POWER TRANSMISSION FOUNDATION DESIGN

Project ID: CEEn_2016CPST_005

by

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A Capstone Project Final Report

Submitted to

Matt Hawley
Kiewit Infrastructure Co.

Department of Civil and Environmental Engineering
Brigham Young University

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Executive Summary

PROJECT TITLE: Power Transmission Foundation Design for Kiewit Engineers
PROJECT ID: CEEn-2016CPST-005
PROJECT SPONSOR: Kiewit Infrastructure Engineers Co.
TEAM NAME: MDT Engineering

Two foundations were designed, one for a lattice tower in the New Jersey meadowlands, and one for a monopole tower in the Kearney CSX South Railyard. Each site presented engineering challenges with soil stratification, load management, and site access. Each foundation is required to achieve an 80-year design life by resisting corrosion and maintaining the minimum settlement or lateral displacement of one inch. The lattice tower foundation will consist of 16 floating steel pipe piles, and the monopole foundation will consist of 8 pre-stressed concrete end-bearing piles. Reinforced concrete design was conducted and each cap was checked against various failure modes. Constructability plans were drawn up to outline the construction plan and access limitations to each site. Each foundation has been designed with factors of safety to ensure that they will meet the project requirements.

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Introduction

In response to the RFP submitted by Kiewit Infrastructure Co., MDT Engineering has designed two foundations for power transmission towers located in East Hanover, NJ. Foundations for these towers have been previously designed and have already been installed. MDT Engineering has completed their own design of one lattice structure foundation in the meadowlands, and one monopole foundation in the rail yard.

The scope of this project consists of a lattice tower located in the meadowlands, and a monopole tower located in an existing railyard. Each site presents challenges in regard to the soil stratification, soil bearing capacity, and site access. The lattice tower is designed to be placed in the meadowlands of the Richard W. De Korte Park. This location for the lattice tower is surrounded by water which presents construction access limitations and additional design considerations. The monopole is designed to be placed at the CSX South Kearney Railyard. Access to this site is limited by traffic in the area, established businesses, and private property.

Cone penetration test results for each site describe challenging soil profiles. The soil at the meadowlands location was found to be mostly normally consolidated to under consolidated silts and clays with the water table located four feet below the surface. The bearing capacity of this type of saturated soil can be difficult to ascertain, which makes pile foundation design challenging. The soil at the rail yard location was found to be mostly sand layers with traces of clays and a bedrock layer at about 110 feet deep. The water table at this location is 8 feet below the surface. Challenges were found in regards to the large tower diameter and the large moment load applied to the foundation.

Project requirements call for an 80-year design life. To achieve this, the concrete needs to be sulfate resistant, vertical settlement must be one inch or less for each foundation, differential settlement must be one inch or less for the lattice tower foundation, and lateral displacement must be less than one inch under the working load conditions. Design codes according to the ACI and the AISC Steel Construction Manual were followed, where possible, to ensure proper design.

Schedule

Throughout the execution of this project, changes were made to the original schedule to accommodate unforeseen challenges such as the steep learning curve that required more research time and left less time for design. Table *1* shows the time line that was followed throughout the completion of the project.

Table 1, Project Schedule

	January					February				March				April		
	2	9	16	23	30	6	13	20	27	6	13	20	27	3	10	13
Preparation and Analysis																
Organize Data	■	■														
Analyze Data		■	■	■												
Perform Calculations			■	■	■	■	■	■	■	■	■	■	■	■	■	
Design																
Lattice Tower Foundation						■	■	■	■	■	■	■				
Monopole Foundation										■	■	■	■	■	■	
Construction Schemes													■	■	■	
Report																
Monthly Progress Reports					■				■				■			
50% Complete Status Report								■	■							
Report Complete													■	■	■	■
Poster Complete														■	■	
Presentation															■	

Assumptions & Limitations

Assumptions were made during the completion of this project to account for unknown variables and to make up for the students' lack of knowledge. Per the project parameters, pile drivability issues were not considered for this project. This assumption limits the design as pile drivability and the effects from hard driving of steel or concrete piles can alter the soil bearing capacity.

In analyzing the soil profile, the soil was divided into four-foot sections where each section was assumed to have consistent properties. The density of each soil layer was estimated based on the CPT results and on reference texts. The soil types for each layer at each site were matched as closely as possible to the available soil types that are preloaded into the GROUP program. These assumptions could affect the values for lateral loading and displacement that were calculated with the GROUP program.

Methods for calculating the bearing capacity for each project site were chosen based on the soil profile and the expected pile size for each foundation. For the lattice tower foundation site, the alpha method was chosen due to the presence of cohesive soils. For the monopole foundation site, the Nordlund method was chosen for the presence of non-cohesive soils. It was assumed that bearing capacity of each pile would not be effected by the corrosive properties of the soil. Corrosion of the steel pipe piles was found to be 0.136 inches over the 80-year design life which would reduce an HSS 20 X 0.5 steel pipe approximately to an HSS 20 X 0.375 steel pipe. Failure calculations were made using an HSS 20 X 0.375 to account for corrosion.

Pile caps were modeled as a network of simple beams and trusses to calculate the internal forces needed for reinforced concrete design. For the lattice tower, a truss configuration for the pile cap was analyzed with a structural engineering software named Visual Analysis. The monopole was designed using two simple beams. These two simple beams were laid across each other, and it was assumed that each beam carried 60 percent of the load.

It was also assumed that the given anchor bolt design for the monopole tower is sufficient to transfer the load from the tower into the foundation. For each foundation, the effective length of each pile under compression was assumed to be zero. This is because each pile will be horizontally supported by the soil around it. Values used in the calculations for any equation were assumed according the best knowledge of the engineering team.

Design, Analysis & Results

Work began by analyzing CPT data for each site. The soil stratification for the lattice tower site was divided into four-foot layers and the soil stratification for the monopole site was divided into two-foot layers. These layers were individually analyzed to calculate the bearing capacity of the soil at each site.

Lattice Tower Foundation Design

Based on pile design recommendations found in the National Highway Administration (FHWA) manual, Design and Construction of Driven Pile Foundations – Volume I (2016), floating steel pipe piles have been chosen for the foundation of the lattice tower. Calculations for bearing capacity and sleeve resistance were conducted using the alpha method, found on page 245 of the FHWA manual. A step by step description of how the alpha method was followed is located in Appendix B.

The lattice tower foundation consists of four individual foundations to support each foot of the tower. The total axial load applied to each foundation is 450 kips, which is a combination of the demand of 415.8 kips for the tower and 34.2 kips for a rough estimation of the weight of the pile cap. Using the calculated undrained shear strength of the soil and charts found in the FHWA manual, shaft resistance and nominal shaft resistance were calculated. Figure *1* shows a plot of shaft resistance (Rs) versus depth. A pile depth of 100 feet using a steel pile diameter of 20 inches provides a nominal shaft resistance of 449.7 kips per pile. This depth was chosen because it allows the design to operate at a factor of safety of four, which was considered necessary due to the degrading effect piles have when in proximity of each other, and the natural ambiguity of soil. This nominal shaft resistance allows for the highest magnitude of uplift, which is 338.8 kips. The current design places each pile 10 feet apart, center to center, to ensure minimum interaction between piles.

Piles for the lattice tower foundation will be steel pipe piles, 20-inch in diameter and 1/2-inch-thick (HSS 20 X 0.5). Since adequate nominal shaft resistance was obtained well before bedrock depth, floating piles were chosen for this design. Piles will be 97.5 feet in length and extend to a depth of 100 feet below the surface. Each foot of the lattice tower will be supported by four piles for a total of 16 piles. Steel pipe piles will maintain structural integrity under the given load conditions with this thickness decrease.

Ensoft programs, LPILE and GROUP, were used to analyze the expected lateral loads. Individual pile reactions were analyzed using LPILE and the reactions for the pile groups were analyzed using the GROUP program. Figure *2* is a plot of lateral pile deflection for one pile at the lattice tower site. Figure *3* is a plot of resultant lateral deflection for one pile group of the lattice tower foundation. These values are less than the maximum settlement for the project. Resultant lateral deflection is less than one inch as required by the project specifications.

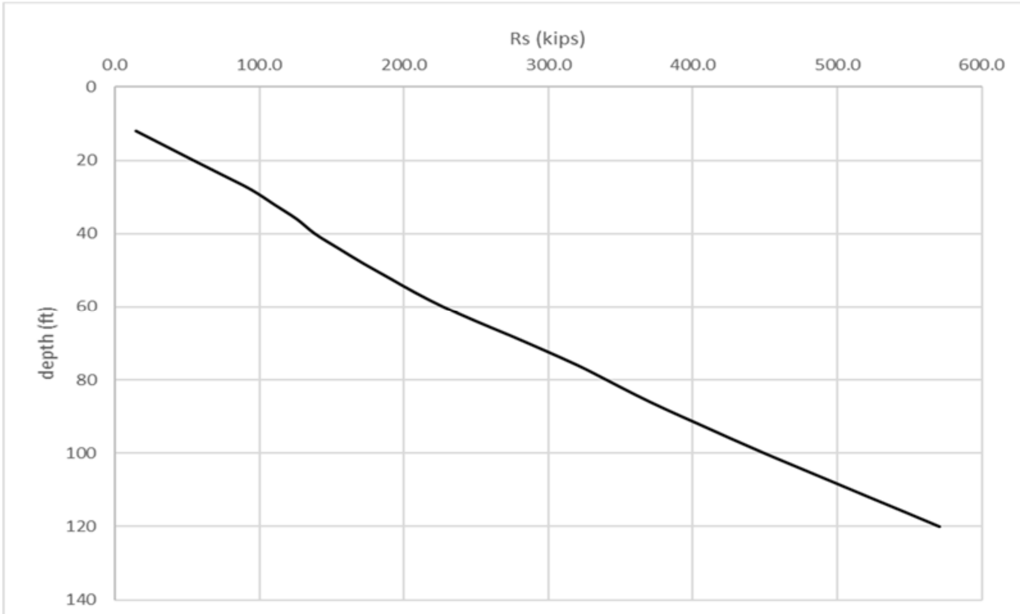

Figure 1, Rs versus depth for the lattice tower foundation site

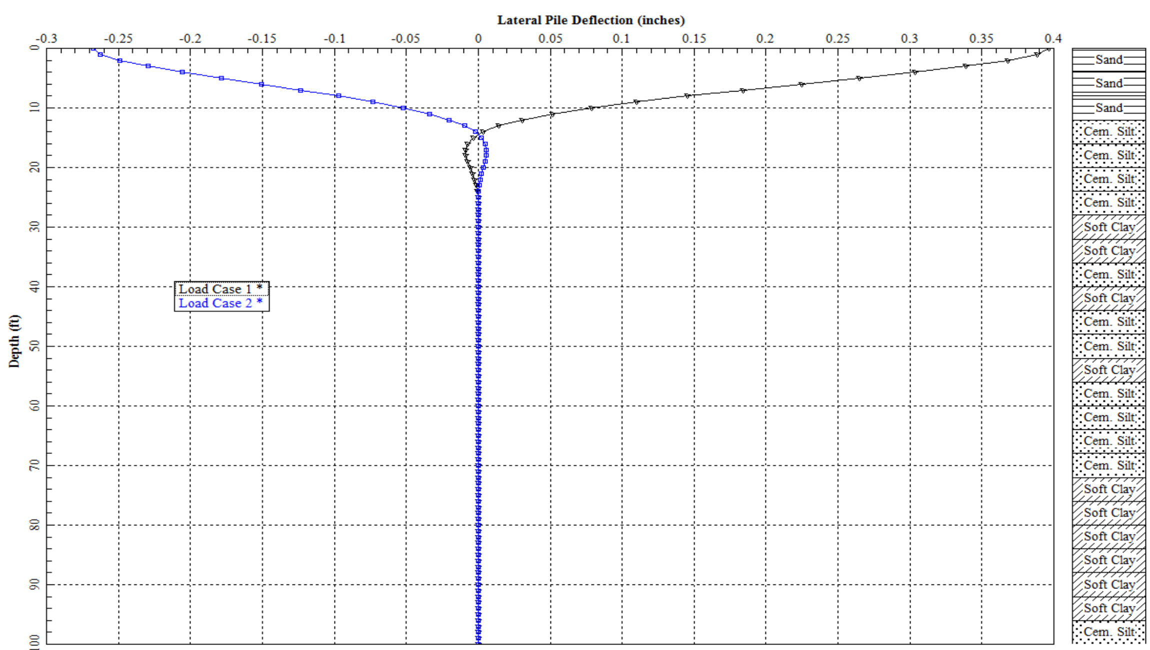

Figure 2, Lateral pile deflection at the lattice tower site

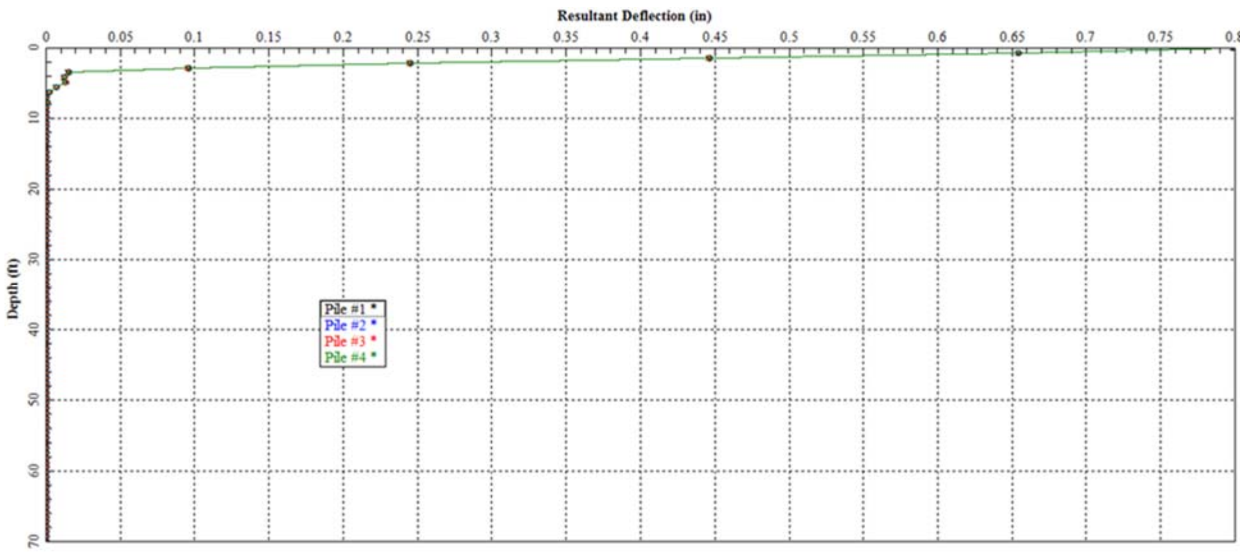

Figure 3, resultant deflection for one pile group at the lattice tower site

Equations for calculating the settlement of each pile group were found in the U.S. Army Corps of Engineers Design of Pile Foundations manual on page 4-21. Overall settlement for the designed pile group was found to be 0.57 inches, which is within the given maximum settlement of one inch. Table *3* shows the values used to calculate settlement for both project sites.

Table 2, Settlement calculations for the lattice tower foundation

Meadowland	w_s	0.000781	ft	w_{pp}	0.009126	ft	w_{ps}	0.006429	w	0.20	ζ_g	2.449245	S	0.480135	(in.)
	Q_p	5.10	kips	C_p	0.02						Bbar	10			
	α_s	0.67		B	1.667	ft									
	Q_s	98.6	kips	q	6.705599	kips	S:	group settlement							
	L	100	ft	C_s	0.04326		ζg:	group settlement factor							
	A	2.182534	ft^2	D	99	ft	Bbar:	width of pile group							
	E	4176000	ksf												

Foundation Analysis and Design (Bowles, 1996). The calculations determined that three feet of concrete would be sufficient for the pile cap to contain all the shear, punching shear, and moment loads. This size was altered to four feet in order to make sure there is enough room for development lengths in the rebar. This design was checked using GROUP, which showed no effect on the pile group settlement. The design of the simple beams required at least four bars of #9 rebar on the bottom side of the beam. Because the loading within the cap could be reversed as uplift is created, the minimum four #9 bars will be placed along the top as well. The exact rebar layout can be seen in Figure *4*. The plate that the tower will be welded on is anchored into the slab with a 1.25-inch anchor bolts with a normal nut at the end. The cap design provides overhang to prevent punching shear, and to account for the inaccuracy of pile placement. The concrete is specified as 4000 psi and Type 5.

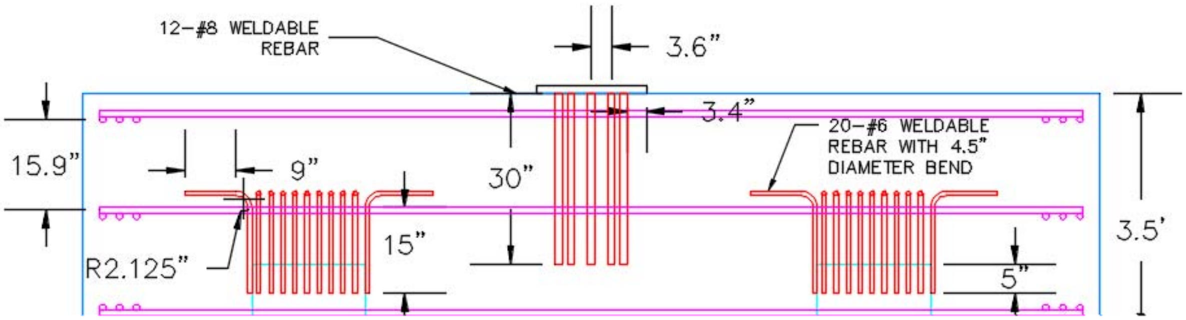

Figure 4, Lattice tower pile cap dimensions and reinforcement layout

Monopole Foundation Design

Based on pile design recommendations found in the FHWA manual, Design and Construction of Driven Pile Foundations – Volume I (2016), pre-stressed concrete piles of one-foot width have been chosen for the monopole foundation. These piles will be sulfate resistant to reduce corrosion. Calculations for bearing capacity of the soil and for the nominal resistance of a pile were conducted per the Nordlund method found on page 242 of the FHWA manual. A step by step description of how the Nordlund method was implemented can be found in appendix C of this report.

After examining the CPT results for the railyard site, the decision was made to design the piles to rest on the bedrock layer at a depth of 110 feet. Placing the piles into the bedrock layer provides the maximum nominal pile end-baring capacity for the pre-cast concrete piles. This design provides a significant factor of safety for axial and moment loads. The nominal sleeve resistance for these piles was found to be about 34 000 kips per pile. This provides a suitable factor of safety against uplift load cases. Figure 5 shows the nominal sleeve resistance plotted against depth for the monopole site.

The monopole tower will be supported by eight square pre-cast concrete piles, each one-foot in width. Piles will be 107 feet in length and will extend to a depth of 110 feet and will be end-bearing piles. Each pile cap will be cast with Type V concrete cement to achieve sulfate resistance. The corrosive properties found of the soil at the project site require pile to be corrosion resistant to achieve the required 80-year design life.

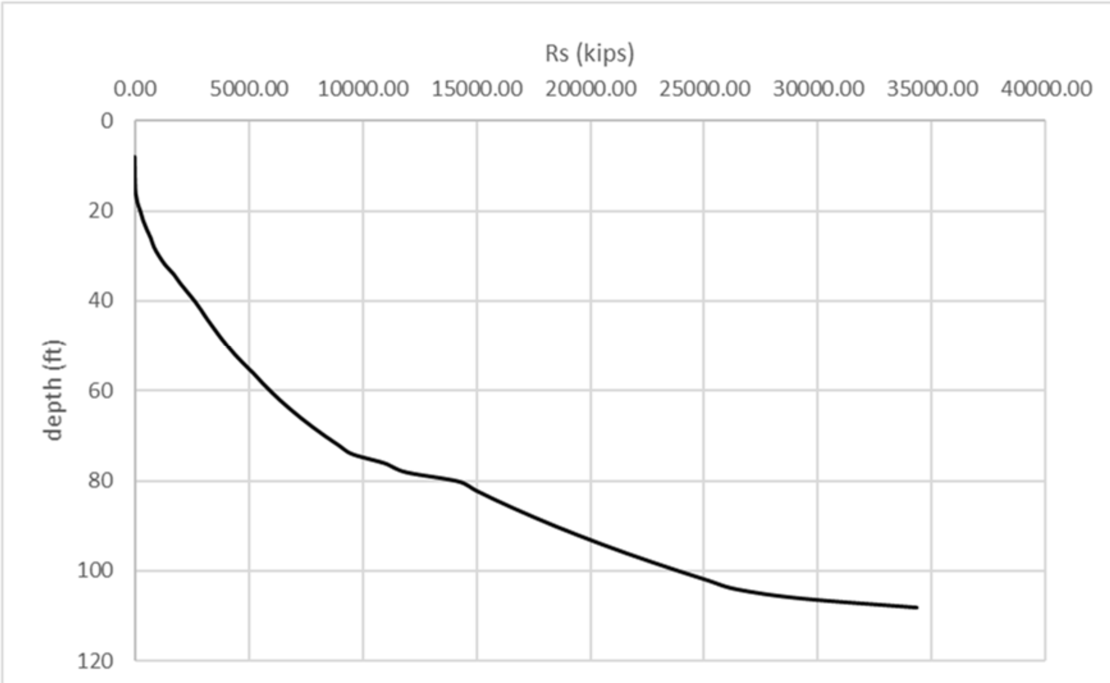

Figure 5, Rs versus depth for the monopole foundation

Lateral deflection analysis was performed with the same program used for the lattice tower site. A plot from GROUP showing the resultant lateral deflection of the monopole pile foundation group is seen in Figure *6*.

