HIGH-MAST TOWER FOUNDATION

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16. Abstract
High-Mast Tower (HMT) foundations have been traditionally designed and constructed using cast-in-place foundation with anchor bolts that are used to secure the tower to the foundation. This type of design requires a base plate that is welded to the tower shaft. The Nebraska Department of Transportation (NDOT) has recently experienced issues with stresses that this type of design presents at the anchor bolt/foundation or base plate/tower shaft interface. This issue in worst cases may lead to a premature failure due to high-cycle fatigue as one of the towers at Milford, Nebraska that fell down during a winter snow storm event in 2018. This research project objective was to develop an alternative design for HMT foundations with direct embedment of HMT which can eliminate fatigue-prone details associated with the pole-to-base plate connection which is the primary location of failure. First, the literature that includes research from academia and industry, current and proposed state of practice from industry, examples of design specifications and guidelines, and corrosion for buried structures were reviewed. Secondly, structural loads for the typical 120- and 140-ft HMTs constructed in Nebraska and the soil resistance for them were calculated. The structural loads were computed using the AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, with a spreadsheet based on the fundamental principles of structural analysis. The geotechnical foundation resistance calculations were made to check the vertical and horizontal soil capacity for the typical HMTs used in Nebraska. In addition, further parametric study was conducted using two numerical software: LPILE and COMSOL for varying soil conditions and foundation systems with different embedment length and backfill diameter for the service level base moment and shear. Required embedment length and backfill diameter are provided as a matrix using the LPILE analysis results. Finally, based on the site considerations and constructability, a draft design and construction specification for soil parameters that can be used for Nebraska soil conditions are provided.

17. Key Words
high-mast tower; steel pole direct embedment; direct embedment foundation; backfill; soil-structure interaction

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TABLE OF CONTENTS

Page

LIST OF TABLES

LIST OF FIGURES

Figure Page

1. INTRODUCTION

1.1 Background

High-Mast Tower (HMT) foundations have been traditionally designed and constructed using cast-in-place foundation with anchor bolts that are used to secure the tower to the foundation. This type of design requires a base plate that is welded to the tower shaft as shown in Figure 1.1(a).

The Nebraska Department of Transportation (NDOT) has recently experienced issues with stresses that this type of design presents at the anchor bolt/foundation or base plate/tower shaft interface. This issue in worst cases may lead to a premature failure due to high-cycle fatigue as shown in one of the towers at Milford, Nebraska that fell down during a winter snow storm event in 2018.

(a) High Mast Tower Base Plate, Anchor, Non-shrink Grout, and Cast-in-Place Foundation

(b) High Mast Tower Failure (Milford, Nebraska, January, 2018)

Figure 1.1: High Mast Lighting Tower in Milford, Nebraska (photo provided by NDOT)

There have been several research efforts in the past decade to evaluate the fatigue behavior of these HMTs (Thompson 2011, Connor et al. 2012) to propose retrofit strategies that could reduce wind-induced vibrations observed in these structures (Ahearn and Puckett, 2010). Goode and van de Lindt (2007) developed a reliability-based design procedure for High Mast Lighting Towers while Connor and Hodgson (2006) conducted field instrumentation and conducted pluck tests on these structures to measure the dynamic characteristics.

While most of these previous studies have focused on the 100-120 ft tall structures, there are limited or no research conducted for the substructure related to these towers. All research whether related to load effects, mitigation of vibrations, and/or resistance of the pole-to-baseplate connection always include the connection to a plate that is bolts for the foundation.

NCHRP Report 176 (2011) is one of the several studied that has examined the fatigue resistance of typical pole-to-baseplate connections. The resistance is dependent upon the connection geometry, relative tube-wall thickness to baseplate thickness, and specific of whether the pole is attached via fillet welds or full penetration welds. Although weld details have been significantly improved with research and associated specification updates, the joint remains a potential weakness in the overall performance.

This research suggests that this connection may be completely eliminated thereby obviating the need for the design of fatigue prone weldments. The joint is removed by directly embedding the pole into the foundation. This approach is novel for the transportation industry and could become an exceptional strategy for NDOT as well as other owners confronting failures in their high-mast luminaire poles.

1.2 Research Objective

This research project aims to develop an alternative design for HMT foundations which can eliminate fatigue-prone details associated with the pole-to-base plate connection which is the primary location of failure. To address critical issues, the objectives are to:

1. Evaluate the various types of foundations used in other structures that are similar in height and shape to the HMTs. This includes evaluating drilled shafts and direct embedment foundations for Power Transmission Line Structures,

2. Evaluate the corrosive environment with steel pole structure being embedded either in soil or concrete and propose mitigation measures for any corrosion issues, and based on these findings,

3. Provide design and construction provisions that may integrate into NDOT specifications for design and construction.

1.3 Research Scope

Three tasks support the objectives:

1. Review the literature review on foundations for structures that are similar in size and shape with the high-mast towers used in Nebraska. One good example is the high-tension power transmission line structures that use various types of foundations. Literature review addressed single steel-shaft poles with direct embedment or drilled-shaft foundations. Construction methods and corrosion protection strategies were reviewed. NDOT was surveyed with respect to any problems associated with their present drilled shafts. The shafts appear to be performing well. Note the shafts involved with the new configuration experiences the same loads as any new

configuration. Although the latter might consider concrete or aggregate backfill with in the foundation, presently only concrete is used.

