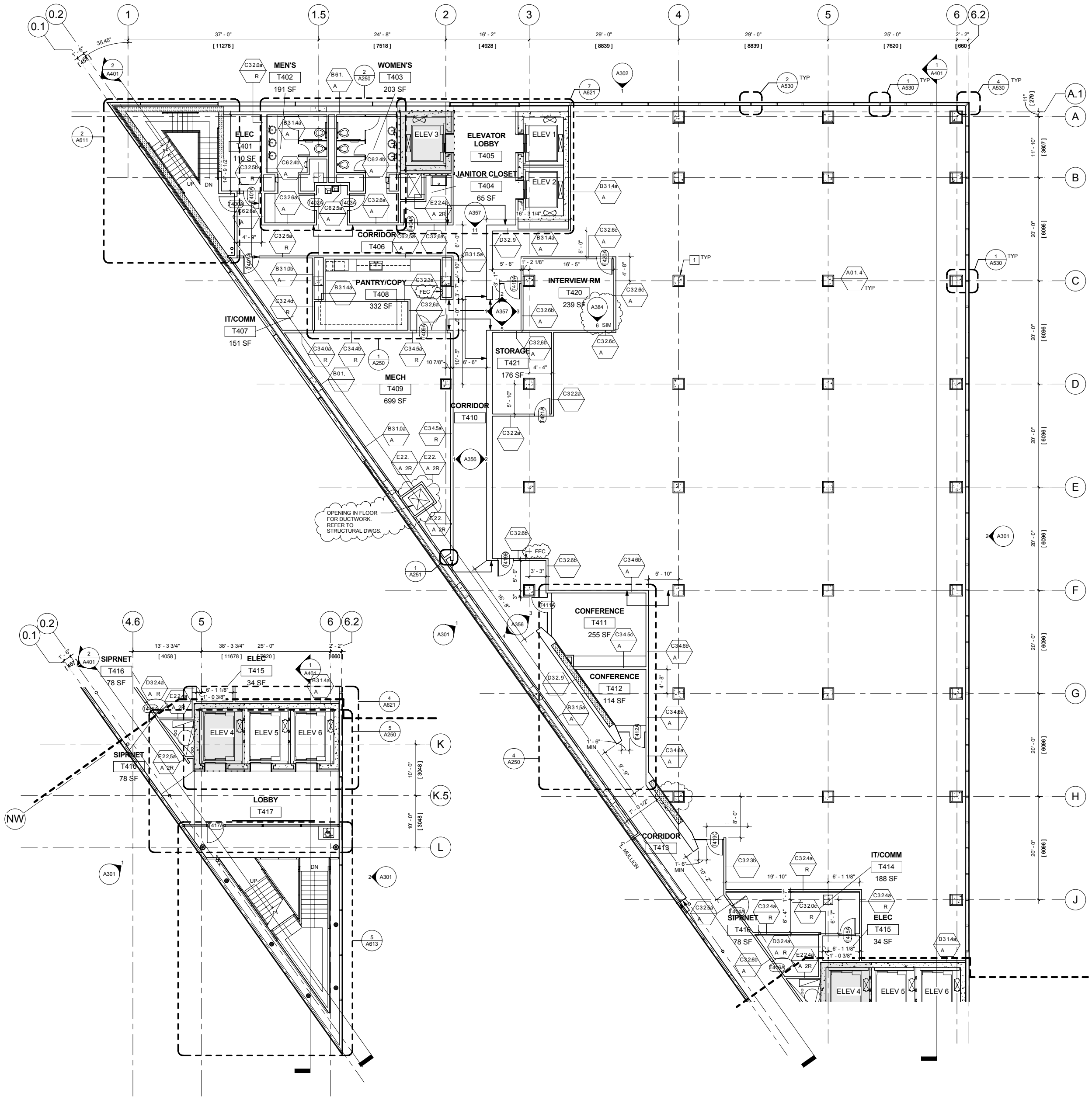
Partial Level 1T Floor Plan



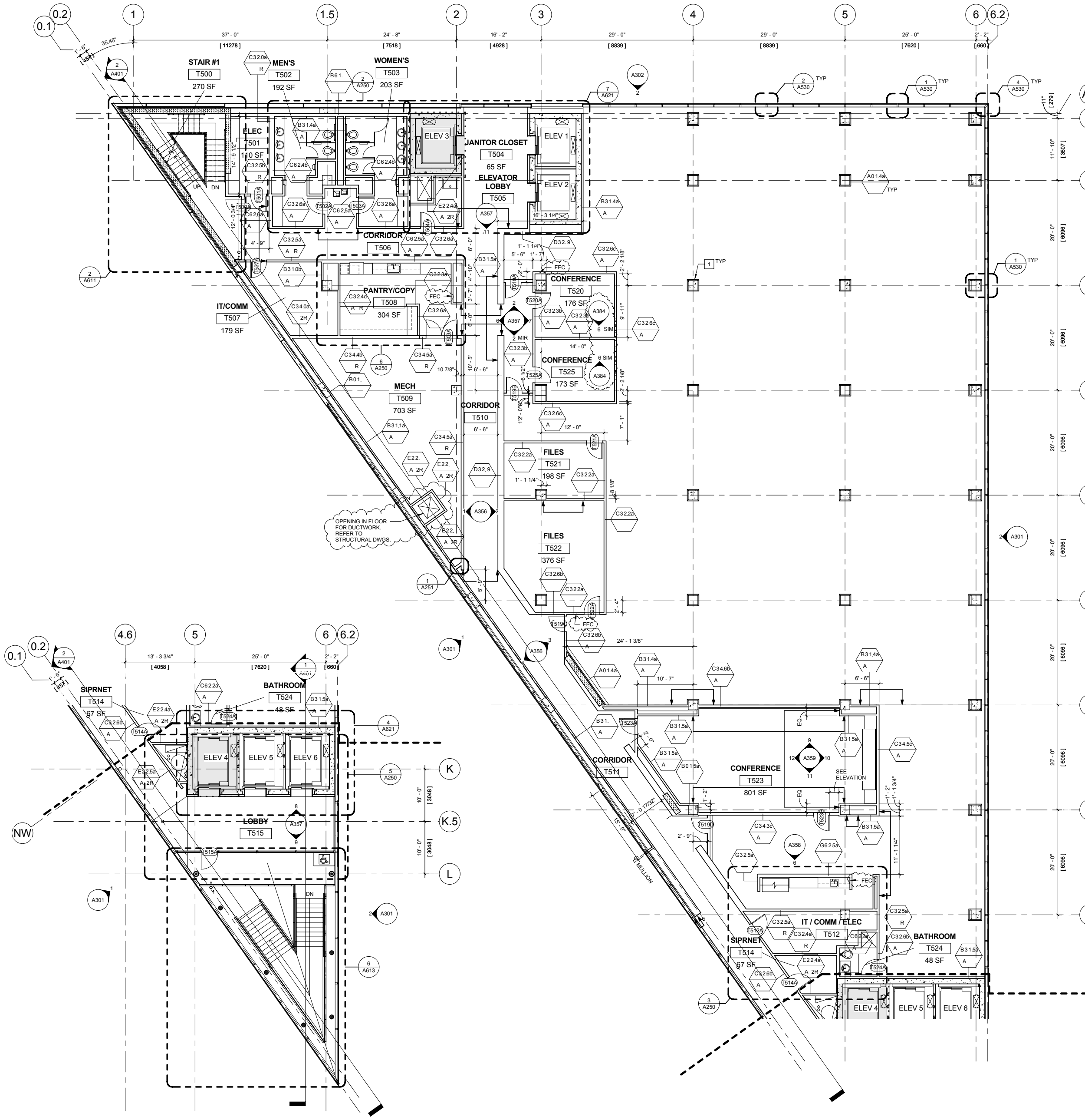
Level 2T Floor Plan



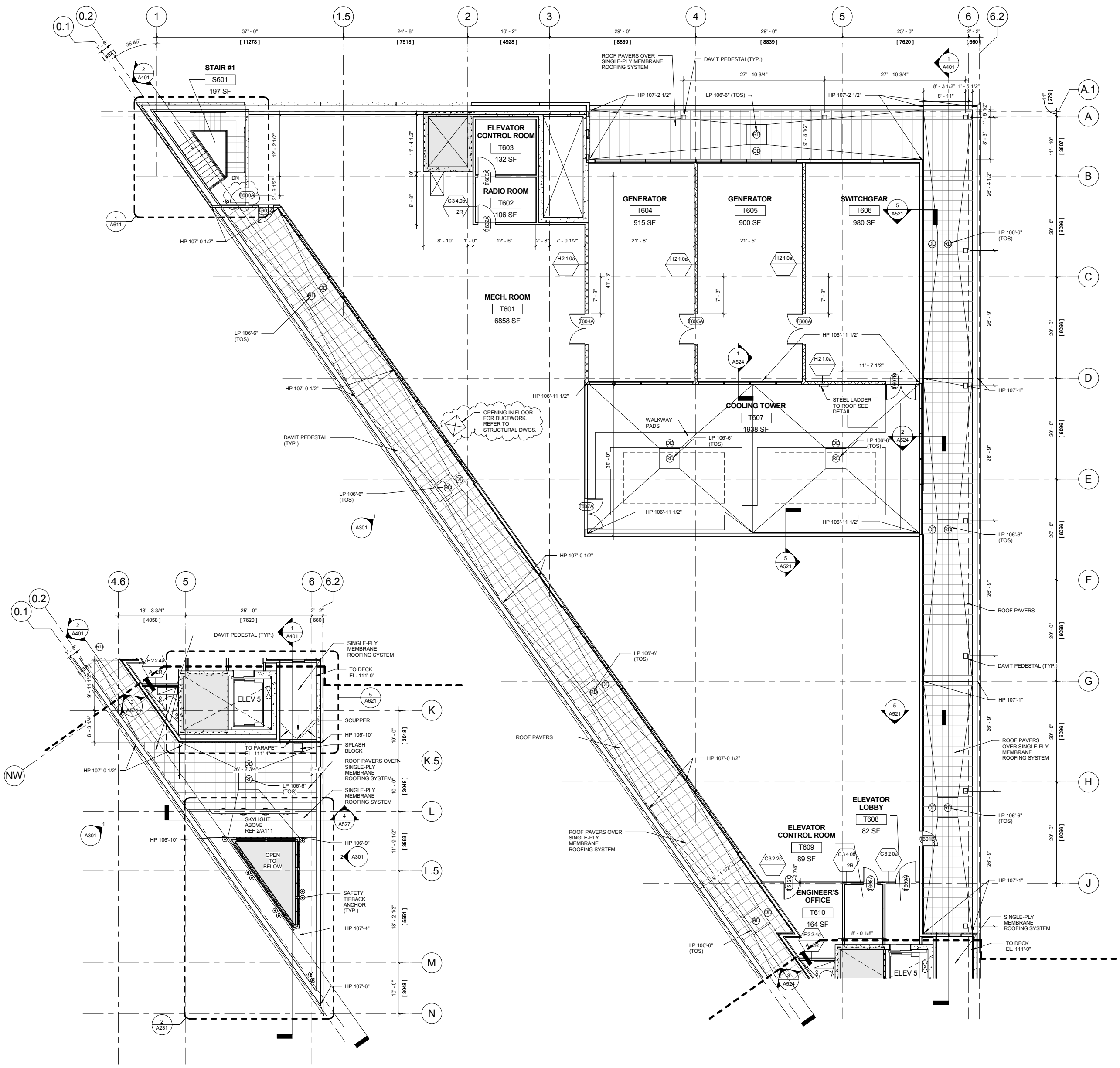

Level 3T Floor Plan



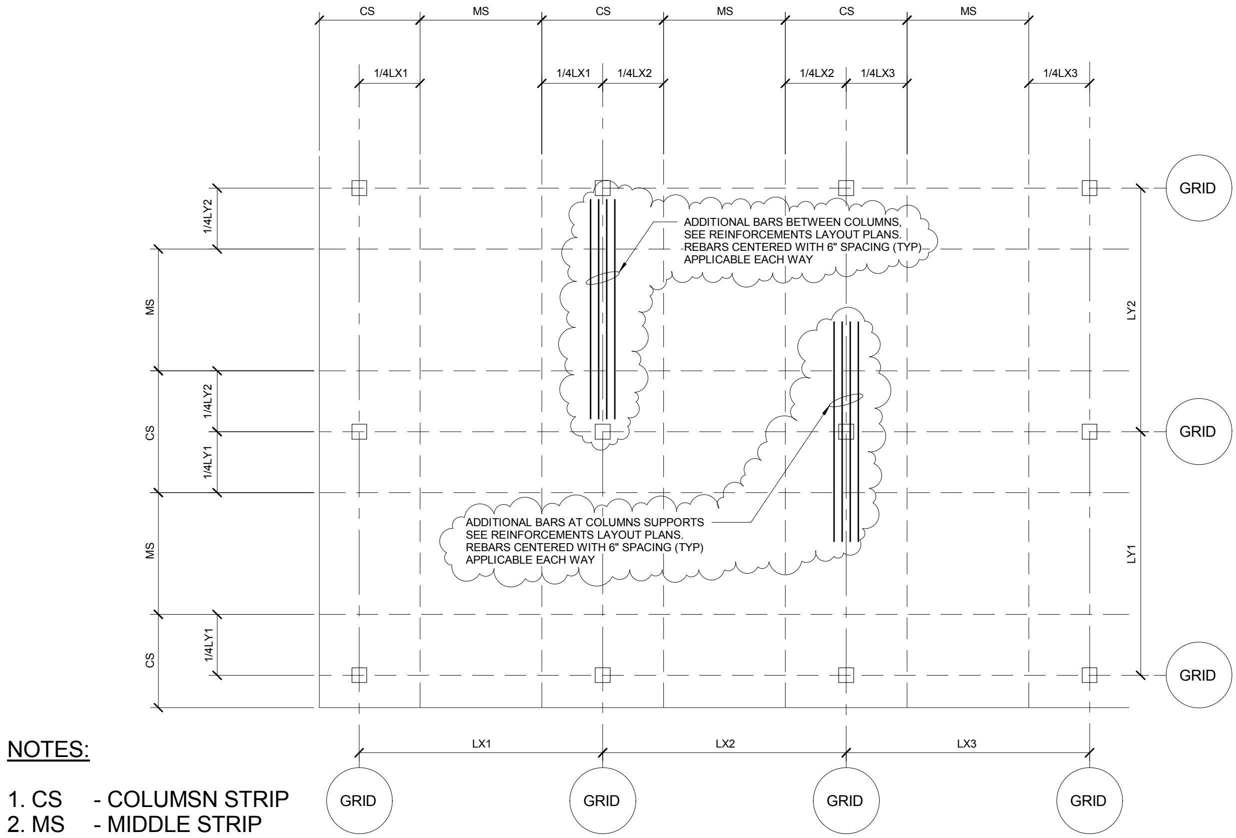
Level 4T Floor Plan



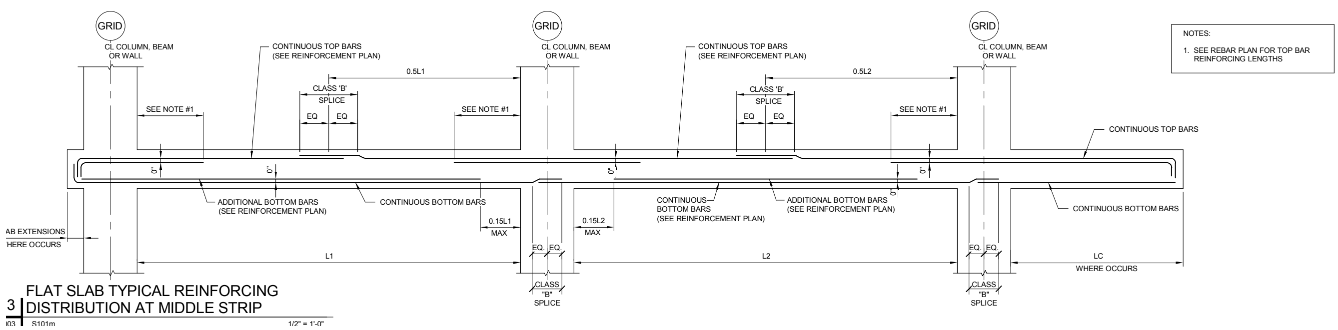

Level 5T Floor Plan



Level PH Floor Plan




Typical Column Strip and Middle Strip Detail




Typical Flat Slab Detail

# APPENDIX B: WIND LOAD CALCULATIONS

Presented in this appendix are summaries of variables and building parameters required to determine wind loads in both the North-South and East West directions. Hand calculations were performed and can be referenced here as well as force distribution tables and diagrams used to determine the base shear and overturning moments caused by wind forces.

TECH REPORT I | WIND DESIGN | ArNG Readiness Center | 1/1

- Determine lateral forces on the building caused by wind using analytical procedure from ASCE 7-05 Chapter 6 (Method 2)

- BASIC WIND SPEED (From Figure 6-1)

  V=90 mph (V < 120 mph therefore not located in wind blown debris zone)

- LOCATION PARAMETERS

  $K_d = 0.85$ (Table 6-4)
  $I = 1.15$ (Table 6-5)
  Exposure Category C
  $K_{zt} = 1.0$ (Table 6-4)

- VELOCITY PRESSURE EXPOSURE COEFFICIENT ($K_z$)
  Building Height = 82 ft (Including Parapet)

| HEIGHT(ft) | $K_{z}$ (Table 6-3) |
|---|---|
| 82 | 0.94 |
| 80 | 0.93 |
| 70 | 0.89 |
| 60 | 0.85 |
| 50 | 0.81 |
| 40 | 0.76 |
| 30 | 0.7 |
| 25 | 0.66 |
| 20 | 0.62 |
| 0-15 | 0.57 |

- VELOCITY PRESSURE

$$q_z = 0.00256\, K_z K_{zt} K_d V^2 I$$

$$q_z = 0.00256 K_z (1.0)(0.85)(90)^2(1.15) = \underline{20.269 K_z}$$

-REFERENCE CHART FOR ACTUAL VALUES

- COMBINED NET PRESSURE COEFFICIENT ($GC_{pn}$)
  § 6.5.12.2.4

  $GC_{pn} = +1.15$ (WINDWARD PARAPET)

  $GC_{pn} = -1.0$ (LEEWARD PARAPET)

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|---|---|---|---|