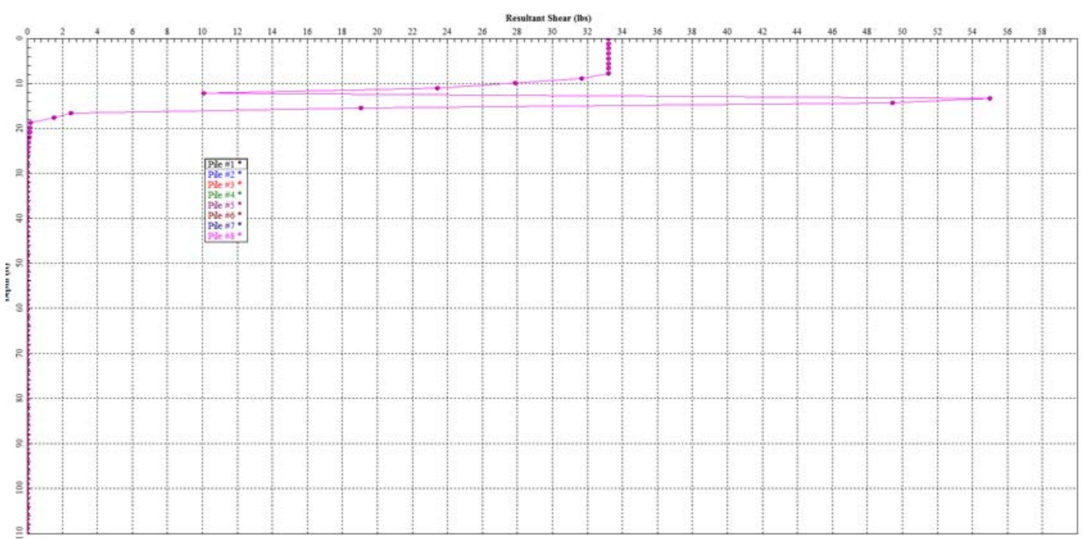

Figure 6, Resultant lateral deflection for the pile group of the monopole foundation

Equations for calculating the vertical settlement of the monopole foundation were obtained from the U.S. Army Corps of Engineers Design of Pile Foundations manual on page 4-21. Overall settlement was found to be 0.85 inches, which is within the allowable tolerance of one inch. Table *3* shows the values used to calculate settlement for each project site.

Table 3, Settlement calculations for the monopole site

Railyard	w_s	0.007937	ft	w_{pp}	0.002628	ft	w_{ps}	0.002628	w	0.158324	ζ_g	5.385165	S =	0.852598	(in)
	Qp	258.3	kips	C_p	0.02						Bbar	29			
	α_s	0.67		B	1	ft									
	Q_s	248.93	kips	q	172.2	kips									
	L	12	ft	C_s	0.029213										
	A	1	ft^2	D	11	ft									
	E	642693	ksf												

S:	group settlement
ζg:	group settlement factor
Bbar:	width of pile group

The concrete cement design for the pile cap will be Type 1 and 4000 psi with ¾ inch aggregate. The pile cap design consists of a square concrete cap with a width of 21 feet and a depth of 4 feet around the outer edge. The center of the pile cap is seven-feet thick to accommodate the anchor bolts for the monopole connection. The pile cap and pile group for this configuration were analyzed in GROUP, and no excessive displacement or settlement is expected. Figure *7* and Figure *8* display the pile cap dimensions and reinforcement layout. Number 10 rebar was chosen for this design to provide the necessary reinforcement and to make space in the design for the anchor bolts. Rebar needed to be place far enough away from each other to allow for the largest aggregate to pass through the cage. As seen in Figure *7*, rebar will be stacked to have enough reinforcement while still maintaining the spacing for the anchor bolts, pile connections, and aggregate. It is assumed that the stacked rebar can be modeled as a number 11 rebar for the purpose of calculations. AutoCAD was used to design the rebar spacing to the optimal fit, and the monopole orientation on the pile cap is rotated 45 degrees from the given orientation to maximize the rebar layout. Following the same procedure for the lattice tower pile caps, calculations were made to check against punching shear and bending moment loads. Wide pile cap design provides overhang to prevent punching shear and to account for the inaccuracy of pile placement.

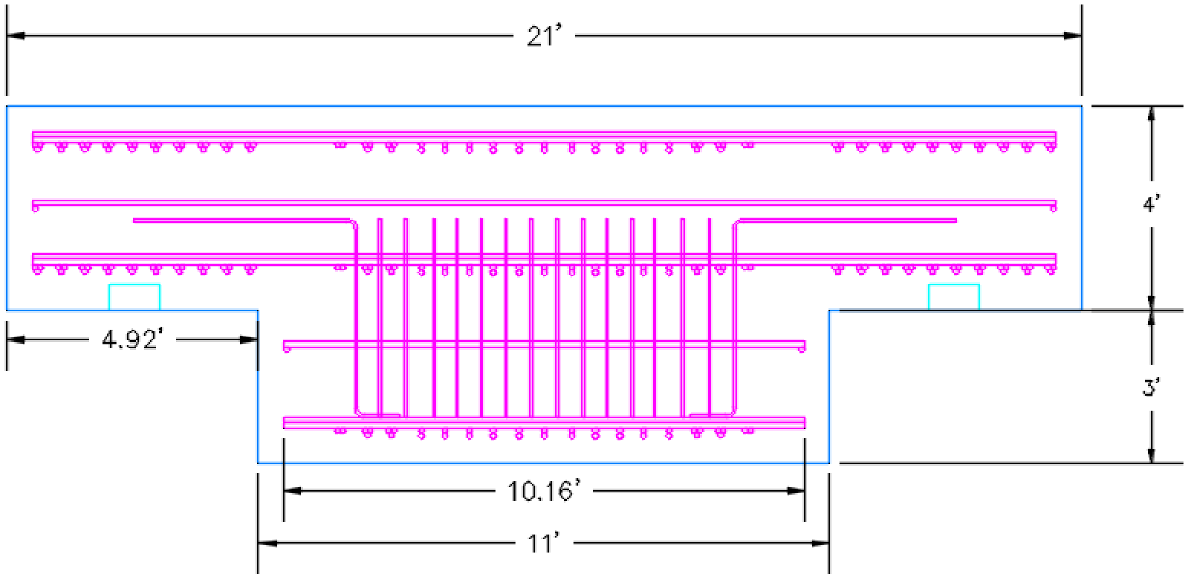

Figure 7, Monopole pile cap dimensions and reinforcement layout

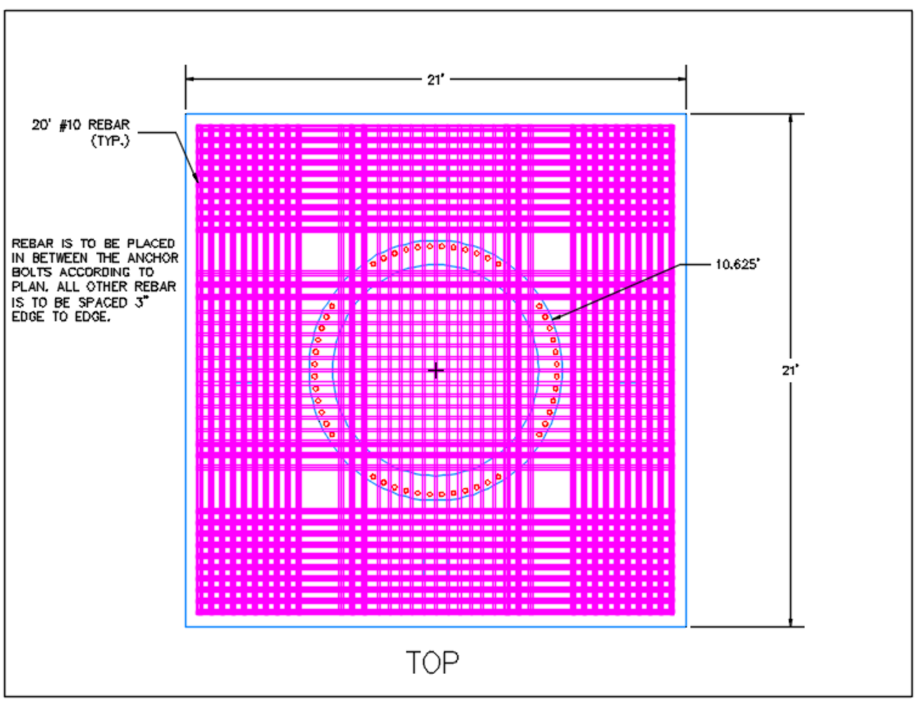

Figure 8, Plan view of the monopole pile cap design

Constructability

Saw Mill Creek Trail is the only access path to the project site across the meadowland. This trail is 13 feet wide and will limit the size of equipment and vehicles that can access the construction site. Construction of the lattice tower foundation will begin with the excavation of the site. Top soil will be removed to a depth of 3.5 feet with an area of 15 square feet for each of the four pile caps. Four piles will be driven for each pile cap with a diesel hammer to a depth of 97.5 feet below the surface. The piles will be spaced 10 feet apart in a square pattern around the center of the pile cap. Following the pile installation, piles will be preloaded with 30 kips per pile. Solid concrete blocks will be stacked on top of a steel frame that will rest on the four piles for each pile group. The concrete brick load will be eight feet tall on the 10 by 10-foot frame to achieve the required load. This load will be maintained on each pile group for 60 days to ensure proper settlement and to allow the subsurface soil to recover from the pile driving process.

Once the preloading process is complete, the end of each pile will be covered to prevent concrete from entering the pile during construction, see appendix D. After the piles have been capped, compaction of the subsurface will proceed the pile cap reinforcement cage. Each layer of the reinforcement will be constructed from bottom to top and then the anchor bolts and connection shoe will be attached to the cage. Reinforcement has been designed to allow the largest aggregate to pass between bars to ensure a uniform pile cap. Concrete pouring will commence after the reinforcement cage has been constructed. Concrete transportation vehicles will be subject to the same limitations as the pile driving equipment due to the size of Saw Mill Creek Trail.

Construction of the monopole foundation will begin with the excavation of the site. Top soil, including the asphalt layer and base layer for the parking lot will be removed to a depth of seven feet in the center of the pile cap and a depth of three feet around the center. See Figure *7* for specific dimensions of the pile cap. Access to this construction site will be limited by existing power lines and traffic. Coordination with the CSX South Kearny Yard will be necessary to ensure that parking lot on the construction site will be cleared of vehicles prior to excavation.

Pile driving will commence after the excavation process has been completed. A diesel hammer will be employed to drive the eight pre-stressed concrete piles to a depth of 110 ft. The soil around the base of each pile will be compacted to prevent the pile from shifting. Preloading is not required for these piles as they will be driven down to the bedrock layer.

After the piles are in place the construction of the pile cap will begin with building the reinforcement cage. Each pile will need to be epoxy drilled and have nine #9 rebar attached to the top of the pile to resist uplift loads between the pile to the pile cap. These bars will have 90 degree hooks and be 23 inches tall and the normal section will be 13.5 inches for the proper development. Spacing of the reinforcements has been carefully designed to allow space for the anchor bolts and to allow the largest aggregate in the concrete to pass between bars. In the field, it is expected that reinforcement bars will need to be adjusted to make them fit within the necessary tolerances. Details for each pile cap and for the attachment points can be found in appendix F.

Lessons Learned

None of the students working on this project had much experience with foundations or reinforced concrete design. Extensive research was conducted to understand the basics of driven pile design and reinforced concrete cement design. Reference manuals were obtained that guided the design for each foundation. Reference texts and manuals such as the FHWA Driven Pile Foundation Manual and the Army Corps of Engineers Pile Design Manual were found to be instructive and reliable for a design guide.

Constant and effective communication is essential for any project to be completed successfully. Time could have been saved on this project if more consistent communication was maintained with project sponsors and with faculty advisors. Communication within the project group could have been improved but was consistent overall. Team members consistently attended meetings and spent the required time to complete scheduled tasks. One month into the project an additional team member was added to the project. Communication with this new team member could have been improved, but due to schedule conflicts and health issues, this team member had to leave the project shortly after being assigned. This may have been worked out with better communication to facilitate remote participation with the design.

Design processes in the professional world are not as simple as they might seem. From correspondence with professionals, it has become apparent that extensive research, learning, and design iterations are required for the design process. Credible reference sources are crucial for obtaining the correct equations and coefficients for design calculations. College level text books are good sources of conceptual information but professional design manuals, such as the FHWA Pile Design Manual, were necessary for this project.

Conclusions

Two foundations were designed, one for a lattice tower in New Jersey meadowlands and one for a monopole tower in the Kearney CSX South Railyard. Each site presented engineering challenges with soil stratification, load management, and site access. The project began with extensive research into deep foundation design. It was quickly ascertained that textbooks were not as much help in the design process as professional manuals. The FHWA Pile Design Manual and the Army Corps of Engineers Pile Design Manual were used to determine the bearing capacity of the soil for each site and of the piles for each site. Reference manuals, such as the Civil Engineering Reference Manual (Lindberg, 1992), were used to guide the reinforced concrete design calculations. Completed calculations can be found in the appendices of this report. Assumptions were made to make up for some of the unknown information or lack of knowledge on the part of the design team. After the completion of the foundation calculations and design, construction schemes were developed to install these foundations at their sites despite challenges. Efforts were made to develop a conservative design to ensure the integrity of each foundation. It has been determined that the current design is suitable to withstand the given loads required to meet an 80-year design life.

References

Bowles, J. E. (1988). *Foundation analysis and design* (4th ed.) McGraw-Hill.

Hahs, J. D., Erlemann, G. G., Beutnagel, C. H., & et al. (2002). *CRSI design handbook* (9th ed.) CRSI.

Hannigan, P. J., Rausche, F., Likins, G. E., & et, a. (2014). *Design and construction of driven pile foundations - volume I* (7th ed.) FHWA.

Herndon, R. L. (1991). *Driven pile foundations* (EM 1110-2-2906 ed.) US Army Corps of Engineers.

Lindeburg, M. R. (1992). *Civil engineering reference manual* (6th ed.) Professional Publications, Inc.

McCormac, J. C., & Brown, R. H. (2008). *Design of reinforced concrete* Wiley.

Equations in Appendices:

- A – pg. 89
- B – pg. 88
- C – pg. 88
- D – pg. 133
- E – pg. 133
- F – pg. 133
- G – Equation 12.2

Thornton, W. A., Holland, M. V., Aminmansour, A., & et, a. (2011). *Steel construction manual* (14th ed.) AISC.

Equations in Appendices:

- K – Equations J2-4 and J2-5
- M - Equations D2-1 and D2-2
- O – Table 4-5

Wight, J. K., & MacGregor, J. G. (2009). *Reinforced concrete mechanics and design* (6th ed.) Pearson.

Equations in Appendices:

- L – Equation 8-18, pg. 383
- N - pg. 382

Building code requirements for structural concrete (ACI 318-14): an ACI standard: commentary on building code requirements for structural concrete (ACI 318R-14), an ACI report

Equations in Appendices:

- H – 17.5.1.26
- I – 17.4.1.2
- J – 17.4.3.1

Appendix A

Team Member Resumes

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(760) 462-5002 dannyjp90@gmail.com

EDUCATION

Brigham Young University Provo, UT
Bachelor of Science, Civil Engineering April 2017

- Relevant coursework in Materials Science, Fluid Mechanics, Chemistry, Geology, Surveying, and Environmental Science
- Member of Air Force Reserve Officer Training, extensive military professional leadership training

EXPERIENCE

Missionary Training Center Provo, UT
Facility Services Student Supervisor Aug 2014 - Present

- Efficiently manage facilities coordination for international and religious dignitaries, improving processes and saving thousands of dollars for event budget expenditures
- Promoted to supervisor over 20+ employees and 19 buildings after 3 months on the job
- Provides policy and HAZCOM training for new employees each month

Merrill Construction Apple Valley, CA
Construction Crew Leader Nov 2011 - Aug 2012

- Led a three-man crew to renovate and repair six residential homes in the Apple Valley area
- Provided on the job training for new employees
- Provided regular maintenance for 38 properties across the valley

Jake's Archery Orem, UT
Sales Associate and Technician April 2013 - Aug 2014

- Provided quality customer service to a global list of clients via email and telephone communication
- Assessed and repaired damaged hunting equipment for a variety of customers
- Managed the daily shipping of hundreds of packages across the country

VOLUNTEER SERVICE

Provo City School District Provo, Utah
Volunteer Tutor Jan 2013 – April 2013

- Worked with elementary school students in need of reading, writing, and math tutoring 2 hours per week

The Church of Jesus Christ of Latter-day Saints Winnipeg, Canada
Volunteer Representative Oct 2009- Oct 2011

- Adapted to a new culture in order to provide quality service to the local people
- Executed various humanitarian work projects including a soup kitchen
- Maintained 70-hour work weeks providing service and leading 6 other volunteers

SKILLS AND INTERESTS

- Proficient in VBA, Microsoft Excel, and Microsoft Word experience designing sheets for open channel flow
- Basic in GIS mapping and analysis software, experience in educational setting designing multiple analysis maps
- Proficient in Structural Analysis (statics, flexibility method, stiffness method, and moment distribution)
- Certified HAM radio operator, trained by red cross for communications in natural disasters

Todd L. Weichers II

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EDUCATION

BS: Civil Engineering (Anticipated) June 2017
Brigham Young University Provo, UT
- GPA: 3.2/4.0
- Recipient of the California Highway Patrol 11-99 Foundation Scholarship for academic excellence
- Relevant Courses: AutoCAD, Hydrology, Fluid Mechanics, Hydrologic Modeling

AS: General Studies Dec. 2012
Brigham Young University-Idaho Rexburg, ID
- GPA: 3.8/4.0

EXPERIENCE

Technical Support August 2016- Present
Aquaveo Provo, UT
- Solve customers' questions about SMS, GMS, WMS and ArcHydro Groundwater software
- Communicate complex solutions and instructions through various forms of communication

Civil Engineer Analyst April 2016- August 2016
Kimley-Horn and Associates Las Vegas, NV
- Designed utility, grading, and general civil improvement plans
- Performed due diligence for project starts
- Maintained close relationships with local municipalities in order to progress client's needs

Crane Engineering Intern April 2015- August 2015
Mountain Crane Services Salt Lake City, UT
- Developed presentations for new client meetings
- Supervised multiple construction projects and crane jobs

Civil Engineering Intern Sept. 2014- Dec. 2014
Merrell Johnson Companies Apple Valley, CA
- Drafted AutoCAD Civil 3D drawings including: street plans, grading plans, and sewer improvement plans
- Tested samples of soil and concrete for strength and adequacy of foundation

McKay Harper

Civil Engineering with Construction Emphasis

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Experience

Harper Construction ● Blackfoot, Idaho

Operator and Laborer (May 2006 - August 2010, June 2014 - Sep 2014)

- Learned work smart and diligently with others after stripping and re-applying 32 roofs in 40 days
- Proficient in operating heavy equipment such as track hoes, backhoes, dump trucks, and skid loaders, and lifts
- Framed, wired, plumbed, laid brick, and was involved with every aspect of building a home from start to finish
- Poured over 300 yards of concrete pads, foundations, curbs, gutters, and sidewalks and learned how to finish concrete
- Attended pre-bids and bids over 1.3 million dollars in behalf of company and learned about commercial construction

Harper Farms ● Blackfoot, Idaho

Laborer and Manager (May 2006 - August 2010, June 2014 - Sep 2014)

- Moved and managed water irrigation on 500 acres daily and learned to complete, communicate, and delegate tasks
- Operated and repaired large heavy equipment and machinery such as tractors, loaders, and tractor cabs
- Learned how to improvise and stay composed when immediate solutions were needed during emergencies
- Expected to cut and bail 500 acres of hay with no supervision and learned to provide quality work and progress

The Church of Jesus Christ of Latter-day Saints ● Kenya Nairobi Mission

Missionary and Mission Leader (June 2011 - June 2013)

- Assigned to lead 10 to 15 missionaries 600 miles away from Mission President and learned independence and trust
- Reported daily and weekly to mission president on success and needs of the missionaries I lead
- Learned to solve temporal and social problems among the people I lead and help them work together
- Volunteered 2 years of service in teaching doctrine for the church and learned to give meaningful service and love

Snake River High School ● Blackfoot, Idaho

Student Body Secretary (September 2009- May 2010)

- Helped supervise and organize the funds for activities, parties, and student-school functions
- Provided notes and minutes to help the president and other leaders make more informed decisions

Appendix B

Lattice Tower Bearing Capacity Calculations

Alpha Method

Alpha Method for calculating bearing capacity of a single pile at the meadowlands site. Found on page 245 of *Design and Construction of Driven Pile Foundations - Volume 1*, U.S. Department of Transportation Federal Highway Administration Publication No. FHWA-NHI-16-009. Page references in this section of the report refer to the FHWA manual.
Step 1: Delineate the soil profile into layers as seen in Table *4*.