2. Review site-specific corrosion investigations to evaluate the corrosivity of the soil and provide mitigation strategies. This is possible through investigating the resistivity, pH, chloride, sulfate, and oxygen content of the soils and water at potential sites or in the concrete as indicated in the Caltrans (2018) report. Some NDOT districts have corrosive soil, and because the foundations for these HMTs are close to the surface where oxygen content may be higher, protective measures may be required. The web soil survey system provided by the USDA Natural Resources Conservation Service (NRCS, 2018) has soil maps and data available online for Nebraska counties and can be used as a source for soil survey information. The most typical protection for steel poles embedded in soil in a corrosive environment may be achieved by providing by: 1) cathodic protection, 2) protective coatings (epoxy, bituminous coating, etc.), 3) increasing cross-section area of steel, or 4) adding a reinforced concrete jacket.
3. The final task includes use of the outcomes from the first two tasks and integrate them into design and construction recommendations. All poles are assumed to be galvanized. The recommendations informed the integration of materials into the NDOT Inspection Guide for Installation of High-Mast Lighting and Sign Structures (2008) manual and the NDOT Standard Specifications for Highway Construction (2017) that can be applied statewide.

2. LITERATURE REVIEW

2.1 Introduction

This chapter provides a literature review focusing on steel poles with direct embedment foundations or drilled shafts: 1) Academic and industry research results, 2) Drawings from the industry practice, and 3) Design specifications, guidelines, or research reports published from technical committees or societies were reviewed.

2.1 Research Literature

2.1.1 EPRI Report EL-6309 – Direct Embedment Foundation Research (1989)

This report delivers research results of numerical analysis and field load testing conducted by the Electric Power Research Institute (EPRI) and the Empire State Electric Energy Research Corporation (ESEERCO). The motivation of this study starts from introducing the "10 percent plus 2 feet" rule-of-thumb design (illustrated in Figure 2.1) for direct embedment foundations of transmission towers. The report states that this rule-of-thumb works well for wood poles but for steel and concrete poles with various backfill conditions, there is a need for a better model representation and design criteria to resist the possible load conditions.

The study presents analytical models developed for direct embedment foundations and field test results to predict the ultimate capacity and load-displacement behavior of direct embedment foundations under full-scale load tests. Twelve full-scale direct embedment foundations were loaded at seven different test sites. The pole height of the full-scale test specimens varied between 65 to 115 ft with a maximum outer diameter varying from 28 to 38 in. The embedment depth varied from 7.7 to 11.5 ft and the average hole diameter for the backfill ranged from 41 to 57 in. Various backfills were used for the test specimens: 1) lightly compacted silty clay, 2) loose crushed stone, 3) well-compacted silted clay, and 4) well-compacted crushed stone. From each load tests, the subsurface investigation, foundation design, pole installation and instrumentation, load testing, and data analysis were reported. The full-scale foundation load test demonstrated that when loose or poorly compacted backfill was used, the backfill dominated the load-deflection behavior with excessive ground-line deflection and rotation at a small percentage of the ultimate capacity of the foundation. The test results also indicated that the load-deflection response of foundations embedded into rock is significantly different than soil-supported foundations. The direct embedment foundations with rock backfill showed nearly linear moment deflection or rotation relationships.

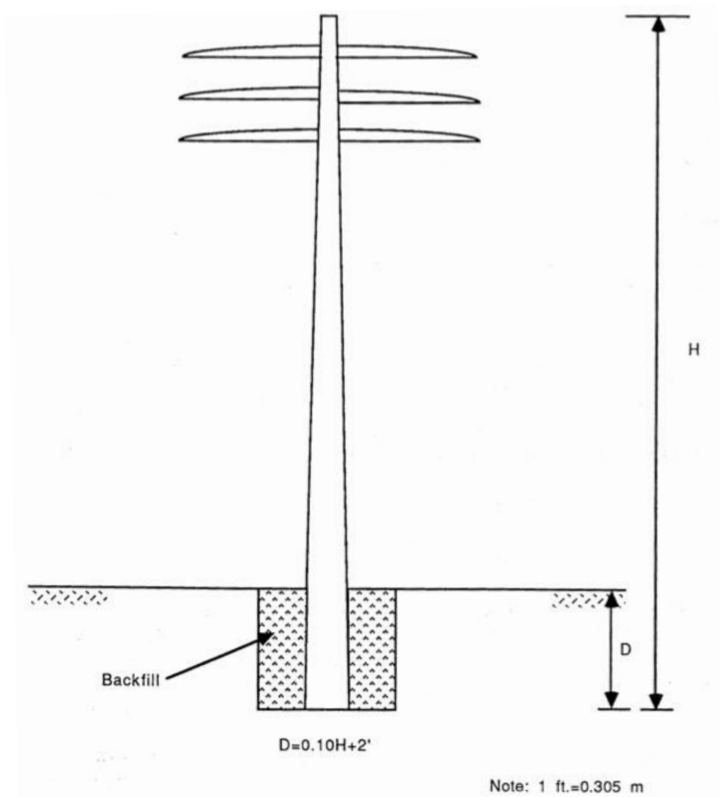

Figure 2.1: Rule-of-Thumb Design of Direct Embedment Foundation for Transmission Towers (retrieved from the EPRI EL-6309 Report, 1989)

The analytical models used in this study provided reasonable predictions compared to the test results regarding the load-deflection response and the ultimate capacity of the direct embedment foundations only when the foundations were soil-supported with well-compacted backfill. This study also stated that analytical models used for drilled shaft foundations can also be used for the direct embedment foundations when the backfill of direct embedment foundations have similar or higher shear strength and stiffness characteristics than the native soil. However, for the case where the backfill has lower shear strength and stiffness than the surrounding soil, models used for drilled shafts do not work well for direct embedment foundations because they do not consider variations in strength and stiffness in the radial direction from the perimeter of the foundation. The study also indicated that the analytical models used for drilled shafts typically only consider horizontal soil layering but a vertical layer surrounding the pole must be modeled if a direct embedment foundation is used by assigning the shear strength and stiffness parameters of either the backfill or the native soil.