• COMBINED NET DESIGN PRESSURE ON PARAPET ($P_p$)

$$P_p = q_p \, GC_{pn}$$

$$P_p = 19.05\,(1.5) = 28.58 \text{ psf} \quad \text{(WINDWARD)}$$
$$P_p = 19.05\,(-1.0) = -19.05 \text{ psf} \quad \text{(LEEWARD)}$$

• FORCES ON PARAPET

HEIGHT OF PARAPET = 4'

$$F = 28.28 \text{ psf } (4') = 113.10 \text{ plf} \quad \text{(WINDWARD)}$$
$$F = 18.85 \text{ psf } (4') = 75.40 \text{ plf} \quad \text{(LEEWARD)}$$

• NATURAL FREQUENCY OF BUILDING ($\eta_1$)

$$\eta_1 = \frac{43.5}{h^{0.9}} = \frac{43.5}{(80.79)^{0.9}} = 0.8242 \text{ Hz}$$

$\eta_1 < 1.0\text{Hz}$ ∴ <u>FLEXIBLE BUILDING</u>

• GUST FACTOR

$$g_Q = g_v = \underline{4.3}$$

$$g_R = \sqrt{2\ln(3600\eta_1)} + \frac{0.577}{\sqrt{2\ln\sqrt{3600\eta_1}}} = \underline{4.146}$$

$$\bar{z} = 0.6h = 49.2'$$

$z_{min} = 30' < 49.2'$ ∴ <u>OK</u>

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = \underline{0.281}$$

$$Q = \sqrt{\frac{1}{\left(1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}\right)}}$$

$$\boxed{\begin{array}{l} Q = 0.791 \text{ FOR N-S DIRECTION} \\ Q = 0.768 \text{ FOR E-W DIRECTION} \end{array}}$$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V \left(\frac{88}{60}\right) = \underline{65.4}$$

$$N_1 = \frac{\eta_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \underline{3.83}$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \underline{0.063}$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) = \underline{0.188}$$

$$\eta_h = \frac{4.6\,\eta_1 h}{\bar{V}_{\bar{z}}} = 4.75$$

$R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta})$

$\eta_B = \frac{4.6 n_1 B}{\bar{V}_z} \Rightarrow$ $\eta_B = 9.58$ [N-S DIRECTION]
$\eta_B = 13.67$ [E-W DIRECTION]

$R_B = 0.099$ [N-S DIRECTION]
$R_B = 0.031$ [E-W DIRECTION]

$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta})$

$\eta_L = \frac{15.4 n_1 L}{\bar{V}_z} \Rightarrow$ $\eta_L = 45.78$ [N-S DIRECTION]
$\eta_L = 32.06$ [E-W DIRECTION]