Table 4, delineation of soil profile at the lattice tower site

Depth (ft)	γ (pcf)	Depth (ft)	γ (pcf)
4	90	56	120
8	100	60	120
12	115	64	120
16	108	68	120
20	115	72	120
24	120	76	125
28	115	80	125
32	115	84	125
36	125	88	125
40	120	92	125
44	125	96	125
48	122	100	120
52	122	104	120

Step 1: Determine the pile adhesion, C_a from figure 7-17 on page 248. The results for each soil layer can be found in Table *5*.

Table 5, Depth and Ca values determined from FHWA manual figure 7-17

Depth (ft)	C_a (ksf)	Depth (ft)	C_a (ksf)
12	0.7	60	0.95
16	0.95	64	1.08
20	0.95	68	1.16
24	0.96	72	1.11
28	0.95	76	1.08
32	0.75	80	0.96
36	0.75	84	0.94
40	0.6	88	1.01
44	0.78	92	1.08
48	0.8	96	1.08
52	0.86	100	1.11
56	0.86	104	1.15

Step 2: Compute unit shaft resistance (f_s) for each soil layer according to Equation *1*. Unit shaft resistance for each soil layer can be found in Table *6*.

Equation 1, unite shaft resistance

$$f_s = C_a = \alpha\, s_u$$

Table 6, Depth and sleeve resistance per soil layer

Depth (ft)	f_s(kips)	Depth (ft)	f_s(kips)
12	14.7	60	19.9
16	19.9	64	22.6
20	19.9	68	24.3
24	20.1	72	23.2
28	19.9	76	22.6
32	15.7	80	20.1
36	15.7	84	19.7
40	12.6	88	21.2
44	16.3	92	22.6
48	16.8	96	22.6
52	18.0	100	23.2
56	18.0	104	24.1

Step 3: Compute shaft resistance in each soil layer and the nominal shaft resistance, R_S, in kips. Multiplying the shaft resistance by the pile area for each layer yields the shaft resistance per layer in kips, see Table *7*. Nominal sleeve resistance is calculated by the sum of layer shaft resistances.

Table 7, depth and nominal sleeve resistance

Depth (ft)	R_s (kips)	Depth (ft)	R_s (kips)
12	14.7	60	227.5
16	34.6	64	250.1
20	54.5	68	274.4
24	74.6	72	297.6
28	94.5	76	320.2
32	110.2	80	340.3
36	125.9	84	360.0
40	138.4	88	381.2
44	154.8	92	403.8
48	171.5	96	426.4
52	189.5	100	449.7
56	207.6	104	473.8

Step 4: Compute the unit toe resistance using equation 7-11 which is seen in Equation *2*. Undrained shear strength is found using Equation *3*.

Equation 2, unite toe resistance

$$q_p = \mathrm{N}_c \, s_u$$

Equation 3, undrained shear strength

$$S_u = \frac{(q_t - \sigma)}{N_k}$$

The value of q_t is obtained from the CPT results and σ is found from γ values of table 1 multiplied by the depth of the layer of soil. The term N_c is a dimensionless bearing capacity factor which depends on the pile diameter and the depth of embedment. The bearing capacity factor, N_c, is usually taken as 9 for deep foundations. (FHWA volume 1 page 248) Values for q_p were found to be very small compared to values for R_s. For conservative calculations q_p was ignored in bearing capacity for the lattice tower site.

Step 5: Compute the nominal toe resistance, R_p (kips). This step was ignored due to negligible results for nominal toe resistance.

Step 6: Compute the nominal pile resistance, R_n from the sum of the shaft and toe resistances. This formula can be seen in Equation *1*. The value of R_n was found to be 449.8 kips, which is the same magnitude as R_s.

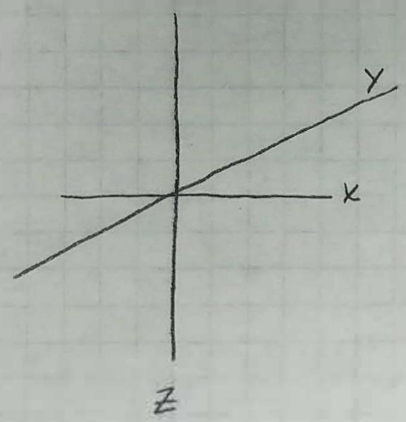
Equation 4, nominal pile resistance, Rn

$$R_n = R_s + R_p$$

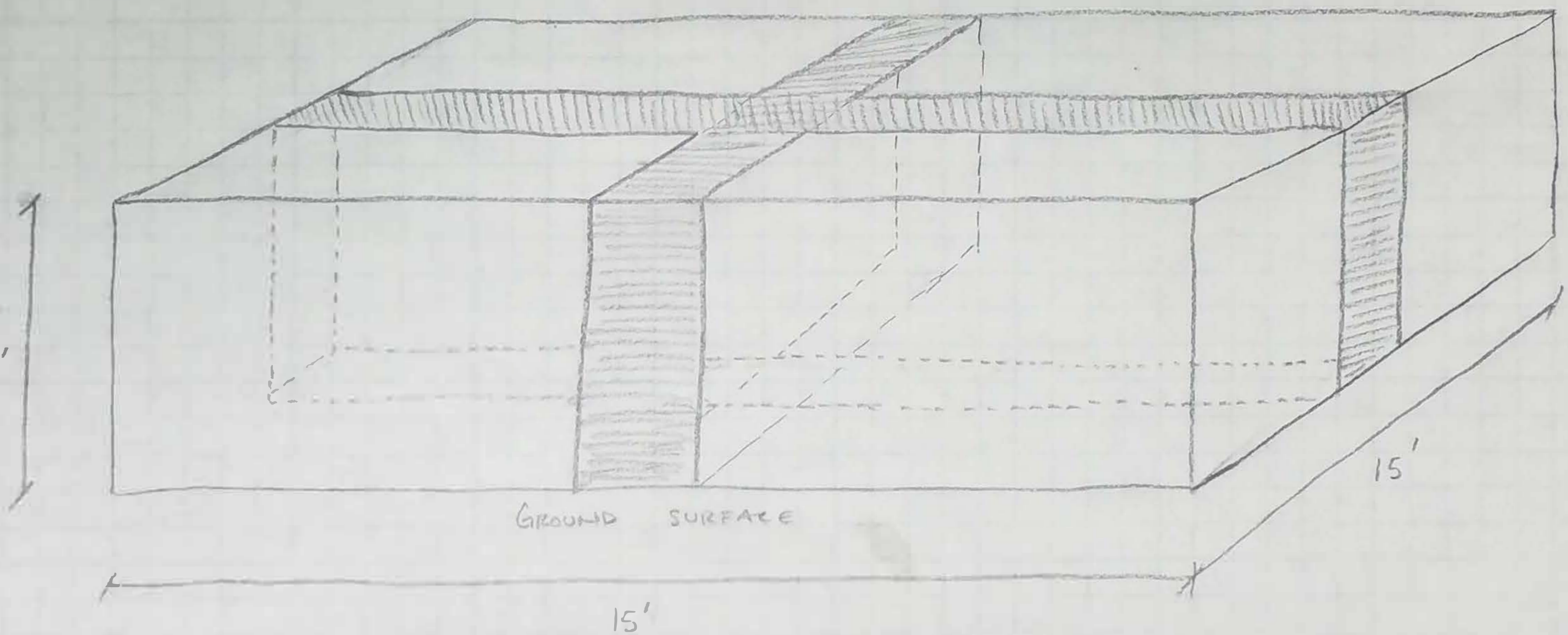
Step 7: Compute the factored resistance, R_r (kips). Φ_{stat} for alpha method with single pile in compression = 0.35. Factored resistance was considered unnecessary based on this statement found in the FHWA manual and the fact that planned pile diameters are 20 inches. "Therefore the designer should consider this factor if performing analyses for piles larger than 24 inches in diameter regardless of pile type." An overall factor of safety of four for each pile group also provides the required level of confidence for this design.

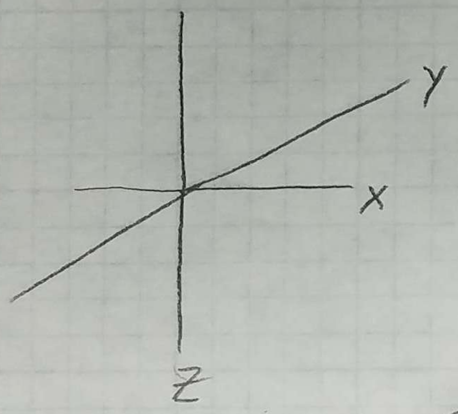
Appendix C

Lattice Tower Pile Cap Design Calculations

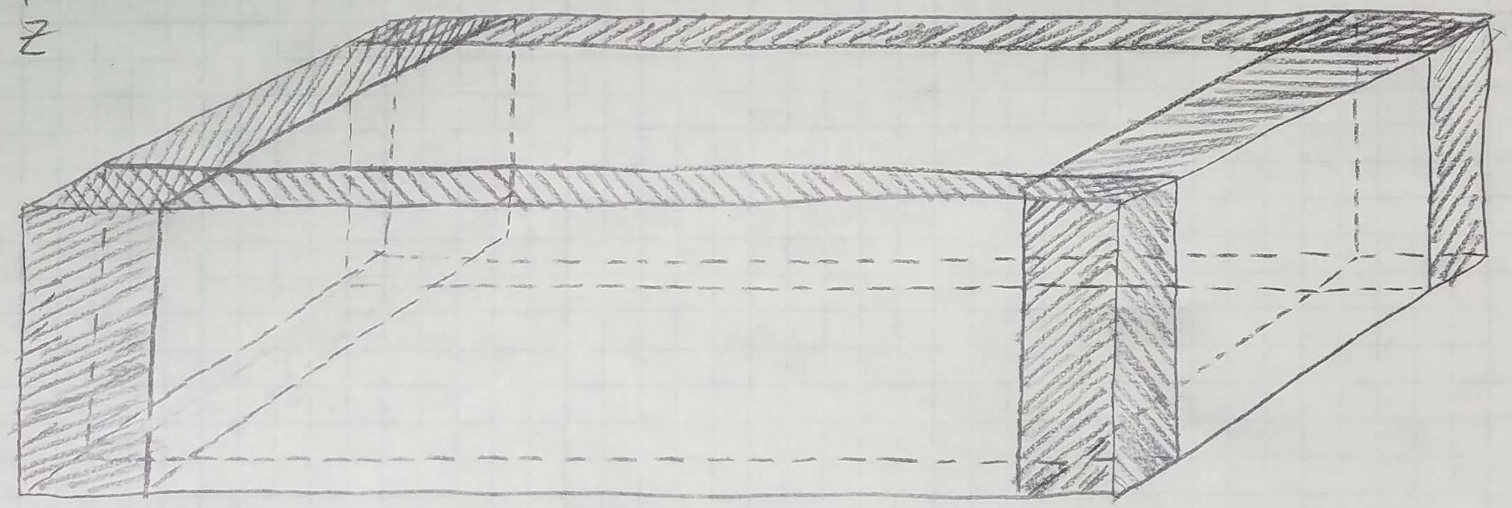


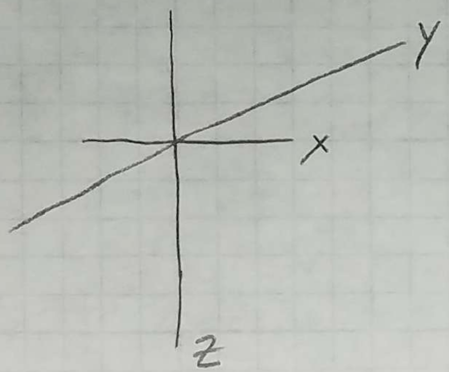
ORIENTATION OF BEAMS ANALYZED IN PILE CAP
FOR CROSS ORIENTATION
or (plus)



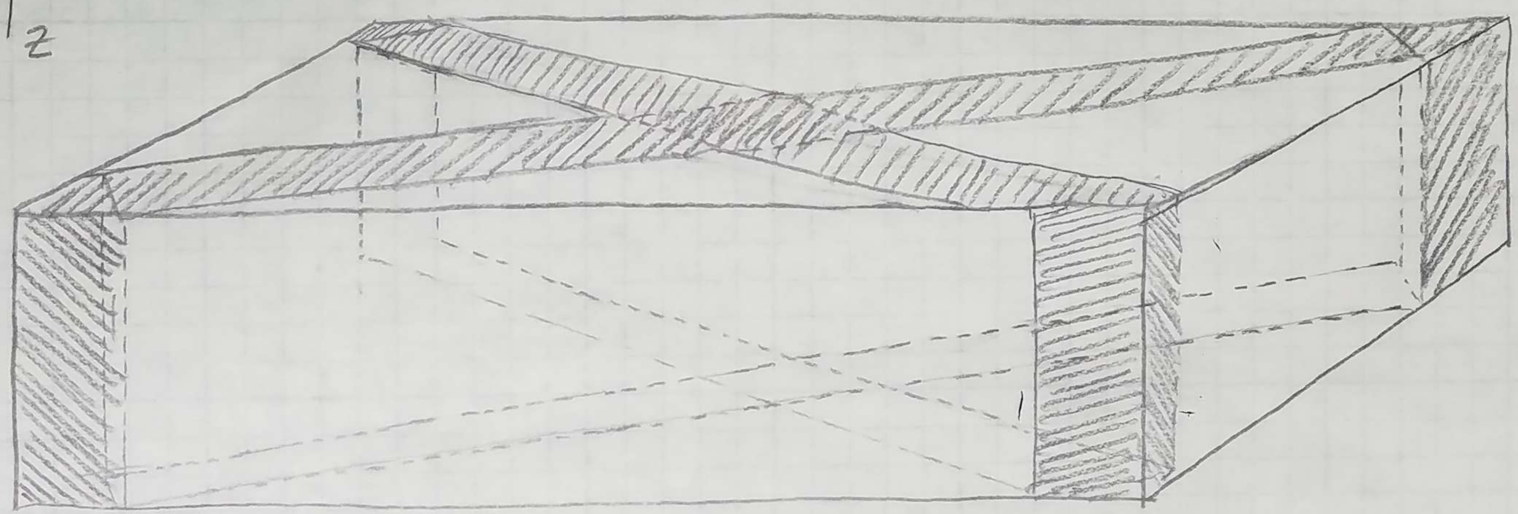


ORIENTATION OF BEAMS ANALYZED
IN PILE CAP FOR
EDGE BEAMS





ORIENTATION OF BEAMS ANALYZED IN PILE CAP FOR "X" ORIENTATION
or
DIAGONAL



[415.8 KIP LOAD TRANSFERING FROM THE TOWER TO THE SHOE CONNECTION. SHOE = 19½" × 19½" × 1½"

50.1 kip and 30.9 kip shear in x and y directions.
Z IS VERTICAL DIRECTION

+ UPLIFT OF 338.8 kip uplift with shear below 50.1 and 30.9 kip

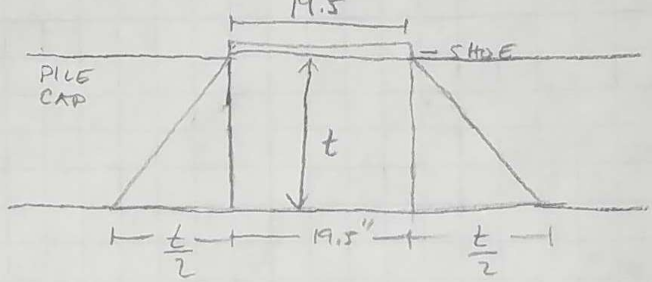
▶ CHECKING PUNCHING SHEAR OF FOOT LOAD ONTO PILE CAP

FOOT PERIMETER (D) = 4(19.5") = 78"

Force (F) = 416 kip =

THICKNESS (t) = 3.5 ft = 42 in

$f'_c = 4000$ psi



– DEPTH OF SHEAR (d)
b_o = perimeter of shear
$b_o = 4(t + 19.5\text{ in}) = 4(42\text{ in} + 19.5\text{ in}) = 246\text{ in}$

(G) $d = \dfrac{F}{\phi 4 \sqrt{f'_c}(b_o)} = \dfrac{416\text{ kip}}{(.75)(4)(\sqrt{4000}\text{ psi})(246\text{ in})} = 8.9\text{ in}$

8.9 in < 42 in → NO 2WAY PUNCHING OF ✓

BOLTS HOLDING PLATE TO CAP FOR ATTACH ? TOWER FOOT POINT.

▶ ANCHOR BOLT PULLOUT YIELDING & SHEAR
- USE 1.25 in diameter bolts (D) at 30 in deep (12 bolts)

(P_u) PULLOUT = 340 kip
(V_u) SHEAR = $\sqrt{50.1^2 + 39.1^2} = 63.6$ kip

- SHEAR

$A_{se,v} = \frac{\pi}{4}(1.25)^2 = 1.23\text{ in}^2$ $f_{uta} = 1.9 f_y = 1.9(60{,}000) = 114{,}000$ ksi

(H) $V_i = A_{se,v} f_{uta} = (1.23\text{ in}^2)(114{,}000\text{ ksi}) = 139.9$ kip

$V_T = (V_i)(12\text{ bolts}) = (139.9\text{ kip})(12) = 1678.8$ kip

$V_u < V_T$ — BOLT SHEAR OK ✓

- YEILDING OF STEEL IN TENSION (P_i)

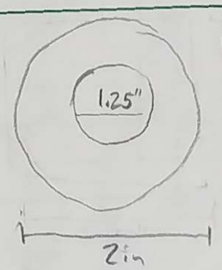
(I) $N_{sa} = (A_{se,N})(f_{uta}) = (1.23\text{ in}^2)(114{,}000\text{ ksi}) = 139.9$ kip ACI

$P_i = (N_{sa})(12) = 1678.8$ kip

ACI 17.4.1.2

$P_u > P_i$ – BOLT YEILDING OK ✓

- PULLOUT OF ANCHORS (STUDDED) (N_p)



BOLT DIAMETER = 1.25 in
NUT DIAMETER = 2 in
$f'_c = 4000$ psi

(J) $N_p = 8 A_{brg} f'_c$

$N_p = 8(1.9\ in^2)(4000\ psi)$ $\quad A_{brg} = \frac{\pi}{4}(2^2 - 1.25^2) = 1.9\ in^2$

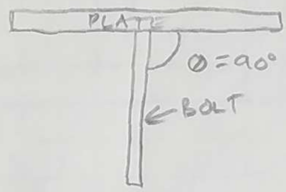
$N_p = 61.3$ kip

$N_{p_u} = (61.3\ kip)(12\ bolt) = 735\ kip$

$N_{p_u} > P_u$ – NO PULLOUT, OK ✓

▶ WELD OF ANCHOR BOLTS TO LATTICE TOWER PLATE

- FORCE ON EACH BOLT



$P_u = 340$ kip, pullout

$R_n = \frac{340\ kip}{12\ bolts} = 28.3\ kip/bolt$

- WELD DESIGN

(K) $\phi R_n = \phi(.6)(F_{EXX})(1.0 + 0.5 \sin\theta^{1.5}) A_{we}$

$A_{we} = t\ell$

t = throaght of weld = $\sin(45)\, d$

$d = 5/16$ [TABLE J2.5, AISC]

ℓ = circumference of bolt = πD

$A_{we} = \sin(45)(5/16)(\pi)(1.25)$

$A_{we} = .868\ in^2$ $\quad \theta = 90°$ $\quad F_{EXX} = 70$ ksi

(K) $\phi R_n = (.75)(.6)(70\ ksi)(1.0 + 0.5 \sin(90)^{1.5})(.868)$

$\phi R_n = 41.0$ kip

$\phi R_n > R_n$ ok ✓

- USE 5/16 inch weld of 70 ksi weld around circumference of BOLT.

FROM ANALYSIS RUN ON "VISUAL ANALYSIS" THE MAXIMUM MOMENT IN Z IS 542 KIP-ft AND THE MAXIMUM MOMENT IN X IS 5.5 KIP-ft.