For future research, this report asks the following questions to be answered: 1) What backfill materials are most suitable for use in construction the direct embedment foundations? 2) What amount of compaction is required for backfill to obtain satisfactory load-deflection performance? 3)What types of compaction equipment are most suitable for compacting the backfill in the relatively thin annulus surrounding a direct embedment foundation? and 4) Can flowable backfill materials not requiring compaction such as cement-fly ash mixtures be considered as material for backfill in the direct embedment foundations?

2.1.2 Rojas-Gonzalez, DiGioia, Jr., and Longo (1991)

This IEEE Transactions on Power Delivery article was prepared by two engineers from GAI Consultants and one engineer from EPRI who were involved in the 1989 EPRI EL-6309 report and test program. The paper summarizes the results of the ten full-scale load tests and analysis results using the direct embedment foundation model introduced in the EPRI EL-6309 report (see Figure 2.2). Test sites No. 3, 7, and 10 which had steel poles that are 115 ft high with embedment lengths between 10.5 ft and 11.5 ft with an auger hole diameter of 52 and 54 in. with well-compacted crushed stone or well-compacted sand with gravel as backfill material were selected tests that were of interest due to the fact that Nebraska typically uses either a 120 ft or 140 ft high steel poles. However, HMT loads are typically lower than transmission structures. Test site No. 3 with well-compacted crushed stone had some stiff clayey silt near the ground-line and some sand below 3 ft to the bottom of the pole. Test site No. 7 had well-compacted crushed stone mix for backfill material surrounded by very loose to loose gravelly silty sand from the ground-line to 3 ft depth and medium dense sand and some gravel below 3 ft. Test site No. 10 had well-compacted crushed stone as backfill material surrounded by silty sand and dolomite fragments in the upper 4 ft soil and Dolostone below 4 ft to the bottom of the pole. Two figures from the original paper. Figure 2.3 shows how well the analytical model matches with the full-scale test results. Figure 2.4 is the result of the analytical prediction versus the applied moment measured at the test site with 0.5 in., 1 in., and 2 in. deflection at the ground-line. Figure 2.4 also demonstrates that the prediction based on the analytical model matches well with the measured applied moment at the full-scale test sites. Based on the model and the test results, at test site No. 3, 0.5-in. ground-line deflection is observed at approximately 400 ft-kip moment while 1 in. deflection is observed at approximately 600 ft-kip moment. At test site No. 7 and 10, ½ in. ground-line deflection is observed at approximately 200 ft-kip moment, 1 in. deflection at 300 ft-kip, and 2 in. deflection at 400 ft-kip moment. These test and analysis results can be served as a reference for the High Mast Towers typically used in Nebraska (120 ft or 140 ft) which is subjected to a similar load and ground-line displacement.

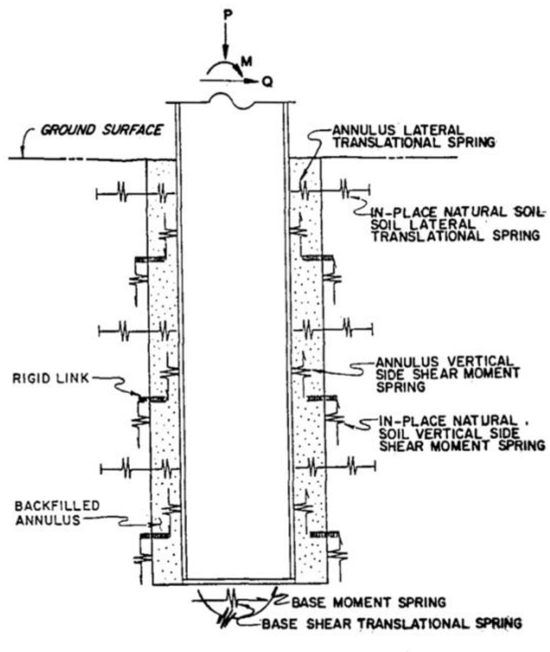

Figure 2.2 Direct Embedment Foundation Model developed from the EPRI research (retrieved from Rojas-Gonzalez et al. 1991)