$R_L = 0.022$ [N-S DIRECTION]
$R_L = 0.031$ [E-W DIRECTION]

$R = \sqrt{\frac{1}{\beta}(R_n R_h R_B)(0.53 + 0.47 R_L)}$ WHERE $\beta = 0.05$

$R = 0.0113$ [N-S DIRECTION]
$R = 0.095$ [E-W DIRECTION]

$G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$

$G_f = 0.87$ [N-S DIRECTION]
$G_f = 0.86$ [E-W DIRECTION]

- PRESSURE COEFFICIENTS FOR WALLS ($C_p$)
  From Figure 6-6

  - WIND IN NORTH-SOUTH DIRECTION
    Windward Wall: $C_p = 0.8$ (USE WITH $q_z$)
    Leeward Wall: ($L/B = 0.7$) $C_p = -0.3$ (USE WITH $q_h$)
    Side Wall: $C_p = -0.7$ (USE WITH $q_h$)

  - WIND IN EAST WEST DIRECTION
    Windward Wall: $C_p = 0.8$ (USE WITH $q_z$)
    Leeward Wall: ($L/B = 1.43$) $C_p = -0.5$ (USE WITH $q_h$)
    Side Wall: $C_p = -0.7$ (USE WITH $q_h$)

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• DESIGN WIND PRESSURE (Equation 6-17)

- WINDWARD WALLS

$P_z = q_z G C_p - q_h (GC_{pi})$

$P_z = (0.87)(0.8)q_z \pm q_h(0.18) = (0.696q_z \pm 3.42)$ psf [N-S]

$P_z = (0.86)(0.8)q_z \pm q_h(0.18) = (0.688q_z \pm 3.42)$ psf [E-W]

- LEEWARD WALLS

$P_z = q_h G C_p - q_h (GC_{pi})$

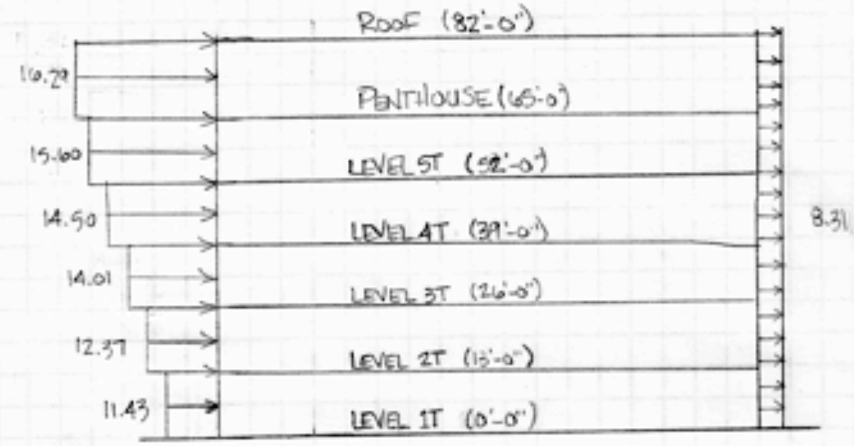
$P_z = 19.03(0.87)C_p \pm 19.03(0.18) = (16.6C_p \pm 3.42)$ psf [N-S]

$P_z = 19.03(0.86)C_p \pm 19.03(0.18) = (16.4C_p \pm 3.42)$ psf [E-W]

*REFERENCE TABLE FOR VALUES

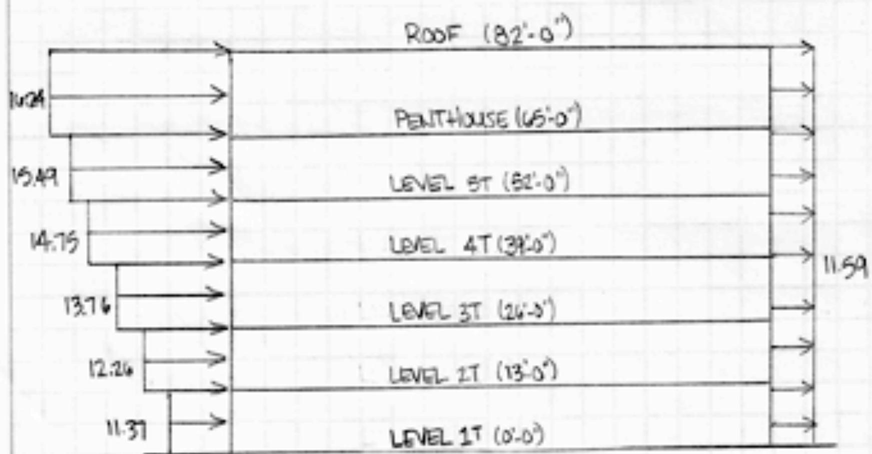
TECH REPORT I | WIND DESIGN | ArNG Readiness Center | 5/7

## WIND PRESSURE DIAGRAMS




* All Values in Pounds Per Sq. Ft. (psf)

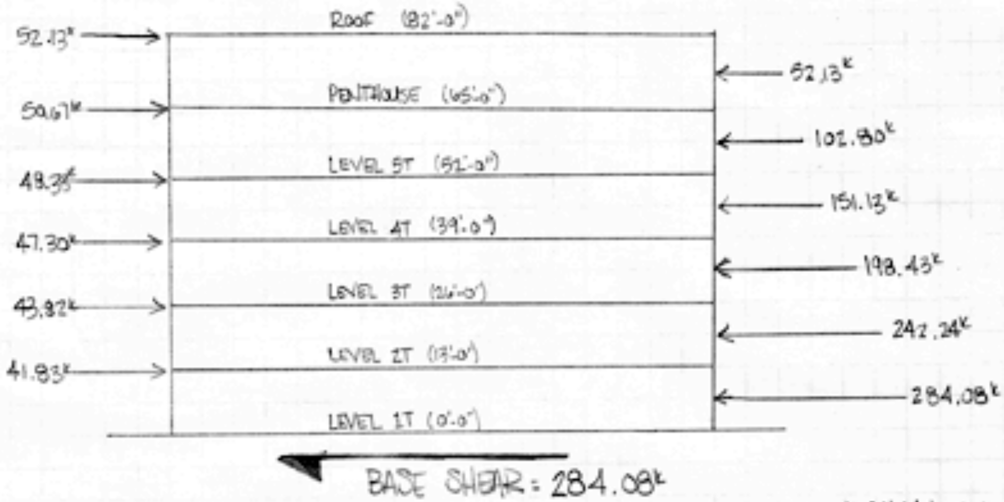
PRESSURE DISTRIBUTION-NS DIRECTION




* All Values in Pounds Per Sq. Ft. (psf)

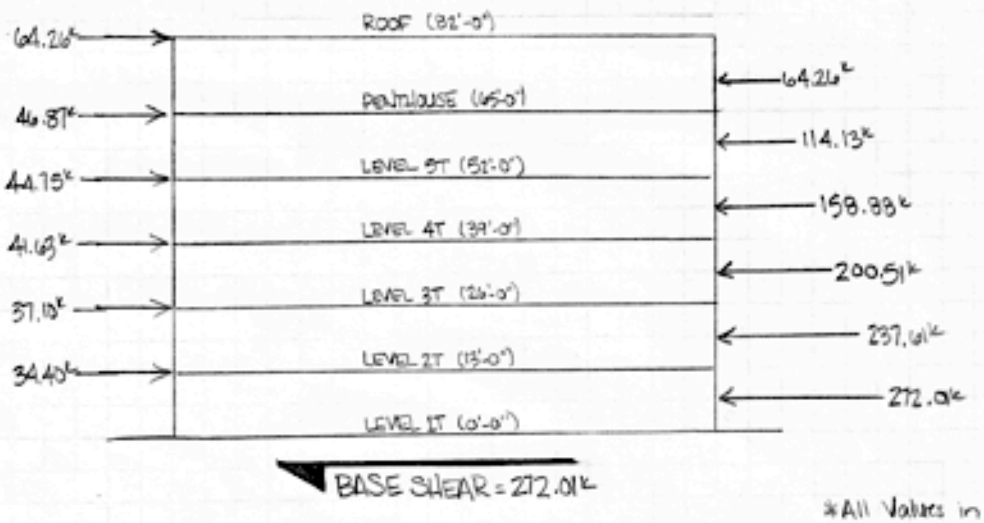
TECH REPORT I | WIND DESIGN | ANG Readiness Center | 6/1

WIND STORY LOAD DIAGRAMS




STORY FORCES & SHEAR → N-S DIRECTION

* All Values in Kilopounds (kips)




STORY FORCES & SHEAR → E-W DIRECTION

* All Values in Kilopounds (kips)

TECH REPORT I | WIND DESIGN | ArNG Readiness Center | 7/7

MOMENT CALCULATIONS

N-S DIRECTION:

ROOF (@ 73.5'): $M = 52.13^k \times 8.5' = 443.11^{k\text{-}ft}$

PH (@ 58.5'): $M = 52.13^k(23.5') + 50.67(6.5') = 1554.41^{k\text{-}ft}$

5T (@ 45.5'): $M = 52.13^k(36.5') + 50.67(19.5') + 48.33^k(6.5') = 3204.96^{k\text{-}ft}$

4T (@ 32.5'): $M = 52.13^k(49.5') + 50.67(32.5') + 48.33(19.5') + 47.30(6.5') = 5477.10^{k\text{-}ft}$

3T (@ 19.5'): $M = 52.13^k(62.5') + 50.67^k(45.5') + 48.33(32.5') + 47.30^k(19.5) + 43.82^k(6.5) = 8241.52^{k\text{-}ft}$

2T (@ 6.5'): $M = 52.13(75.5) + 50.67(58.5) + 48.33(45.5) + 47.30(32.5) + 43.82(19.5) + 41.83(6.5)$

$= \underline{11762.66^{k\text{-}ft}}$

E-W DIRECTION

ROOF (@ 73.5'): $M = 64.26^k \times 8.5' = 546.21^{k\text{-}ft}$

PH (@ 58.5'): $M = 64.26(23.5) + 114.13(6.5) = 2251.96^{k\text{-}ft}$

5T (@ 45.5'): $M = 64.26(36.5') + 114.13(19.5) + 158.88(6.5) = 5603.75^{k\text{-}ft}$

4T (@ 32.5'): $M = 64.26(49.5) + 114.13(32.5) + 158.88(19.5) + 200.51(6.5) = 11291.57^{k\text{-}ft}$

3T (@ 19.5'): $M = 64.26(62.5) + 114.13(45.5) + 158.88(32.5) + 200.5(19.5) + 237.61(6.5) = 19826.98^{k\text{-}ft}$