(B) MAX Z MOMENT FOR FLEXURE FOR DIAGONAL MEMBER

$$R_N = \frac{M_U}{\phi b d^2}$$

b = WIDTH OF MEMBER CAP
d = DEPTH OF PILE MINUS 6 IN FOR REBAR COVER.

$$R_N = \frac{542\ k\text{-}ft(12\ in/ft)(1000\ lb/kip)}{(.9)(24in)(36in)^2}$$

$$R_N = 232.34\ psi$$

PERCENTAGE OF STEEL REQUIRED

(C)
$$\rho = \frac{.85 f'_c}{f_y}\left(1 - \sqrt{1 - \frac{2R_N}{.85 f'_c}}\right)$$

$$\rho = \frac{.85(4000psi)}{(60,000\ psi)}\left(1 - \sqrt{1 - \frac{2(232.34psi)}{.85(4000psi)}}\right)$$

$$\rho = .0040$$

AREA OF STEEL REQUIRED

(A)
$$A_s = \rho b d$$

$$A_s = .0040(36in)(24in)$$

$$A_s = 3.47\ in^2$$

▸ DETERMINING TYPE AND AMOUNT OF REBAR

$$\#9\ REBAR = A_g = 1 in^2$$
$$D = 1.128\ in$$

$$N = \frac{A_s}{A_g} = \frac{3.47}{1} = 3.47\ BAR$$

$\boxed{\text{USE 4, \#9 REBAR}}$

CHECK FOR YIELDING OF STEEL

(D)
$$a = \frac{A_s f_y}{.85 f'_c b} = \frac{(3.47in^2)(60,000psi)}{.85(4000psi)(24in)}$$

$$a = 2.55\ in$$

(E)
$$C = \frac{a}{.85} = \frac{2.55}{.85}$$

$$C = 3.00\ in$$

(F)
$$\varepsilon'_s = \frac{c - d'}{c}(.003)$$

▸ ASSUME 3 in COVER OF REBAR

$$\varepsilon'_s = \frac{3\ in - 3\ in}{3\ in}(.003)$$

$$\varepsilon_y = \frac{60\ ksi}{29000\ ksi} = .00207$$

$$\varepsilon'_s = 0$$

$\varepsilon'_s < \varepsilon_y$ THEREFORE STEEL WILL NOT YIELD

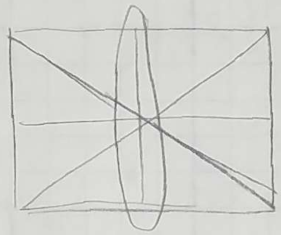
REQUIRED % OF STEEL FOR FLEXURE FOR VERTICAL MEMBER

$$R_N = \frac{(12\ in/ft)(1000\ lb/kip)(133\ kip\ ft)}{(.9)(24\ in)(30\ in)^2}$$

$R_N = 82.1\ psi$

% STEEL REQ'D

$$\rho = \frac{.85(4000psi)}{(6000)psi}\left(1 - \sqrt{1 - \frac{2(82.1)^{psi}}{.85(4000psi)}}\right)$$



$\rho = .00138$

AREA OF STEEL REQUIRED

$A_s = .00138(24in)(30in)$

$A_s = .997\ in^2$

CHECK FOR YIELDING

$$a = \frac{(.997\ in^2)(60000psi)}{(.85)(4000psi)(24in)}$$

$a = .733in$

$$c = \frac{.733}{.85}$$

$c = .863in$

$$\varepsilon_s' = \frac{.863 - 3in}{.863}(.003)$$

$\varepsilon_s' = .018$ WILL YIELD.

$A_s = 2.5in^2$

$$a = \frac{(2.5in^2)(60000psi)}{.85(4000psi)(24in)}$$

$a = 1.838in$

$$c = \frac{1.838in}{.85}$$

$c = 2.16in$

$$\varepsilon_s' = \frac{2.16in - 3in}{2.16in}(.003)$$

$\varepsilon_s = .00116$

$\varepsilon_s' < \varepsilon_y$ WILL NOT YIELD.

$\boxed{8 \# 5\ \text{BARS}}$

TYP FOR ALL EXCEPT MAX Z MOMENT

$A_s = 2.5 \text{ in}^2$

$a = 1.839 \text{ in}$
$c = 2.16 \text{ in}$

$\varepsilon_s' = .0016$

$\varepsilon_s' < \varepsilon_y$

WILL NOT YIELD

$\boxed{8 \text{ \#5 BARS}}$

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

COMET

MAX X MOMENT FOR FLEXURE

(B) $R_N = \dfrac{M_u}{\phi b d^2}$

$$R_N = \frac{5.5\,\text{kipft}(12\,\text{in/ft})(1000\,\text{lb/kip})}{(0.9)(36\,\text{in})(24\,\text{in})^2}$$

$R_N = 3.536\,\text{psi}$

PERCENTAGE OF STEEL REQUIRED

(C) $\rho = \dfrac{.85 f'_c}{f_y}\left(1 - \sqrt{1 - \dfrac{2R_N}{.85 f'_c}}\right)$

$$\rho = \frac{.85(4000\,\text{psi})}{60000\,\text{psi}}\left(1 - \sqrt{1 - \frac{2(3.536\,\text{psi})}{.85(4000\,\text{psi})}}\right)$$

$\rho = .000059$

AREA OF STEEL REQUIRED

(A) $A_s = \rho b d$

$A_s = .000059(36\,\text{in})(24\,\text{in})$

$A_s = .0509\,\text{in}^2$

DOES NOT MEET ACI CODE 8.6.1.1

$A_s = 3.47\,\text{in}$

$\boxed{4 \# 9 \text{ REBAR}}$

CHECK FOR YIELDING

(D) $a = \dfrac{A_s f_y}{.85 f'_c b} = \dfrac{3.47\,\text{in}^2(60000\,\text{psi})}{.85(4000\,\text{psi})(36\,\text{in})}$

$a = 2.55\,\text{in}$

(E) $c = \dfrac{a}{.85} = \dfrac{2.55}{.85}$

$c = 3.0\,\text{in}$

ASSUME 3in COVER OF REBAR

(F) $\varepsilon'_s = \dfrac{3.0\,\text{in} - 3\,\text{in}}{1.038}(.003)$

$\varepsilon'_s = 0 < \varepsilon_y$

WILL NOT YIELD

MOMENTS FOR THE "X" ORIENTATION ARE GOVERNING, MOMENTS FOR THE CROSS ORIENTATION ARE MODELED AFTER MOMENTS FROM THE "X" ORIENTATION.

▸ DEVELOPMENT LENGTH FOR # 9 REBAR

$\psi_e = 1 \quad d_a = 1.128$ in

$F_y = 60$ ksi $\quad f'_c = 4$ ksi

$\lambda = 1 \quad F_a = .7$ [TABLE 8-2]

L $\quad \ell dh = \left(\dfrac{.02\,\psi_e f_y}{\lambda \sqrt{f'_c}}\right) d_a F_a$

$\ell dh = \left(\dfrac{(.02)(1)(60000)}{(1)(\sqrt{4000})}\right)(1.128)(.7)$

$\ell dh = 14.98$ in ≈ 15 in

▸ BECAUSE DEVELOPMENT LENGTH IS 15 inches, EXTEND REBAR 15 INCHES PAST DESIGNED LENGTH (PAST PILE OR SIDE BEAM)

▸ PUNCHING SHEAR FOR PILES UNDER PILE CAP

PILE DIAMETER (D) = 20 in

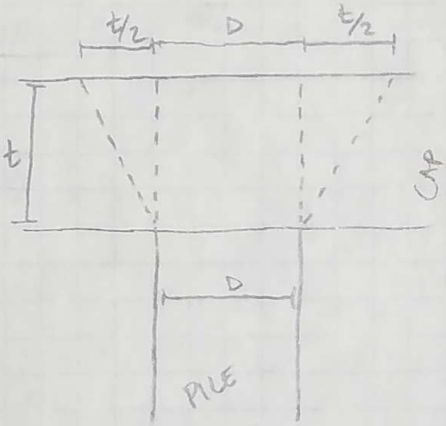
PILE CIRCUMFERENCE (c) = 62.8 in

FORCE (F) = 450 KIP (AS SAFETY FACTOR)

CAP THICKNESS (t) = 3.5 ft = 42 in

$f'_c = 4000$ psi

– DEPTH OF PUNCHING SHEAR (d)



G $\quad b_o = (D + t)\pi$

$b_o = (20 + 42\text{ in})\pi = 194.8$ in

$d = \dfrac{F}{\phi\, 4 \sqrt{f'_c}\, b_o} = \dfrac{450{,}000 \text{ lbs}}{(.75)\, 4 \sqrt{4000}\,(194.8)}$

$d = 12.18$ in

$d < t \quad$ ok ✓ – NO 2 WAY PUNCHING SHEAR

$12.18 < 42$ in ✓

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

▸ DETERMINING REBAR LAYOUT FOR CONNECTION OF PILE TO PILECAP

▸ FINDING AREA OF REBAR NEEDED FOR CONNECTION

$\phi P_n = 450$ kip.

★ all references to tables, figures, & pages come from "Reinforced concrete Mechanics & design 6E"

M $\phi P_n = \phi F_y A_g$ $\phi = 0.9$, $F_y = 60$ ksi (steel)

$$A_g = \frac{\phi P_n}{\phi F_y}$$

$$A_g = \frac{450 \text{ kip}}{(0.9)(60 \text{ ksi})} = 8.33 \text{ in}^2 \rightarrow 8.5 \text{ in}^2$$

▸ DETERMINING AMOUNT OF REBAR AND SIZE

USING #6 REBAR $\left(A_6 = .44 \text{ in}^2,\ d_6 = 3/4 \text{ in} \right)$

$$\frac{A_g}{A_6} \quad \frac{8.5 \text{ in}^2}{.44 \text{ in}^2} = 19.3 \rightarrow \boxed{20 \text{ sicks of \#6 rebar}}$$

▸ CIRCUMference of Pile with D= 20"

$$C = \pi D = (3.14)(20") = 62.8 \text{ in}$$

▸ DISTANCE OF REBAR THICKNESS

$$N = (20 \text{ sticks})\left(\frac{3/4 \text{ in}}{\text{stick}} \right) = 15 \text{ in}$$

$$62.8 \text{ in} - 15 \text{ in} = 47.8 \text{ in}$$

▸ SPACE BETWEEN EACH REBAR

$$\frac{N}{20} = \frac{47.8 \text{ in}}{20} = 2.39 \text{ in}$$

▸ DISTANCE TO EACH REBAR - ON CENTER

$$2.39 \text{ in} + d = 2.39 \text{ in} + .75 \text{ in} = 3.14 \text{ in} \approx 3\tfrac{1}{8}"$$

▸ DEVELOPMENT LENGTH FOR #6 REBAR

L $$l_{dh} = \left(\frac{.02 \Psi_e f_y}{\lambda \sqrt{f'c}} \right) d_6 F_a$$

$\Psi_e = 1$ $d_6 = .75$ in (#6)

$F_y = 60$ ksi $f'c = 4$ ksi

$\lambda = 1$ $F_a = .7$ [TABLE 9-2]

$$l_{dh} = \left(\frac{.02(1)(60 \text{ ksi})}{1 \sqrt{4000}} \right)(.75)(.7)$$

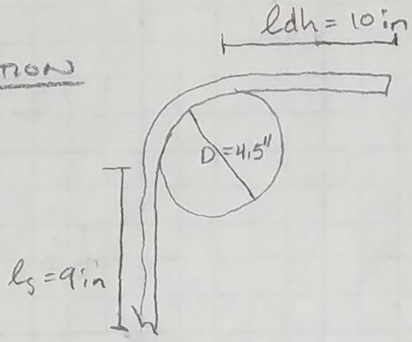
$$l_{dh} = 9.96 \text{ in} \approx 10 \text{ in}$$

▸ DIAMETER OF BENT REBAR for #6 bar

(N) $D = 6d_b$ (ACI CODE 7.1-7.2)

$D = (6)(.75) = 4.5$ in

▸ LENGTH OF HOOK FOR PILE CONNECTION

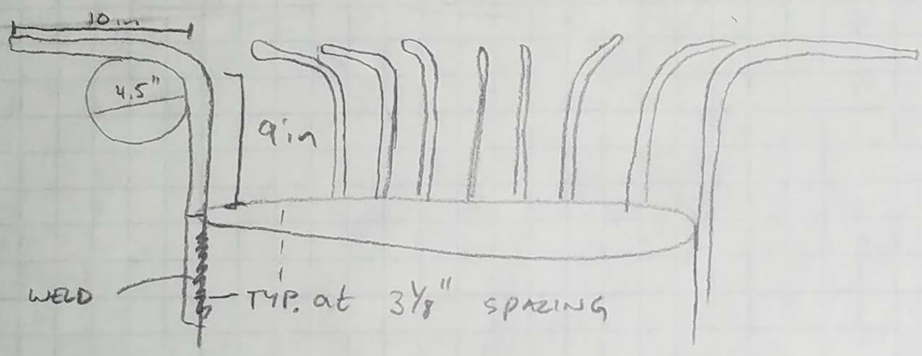


$l_{dh} = 10$ in

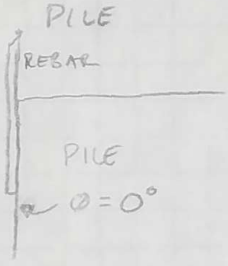
(N) $l_s = 12 d_b$ [ACI CODE]

$l_s = 12(.75) = 9$ in

▸ BRIEF DESCRIPTION OF PILE CONNECTION



▸ WELD CONNECTION FOR ANCHORING HOOK ONTO PILE



– FIND FORCE ON EACH REBAR

$$R_n = \frac{P_u}{20} = \frac{450 \text{ kip}}{20 \text{ bar}} = 22.5 \text{ kip/bar}$$

– FIND WELD LENGTH –

(K) $\phi R_n = \phi(.6)(F_{EXX})(1.0 + 0.5\sin(\theta)^{1.5})(A_{we})$ [AISC Eq J2-4 → J2-5]

$A_{we} = t\,l$

$d = 3/16$

$l = 5.5$ in (LENGTH OF WELD)

t = THROAT OF WELD $= \sin(45)\,d$

$A_{we} = \sin(45)(3/16)(5.5)$

$A_{we} = 0.729 \text{ in}^2$

8/ 13/

(K) $\phi R_n = (.75)(.6)(70\text{ ksi})(1.0 + 0.5\sin(0)^{1.5})(.729)$ $F_{EXX} = 70$ ksi

$\phi R_n = 22.97$ kip

$\phi R_n > R_n$ WELD OK ✓

- USE 3/16 inch weld, 2.25 in long on each side of the #6 rebar that connects to the pile cap.

▶ CHECK STEEL PILE FOR YEILDING (USE HSS 20 x0.5)

(M) $\phi P_n = \phi F_y A_g$ $F_y = 42$ ksi (AISC D2-1)

$\phi P_n = (.75)(42)(28.5)$ $A_g = 28.5$ ksi

$\phi P_n = 897.75$ kip

$\phi P_n > P_n$

$897.75 > 450$ ✓ OK

▶ CHECK PILE FOR FAILURE IN COMPRESSION

- ASSUME EFFECTIVE LENGTH OF PILE IS ESSENTIALL 0.

(O) $\frac{K \cdot L}{r}$ = EFFECTIVE STRENGTH = 0

$\phi P_n = 1080$ kip [TABLE 4-5, AISC 2014]

$\phi P_n > P_n$

$1080 > 450$ ✓ OK

▶ CHECK YIELDING AND COMPRESSION FAILURE FOR CORROSION

- CORROSION OF STEEL

λ = CORROSIVE VALUE OF 1.7 mil/year in saturated SOIL AND TIDAL ZONE, TO BE CONSERVATIVE

$$\lambda = \left(1.7\,\frac{\text{mil}}{\text{Year}}\right)(80\text{ year})\left(\frac{1\text{ in}}{1000\text{ mil}}\right) = .136\text{ in}$$

$t - \lambda = 0.5 - .136\text{ in} = .364$

☆ BECAUSE P_u is 4 times the needed strength and ϕP_n for HSS 20 X 0.5 IS SO LARGE — ASSUME CORROSION TURNS HSS 20 X 0.5 IN TO HSS 20 X 0.375

▸ CHECK HSS 20X 0.375 FOR YEILDING

(M) $\phi P_n = \phi F_y A_g$

$F_y = 42 ksi$
$A_g = 21.5 in^2$

$\phi P_n = (.75)(42 ksi)(21.5 in^2) = 677.25$ in

$\phi P_n > P_u$
$677.25 > 450$ ✓ OK

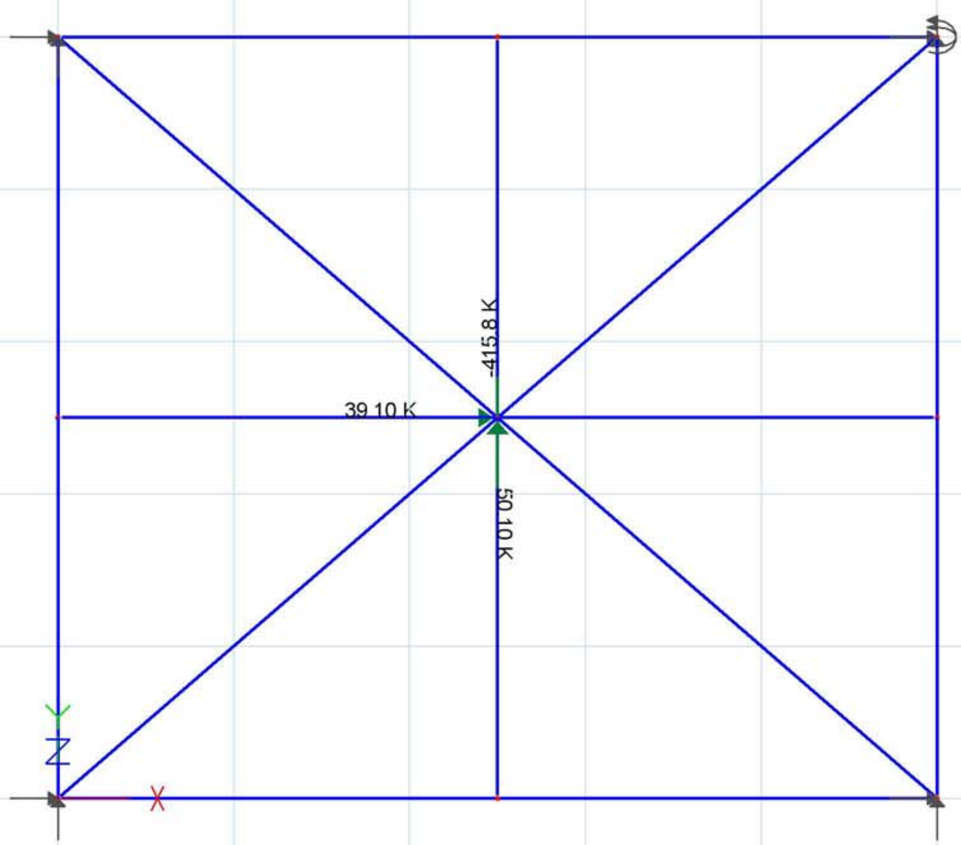
▸ CHECK HSS 20 X 0.375 FOR COMPRESSION FAILURE