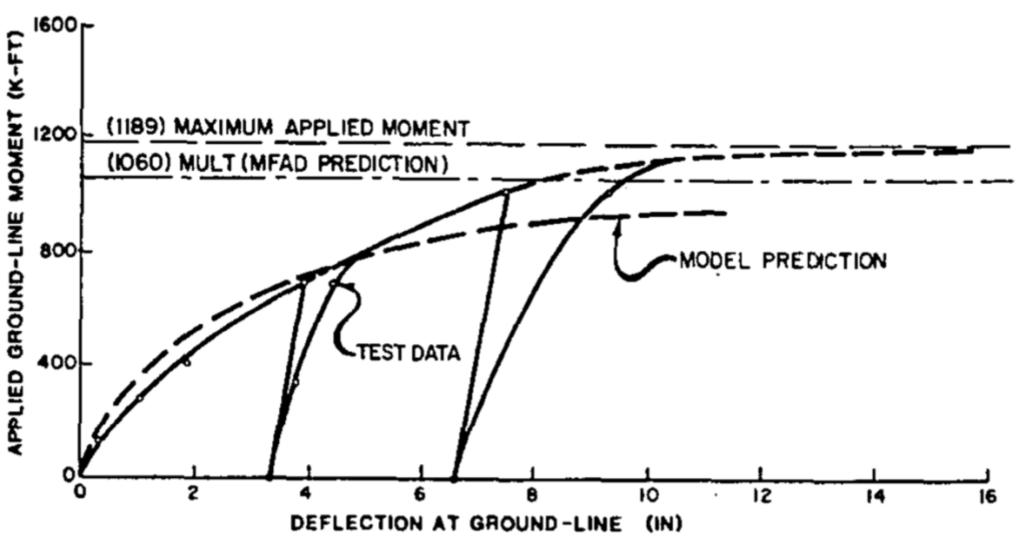

Figure 2.3: Applied Ground-Line Moment vs. Deflection at Ground-Line from the Full-Scale Testing and Analysis (retrieved from Rojas-Gonzalez et al. 1991)

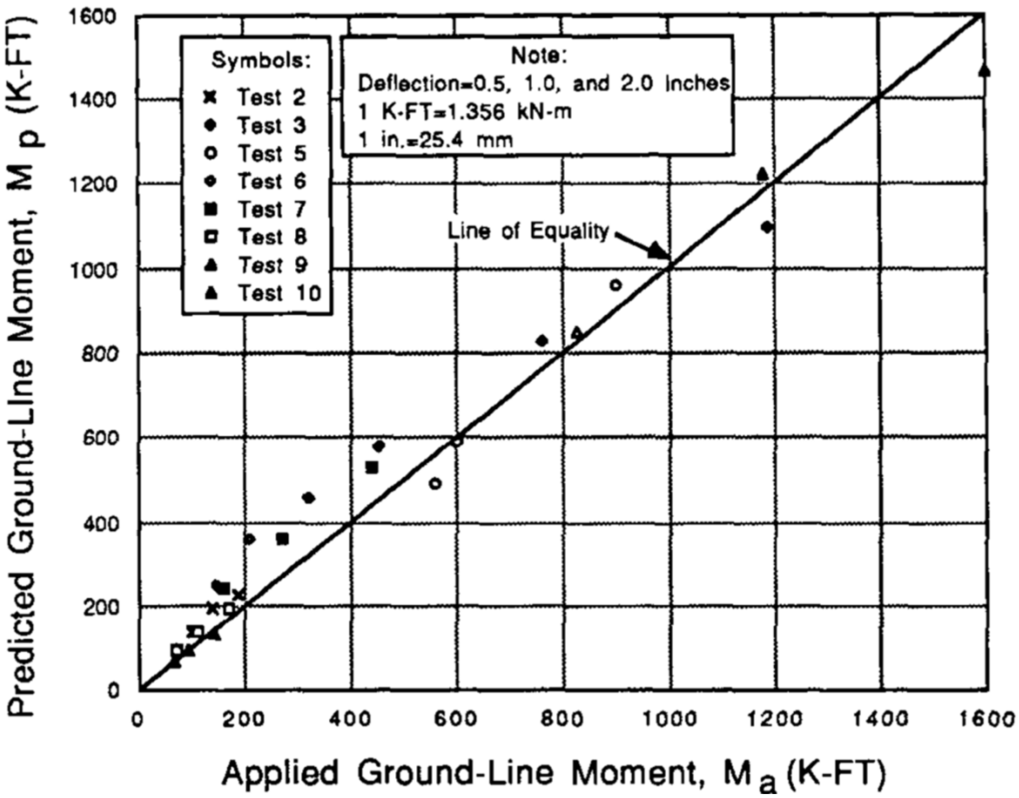

Figure 2.4: Predicted vs. Applied Ground-Line Moment for Ground-Line Deflections of 0.5, 1.0, and 2.0 inches (retrieved from Rojas-Gonzalez et al. 1991)

2.1.3 Ong et al. (2006)

This study was presented in the conference proceedings of the ASCE Electrical Transmission and Substation Structures Conference and provided interesting results using spectral analysis of surface waves to conduct the stability design of direct embedment pole structures. The study highlights that the widely used Brom's method for lateral stability would only be valid for either cohesive or cohesionless soils. For that reason, when dealing with natural soil which is mixed cohesive/cohesionless soil, empirical adjustments have to be made and there is inconsistency with these theoretical geotechnical analyses. The study also stressed out that the analytical models (MFAD by EPRI) require classical soil parameters as the friction angle or cohesive strength and to obtain these properties, an intensive soil sampling should be conducted at isolated points near the project site which is not easy and time consuming. Therefore, this study offers an alternative method in obtaining the elastic material constants of different soil layers by using mechanically induced stress waves and high-resolution transducers. This research identifies that the geophysical technique introduced in this paper is low-cost and the design approach using this geophysical method will provide reliable prediction of ground-line deflection for directly embedded poles. This analytical method can determine pole stability and soil strain by limiting the ground-line deflection based on easy to conduct, inexpensive, rapid site characterization.