2T (@ 6.5'): $M = 64.26(75.5) + 114.13(58.5) + 158.88(45.5) + 200.5(32.5) + 237.61(19.5) + 212.0(6.5)$

$= \underline{31675^{k\text{-}ft}}$

| Table 1B: Building Location Parameters | |
|---|---|
| Basic Wind Speed (V) | 90 mph |
| Wind Enclosure Category | C |
| Importance Factor | 1.15 |
| Wind Directionality Factor ($K_d$) | 0.85 |
| Topographic Factor ($K_{zt}$) | 1 |

| Table 2B: Building Information | |
|---|---|
| Number of Stories | 5 |
| Building Height (Feet) | 82 |
| N-S Building Length (Feet) | 163 |
| E-W Building Length (Feet) | 232.75 |
| L/B in N-S Direction | 1.43 |
| L/B in E-W Direction | 0.7 |

**Table 3B: Gust Factor**

| Variable | Wind Direction | |
|---|---|---|
| | N-S | E-W |
| Stiffness | Flexible | |
| B (Feet) | 162 | 232.75 |
| L (Feet) | 232.75 | 162 |
| h (Feet) | 82 | 82 |
| $n_1$ | 0.8242 | 0.8242 |
| c | 0.20 | 0.20 |
| $z_{BAR}$ | 15 | 15 |
| $I_{Z(BAR)}$ | 0.281 | 0.281 |
| $g_Q$ & $g_V$ | 3.4 | 3.4 |
| $g_r$ | 4.146 | 4.146 |
| Q | 0.791 | 0.768 |
| $V_{BAR}$ | 65.4 | 65.4 |
| $N_1$ | 3.83 | 3.83 |
| $R_n$ | 0.063 | 0.063 |
| $R_h$ | 0.188 | 0.188 |
| $N_h$ | 4.75 | 4.75 |
| $R_B$ | 0.099 | 0.031 |
| $N_B$ | 9.58 | 13.67 |
| $R_L$ | 0.022 | 0.031 |
| $N_L$ | 45.78 | 32.06 |
| R | 0.0113 | 0.095 |
| **G** | **0.87** | **0.86** |

| Table 4B: Gust Factors | | | | |
|---|---|---|---|---|
| Wind Direction | $C_p$; Windward | $C_p$; Leeward | Gust Factor | $GC_{pi}$ |
| N-S Direction | 0.8 | -0.3 | 0.87 | 0.18 |
| E-W Direction | 0.8 | -0.5 | 0.86 | 0.18 |

| Table 5B-1: Typical Wind Pressures in North-South Direction | | | | | |
|---|---|---|---|---|---|
| Height (Feet) | KZ | qZ | Wind Pressures (psf) | | |
| | | | N-S Windward | N-S Leeward | N-S Total |
| 82 | 0.94 | 19.05 | 16.65 | -8.31 | 24.96 |
| 80 | 0.93 | 18.85 | 16.51 | -8.31 | 24.82 |
| 70 | 0.89 | 18.04 | 15.95 | -8.31 | 24.26 |
| 60 | 0.85 | 17.23 | 15.38 | -8.31 | 23.69 |
| 50 | 0.81 | 16.42 | 14.82 | -8.31 | 23.13 |
| 40 | 0.76 | 15.40 | 14.11 | -8.31 | 22.42 |
| 30 | 0.7 | 14.19 | 13.77 | -8.31 | 22.08 |
| 25 | 0.66 | 13.38 | 12.70 | -8.31 | 21.01 |
| 20 | 0.62 | 12.57 | 12.14 | -8.31 | 20.45 |
| 0-15 | 0.57 | 11.55 | 11.43 | -8.31 | 19.74 |

| Table 5B-2: Typical Wind Pressures in East-West Direction | | | | | |
|---|---|---|---|---|---|
| Height (Feet) | KZ | qZ | Wind Pressures (psf) | | |
| | | | E-W Windward | E-W Leeward | E-W Total |
| 82 | 0.94 | 19.05 | 16.51 | -11.62 | 28.13 |
| 80 | 0.93 | 18.85 | 16.39 | -11.62 | 28.01 |
| 70 | 0.89 | 18.04 | 15.83 | -11.62 | 27.45 |
| 60 | 0.85 | 17.23 | 15.27 | -11.62 | 26.89 |
| 50 | 0.81 | 16.42 | 14.72 | -11.62 | 26.34 |
| 40 | 0.76 | 15.40 | 14.02 | -11.62 | 25.64 |
| 30 | 0.7 | 14.19 | 13.18 | -11.62 | 24.8 |
| 25 | 0.66 | 13.38 | 12.63 | -11.62 | 24.25 |
| 20 | 0.62 | 12.57 | 12.07 | -11.62 | 23.69 |
| 0-15 | 0.57 | 11.55 | 11.37 | -11.62 | 22.99 |

| Table 6B: Wind Load Distribution in North-South Direction | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Level | Height (Feet) | Tributary Height (Feet) | Windward (psf) | Leeward (psf) | Total (psf) | Story Force (Kips) | Story Shear (Kips) | Overturning Moment (Ft-Kips) |
| Roof | 82 | 13 | 16.29 | -8.31 | 24.60 | 52.13 | 0 | 443.11 |
| Penthouse | 65 | 13 | 15.60 | -8.31 | 23.91 | 50.67 | 52.13 | 1554.41 |
| 5T | 52 | 13 | 14.50 | -8.31 | 22.81 | 48.33 | 102.80 | 3204.96 |
| 4T | 39 | 13 | 14.01 | -8.31 | 22.32 | 47.30 | 151.13 | 5477.1 |
| 3T | 26 | 13 | 12.37 | -8.31 | 20.68 | 43.82 | 198.43 | 8341.52 |
| 2T | 13 | 13 | 11.43 | -8.31 | 19.74 | 41.83 | 242.24 | 11762.66 |
| 1T | 0 | 0 | 0 | 0 | 0 | 0 | **284.08** | **11762.66** |

| Table 7B: Wind Load Distribution in East-West Direction | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Level | Height (Feet) | Tributary Area (Feet) | Windward (psf) | Leeward (psf) | Total (psf) | Story Force (Kips) | Story Shear (Kips) | Overturning Moment (Ft-Kips) |
| Roof | 82 | 13 | 16.24 | -11.62 | 27.86 | 64.26 | 64.26 | 546.21 |
| Penthouse | 65 | 13 | 15.49 | -11.62 | 27.11 | 46.87 | 114.13 | 2251.96 |
| 5T | 52 | 13 | 14.75 | -11.62 | 26.37 | 44.75 | 158.88 | 5603.75 |
| 4T | 39 | 13 | 13.76 | -11.62 | 25.38 | 41.63 | 200.51 | 11291.57 |
| 3T | 26 | 13 | 12.26 | -11.62 | 23.88 | 37.1 | 237.61 | 19826.98 |
| 2T | 13 | 13 | 11.37 | -11.62 | 22.99 | 34.4 | 272.01 | 31675 |
| 1T | 0 | 0 | 0 | 0 | 0 | 0 | **272.01** | **31675** |

# APPENDIX C: SEISMIC LOAD CALCULATIONS

Presented in this appendix are summaries of variables and building parameters required to determine the seismic loads on the Army National Guard Readiness Center. Hand calculations were performed and can be referenced here as well as force distribution tables and diagrams used to determine the base shear and overturning moments caused by seismic forces.

TECH REPORT I | SEISMIC DESIGN | ArNG Readiness Center | 1

- SEISMIC GROUND MOTION VALUES

$S_1 = 0.0639$
$S_s = 0.1799$

SITE CLASS D

- SOIL MODIFIED ACCELERATIONS
  - SITE COEFFICIENTS

FOR SITE CLASS D AND $S_s \le 2.5$

$F_a = 1.6$ (From Table 11.4-1)

FOR SITE CLASS D AND $S_1 \le 0.1$

$F_v = 2.4$ (From Table 11.4-2)

$S_{MS} = F_a S_s = 1.6 \times 0.1799 = 0.288$

$S_{M1} = F_v S_1 = 2.4 \times 0.0639 = \underline{0.1534}$

- DESIGN ACCELERATIONS

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.288) = 0.192$

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.1534) = 0.102$

- SEISMIC DESIGN CATEGORY D WAS USED

- DESIGN COEFFICIENTS FOR SEISMIC FORCE RESISTING SYSTEM
  - Ordinary reinforced Concrete shear walls with steel elements

$R = 5.0$ $\Omega_o = 2.5$ $C_d = 4.5$

$C_t = 0.016$
$x = 0.9$ (From Table 12.8-2)

$T_s = S_{D1}/S_{DS} = \frac{0.102}{0.192} = 0.53$ sec

$T_a = C_t h_n^x = 0.016\,(82')^{0.9} = 0.844$ sec
$> 0.8T_s = 0.8(0.53) = 0.424$ sec $\underline{OK}$

$T_L = 12$ [From Figure 22-15]

TECH REPORT I | SEISMIC DESIGN | ArNG Readiness Center | 2

$$C_s = \min \begin{cases} \dfrac{S_{DS}}{R/I} = \dfrac{0.192}{5.0/1.5} = 0.0256 \leftarrow \text{CONTROLS} \\ \dfrac{S_{D1}}{T(R/I)} = \dfrac{0.192}{0.944(5.0/1.5)} = 0.0363 \\ \dfrac{S_{D1} \cdot T_L}{T^2(R/I)} = \dfrac{(0.192)(0.944)}{(0.944)^2(5.0/1.5)} = 0.515 \end{cases}$$

$C_s = 0.026$

$\rho = 1/T = \dfrac{1}{0.844} = 1.185 > 1.0$ ∴ RIGID DIAPHRAGM

SEE EXCEL SPREAD SHEET FOR FLOOR WEIGHTS

LEVEL 1T : 18970 sq ft ........... 13.95 psf
LEVEL 2T : 18970 sq ft ........... 160.5 psf
LEVEL 3T : 18970 sq ft ........... 160.5 psf
LEVEL 4T : 18970 sq ft ........... 160.5 psf
LEVEL 5T : 18970 sq ft ........... 160.5 psf
PH/ROOF : 18970 sq ft ........... 132.5 psf

TOTAL BUILDING WEIGHT ($w_T$)

$w_T = 18970(13.95) + 18970(160.5) \times 4 \text{ floors} + 18970(132.5)$

$w_T = 14{,}956{,}896.5 \text{ lbs} \rightarrow \underline{14{,}957^k}$

$V = C_s w_T = 0.026 \times 14957 = 389^k$ ← BASE SHEAR

$k = 0.75 + 0.5(0.844) = 1.172$

$w_x h_x^k$ → Varies with height (See Excel Spreadsheet)