(O) $\frac{KL}{r} = 0$

$\phi P_n = 813$ kip

$\phi P_n > P_u$

$813 > 450$ ✓ OK

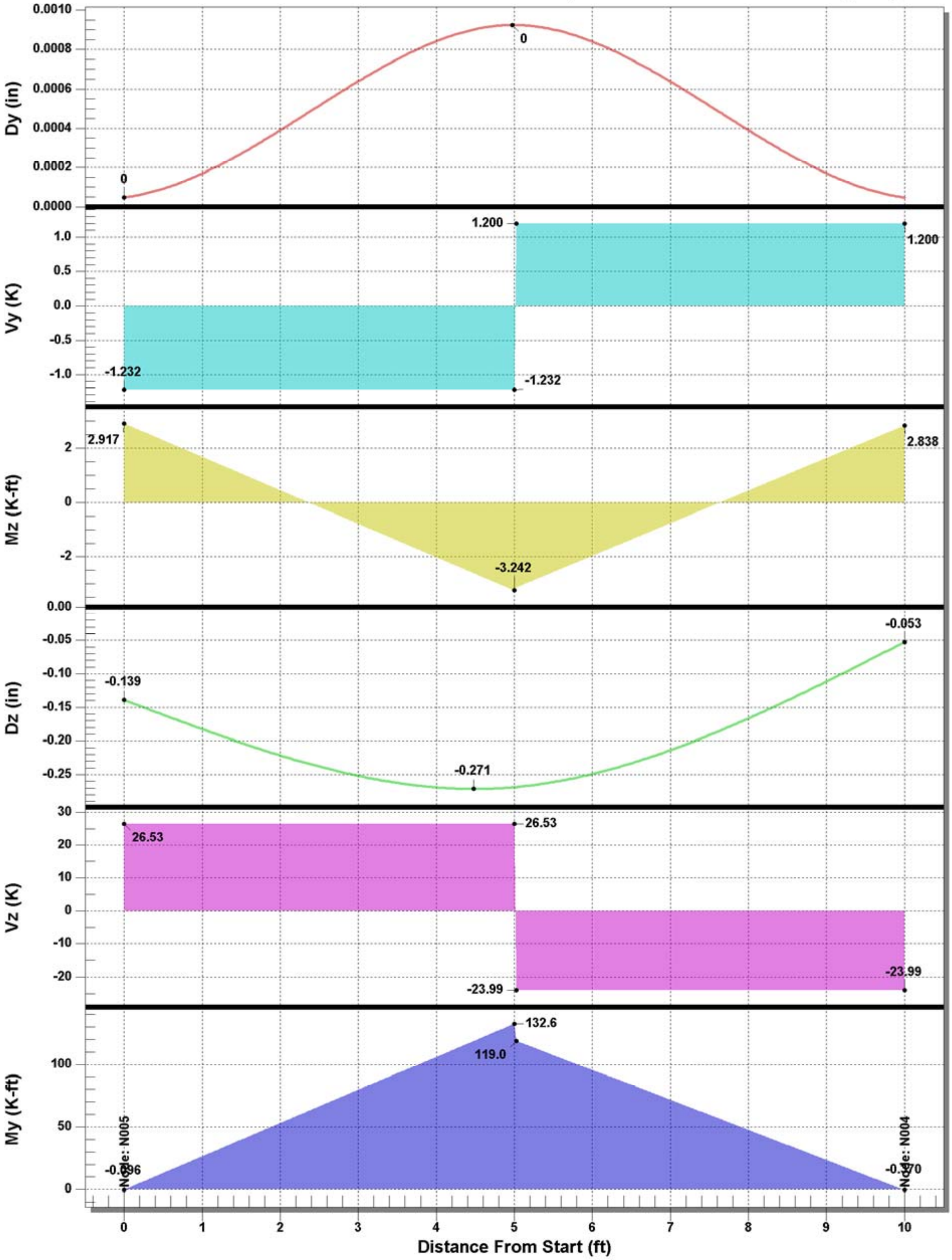


-415.8 K
39.10 K
50.10 K
Y
Z
X

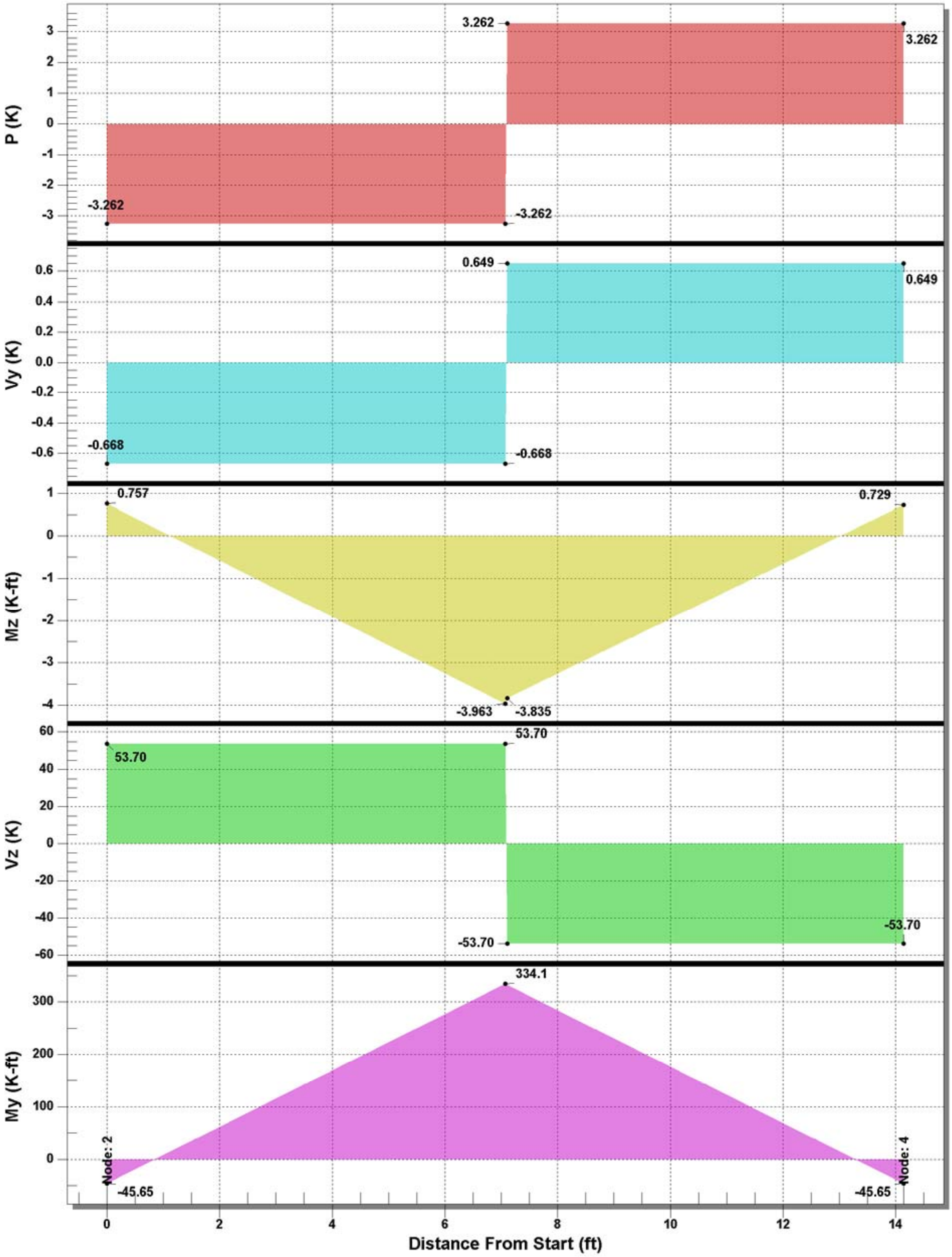
Member: BmX006
Load Case: L

Cap footing
HARPER ENGINEERING, INC.

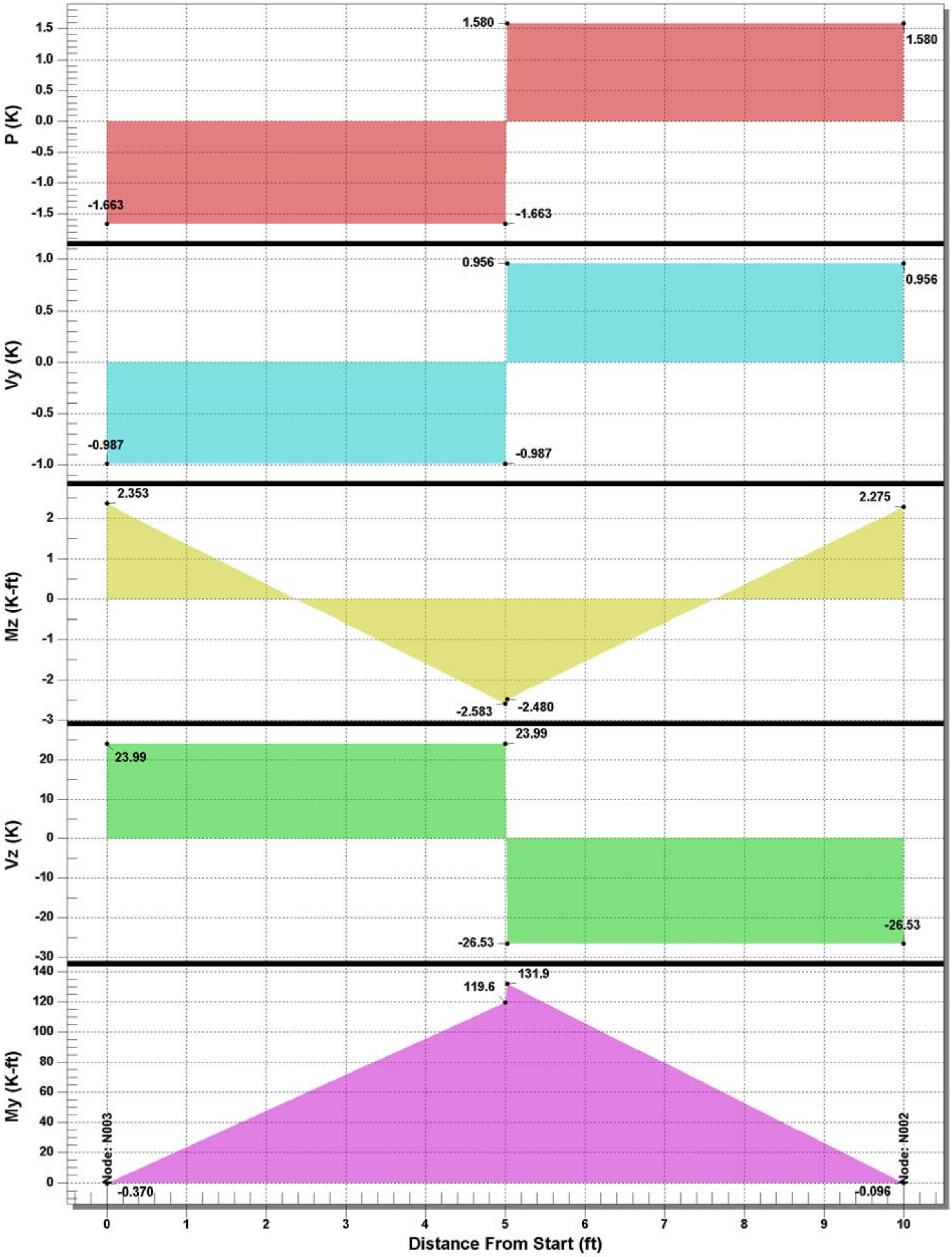
Bryce Harper
Mar 03, 2017; 02:23 PM



VisualAnalysis 12.00.0020
www.iesweb.com


P (K)
3.262
3.262
-3.262
-3.262
Vy (K)
0.649
0.649
-0.668
-0.668
Mz (K-ft)
0.757
0.729
-3.963
-3.835
Vz (K)
53.70
53.70
-53.70
-53.70
My (K-ft)
334.1
Node: 2
Node: 4
-45.65
-45.65
Distance From Start (ft)

Member: COL006	Cap footing	Bryce Harper
Load Case: L	HARPER ENGINEERING, INC.	Mar 03, 2017; 02:24 PM

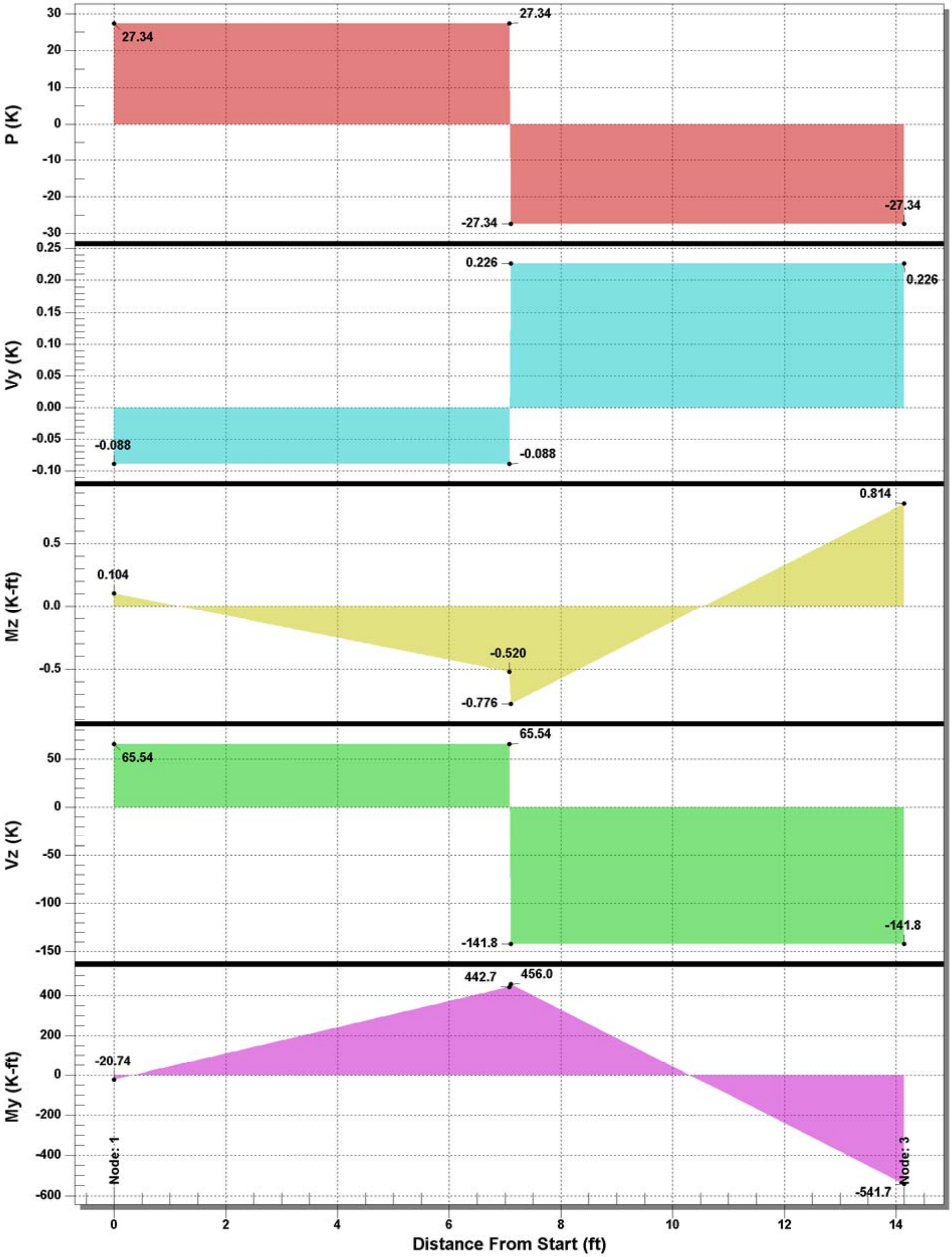


VisualAnalysis 12.00.0020
www.iesweb.com

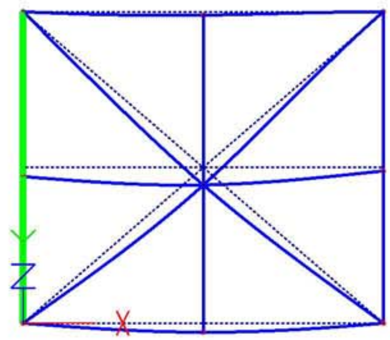
Member: V006
Load Case: L

Cap footing
HARPER ENGINEERING, INC.

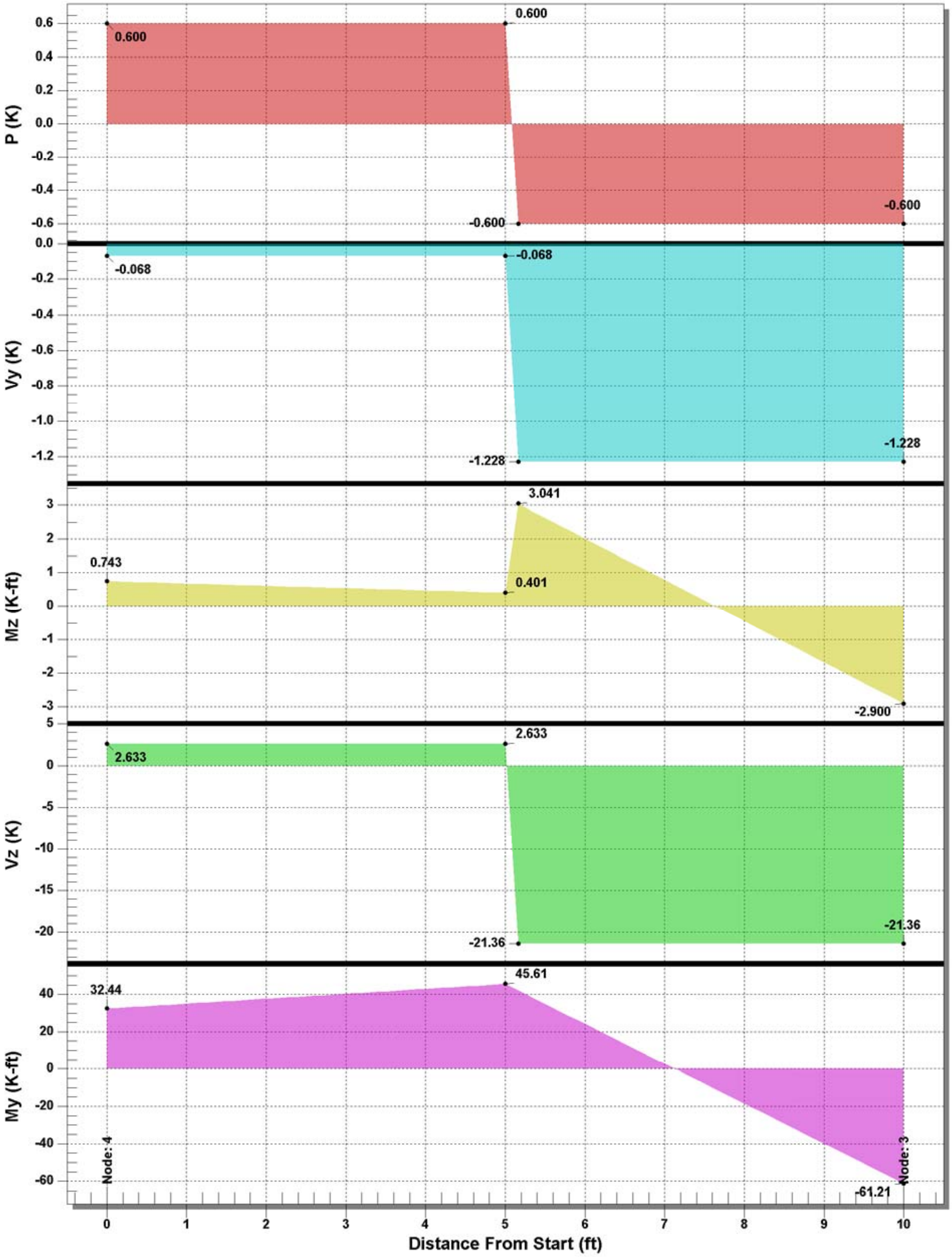
Bryce Harper
Mar 03, 2017; 02:24 PM

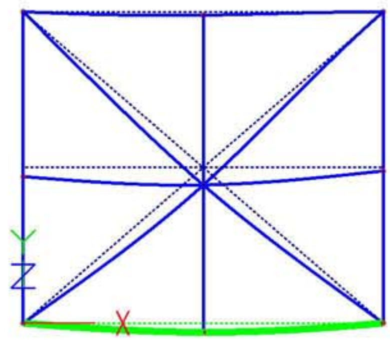


VisualAnalysis 12.00.0020
www.iesweb.com



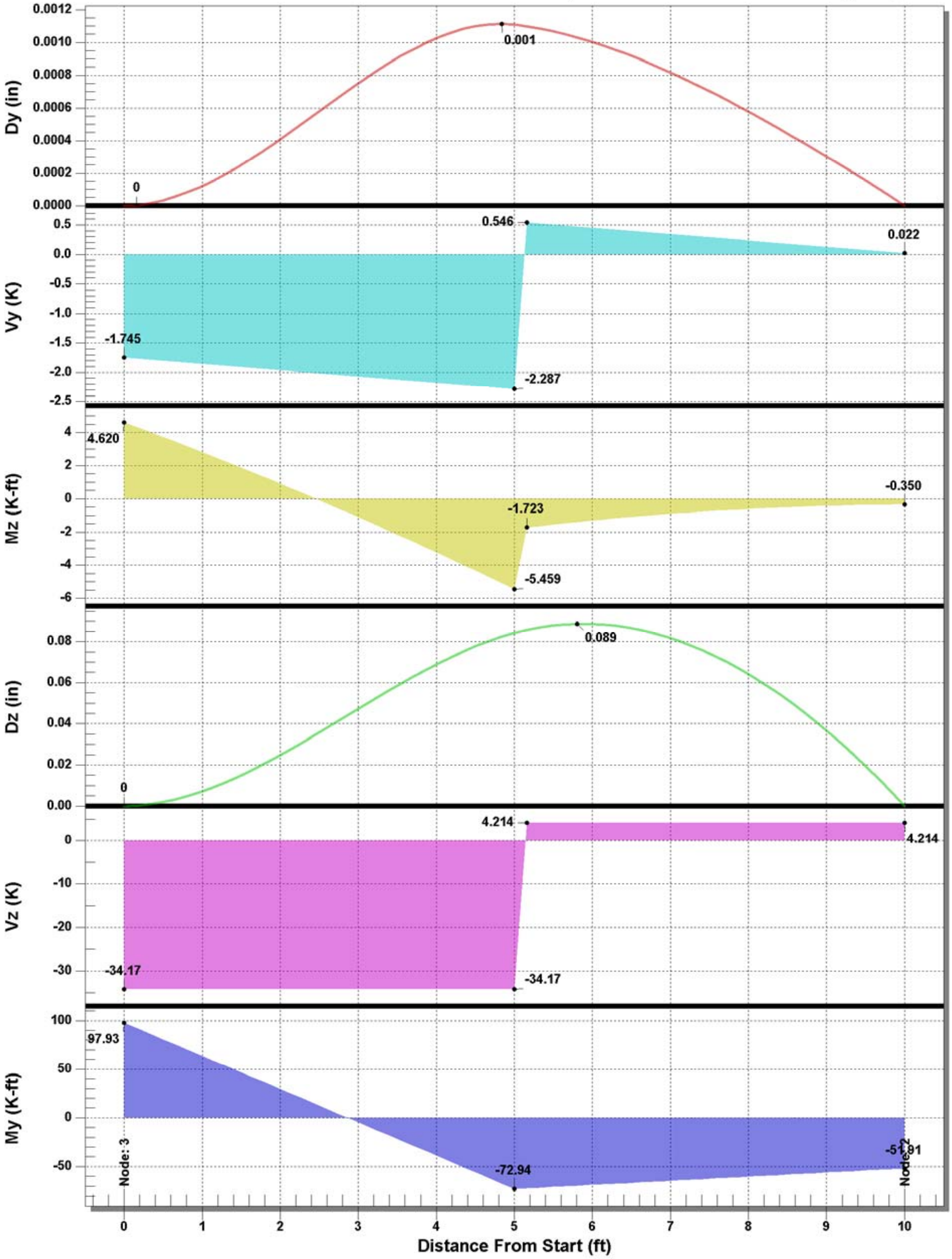
Y
Z
X


P (K)
0.6
0.4
0.2
0.0
-0.2
-0.4
-0.6
0.600
0.600
-0.600
-0.600
Vy (K)
0.0
-0.2
-0.4
-0.6
-0.8
-1.0
-1.2
-0.068
-0.068
-1.228
-1.228
Mz (K-ft)
3
2
1
0
-1
-2
-3
3.041
0.743
0.401
-2.900
Vz (K)
5
0
-5
-10
-15
-20
2.633
2.633
-21.36
-21.36
My (K-ft)
40
20
0
-20
-40
-60
32.44
45.61
-61.21
Node: 4
Node: 3
0
1
2
3
4
5
6
7
8
9
10
Distance From Start (ft)