2.1.4 Bingel III and Niles (2009)

This paper included in the proceedings of the ASCE Electrical Transmission and Substation Structures present methods for assessment and repair of steel tower and steel pole foundations. The paper introduces both direct inspection procedures (visual assessment, physical measurements, excavation, ultrasound measurements, electromagnetic acoustic transducer) and indirect inspection methods (measurements of pH, Redox, soil resistivity, half-cell measurements) to develop corrosion inspection on steel poles under soil. Common assessment methods, possible corrosion ratings, and repair techniques are introduced in this summary paper that can be adapted for the steel poles that will be directly embedded in soil. For example, the study suggests using the “SSPC VIS2 Standard Method for Evaluating Degree of Rusting on Painted Steel Surfaces” developed by the Society for Protective coatings that can be used to classify the type of corrosion and severity for the samples collected and quantify the inspection result following the standard. The study states that directly embedded poles often requires necessary excavation for a shallow depth between 18 in. to 24 in. to allow the inspector to make visual assessments and physical measurements to determine the amount of section loss caused by corrosion. The study suggests when direct measurements or physical measurements from excavation is not easy to conduct, that nondestructive testing methods using ultrasound or electromagnetic acoustic transducers could also be an option to detect the corrosion on steel members buried under ground. In addition, the study suggests that based on the direct or indirect assessment, unless severe corrosion with metal loss, pitting, thinning, or perforation is observed, if the corrosion rating is between low and moderate, the next inspection can be conducted in ten years.

2.1.5 Kandaris and Davidow (2015)

This study was presented in the conference proceedings of the ASCE Electrical Transmission and Substation Structures Conference and provided interesting results of the survey conducted by an ad hoc task force team assembled by the Deep Foundation Institute with experienced individuals from the electric power industry, utility consultants, utility companies and academia. The conference proceedings paper summarizes the survey results of 45 questions about the designer demographics, design approach, deep foundation design practices, geotechnical exploration practices, reinforced concrete design practices, direct embedment pole foundation design, alternate foundation types, foundation field testing and validation, and construction considerations.

A total number of 22 surveys were collected which is a low number to generalize, but there are some points we can still learn from this survey result that there is little consistency in the foundation design of transmission lines for designers, consultants, and utilities. Many of the utilities and consultants had their own internal design manuals due to the lack of a uniform guidance or specifications. The top deflection of the foundation used as performance factors at the ground-line varied between zero to six inches and rotation between zero to two degrees. Half of the respondents used factored loads when evaluation foundation performance criteria, while the rest used service loads or do not identify the load type. And, 73% of the designers stated that they use some reduced foundation resistance due to the near surface soil conditions but no more than one quarter agreed on the method for determining the reduced resistance. Regarding direct embedment foundations, the survey results demonstrate that the backfill material was compacted native soil (57%), engineered aggregate (79%), and concrete (74%). Most designers used analytical (84%) and general practice (79%) methods for the embedment depth design. The participants of the survey also addressed that 36% of the respondents have no limits to free fall placement of concrete within drilled shafts and 54% do not allow cold joints within concrete drilled shaft foundations.

2.2 Current State of Practice

2.2.1 Industry practice

The current state of US practice for high-mast tower foundations are drilled-shaft foundations. These foundations consist of a drilled-shaft excavation with concrete, a rebar cage containing longitudinal (vertical) and transverse steel, wiring conduit, and anchor bolts. The anchor bolts are embedded with proper length and spacing to transfer structural loads to the concrete. These bolts attach to a transverse plate that is welded to the pole.

The excavations are typically bored with a large excavator-mounted drill rig and appropriately sized tooling for the shaft with diameters in 6-in increments typical. The anchor bolts are set with a template. With the current state of practice, the foundation contractor never handles the pole.

The pole erection is performed by another contractor who may self-perform or subcontract. Frequently, the electrical contractor will perform or supervise this work and this can be source malfunction as they typically do not perform foundation and structural work. Owner inspections of in-service pole often reveal that bolts are loose, and likely, not tightened properly during erection. Research is ongoing to address this important issue.

2.2.2 Direct embedment for cellular tower steel poles

Table 2.1 and Table 2.2 are the physical properties, effective projected area (EPA) capacity of the pole, and maximum reactions at the base for a typical 120-ft direct embed steel pole, and 140-ft direct embed steel pole, respectively, used for cellular towers in US designed by Rohn design engineers for the TESSCO Technologies (https://www.tessco.com/). The embedment depth ranges between 24 to 26 ft with aggregate backfill placed between the soil and the steel pole for a 140-ft tower. The embedment depth for the 120-ft tower range between 19 to 23 ft with aggregate backfill. The 140-ft tower has a base shear ranging between 9.6 kips to 14.9 kips while the 120-ft tower has base shear ranging between 7.1 kips and 11.5 kips. These loads are approximately two to three times the load observed in typical High Mast Towers with the identical height of 120 and 140 ft. The auger diameter for the backfill ranged between 3.5 to 4.0 ft for 120-ft tower and 4.0 to 4.5 ft for 140-ft tower. The calculations shown in both tables have notes that indicate that the tabulated EPA values are based on the assumption that 80% of the total EPA is located at the top of the pole and the remaining is located 20-ft below the top. The notes provided by the Rohn design engineers indicate that if all loads are located at the top of the pole, the tabulated EPA capacity should be reduced by 20%.