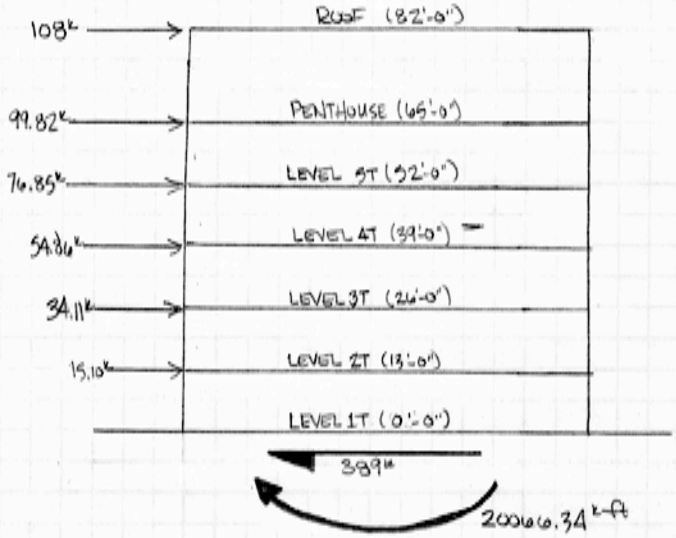
$\sum w_x h_x^k = 1580367.38$

$C_{vx} = \dfrac{w_x h_x^k}{\sum w_i h_i^k}$ → Varies at height (See Excel Spreadsheet)

$F_x = C_{vx} V$ → Varies (See Spreadsheet)

| TECH REPORT I | SEISMIC DESIGN | ArNG Readiness Center | 3 |
|---|---|---|---|

OVERTURNING MOMENT




PH: $OM@\,73.5' = (108)(8.5) = 918.0^{k\text{-}ft}$

5T: $OM@\,58.5' = (108)(23.5)+(99.82)(6.5) = 3186.83^{k\text{-}ft}$

4T: $OM@\,45.5' = (108)(49.5)+(99.82)(19.5)+(76.85)(6.5) = 6388.02^{k\text{-}ft}$

3T: $OM@\,32.5' = (108)(62.5)+(99.82)(32.5)+(76.85)(19.5)+(54.86)(6.5) = 10445.32^{k\text{-}ft}$

2T: $OM@\,19.5' = (108)(62.5)+(99.82)(45.5)+(76.85)(32.5)+(54.86)(19.5)+(34.11)(6.5) = 15080.92^{k\text{-}ft}$

1T: $OM@\,6.5' = (108)(75.5)+(99.82)(58.8)+(76.85)(45.5)+(54.86)(32.5)+(34.11)(19.5)+(15.10)(6.5) = \underline{20066.34^{k\text{-}ft}}$

| Table 1D: General Seismic Information | | |
|---|---|---|
| Occupancy Category | | III |
| Site Class | | D |
| Seismic Design Category | | B |
| Short Period Spectral Response | $S_S$ | 0.1799 |
| Spectral Response (1Sec) | $S_1$ | 0.0639 |
| Maximum Short Period Spectral Response | $S_{MS}$ | 0.288 |
| Maximum Spectral Response (1 Sec) | $S_{M1}$ | 0.1534 |
| Design Short Spectral Response | $S_{DS}$ | 0.192 |
| Design Spectral Response (1 Sec) | $S_{D1}$ | 0.102 |
| Response Modification Coefficient | R | 5 |
| Seismic Response Coefficent | $C_S$ | 0.026 |
| Effective Period | T | 0.844 |
| **Base Shear** | | **389 k** |
| | | |
| **Overturning Moment** | | **20066.34** |

**Table 2C: Seismic Loads**

| Level | Height $h_x$ (ft) | Tributary Height (Ft) | Story Weight $w_x$ (Kips) | $h_x^k$ | $w_x h_x^k$ | $C_{vx}$ | Lateral Force $F_x$ (kips) | Story Shear $V_x$ (kips) | Moments $M_x$ (ft-kips) |
|---|---|---|---|---|---|---|---|---|---|
| Roof | 82 | 8.5 | 2513.5 | 174.98 | 439813.67 | 0.28 | 108.22 | 0.00 | 0.00 |
| Penthouse | 65 | 13 | 3044 | 133.27 | 405675.80 | 0.26 | 99.82 | 108.22 | 918.01 |
| 5T | 52 | 13 | 3044 | 102.60 | 312320.55 | 0.20 | 76.85 | 208.04 | 3186.83 |
| 4T | 39 | 13 | 3044 | 73.24 | 222931.98 | 0.14 | 54.86 | 284.89 | 6388.02 |
| 3T | 26 | 13 | 3044 | 45.54 | 138609.63 | 0.09 | 34.11 | 339.75 | 10445.32 |
| 2T | 13 | 13 | 3044 | 20.21 | 61515.74 | 0.04 | 15.14 | 373.86 | 15080.92 |
| 1T* | 0 | 6.5 | 264.5 | 0.00 | 0.00 | 0.00 | 0.00 | **389** | **20066.34** |
| **$\sum(w_i h_i^k)$ = 1580867.38** | | **$\sum(F_x)$ = V = 389 Kips** | | | **$\sum M$ = 20066.34 K-Ft** | | | | |
| Total Building Weight(Above Grade) = 14,957 kips | | | | | | | | | |
| * The Level 1T story weight is only weight of the columns whose base is at the ground floor. Weights of slabs, beams, and superimposed deads loads are not considered at the ground floor because the base shear is related only to the levels above grade and the components mentioned are at grade level. | | | | | | | | | |

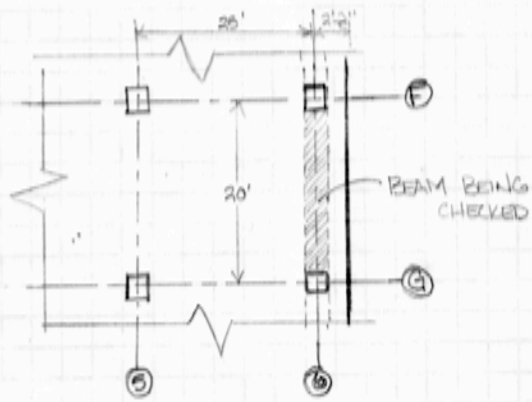
# APPENDIX D: SPOT CHECKS

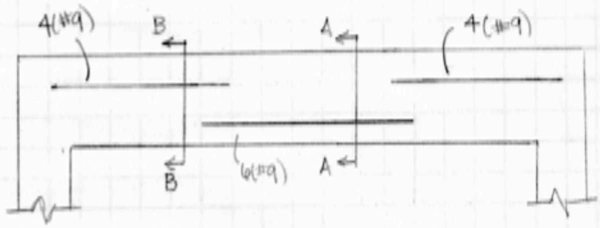
Presented in this appendix are the hand calculations that were performed to analyze individual members of the structural system of the Army National Guard Building to verify the structural engineer's design. The first part consists of the spot check that was completed on a typical edge beam at the 3T level of the building. The second part of this appendix is the spot check for a typical 22" by 22", interior column checked at three different levels.

# *Beam Spot Check*

TECH REPORT I | BEAM SPOT CHECK | ArNG Readiness Center | 1/4

TYPICAL EDGE BEAM AT LEVEL 3T




#3 TIES @ 6" OC

$f'_c = 4000$ psi
$f_y = 60\,000$ psi
$w = 150$ pcf

DEAD LOADS:

Slab → $\frac{9''(150\text{pcf})}{12\text{ in/ft}} = 112.5$ psf

Beam → $\frac{(18''\times15'')(150\text{pcf})}{144\text{ in}^2/\text{ft}^2} = 281.25$ plf

Misc → 35 psf (Raised Floor Area - As noted in Table)

$DL = (112.5+35)(14.667)+281.25 = \underline{2445 \text{ plf}}$

LIVE LOADS:

Office Space (As noted in Table)

$LL = 50 \text{ psf } (14.667') = \underline{733.4 \text{ plf}}$

ASCE 7-05 LOAD COMBINATIONS

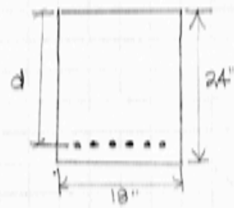
$1.2DL + 1.6LL$ (Worst Case)

$1.2(2445) + 1.6(733.4) = 4107.4 \text{ lb/ft} = \underline{4.11 \text{ k/ft}}$

TECH REPORT I | BEAM SPOT CHECK | ArNG Readiness Center | 2/4

USING ACI MOMENT COEFFICIENTS

- check Section A-A

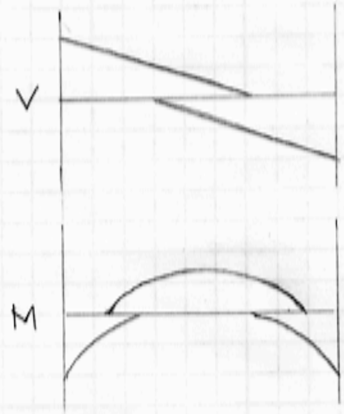



6 (#9) BARS

$d = 24 - 1.5 - \frac{1.128}{2} = 21.936''$

$A_s = 6(1.00) = \underline{6\ in^2}$

$\beta_1 = 0.75$




$M^+ = \frac{w_u l_n^2}{16} = \frac{(4.11)(20)^2}{16} = 102.75^{k\text{-}ft}$

$M^- = \frac{w_u l_n^2}{11} = \frac{(4.11)(20)^2}{11} = 149.5^{k\text{-}ft}$

$V = \frac{w_u l_n}{2} = \frac{(4.11)(20)}{2} = 41.1^k$

$\rho_{max} = 0.85\beta_1 \frac{f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + 0.004}$

$= 0.85(0.75)\left(\frac{4000}{60000}\right)\left(\frac{0.003}{0.007}\right) = 0.0182$

$A_{s_{min}} = \begin{cases} \frac{3\sqrt{f'_c}\,bd}{f_y} = \frac{3\sqrt{4000}(18)(21.936)}{60\,000} = 1.25 in^2 \\ \frac{200bd}{f_y} = \frac{200(18)(21.936)}{60\,000} = \underline{1.32 in^2} \leftarrow \text{CONTROLS} \end{cases}$

$A_{s_{max}} = \rho_{max} bd = 0.0182(18)(21.936) = \underline{7.2\ in^2}$

$A_{s_{min}} < A_s < A_{s_{max}}$ ∴ REINFORCEMENT IS GOOD

$\underline{M_n}$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(6)(60000)}{0.85(4000)(18)} = 5.88''$