Y
Z
X

Member: BmX008 | Cap footing | Bryce Harper
Load Case: 1.2D+1.6L+.2Di+.5S | HARPER ENGINEERING, INC. | Mar 03, 2017; 02:21 PM



VisualAnalysis 12.00.0020
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Appendix D

Monopole Foundation Calculations

Nordlund Method

The Nordlund method for soil bearing capacity was used to determine pile parameters for the railyard site foundation. The steps for this method were followed using the FHWA manual on *Design and Construction of Driven Pile Foundations – Volume 1.*

Step 1: Delineate the soil profile into layers and determine φ angle for each layer. Values for the specific weight of each soil layer were estimated from tables found in *Geotechnical Engineering Foundation Design* by John N. Cernica. Friction angles were determined based off of Table 5-5 found in FHWA manual *Design and Construction of Driven Pile Foundations – Volume 1. An effective stress diagram was computed as part of this step and can be seen in* Figure *9.*

Table 8, Specific weight and friction angle values for soil layers at the railyard site

Depth	γ (pcf)	ϕ' (degrees)
8	114.6	
10	114.6	40.074
12	114.6	37.789
14	114.6	31.102
16	111.4	31.081
18	111.4	32.495
20	114.6	41.525
22	114.6	43.415
24	114.6	42.310
26	114.6	42.655
28	114.6	40.570
30	114.6	36.706

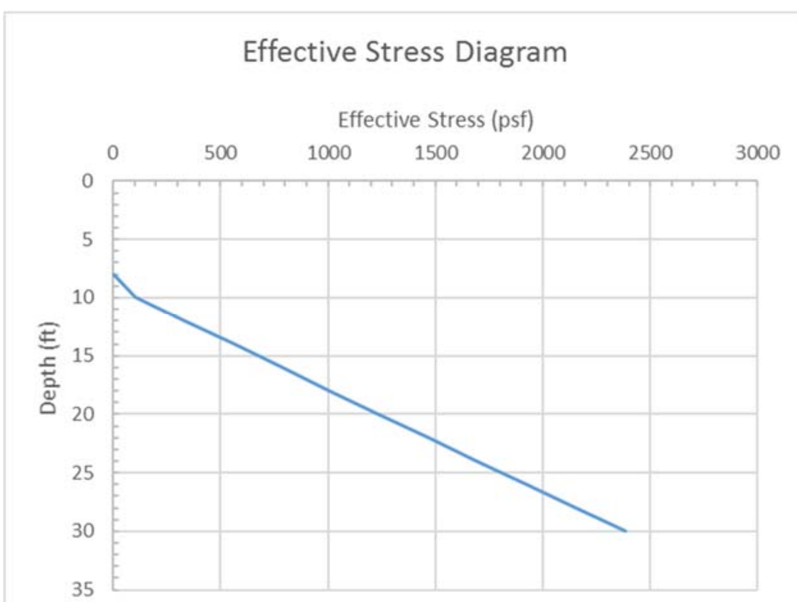


Figure 9, Effective stress diagram for the monopole foundation site

Step 2: Determine Ϭ, the friction angle between pile and soil, based on displaced soil volume and the soil friction angle. The computed value for displaced volume per unit length of the pile was found to be 0.79 ft^3/ft. The volume and ration Ϭ/φ were used in conjunction with Figure 7-9 in the FHWA manual to determine Ϭ for each soil layer.

Table 9, Sigma values from figure 7-9 of the FHWA pile design manual

Depth	Ϭ
8	
10	0.54
12	0.51
14	0.42
16	0.42
18	0.44
20	0.56
22	0.59
24	0.57
26	0.58
28	0.55
30	0.50

Step 3: Determine the coefficient of lateral earth pressure, $K_Ϭ$, for each φ angle. Since the displaced unit volume of the selected piles did not correspond to one of the figures in the FHWA manual linear, interpolation was used to determine values for $K_Ϭ$ at our displaced volume and φ angles. Tables 7-6 and 7-7 in the FHWA manual were used.

Step 4: Determine the correction factor, C_F, to be applied to $K_Ϭ$ if Ϭ ≠ φ. Figure 7-14 in the FHWA manual was used to determine the correction factors by entering the φ angle and Ϭ/φ.

Table 10, Correction factors for soil layers of the monopole foundation site

Depth	C_F
8	
10	0.88
12	0.85
14	0.90
16	0.90
18	0.88
20	0.88
22	0.87
24	0.87
26	0.87
28	0.87
30	0.89

Step 5: Compute the average vertical effective stress at the midpoint of each soil layer. Table *11* displays the calculated values from this step.

Step 6: Compute the shaft resistance in each soil layer. Sum the shaft resistance from each soil layer to obtain the nominal shaft resistance R_s (kips). Equation *5* was used to find the values seen in Table *12*.

Table 11, Vertical effective stress for each soil layer at the monopole foundation site

Depth	Vertical Stress (psf)
8	0
10	229.2
12	458.4
14	687.6
16	910.4
18	1133.2
20	1362.4
22	1591.6
24	1820.8
26	2050
28	2279.2
30	2508.4

Equation 5, Shaft resistance

$$R_s = K_\delta \, C_F \, \sigma'_d \sin(\delta) \, C_d \, D$$

Depth	Rs (kips)
8	
10	0.98
12	5.65
14	11.34
16	21.93
18	40.32
20	114.90
22	248.93
24	369.43
26	524.43
28	708.72
30	848.40

Table 12, Shaft resistance for soil layers at the monopole foundation site

Step 7: Determine the α_t coefficient and the bearing capacity factor, N'_q, from the φ angle near the pile toe. Figure 7-16(a) and (b) were used with the φ angle near the pile toe to determine α_t and N_q. The value of α_t was found to be 0.8 and the value of N'_q was found to be 115.

Step 8: Compute the vertical effective stress at the pile toe, σ'_p (ksf). The value of vertical effective stress at the pile toe was found to be 2.2 ksf.
Step 9: Compute the nominal toe resistance, R_p (kips). Equation *6* was used to find the nominal toe resistance. The value of R_p was found to be 155.7 kips at 28 feet deep.

Equation 6, Nominal toe resistance

$$R_p = \alpha_t N'_t A_t \sigma'_p$$

Equation 7, Sum of shaft and toe resistances

$$R_n = R_s + R_p.$$

Step 10: Compute the nominal resistance, R_n, from the sum of the shaft and toe resistances. Equation *7* was used to determine the value for R_n. The value of R_n at 110 feet deep was found to be 35,166 kips.

Based on these calculations and lateral displacement and vertical settlement computations a pile array of 36 1ft wide prestressed concrete piles was designed. Lateral displacement analysis was performed on the designed pile group using the Ensoft program GROUP. Lateral displacements were found to be within prescribed tolerances. The following charts are plots from the GROUP program.

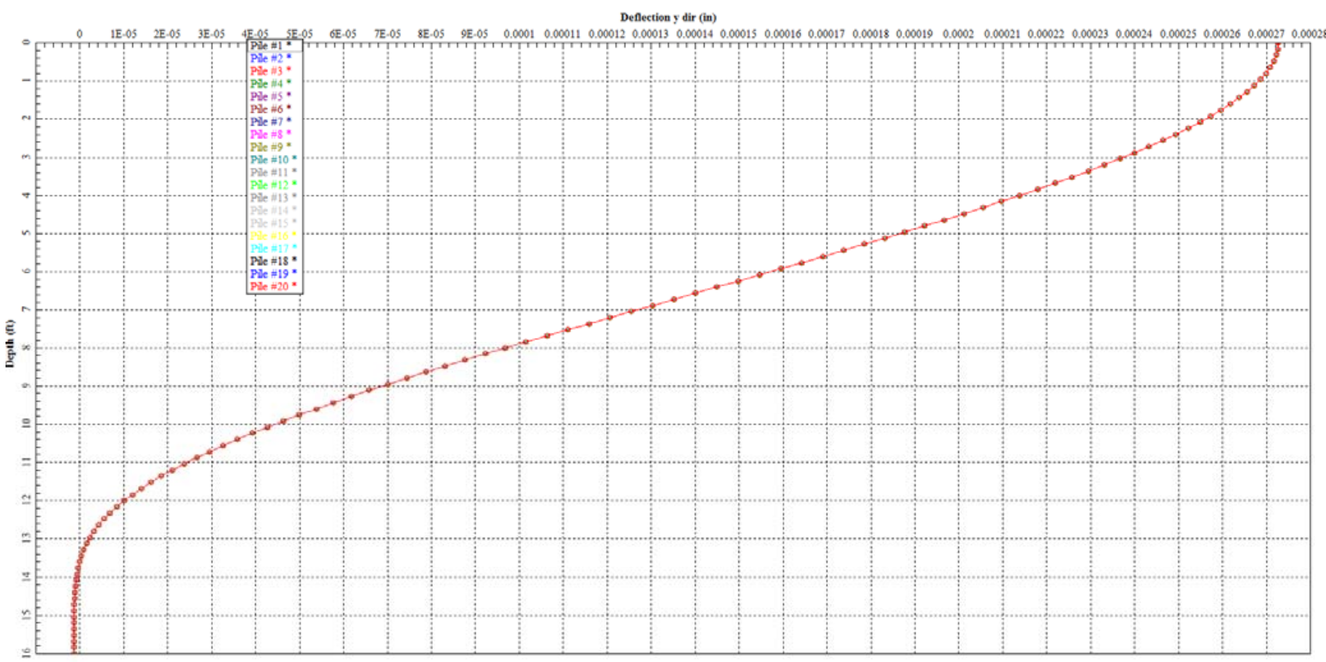


Figure 10, Deflection in y direction (in).

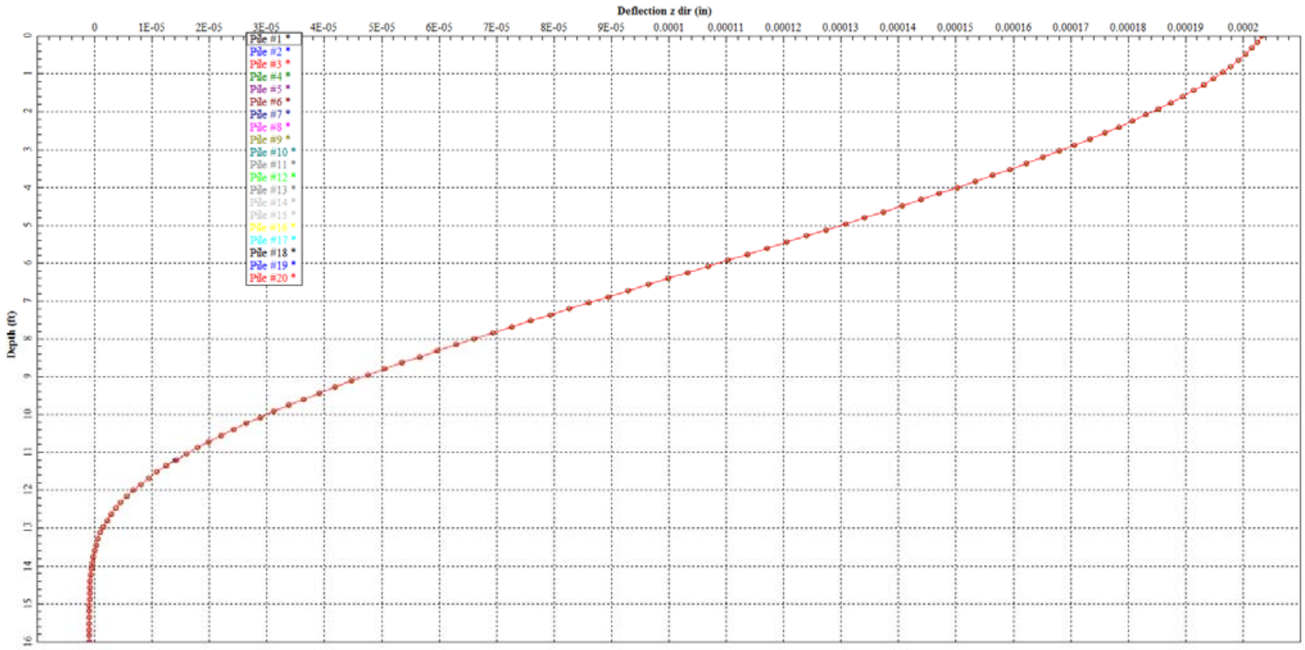


Figure 11, Deflection in the z direction (in).

Vertical settlement was computed in accordance with the Army Corps of Engineers manual, *Design of Pile Foundations* Engineer Manual 1110-2-2906.
The following table displays the values used to compute vertical settlement of the pile group. Vertical settlement was found to be 0.79 inches. Equations were obtained on pages 4-22 through 4-26.

Railyard	w_s	0.007937	ft	w_{pp}	0.002628	ft	w_{ps}	0.002628	w	0.158324	ζ_g	5	S = 0.791618	(in)
	Qp	258.3	kips	C_p	0.02						Bbar	25		
	α_s	0.67		B	1	ft								
	Q_s	248.93	kips	q	172.2	kips								
	L	12	ft	C_s	0.029213									
	A	1	ft^2	D	11	ft								
	E	642693	ksf											

Appendix E

Monopole Pile Cap Design Calculations

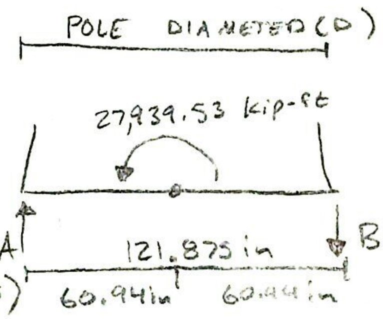
DESIGN LOADS FOR MONOPOLE

- 226.6 kip vertical load
- 224.51 kip SHEAR
- 27,939.53 kip-ft MOMENT $\approx$ 28,000 kip-ft

- ASSUME ANCHOR BOLT DESIGN GIVEN IS SUFFICIENT TO TRANSFER LOAD INTO FOUNDATION.
- CHECK PUNCHING SHEAR OF MONOPOLE ON PILECAP.

POLE DIAMETER (D) = 121.875 in

POLE CIRCUMFERENCE (C) = 382.88 in



– FIND FORCE (F)

$F = 226.6\text{ kip} + B$

$\Sigma M_A = 0 = 27939.53 - B(121.875)$

$\therefore B = 229.25\text{ kip}$

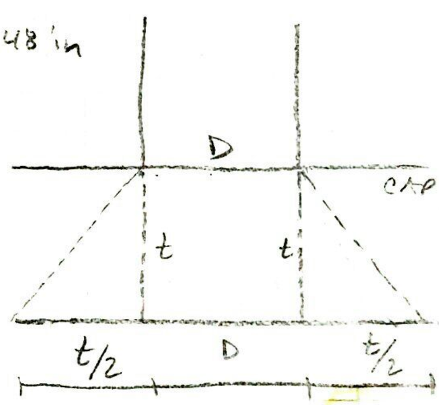
$F = 226.6\text{ kip} + 229.25\text{ kip}$

$F = 455.85\text{ kip}$ $\quad f'c = 4000\text{ psi} \quad \phi = 0.75$

PILECAP THICKNESS $(t) = 4\text{ ft} = 48\text{ in}$

$b_o = (D + t)\pi$

$b_o = (121.875 + 48)\pi = 533.7\text{ in}$



– DEPTH OF PUNCHING SHEAR (d)

G

$$d = \frac{F}{\phi\, 4\sqrt{f'c}\, b_o} = \frac{455{,}850\text{ lbs}}{(0.75)\, 4\sqrt{4000}\,(533.7)}$$

$d = 4.5\text{ in}$

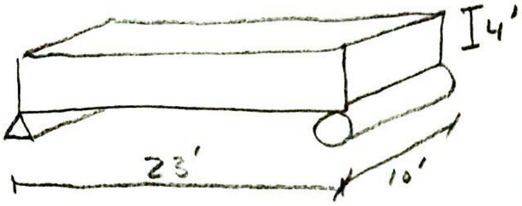
$d < t$ $\quad$ ok ✓ $\quad$ – NO PUNCHING SHEAR

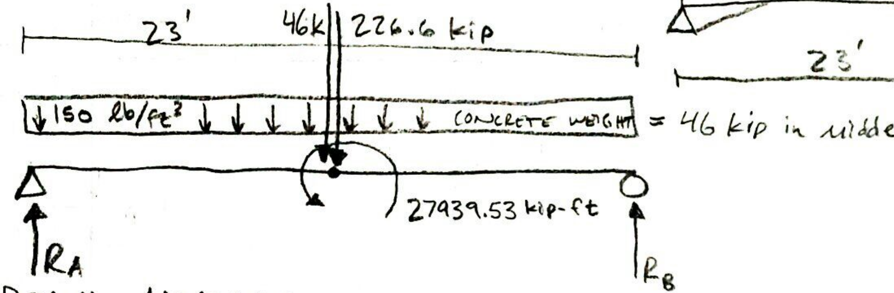
$4.5 < 48\text{ in}$

MONOPOLE FLEXURE

- ACTUAL PILECAP DIMENSIONS ARE 21 ft X 21 ft X 4 ft
- CONSERVITIVE PILECAP DIMENSIONS ARE 23 ft X 23 ft X 4 ft (CONSERITIVE)
- INTERNAL FORCES OF PILECAP DETERMINED BY DIVIDING UP INTO two SIMPLE BEAMS WHERE EACH BEAM IS ASSUMED TO CARRY 65% OF LOAD.

▶ SIMPLE BEAM SHAPE:





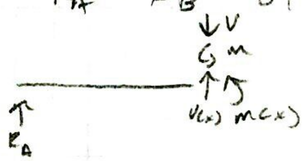
▶ BEAM ANALYSIS

$\Sigma M_A = 0$

$0 = 27939.53\,Kft - 270.6\,kip(11.5\,ft) - R_B(23\,ft)$

$R_B = 1078.5\,kip$

$R_A = R_B$ BY SYMMETRY



$\Sigma M_{CUT} = 0$

$0 = 27939.53\,Kft - m(x) - 1078.5\,kip(11.5\,ft)$

$M(x) = 15537.215\,kip\,ft$

▶ ASSUME EACH BEAM HOLDS 65% OF M(x) BASED ON THE MULTIPLE "BEAMS" IT TAKES TO DESIGN THAT PILE CAP.