Table 2.1: Design Information of a 120-ft Direct Embed Steel Cellular Pole

Physical Properties for 120-ft Direct Embed Steel Poles

	Light	Medium	Heavy
TESSCO SKU	-	371245	301734
Design Number	T120LA	T120MA	T120HA
Tip OD, in (cm)	6.50 (16.51)	9.00 (22.86)	12.00 (30.48)
OD @ Grade, in (cm)	26.00 (66.04)	28.00 (71.12)	31.00 (78.74)
Butt OD, in (cm)	29.26 (74.32)	31.77 (80.70)	34.95 (88.77)
Number of Sides	12	16	18
Δ Dia, in/ft (cm/m)	0.1714 (1.428)	0.1717 (1.431)	0.1717 (1.431)
Side Taper, in/ft (cm/m)	0.0857 (0.7142)	0.0857 (0.7142)	0.0857 (0.7142)
Embedment, ft (m)	19.0 (5.8)	22.0 (6.7)	23.0 (7.0)
Auger Dia, ft (m)	3.5 (1.1)	4.0 (1.2)	4.0 (1.2)
Backfill Type	Aggregate	Aggregate	Aggregate
Total Length, ft (m)	139.0 (42.4)	142.0 (43.3)	143.0 (43.6)
Bare Pole Wt, lbs (kg)	5,538 (2,512)	6,418 (2,911)	8,545 (3,876)
No. of Sections	3	4	4

EPA (ft^2) for 120-ft Tapered Steel Poles

Wind Speed, mph		Light			Medium			Heavy		
		Sway Limit			Sway Limit			Sway Limit		
Fastest Mile	3-sec Gust	4°	3°	2°	4°	3°	2°	4°	3°	2°
70	85	18	10	-	39	24	6	90	62	35
80	100	18	10	-	36	24	6	80	62	35
90	110	5	5	-	15	15	6	55	55	35
100	120	-	-	-	-	-	-	36	36	35
110	130	-	-	-	-	-	-	23	23	23
120	140	-	-	-	-	-	-	14	14	14

	Maximum Reactions		
	Light	Medium	Heavy
Design Number	T120LA	T120MA	T120HA
Download, kips	7.7	9.1	11.5
OTM, ft-kips	405.5	489.5	720.6
Shear, kips	7.1	9.0	11.5

(Information retrieved directly from https://resources.tessco.com/attachments/394772_ROHN-120-ft-DEP-Design-Overview-193192.pdf)

Table 2.2: Design Information of a 140-ft Direct Embed Steel Cellular Pole

Physical Properties for 140-ft Direct Embed Steel Poles

	Light	Medium	Heavy
TESSCO SKU	-	391214	321746
Design Number	**T140LA**	**T140MA**	**T140HA**
Tip OD, in (cm)	6.50 (16.51)	9.00 (22.86)	12.00 (30.48)
OD @ Grade, in (cm)	30.00 (76.20)	32.50 (82.55)	35.50 (90.17)
Butt OD, in (cm)	34.30 (87.12)	37.00 (93.98)	40.18 (102.06)
Number of Sides	12	16	18
Δ Dia, in/ft (cm/m)	0.1793 (1.494)	0.1803 (1.503)	0.1802 (1.502)
Side Taper, in/ft (cm/m)	0.0897 (0.7475)	0.0902 (0.7517)	0.0901 (0.7508)
Embedment, ft (m)	24.0 (7.3)	25.0 (7.6)	26.0 (7.9)
Auger Dia, ft (m)	4.0 (1.2)	4.5 (1.4)	4.5 (1.4)
Backfill Type	Aggregate	Aggregate	Aggregate
Total Length, ft (m)	164.0 (49.99)	165.0 (50.3)	166.0 (50.6)
Bare Pole Wt, lbs (kg)	8,534 (3,871)	10,469 (4,749)	11,941 (5,416)
No. of Sections	4	4	4

EPA (ft^2) for 140-ft Tapered Steel Poles

Wind Speed, mph		Light			Medium			Heavy		
		Sway Limit			Sway Limit			Sway Limit		
Fastest Mile	3-sec Gust	4°	3°	2°	4°	3°	2°	4°	3°	2°
70	85	16	5	-	42	26	6	86	62	31
80	100	16	5	-	42	26	6	86	62	31
90	110	8	5	-	36	26	6	66	62	31
100	120	-	-	-	16	16	6	45	45	31
110	130	-	-	-	-	-	-	28	28	28
120	140	-	-	-	-	-	-	13	13	13

	Maximum Reactions		
	Light	Medium	Heavy
Design Number	**T140LA**	**T140MA**	**T140HA**
Download, kips	11.1	13.6	15.4
OTM, ft-kips	614.7	887.8	1046.5
Shear, kips	9.6	13.1	14.9

(Information retrieved directly from https://resources.tessco.com/attachments/304031_ROHN-140-ft-DEP-Design-Overview-193193.pdf)

2.3 Proposed State of Practice

The proposed state of practice for high-mast towers is to adopt a similar system that the electrical utility industry uses for transmission and distribution pole. Direct embedment eliminates the fatigue prone weld details at the baseplate-to-tube connection. It also eliminates the need to rebar cages, anchor bolt placement, bolt tightening, secondary issues such a gap under the baseplate permitting animal nesting and moisture intrusion (but, Nebraska require a non-shrink grout fill between baseplate and foundation).

This method requires the contractor to drill the same excavation but instead of setting a rebar cage and an anchor bolt cage, the pole base positioned at the bottom of the shaft and the pole is properly plumbed from the top. The lowest pole section will be approximately 15 to 20 ft greater than the shaft depth. This section accommodates conduit holes and above-grade handholes that are fabricated off site. The shaft is filled with either concrete or aggregate backfill. The remaining sections of the pole attached to the bottom section with a slip-fit connection that is standard practice. Various drilling and placement procedures are outlined in the draft specification in Appendix A. The soil type and condition play a large role here.