$c = a/\beta_1 = \frac{5.88}{0.75} = 7.84''$

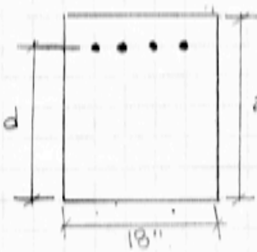
$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{7.84}(21.936 - 7.84) = 0.005 \geq 0.004$ ∴ $\underline{\phi = 0.9}$

$\phi M_n = \phi A_s f_y (d - a/2) = \frac{0.9(6)(60000)(21.936 - \frac{5.88}{2})}{12} = \underline{512.89^{k\text{-}ft}}$

$\boxed{\phi M_n > 102.75 \text{ ∴ DESIGN IS GOOD}}$

TECH REPORT I | BEAM SPOT CHECK | A.NG Readiness Center | 3/4

• Check Section B-B




4(#9) BARS

$A_s = 4(1.00) = 4in^2$
$\beta_1 = 0.75$

$A_{s\,min} = 1.32in^2$ (SAME)
$A_{s\,max} = 7.2in^2$ (SAME)

$A_{s\,min} < A_s < A_{s\,max}$ ∴ GOOD

$M_n$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{4(60000)}{0.85(4000)(18)} = 3.92"$

$c = a/\beta_1 = 3.92/0.75 = 5.23"$

$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{0.003}{5.23}(21.936 - 5.23) = 0.01 > 0.004$ ∴ $\phi = 0.9$

$\phi M_n = \phi A_s f_y (d - \frac{a}{2}) = \frac{0.9(4)(60)(21.936 - \frac{3.92}{2})}{12} = \underline{359.6^{k\text{-}ft}}$

$\boxed{\phi M_n > 149.5^{k\text{-}ft} \therefore \text{GOOD DESIGN}}$

*BEAM PASSES MOMENT CAPACITY FOR BOTH SECTIONS

- Check Shear Strength

$$V_c = 2\sqrt{f'_c}\,bd$$
$$= 2\sqrt{4000}\,(18)(21.936) = \underline{50^k}$$

$$\phi V_n = 0.5\phi V_c$$
$$= 0.5(0.75)(50) = \underline{18.75^k}$$

$$V_s = \frac{V_u}{\phi} - V_c = \frac{41.1}{0.75} - 50 = \underline{4.8^k}$$

$$V_s \le 4\sqrt{f'_c}\,bd = 4\sqrt{4000}\,(18)(21.936) = \underline{100^k}$$

$$s_{max} = \min\begin{cases} d/2 = \frac{21.936''}{2} = 10.968'' \Rightarrow 11'' \leftarrow \text{CONTROLS} \\ 24'' \end{cases}$$

$$11'' > 6'' \quad \therefore \text{OK}$$

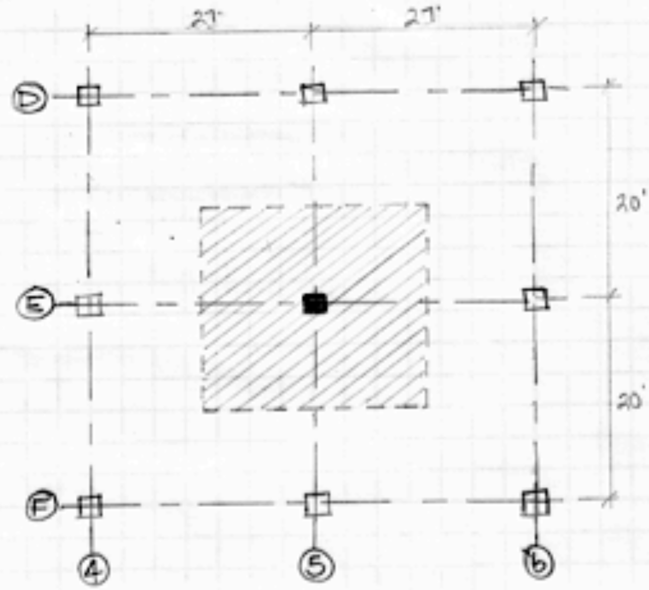
$$A_{v_{min}} = \max\begin{cases} \frac{0.75\sqrt{4000}\,(18)(11)}{60000} = 0.16 \\ \frac{50\,(18)(11)}{60000} = 0.17 \leftarrow \text{CONTROLS} \end{cases}$$

#3 Stirrup $\rightarrow$ $2(0.11) = 0.22 > 0.17 \quad \therefore \underline{\text{GOOD}}$

## *Beam Spot Check*

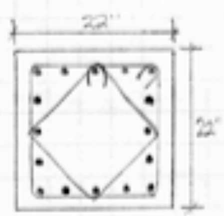
TECH REPORT I | COLUMN SPOT CHECK | ArNG Readiness Center | 1/11

TYPICAL INTERIOR COLUMN - E-5




Clear cover = 1.5"
$f'_c = 4{,}000$ psi
$f_y = 60{,}000$ psi

COLUMN AT LEVEL 3P:




16(#10) BARS
#3 TIES AT 12" OC

$A_s = 16(1.27 in^2) = 20.32 in^2$
$A_g = 22" \times 22" = 484 in^2$

- Pure Axial Strength

$P_o = 0.85 f'_c A_c + A_s f_y$
$= 0.85(4)(484 - 20.32) + (20.32)(60)$

$P_o = 2795.71^k$

TECH REPORT 1 | COLUMN SPOT CHECK | ArNG Readiness Center | 2/3

• Balanced Condition

$\varepsilon_y = f_y/E_s = \frac{60}{29000} = 0.00207$

$c = \frac{\varepsilon_u}{\varepsilon_y+\varepsilon_c}(d_2) = \frac{0.003}{0.003+0.00207}(20.5) = 15.11''$

$a = \beta_1 c = 0.85(15.11) = 12.84''$

$\varepsilon_{s1} = \frac{0.003}{c}(c-d_1) = \frac{0.003}{15.11}(15.11-1.5) = 0.0027$

$\varepsilon_{s1} > \varepsilon_y \therefore f_{s1} = 60^{ksi}$

$\varepsilon_{s2} = \frac{0.003}{c}(c-\frac{h}{2}) = \frac{0.003}{15.11}(15.11-11) = 8.16\times10^{-4}$

$\varepsilon_{s2} < \varepsilon_y \therefore f_{s2} = 23.66^{ksi}$

$\varepsilon_{s3} = \frac{0.003}{c}(c-d_2) = \frac{0.003}{15.11}(15.11-20.5) = -0.001$

$\varepsilon_{s3} < \varepsilon_y \therefore f_{s3} = -31.03^{ksi}$

$M_b = 0.85f'_c b\beta_1 c(\frac{w}{2}-\frac{\beta_1 c}{2}) = 0.85(4)(22)(0.75)(15.11)(\frac{22}{2}-\frac{0.75(15.11)}{2}) = 4521.27^{k\text{-}in}$

$\underline{M_b = 376.77^{k\text{-}ft}}$

$P_b = 0.85f'_c b\beta_1 c + 2f_1 + 2f_2 + 3f_3 = 0.85(4)(22)(0.85)(15.11)+2(60)+2(23.66)+2(-31.03)$

$\underline{P_b = 1065.95^k}$

• Pure Bending

$f_{s1} = \frac{0.003}{c}(c-2.75)(29000)$

$\left.\begin{array}{l} f_{s2} = -60^{ksi} \\ f_{s3} = -60^{ksi} \end{array}\right\}$ ASSUMPTION

$0.85f'_c b\beta_1 c + 2(\frac{\varepsilon_u}{c}(c-2.75)(29000)) - 2(60) - 2(60) \Rightarrow \underline{c = 3.57''}$

$\varepsilon_{s1} = \frac{0.003}{3.57}(3.57-2.75)(29000) = 20^{ksi}$

$\varepsilon_{s2} = \frac{0.003}{3.57}(3.57-11)(29000) = -0.006 > \varepsilon_y \therefore \underline{f_{s2} = 60^{ksi}}$ (GOOD ASSUMPTION)

$\varepsilon_{s3} = \frac{0.003}{3.57}(3.57-20.5)(29000) = -0.013 > \varepsilon_y \therefore \underline{f_{s3} = -60^{ksi}}$ (GOOD ASSUMPTION)

$M_n = 0.85(4)(22)(0.75)(3.57)(\frac{22}{2}-\frac{0.75(3.57)}{2}) + 2(20)(\frac{22}{2}-2.75) + 2(-60)(11-20.5)$

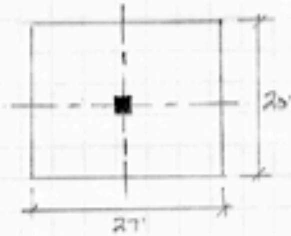
$\underline{M_n = 290^{k\text{-}ft}}$

TECH REPORT 1 | COLUMN SPOT CHECK | ArNG Readiness Center | 3/5

$\phi M_n = 0.9(290) = 261^{k\text{-}ft}$
$\phi P_n = 0.65(2795.71) = 1817.2^{k}$
$\phi M_b = 0.9(376.77) = 339^{k\text{-}ft}$
$\phi P_b = 0.65(1065.95) = 692.87^{k}$

ASSUMPTION → Due to the columns being evenly spaced, any moments that would be transfered into the column could be assumed to be in both direction and would therefore cancel each other out.