▶ MAX Z MOMENT

B $R_N = \dfrac{M_u}{\phi b d^2}$ $\quad b = 10\,ft = 120\,in$
$\quad d = depth - 6\,in = 3.5\,ft = 42\,in$

$$R_N = \frac{.65(15537.215\,kip\,ft)(12\,in/ft)(1000\,lb/kip)}{(.9)(120\,in)(42\,in)^2}$$

$R_N = 636.13\,psi$

▶ PERCENTAGE OF STEEL REQUIRED

(C) $\rho = \frac{.85 f'_c}{f_y}\left(1 - \sqrt{1 - \frac{2R_N}{.85 f'_c}}\right)$

$\rho = \frac{.85(4000psi)}{(60,000\ psi)}\left(1 - \sqrt{1 - \frac{2(636.13psi)}{.85(4000psi)}}\right)$

$\rho = .012$

▶ AREA OF STEEL REQUIRED

(A) $A_s = \rho bd$

$A_s = .012(120in)(42in)$

$A_s = 59.7\ in^2$

▶ CHECK FOR YIELDING OF STEEL

(D) $a = \frac{A_s f_y}{.85 f'_c b} = \frac{59.7in^2(60,000psi)}{.85(4000psi)(120in)}$

$a = 8.77\ in$

(E) $C = \frac{a}{.85} = \frac{8.77}{.85}$

$C = 10.32\ in$

(F) $\varepsilon'_s = \frac{C - 6in}{C}(.003)$

$\varepsilon'_s = \frac{10.32 - 6}{10.32}(.003)$

$\varepsilon'_s = .00125$

$\varepsilon_y = \frac{60}{29,000} = .00207$

$\varepsilon'_s < \varepsilon_y$

THEREFORE THE STEEL WILL NOT YIELD

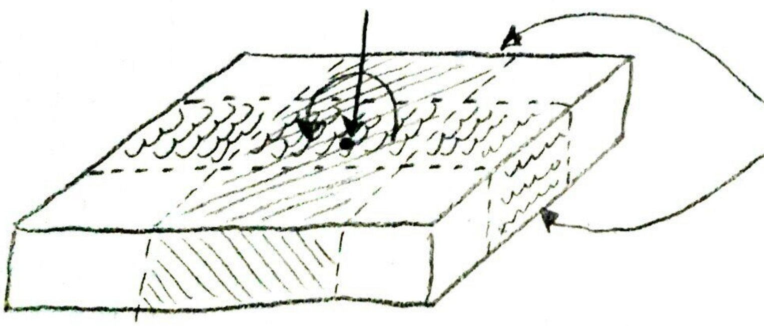
▶ DETERMINING TYPE AND AMOUNT OF REBAR

#10 REBAR: $A_g = 1.27\ in^2$
$D = 1.27$

$N = \frac{A_s}{A_g} = \frac{59.7\ in^2}{1.27} = 47.0$ BARS

$\boxed{\text{USE 47 BARS OF \#10 REBAR}}$

THIS BEAM LAYOUT WILL BE AS SHOWN, BUT REBAR TYPICAL THROUOUT PILE.



BEAMS LOCATED ON PILE CAP

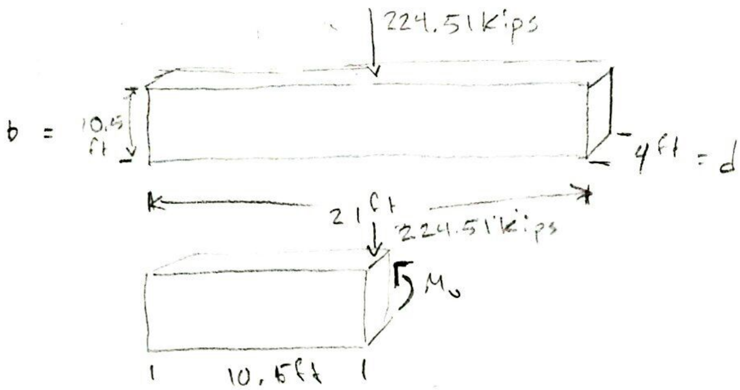
3.5

MONOPOLE PILE CAP

MAX X MOMENT FOR FLEXURE

(B) $R_N = \dfrac{M_u}{\phi b d^2}$

b = WIDTH OF MEMBER
d = DEPTH OF PILE MINUS 6 in FOR REBAR COVER



$M_u = 224.51\text{kips} \cdot 10.5\text{ ft}$
$M_u = 2357.36$ ← MAX MOMENT

$$R_N = \frac{(2357.36\text{ k-ft})(12\text{ in/ft})(1000\text{ lb/kip})}{(0.9)(42\text{ inches})^2(126\text{ inches})} = 141.4\text{ psi}$$

PERCENTAGE OF STEEL REQUIRED

(C) $$p = \frac{.85 f'_c}{f_y}\left(1 - \sqrt{1 - \frac{2R_n}{.85 f'_c}}\right)$$

$$p = \frac{.85(4000\text{ psi})}{60{,}000\text{ psi}}\left(1 - \sqrt{1 - \frac{2(141.4)}{.85(4000)}}\right)$$

$p = .0024$

AREA OF STEEL REQUIRED

(A) $A_s = pbd$
$A_s = .0024(42)(126)$
$A_s = 12.7\text{ in}^2$

CHECK YIELDING OF STEEL

(D) $$a = \frac{A_s f_y}{.85 f'_c b} = \frac{12.7 \cdot 60{,}000}{.85(4000)(126\text{ in})}$$

$a = 1.7787\text{ in} = 1.78\text{ in}$

(E) $c = \dfrac{a}{.85} = \dfrac{1.78}{.85} = 2.09\text{ in}$

(F) $\varepsilon'_y = \dfrac{c - d'}{c}(.003) = 2.09 -$

$\varepsilon'_y = \dfrac{2.09 - 6}{2.09}(.003) = -.0056$

DETERMINING TYPE & AMOUNT OF REBAR

#10 REBAR = $A_g = 1.27\text{ in}^2$
D = 1.27 in

$N = \dfrac{A_s}{A_g} = \dfrac{12.7\text{ in}^2}{1.27} = 10.03$

$\boxed{\text{USE 10 \#10 REBAR}}$

→ Assume 6 in Cover of Rebar

$\varepsilon_y = \dfrac{60\text{ ksi}}{29{,}000\text{ ksi}} = .00207$

$\varepsilon'_s < \varepsilon_y$ THEREFORE STEEL WILL NOT YIELD

- REBAR STACKING AND SPACING DESIGN IS DONE USING AUTO-CAD FOR OPTIMUM FIT AND AGGREGATE PENETRATION.

- MONOPOLE ORIENTATION ON PILE CAP IS 45° OF GIVEN ORIENTATION TO MAXIMIZE REBAR LAYOUT
  (PILE CAP MAY BE ORIENTED TO MEET POLE'S NEEDS)

★ ASSUME NOT MUCH DIFFERENCE IN DEVELOPMENT LENGTH FOR STACKED REBAR — BUT WILL ANALYZE STACKED #10 REBAR AS #11 REBAR

- FIND DEVELOPMENT LENGTH OF #10 REBAR [EFFECTIVELY #11]

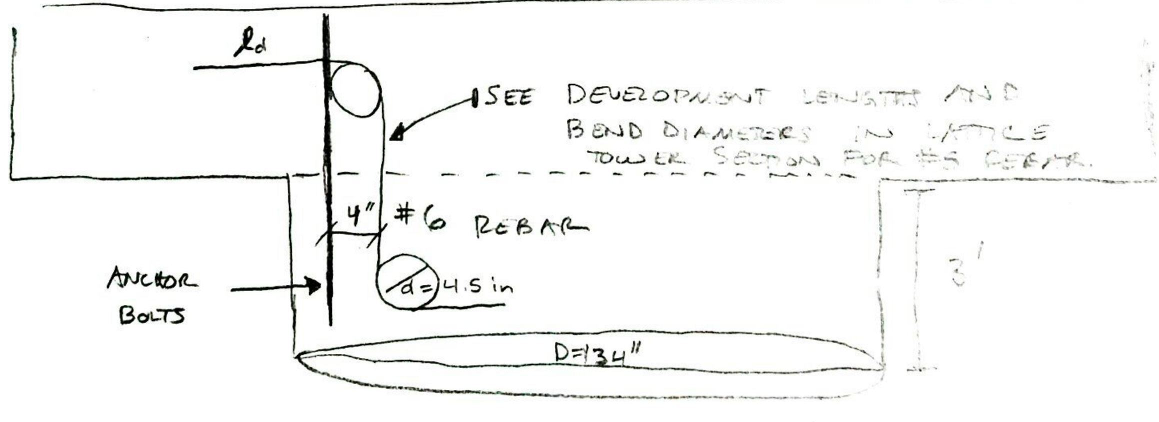
#11 BAR: $A_g = 1.56\ in^2$ $\quad d_{11} = 1.41\ in$ $\quad \psi_e = 1$ $\quad F_y = 60\ ksi$ $\quad F_a = .7$ $\quad f'_c = 4\ ksi$ $\quad \lambda = 1$ [TABLE 8-2]

L $\quad \ell dh = \left(\dfrac{0.02\ \psi_e\ F_y}{\lambda \sqrt{f'c}}\right) d_{11}\ F_a$

$$\ell dh = \left(\frac{0.02(1)(60{,}000)}{1\sqrt{4000}}\right)(1.41)(.7) = 18.7\ in \approx 19\ in$$

- BECAUSE DEVELOPMENT LENGTH IS 19 in, EXTEND REBAR AT LEAST 19 INCHES PAST PILE PERIMETER

- CAGE DESIGN FOR EXTRA 3 FEET OF CONCRETE DIRECTLY UNDER MONOPOLE CONNECTION.



$$P_u = F_{CONCRETE} + F_{STEEL} + 226.6\ kip$$

$$F_{CONCRETE} = \left(\frac{h\pi D^2}{4}\right)\left(150\ \frac{lb}{ft^3}\right) = \left(\frac{(3ft)(\pi)(134\ in)}{4}\right) 150\ \frac{lb}{ft^3} = 40.1\ kip$$

F_{STEEL} = ASSUME 6 KIP OF REBAR AND 4 KIPS FOR ANCHOR BOLTS

$$P_u = 40.1\ kip + 10\ kip + 226.6\ kip = 280.7\ kip$$

- FIND A_s NEEDED TO HOLD 3 FT SECTION

(M) $\phi P_u = \phi A_s F_Y$ $\qquad F_Y = 60$ ksi

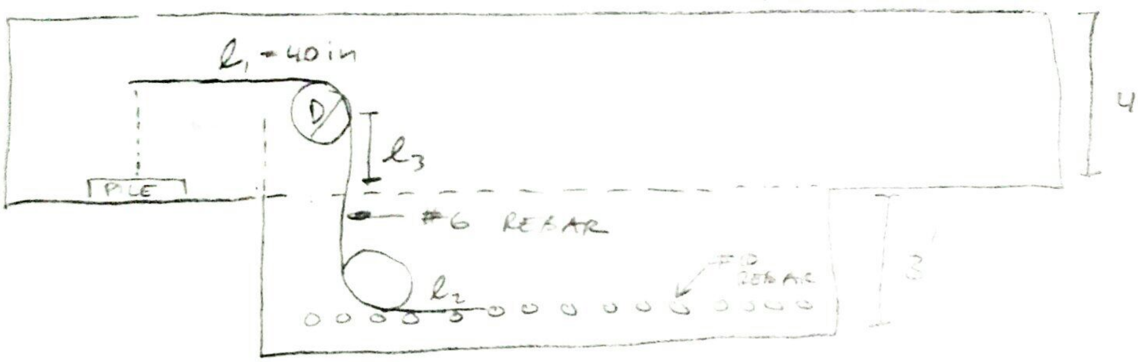
$$P_u = \phi A_s F_Y$$

$$A_s = \frac{P_u}{\phi F_Y} = \frac{280.7 \text{ kip}}{(.75)(60)} = 6.23 \text{ in}^2$$

- FIND NUMBER OF #5 REBAR FOR VERTICAL BARS

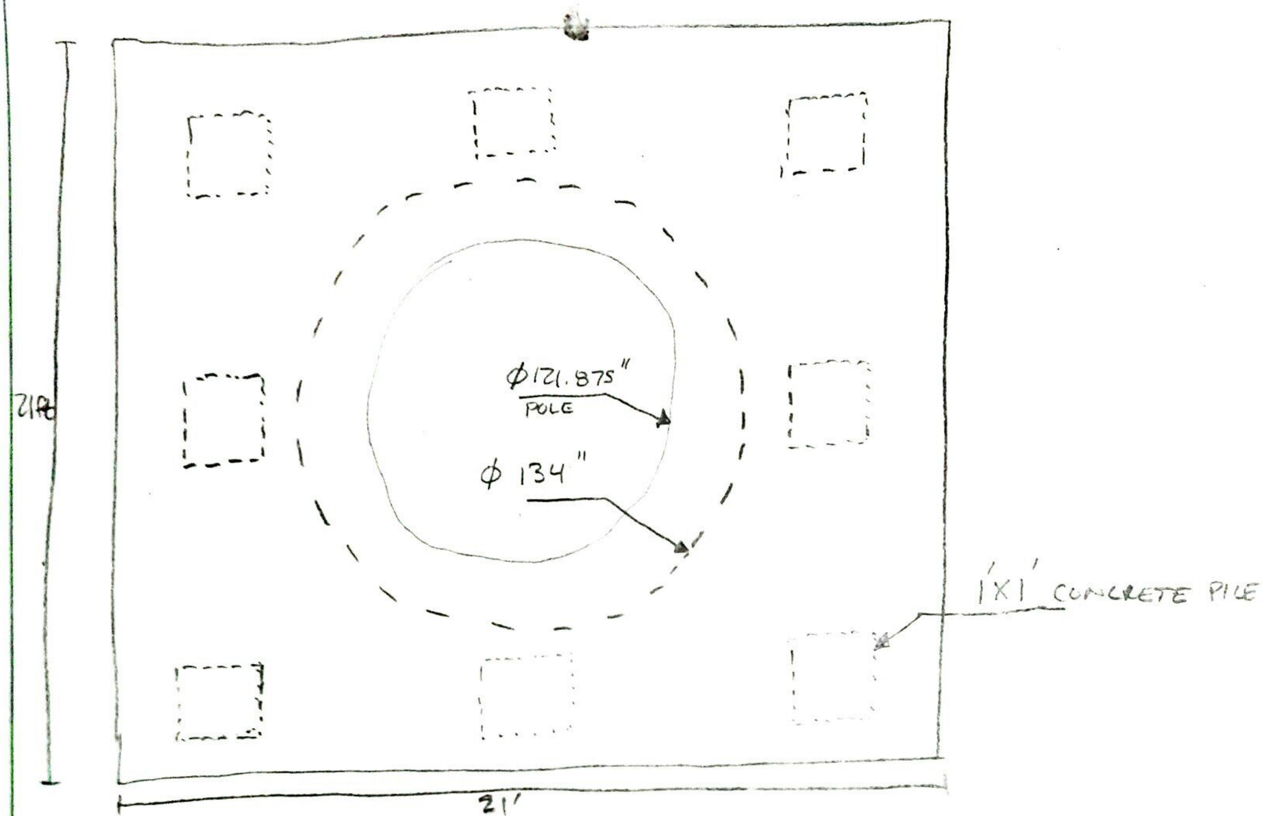
$$N = \frac{A_s}{A_g} = \frac{6.23 \text{ in}^2}{.44 \text{ in}^2} = 14.2 \text{ bars} \approx 15 \text{ BARS} \rightarrow 16 \text{ BARS FOR CONSTRUTIBILITY}$$

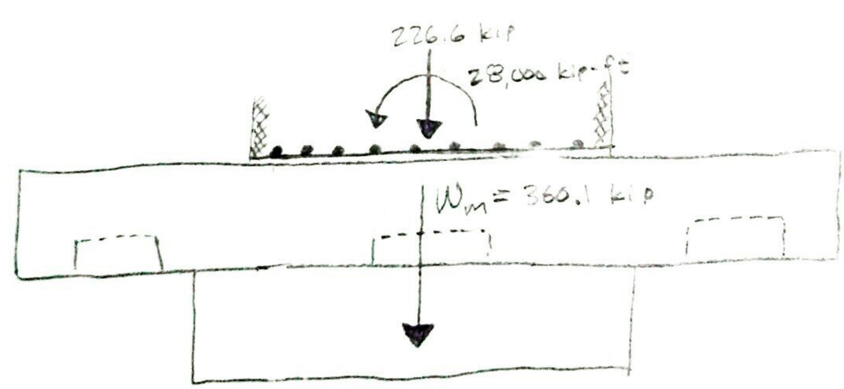
- PLACE HORIZONTAL TIE BAR AROUND ARRAY OF VERTICAL BARS AT LEAST EVERY 18 in.



- l_1 is 10 inches ACCORDING TO MINIMUM. FOR THIS CAP, EXTEND TO 40 INCHES TO HELP RESIST CRACKING AND EXTEND TO PILE
- l_3 is 9 inches ACCORDING TO MINIMUM. FOR THIS CAP, EXTEND l_3 TO 22 INCHES SO THAT REBAR IS NOT EFFECTING MOMENT REBAR IN CAP.
- l_2 MAY EQUAL 10 INCHES AS NORMAL AND CAN BE TIED TO #10 REBAR AT THE BASE

▸ FINDING LOAD EXERTED ONTO PILES





▸ WEIGHT OF MATERIALS (W_m) -

$\gamma_m = 175$ pcf (concrete & steel)

$W_{BOLTS} = 6406$ lbs

$$W_m = W_{CAP} + W_{BOLTS}$$

$$W_{CAP} = [V_{C1} + V_{C2}]\,\gamma_m$$

$$V_{C1} = (4\text{ ft})(21\text{ ft})(21\text{ ft}) = 1764\text{ ft}^3$$

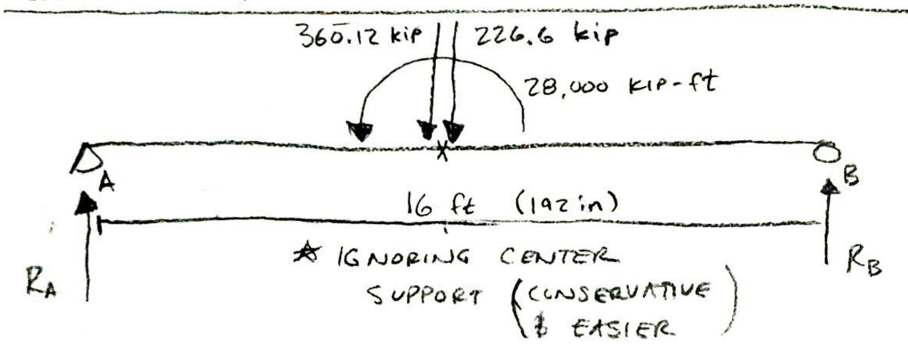
$$V_{C2} = \frac{(3\text{ ft})\,\pi\,(134\text{ in})^2}{4} = 293.8\text{ ft}^3$$

$$W_{CAP} = \left[1764\text{ ft}^3 + 293.8\text{ ft}^3\right] 175\text{ pcf} = 360.12\text{ kip}$$

7/

$W_m = 360.12\ \text{kip} + 6406\ \text{lbs} = 360.12\ \text{kip}$

- DETERMINING FORCES ON PILES AS IF SIMPLE BEAM



$\Sigma M_A = 0 = (28{,}000\ \text{kip-ft}) - (360.12\ \text{kip} + 226.6\ \text{kip})\frac{16\ \text{ft}}{2} + R_B(16\ \text{ft})$

$R_B = -1456.6\ \text{kip}$

$\Sigma M_B = 0 = (28000\ \text{kip-ft}) + (360.12\ \text{kip} + 226.6\ \text{kip})\frac{16\ \text{ft}}{2} - R_A(16\ \text{ft})$

$R_A = 2043.36\ \text{kip}$

- DIVIDING R_A AND R_B BETWEEN 3 PILES

$P_+ = \frac{R_A}{3} = \frac{2043.36\ \text{kip}}{3} = 681.12\ \text{kip/pile}$ (DOWN FORCE)

$P_- = \frac{R_B}{3} = \frac{-1456.6\ \text{kip}}{3} = -485.53\ \text{kip/pile}$ (UP LIFT)

- FINDING AREA OF CONCRETE NEEDED FOR COMPRESSION FORCE

$P_+ = A_c f'_c \qquad f'_c = 7000\ \text{psi} = 7\ \text{ksi}$

$A_c = \frac{P_+}{f'_c} = \frac{681.12\ \text{kip}}{7\ \text{ksi}} = 97.3\ \text{in}^2 \approx .68\ \text{ft}^2$

- USE 1 ft X 1 ft prestressed concrete pile

- MINIMUM REBAR NEEDED FOR TENSION [IF PRESTRESSED PILE CANNOT HANDLE UPLIFT]

$F_y = 60\ \text{ksi}$

M

$P_- = \phi A_s F_y$

$A_s = \frac{P_-}{\phi F_y} = \frac{485.53\ \text{kip}}{(.9)(60\ \text{ksi})} = 8.99\ \text{in}^2$

8/

- DETERMINE REBAR TYPE AND AMOUNT FOR CONCRETE PILE

$$N = \frac{A_s}{A_g} = \frac{8.99 \text{ in}^2}{1 \text{ in}^2} = 8.99 \text{ bars} \approx 9 \text{ bars}$$

USE #9, $A_g = 1 \text{ in}^2$, $d_9 = 1.128$ in

→ USE 9 # 9 BARS IN CONCRETE PILE IF NOT SUITABLE FOR REQUIRED UPLIFT FORCES.

▶ DETERMINE AMOUNT OF REBAR NEEDED TO ATTACH PILE TO PILE CAP

- USING # 9 REBAR, N = 9 BARS
- EPOXY SET THE REBAR, SPACED EVENLY, ON TOP OF PILE WITH 90° HOOKS.