The utility contractor installs the top sections and finishes the electrical work. Different scenarios could be permitted, e.g., the pole might be assembled and then place in the shaft; this would require proper crane and rigging.

2.4 Examples of Design Specifications and Guidelines

2.4.1 ASCE/SEI 48-11 – Design of Steel Transmission Pole Structures

The Structural Engineering Institute (SEI) within American Society of Civil Engineers (ASCE) published a design guideline for steel transmission pole structures. In this document, sections outline design and installation guidelines with commentary for directly embedded steel transmission poles. The design guideline states in Section 9.4 of the guideline that the embedded section shall be designed to resist the overturning moment, shear, and axial loads. In addition, the guideline states that the length of the section of the pole below the ground line shall be determined using a lateral resistance approach and the owner shall be responsible for supplying the structural designer information regarding the embedment depth, allowable foundation rotation, and design point of fixity of the embedded section. In the commentary of for Section 9.4, the document states that a directly embedded pole foundation typically is designed to transfer overturning moments to the in-situ soil, rock, or backfill by means of lateral resistance and axial loads can be resisted by a bearing plate installed on the base of the pole. If additional bearing capacity is required, base-expanding devices can be installed at the bottom of the pole. The commentary includes that the quality of backfill, method of placement, and degree of compaction greatly affects the strength and rotation of the foundation system and, thereby, the design of the embedded pole. In addition, the commentary states that the direct-embedded pole foundations have become popular because of their relatively low installation cost. The guideline also adds a commentary that buoyancy of the pole should be considered when using direct embedded-poles where high water table is present. This document also provides in Section 11.5.2 some installation guidelines that the annular opening around the embedded pole shall be backfilled with soil or concrete and that the soil shall be compacted in accordance with the specification requirements. In the commentary of Section C11.5.2, the guideline provides additional information addressing that care should be taken during the backfilling and compaction process to prevent damage to the protective coating of the embedded pole section. The commentary section for installation also addresses that the Line Designer should provide specific recommendations and requirements for the type of backfill material and the method of backfill placement to ensure that in-service behavior of the pole is in accordance with the design assumptions made regarding pole rotation at the ground line.

2.4.2 IEEE Guide for Transmission Structure Foundation Design and Testing

This guide was jointly prepared by the Institute of Electrical and Electronics Engineers (IEEE) Power Engineering Society Transmission and Distribution Committee and the American Society of Civil Engineers (ASCE) Transmission Structure Foundation Design Standard Committee. The guide states that direct embedment foundations which has been traditionally used for wood pole foundations in distribution lines has recently increased in number for steel and concrete pole foundations in transmission lines. This guide provides the definition of direct embedment: "Foundations that are made by power augering a circular excavation in the ground and inserting the pole (wood, steel, or concrete) directly into the excavation, and backfilling the void between the pole and the sides of the excavation." Then the guide highlights that the quality of backfill, method of placement, and the degree of compaction strongly influences the stiffness and strength of the direct embedment foundation. The guide also provides the principal differences between direct embedment foundations and drilled shaft foundations. The guide indicates that similar analytical models used in drilled shaft design can be used for direct embedment foundations because the response of direct embedment foundations to compression, uplift, and lateral loads is similar to that of drilled shafts. This is important for the present work as well.

However, the major difference between the two types of foundations that this document summarizes is that drilled shafts transfer loads directly into in-situ soil while direct embedment foundations transfer loads to the backfill which in turn transfer the loads to the in-situ soil due to the presence of backfill between the pole and the in-situ soil. The guide also introduces a design/analysis model that can be used for direct embedment foundations which is the model developed by the research conducted and summarized on the EPRI EL-6309 Report introduced in the research literature section.

The following are some of the important aspects highlighted in the report: 1) Backfill clearly constitutes an important element of the direct embedment foundation, 2) Granular backfill can be readily compacted and is generally preferable to a cohesive soil, 3) To obtain proper compaction, the backfill should be placed in layers of 6 in. or less and compacted to the specified density, 4) The uplift capacity of direct embedment foundations is related to the quality of the backfill and the adhesive and friction forces that can be mobilized at the structure-backfill interface or at the backfill-in-situ soil interface, 5) For compression loads, significant end-bearing capacity can only be achieved if the direct embedded pole is closed with a base plate.

2.4.3 Omaha Public Power District (OPPD) technical specifications

The Omaha Public Power District (OPPD) published a 17-page document with technical specifications for galvanized steel transmission structures that includes design and construction requirements. The specification addresses that all stress calculations shall be based on an elastic analysis in accordance with ASCE/SEI 48-11. The following is to highlight some of the design requirements (Section 3) for embedded poles listed in this specification: 1) Embedded poles shall continue the taper at ground line to a point 6 feet below ground line. From this point to the bottom of the section, the taper may continue or the diameter may be held constant. No reverse tapers will be allowed. 2) The base section of all embedded poles shall have a maximum projection 30 ft above the ground line unless otherwise noted on the drawings. 3) The base of all embedded poles shall have a minimum projection of 10 ft above the ground line unless otherwise noted on the drawings. For construction requirements, the specification states that direct embedded poles shall have a minimum of 3/16 in. thick ground sleeve extending above and below ground line as shown on the drawings. In addition, the document states that all direct embedded galvanized steel poles shall be coated with a minimum 16 mil dry thickness of two component hydrocarbon extended polyurethane coating that is resistant to ultraviolet light and conform to ASTM G14-77. Figure 2.5 shows an example of the embedded base section from a typical galvanized steel pole drawing provided by OPPD.