TRIBUTARY AREA:




Trib Area = 20'×27' = 540 ft²

WEIGHT OF UPPER FLOORS DUE TO BEARING LOADS:

Slab: $\frac{9}{12}(150) = 112.5$ psf × 7 floors
Roof: $\frac{10}{12}(150) = 125$ psf
Misc: 15 psf

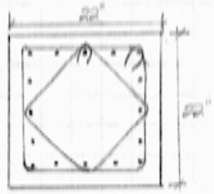
Office Space LL = 50 psf + 15 psf [Partitions]

DL = 7(112.5) + 125 + 15 = 927.5 psf
LL = 65 psf

$P_u = 1.2(927.5) + 1.6(65) = \frac{1217\,(20\times 27)}{1000} = \underline{657.18^{k}}$

TECH REPORT I | COLUMN SPOT CHECKS | A.N.G Readiness Center | 4/5

COLUMN AT 1T LEVEL




16(#8) BARS
#3 TIES @ 12" OC

$A_s = 16(0.79in^2) = 12.64in^2$
$A_g = 22" \times 22" = 484in^2$

* NOTE: Only the pure axial strength will change at this level because the shape of the column is constant, only the reinforcing changes. (See Level 3P Column Calcs)

- Pure Axial Strength

$P_o = 0.85f'_c A_c + A_s f_y$
$= 0.85(4)(484 - 12.64) + 60(12.64)$

$\underline{P_o = 2361.02^k}$

$\phi M_n = 0.9(290) = 261^{k\text{-}ft}$
$\phi P_n = 0.65(2361.02) = 1534.7^k$
$\phi M_b = 0.9(376.77) = 339^{k\text{-}ft}$
$\phi P_b = 0.65(1065.95) = 692.87^k$

TRIBUTARY AREA:

Trib Area = $20' \times 27' = 484 ft^2$

WEIGHT OF UPPER FLOORS DUE TO BEARING LOADS:

* Same Weights as calculated for 3P floor

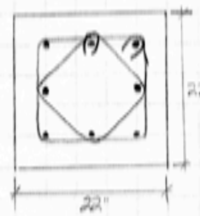
$DL = 4(112.5) + 125 + 15 = 590^{psf}$
$LL = 65^{psf}$

$P_u = 1.2(590^{psf}) + 1.6(65^{psf}) = 812^{psf} \Rightarrow \frac{812(20 \times 27)}{1000}$

$\underline{P_u = 438.5^k}$

TECH REPORT I | COLUMN SPOT CHECKS | ArNG Readiness Center | 5/5

COLUMN AT LEVEL 5T




8(#8) BARS
#3 TIES @ 10" OC

$A_s = 8(0.79) = 6.32 in^2$
$A_g = 22" \times 22" = 484 in^2$

*NOTE: Only the pure axial strength will change at this level because the shape of the column is constant, only the reinforcing changes. (See Level 3P Column calculations)

• Pure Axial Strength

$P_o = 0.85 f'_c A_c + f_y A_s$
$= 0.85(4)(484 - 6.32) + 60(6.32)$

$\underline{P_o = 2003.3^k}$

$\phi M_n = 0.9(290) = 261^{k\text{-}ft}$
$\phi P_n = 0.65(2003.3) = 1302.15^k$
$\phi M_b = 0.9(376.77) = 339^{k\text{-}ft}$
$\phi P_b = 0.65(1065.95) = 692.87^k$

TRIBUTARY AREA

Trib. Area = $20' \times 27' = 540 ft^2$

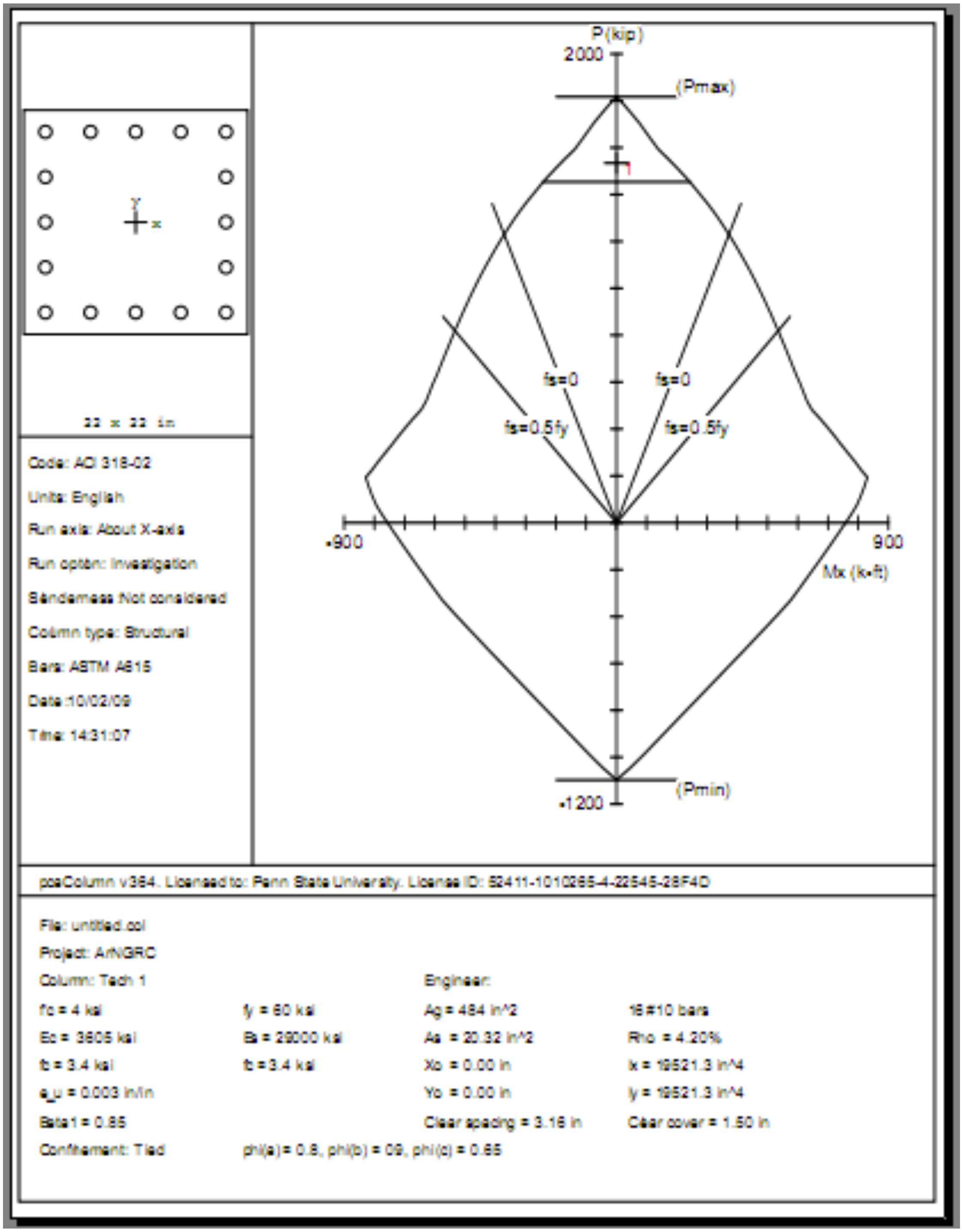
WEIGHT OF UPPER FLOORS DUE TO BEARING LOADS:

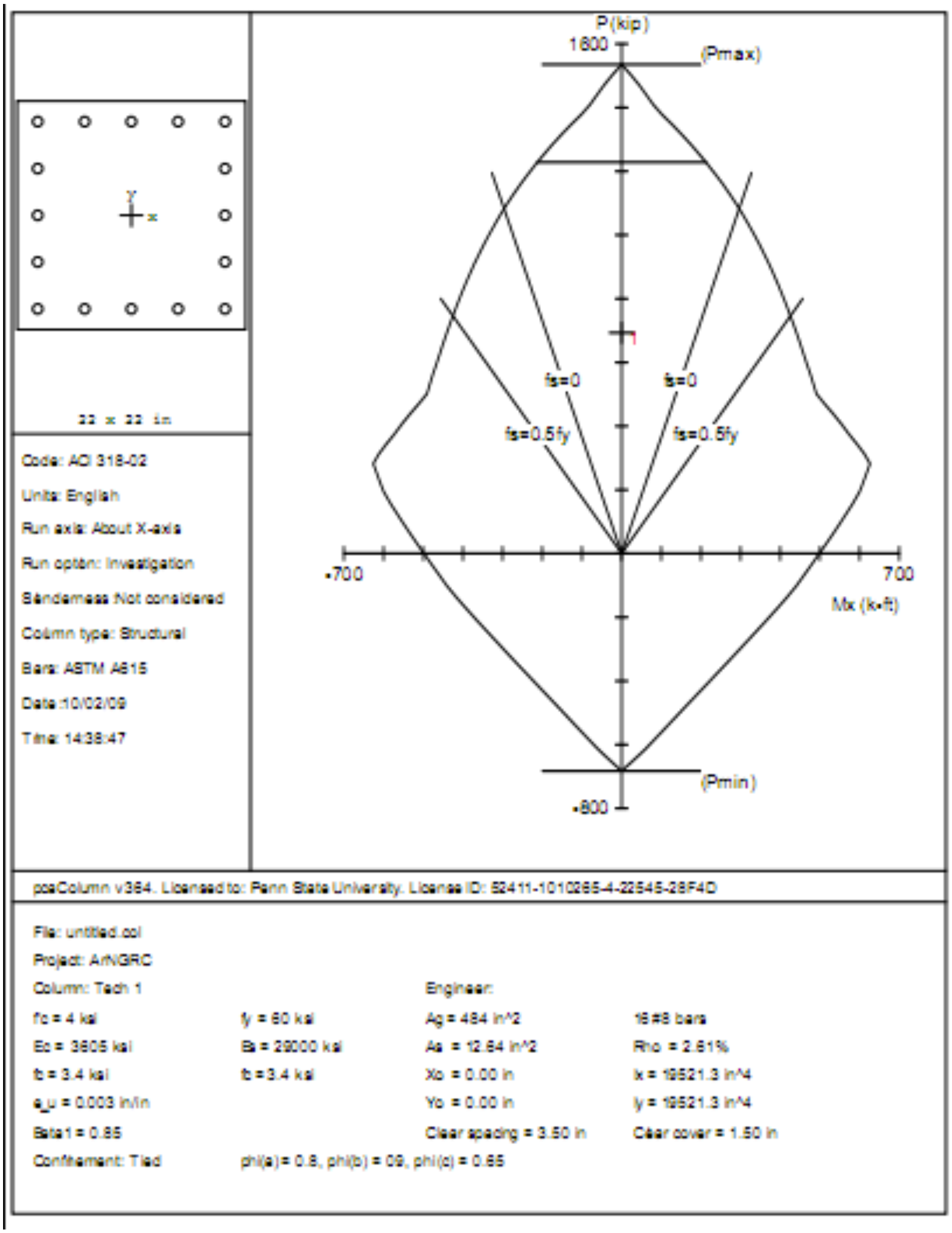
*Same weights as calculated for 3P floor