▶ FINDING DIMENSIONS OF 90° HOOKS

- DEVELOPMENT LENGTH (LENGTH OF TAIL)

L

$$l_{dh} = \left(\frac{0.02\, \psi_e\, F_y}{\lambda \sqrt{f'_c}}\right) d_9 F_a$$

$\psi_e = 1$ $\quad d_9 = 1.128$ in

$F_y = 60$ ksi $\quad A_g = 1 \text{ in}^2$

$f'_c = 4$ ksi $\quad \lambda = 1 \quad F_a = .7$ [TABLE 8-2]

$$l_{dh} = \left(\frac{0.02(1)(60000)}{1\sqrt{4000}}\right)(1.128)(.7)$$

$l_{dh} = 14.98$ in ≈ 15 in

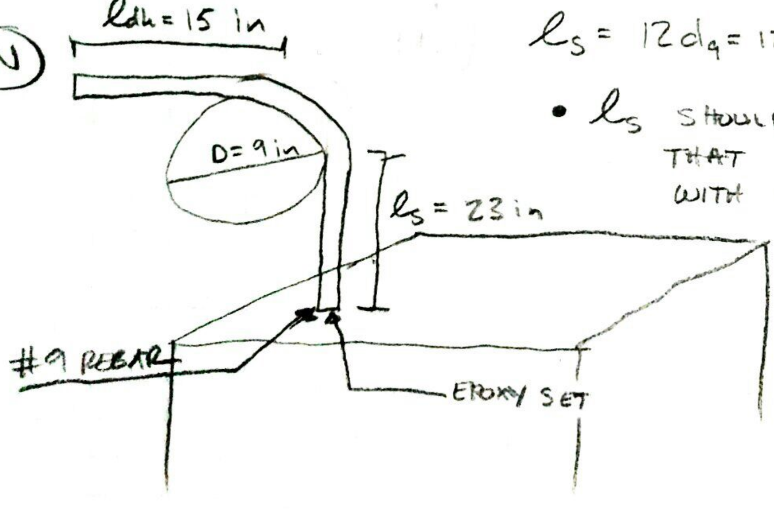
- DIAMETER OF BEND IN HOOK

N

$D = 8 d_9 = 8(1.128 \text{ in}) = 9.0$ in

▶ FINDING LENGTH OF NORMAL SECTIONS (l_s)

N



$l_s = 12 d_9 = 12(1.128) = 13.54$ in

- l_s SHOULD BE 23 INCHES SO THAT REBAR WILL NOT INTERFERE WITH OTHER REINFORCEMENTS AND TRANSFER COMPRESSION OF REBAR PULLOUT OF PILE TO TOP OF PILE CAP.

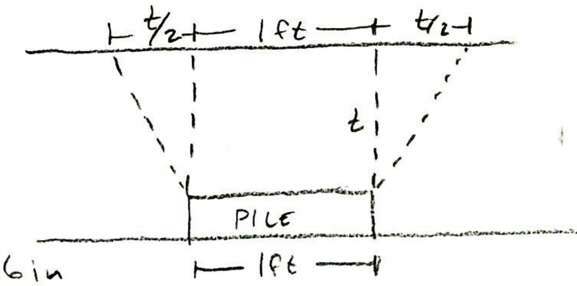
▸ PUNCHING SHEAR OF PILE ON PILE CAP

FOOT PERIMETER (D) = 4(1 ft) = 4 ft = 48 in"

FORCE (F) = P_{-} = 681.12 kip

THICKNESS t = 42 in [PILE RESTS 6 in inside pile cap]

f'_c = 4000 psi $\phi = .9$



– DEPTH OF SHEAR (d)

b_o = PERIMETER OF SHEAR

$b_o = 4(t + 1ft) = 4(42 + 12) = 216\text{ in}$

G

$$d = \frac{F}{\phi 4\sqrt{4000}(b_o)} = \frac{681{,}120\text{ lbs}}{(.9)(4)(\sqrt{4000})(216\text{ in})}$$

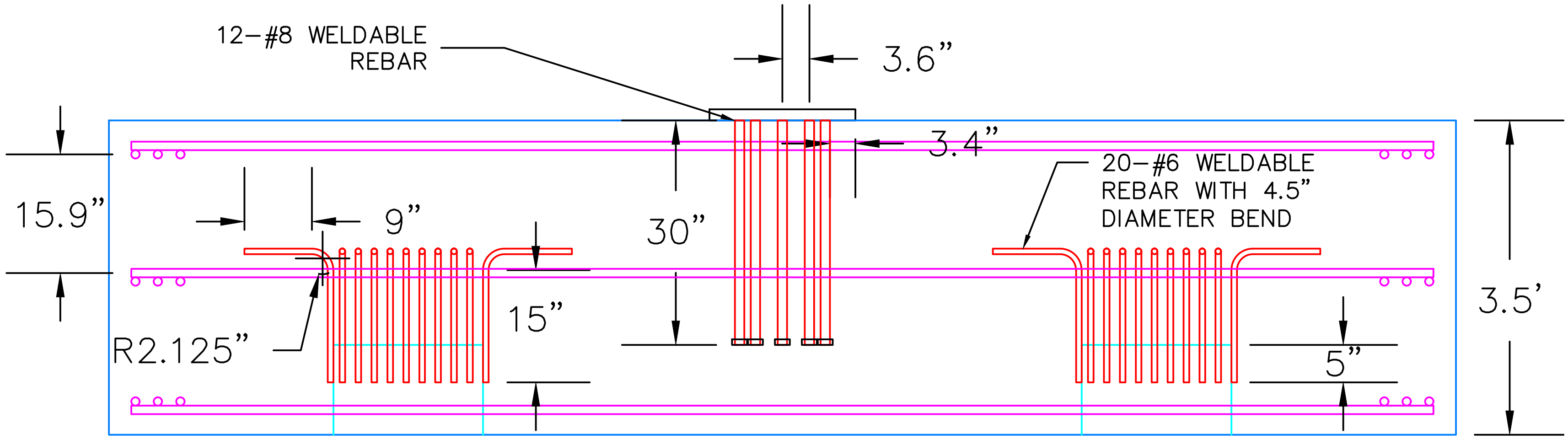
$d = 13.8$ in

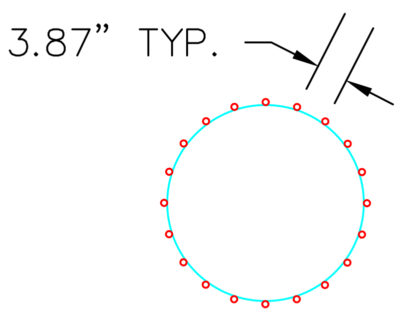
$d < t$

13 in < 42 in ✓ OK, NO PUNCHING SHEAR

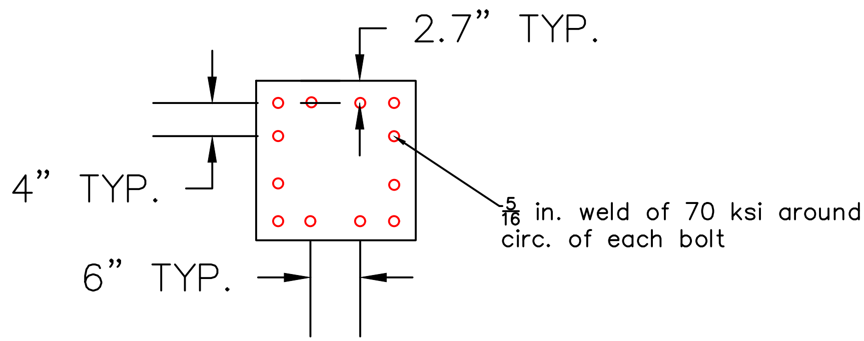
Appendix F

Detail Sheets

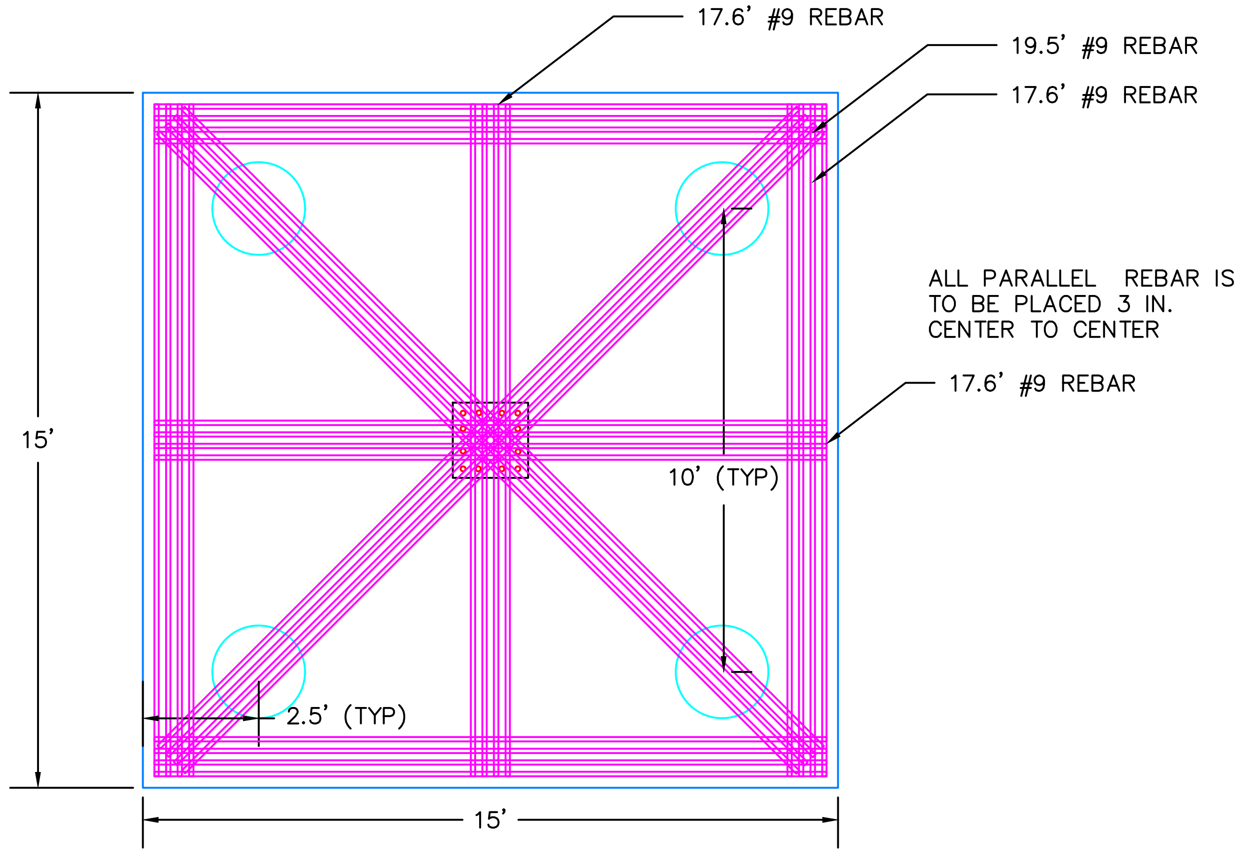




PILE CAP
REBAR DESIGN

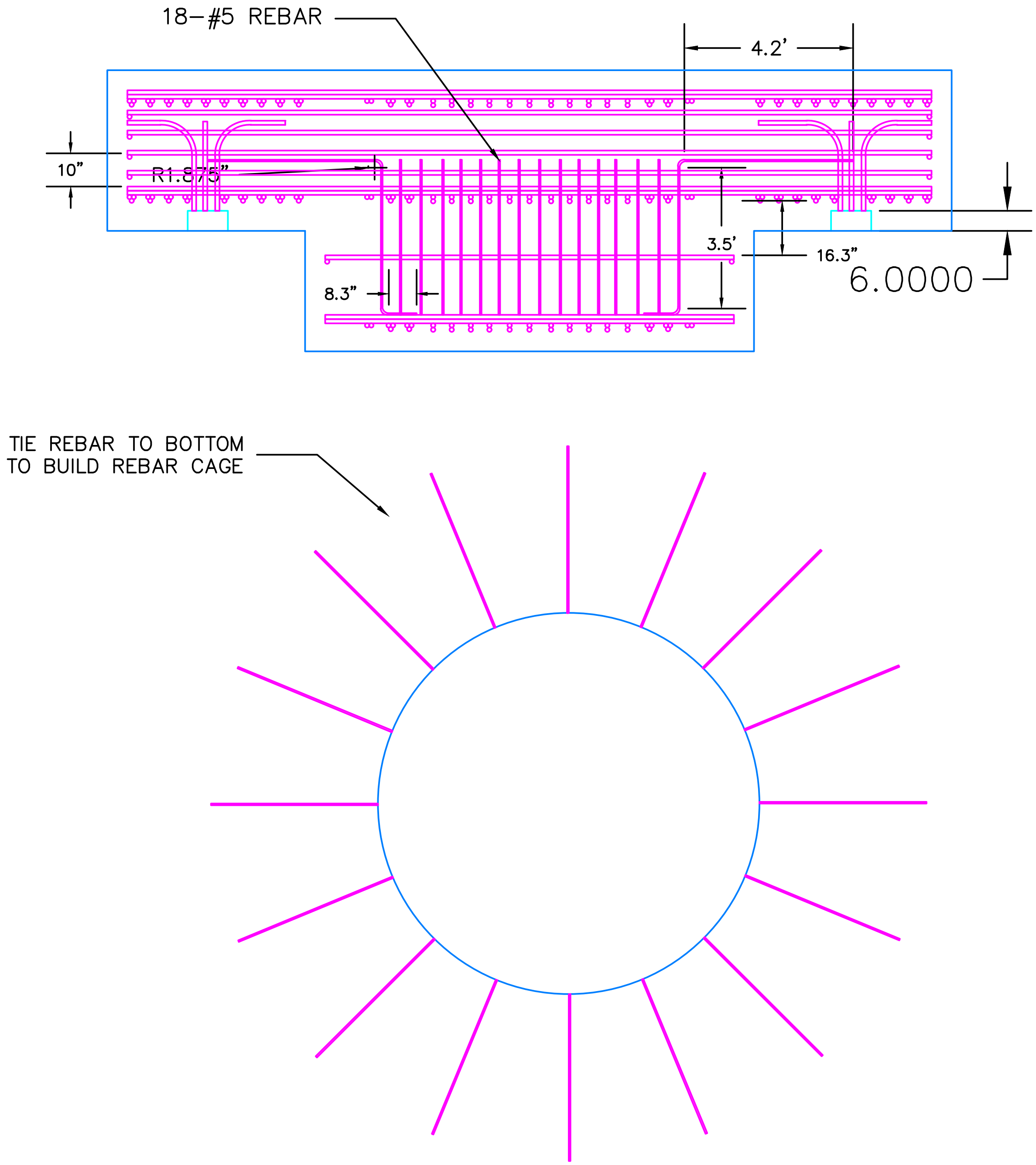


ANCHOR BOLT
DESIGN FOR SHOE

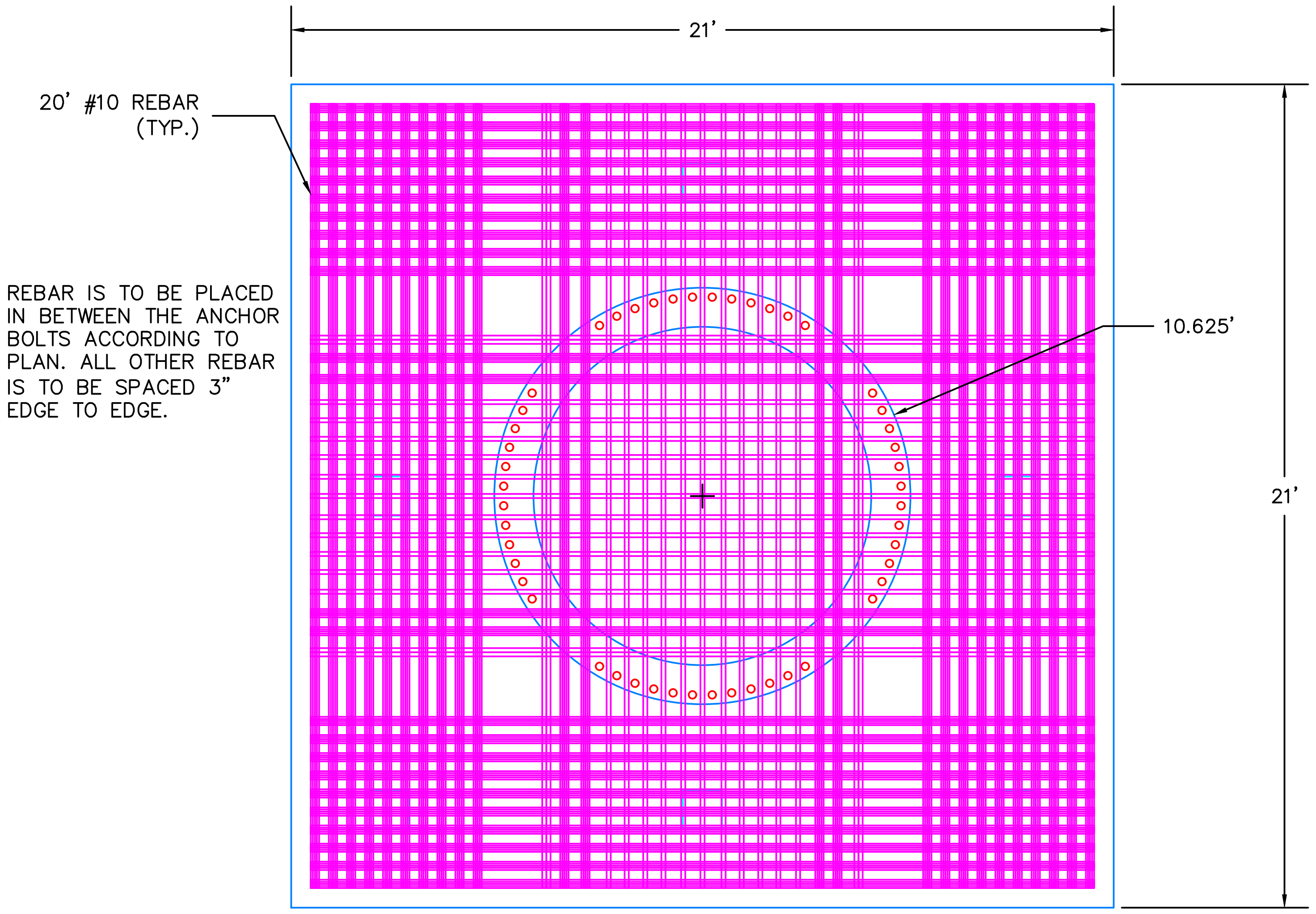


TOP

REBAR LAYOUT



REBAR CAGE
DESIGN



REBAR LAYOUT

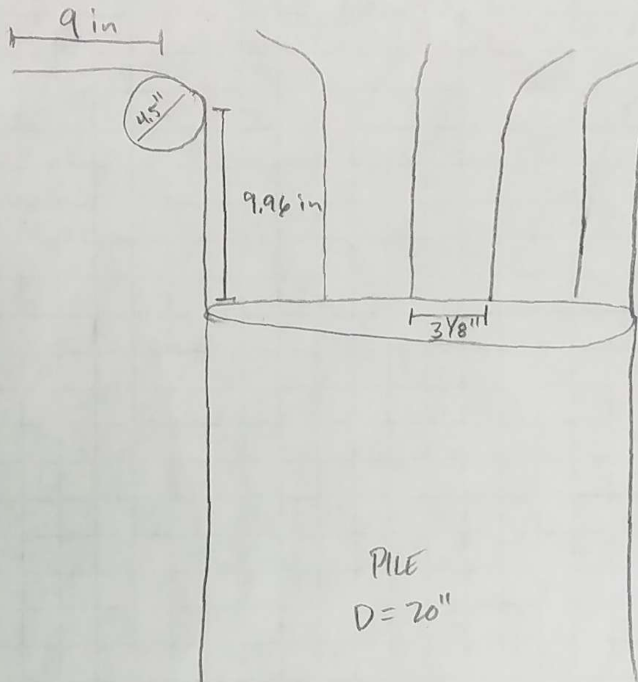
$D = 6d_b$ [FIG 8-12]

$D = (6)(.75)$

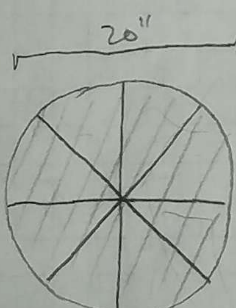
$D = 4.5$ in

$\ell_{tail} = 12d_b$ [ACI CODE]

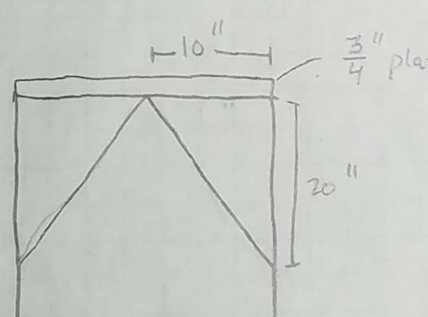
$\ell_{tail} = 12(.75) = 9$ in



WE THOUGH THAT PLACING A 3/4" plate on top of ribs would help transfer loads from the pilecap into the pile — as shown below. TACK WELD THE PLATE TO THE TOP OF THE PILE.



PILE
PLAN VIEW



PILE
PROFILE VIEW