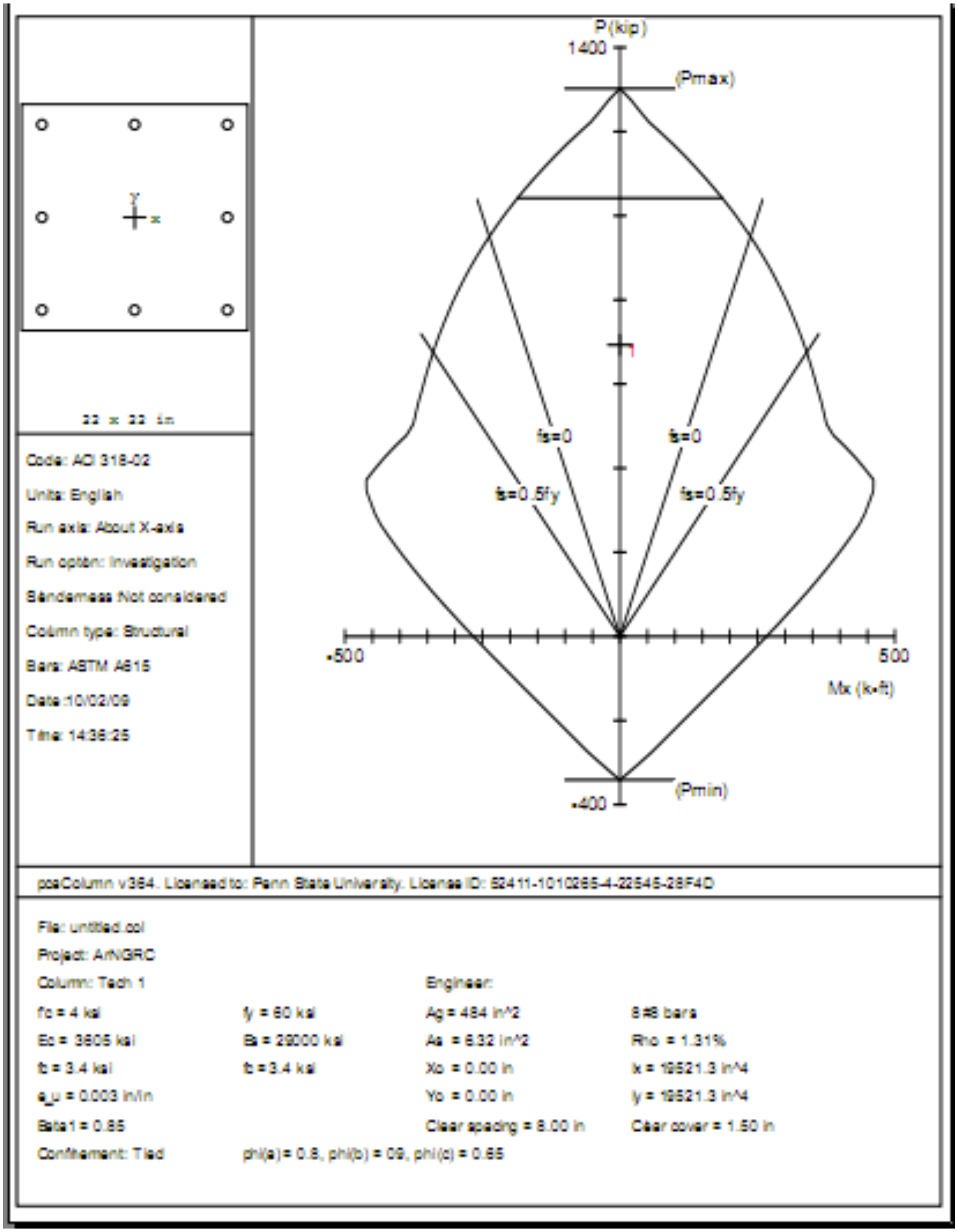
DL = 125psf + 15psf = 140psf
LL = 50psf

$1.2(140) + 1.6(50) = 248psf \Rightarrow \frac{248(540)}{1000}$

$\underline{P_u = 134^k}$

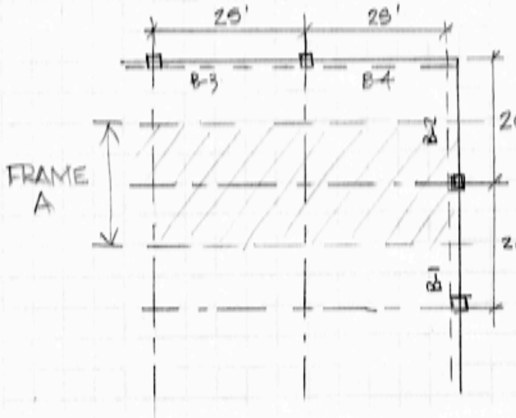

P (kip)
2000
(Pmax)
fs=0
fs=0
fs=0.5fy
fs=0.5fy
-900
900
Mx (k-ft)
(Pmin)
-1200
22 x 22 in
Code: ACI 318-02
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 10/02/09
Time: 14:31:07
pcaColumn v364. Licensed to: Penn State University. License ID: 52411-1010265-4-22545-26F4D
File: untitled.col
Project: ArNGRC
Column: Tech 1
Engineer:
f'c = 4 ksi
fy = 60 ksi
Ag = 484 in^2
16 #10 bars
Ec = 3605 ksi
Es = 29000 ksi
As = 20.32 in^2
Rho = 4.20%
fc = 3.4 ksi
fc = 3.4 ksi
Xo = 0.00 in
Ix = 19521.3 in^4
e_u = 0.003 in/in
Yo = 0.00 in
Iy = 19521.3 in^4
Beta1 = 0.85
Clear spacing = 3.16 in
Clear cover = 1.50 in
Confinement: Tied
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65


22 x 22 in
Code: ACI 318-02
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 10/02/09
Time: 14:38:47
P (kip)
1800
(Pmax)
fs=0
fs=0
fs=0.5fy
fs=0.5fy
-700
700
Mx (k-ft)
(Pmin)
-800
pcaColumn v364. Licensed to: Penn State University. License ID: 52411-1010265-4-22545-28F4D
File: untitled.col
Project: ArNGRC
Column: Tech 1
Engineer:
f'c = 4 ksi
fy = 60 ksi
Ag = 484 in^2
16 #8 bars
Ec = 3605 ksi
Es = 29000 ksi
As = 12.64 in^2
Rho = 2.61%
fc = 3.4 ksi
fc = 3.4 ksi
Xo = 0.00 in
Ix = 19521.3 in^4
e_u = 0.003 in/in
Yo = 0.00 in
Iy = 19521.3 in^4
Beta1 = 0.85
Clear spacing = 3.50 in
Clear cover = 1.50 in
Confinement: Tied
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65


P (kip)
1400
(Pmax)
fs=0
fs=0
fs=0.5fy
fs=0.5fy
-500
500
Mx (k-ft)
(Pmin)
-400
22 x 22 in
Code: ACI 318-02
Units: English
Run axis: About X-axis
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 10/02/09
Time: 14:36:25
spColumn v364. Licensed to: Penn State University. License ID: 52411-1010265-4-22545-28F4D
File: untitled.col
Project: ArNGRC
Column: Tech 1
Engineer:
f'c = 4 ksi
fy = 60 ksi
Ag = 484 in^2
8 #8 bars
Ec = 3605 ksi
Es = 29000 ksi
As = 6.32 in^2
Rho = 1.31%
fc = 3.4 ksi
fc = 3.4 ksi
Xo = 0.00 in
Ix = 19521.3 in^4
e_u = 0.003 in/in
Yo = 0.00 in
Iy = 19521.3 in^4
Beta1 = 0.85
Clear spacing = 8.00 in
Clear cover = 1.50 in
Confinement: Tied
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

## *Slab Spot Check*

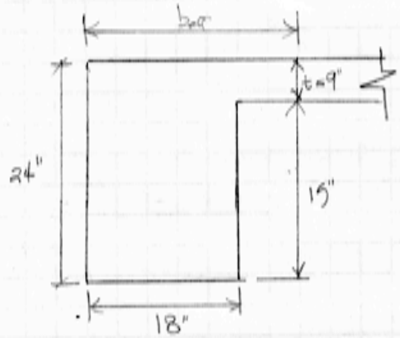
TECH REPORT I | SLAB CHECK | ArNG Readiness Center | 1




TWO-WAY FLAT SLAB

TYPICAL 18"x24" EDGE BEAMS → NO INTERIOR BEAMS

BEAM SECTION
- All 4 Beams have same dimensions




$b_{eff} = b_w + h_w \le b_w + 4t$

$= 18" + 15" = \underline{33"}$

$\le 18" + 4(9") = 54"$ ∴ OK

<u>BEAMS 1 & 2:</u>

$I_b = k \dfrac{bh^3}{12}$

$b_e/b_w = 1.83$

$t/h = 0.375$

USING EQ. 16.4.2b (ACI)

$$k = \frac{1 + (b_e/b_w - 1)(t/h)\left[4 - 6(t/h) + 4(t/h)^2 + (b_{ew}/b_w - 1)(t^3/h^3)\right]}{1 + (b_e/b_w - 1)(t/h)}$$

$k = 1.03$

$I_b = 1.03 \dfrac{(18)(24)^3}{12} = 21358.08 \text{ in}^4$

TECH REPORT I | SLAB CHECK | ArNG Readiness Center | 2

SLAB

$$I_s = \frac{(10' \times 12)(9)^3}{12} = 7290 \text{ in}^4$$

FOR BEAM 1 & 2:

$$\alpha = \frac{EI_s}{EI_b} = \frac{21358.08}{7290} = \underline{2.93}$$

BEAMS 3 & 4

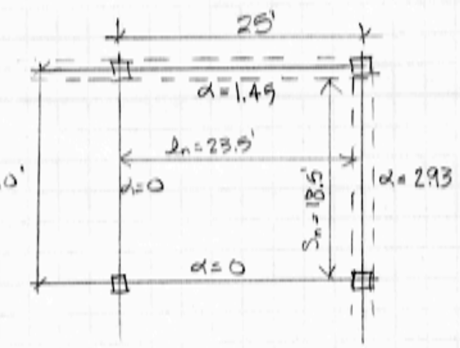
$$I_b = 21358.08 \text{ in}^4 \quad \text{(SAME AS TRANSVERSE)}$$

SLAB

$$I_s = \frac{(25 \times 12)(9)^3}{12} = 9112.5 \text{ in}^4$$

FOR BEAMS 3 & 4

$$\alpha = \frac{EI_s}{EI_b} = \frac{21358.08}{9112.5} = \underline{1.45}$$




$$\beta = \ell_n / S_n = \frac{23.5}{18.5} = 1.27$$

$$\alpha_m = \frac{2.93 + 1.45}{4} = 1.09 \implies 0.2 < \alpha_m < 2.0 \therefore \text{MEDIUM STIFF BEAMS}$$

For medium stiff beams: $t_{min} = \dfrac{\ell_n \left(0.8 + \frac{f_y}{200\,000}\right)}{36 + 5\beta(\alpha_m - 0.2)}$

$$t_{min} = \frac{23.5\left(0.8 + \frac{60000}{200000}\right)}{36 + 5(1.27)(1.09 - 0.2)} = 7.45''$$

$t_{min} < t_{used} \therefore \underline{OK}$

★ Service loads do not need to be calculated