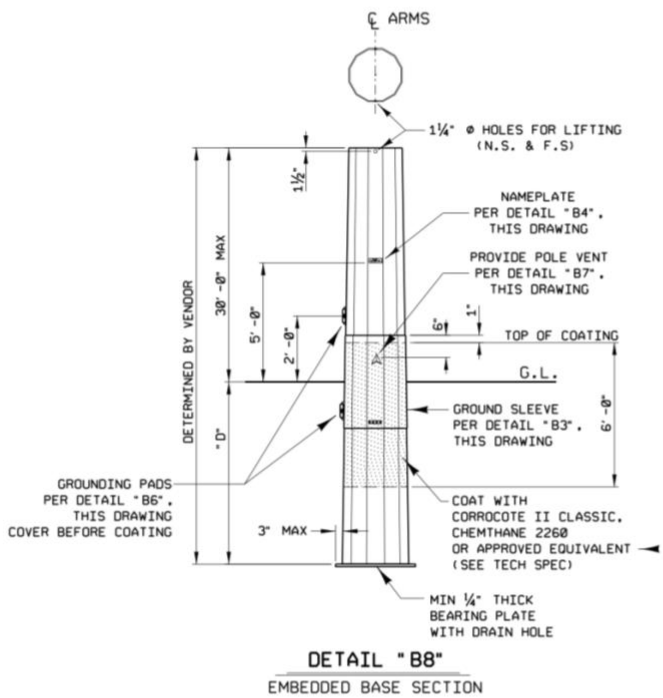

Figure 2.5: Example Drawing of an Embedded Base Section for Galvanized Steel Poles (CAD drawing provided by the Omaha Public Power District)

2.4.3 NDOT technical specifications

The NDOT Standard Specifications for Highway Construction (2017) Section 407 contains material requirements, construction methods, method of measurement, and basis of payment for pole and tower foundations. Either a concrete foundation or a power installed foundation are the options listed in the technical specifications. The size of the concrete foundation should be sufficient to include ground rods, reinforcing steel, anchor bolts, conduit entrance bends, and a spare conduit bend. It is recommended that the contractor shall obtain soil data and design and construct the tower foundation if the foundation details are not shown in the contract. NDOT specifications states that the concrete foundations for the tower installations shall be constructed according to the following steps: a) All foundation excavations shall be free or loose dirt, b) All concrete shall be Class 47B-3000 (47B-20), c) The anchor bolt pattern shall be centered in the foundation, and 3) The contractor shall perform all excavations, backfilling, and placing of reinforcing steel and concrete in accordance with Sections 702 (Excavation for Structures), 704 (Concrete Construction), and 707 (Reinforcement) of NDOT Standard Specifications for Highway Construction.

2.4.4 Other state DOT technical specifications

Table 2.3 was developed by the research team currently working with the Alaska Department of Transportation (Alaska DOT) on evaluation of light pole foundation embedment. The table summarizes the current State Department of Transportation (State DOT) practices for light pole foundation designs. Delaware and Illinois are the only states that require a special requirement for these foundation designs to conduct further structural analysis or soil analysis coordinated with geotechnical engineers. Connecticut, Maine, and Oregon use the AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (AASHTO LRFD LTS) (2015) for their foundation design.

Table 2.3: Summary of State DOT Design Requirements for Light Pole Foundations

State DoT	Design Guidelines on Luminaire Foundation	Remarks
Connecticut	AASHTO LRFD LTS (2015) Standard drawings are provided.	https://portal.ct.gov/DOT/CONNDOT/SECTION-M15#M.15.04
Delaware	Considers structural analysis for cases of nonstandard longer arm or pole height. Foundation design shall be coordinated with Delaware DOT's Geotechnical engineer. Standard drawings are provided.	https://deldot.gov/Business/drc/pdfs/traffic/LightingPolicy.pdf?cache=1599176213610
Illinois	High-mast lighting requires special designs and soil analyses. A 4ft diameter reinforce concrete foundation is recommended	https://idot.illinois.gov/Assets/uploads/files/Doing-Business/Manuals-Split/Design-And-Environment/BDE-Manual/Chapter 56 Highway Lighting.pdf
Kansas	Special condition for non-typical soil condition (frozen, saturated or soft soils) – engineer's directions to remedy unsuitable foundations shall be followed.	https://www.wycokck.org/WycoKCK/media/Public-Works/Engineering/Documents/Technical-Provisions-2008-Edition.pdf
Maine	PE shall design foundations AASHTO LRFD LTS (2015) Standard drawings are provided.	https://www.maine.gov/mdot/contractors/publications/standardspec/docs/2014/StandardSpecification-full.pdf
Michigan	Standard drawings are provided	https://mdotjboss.state.mi.us/TSSD/getCategoryDocuments.htm?categoryPrjNumbers=1403886,2028779,1403887,1403888,1403889,1403890,1797786&category=Traffic%20Signing
Minnesota	Standard drawings are provided.	https://www.dot.state.mn.us/trafficeng/lighting/2010_Roadway%20Lighting_Design_Manual2.pdf
New Hampshire	Precast or cast in place bases. Standard drawings are provided.	https://www.nh.gov/dot/org/projectdevelopment/highwaydesign/specifications/documents/2016NHDOTSpecBookWeb.pdf
Ohio	Standard drawings are provided.	http://www.dot.state.oh.us/Divisions/Engineering/Roadway/DesignStandards/traffic/SCD/Documents/HL_02011_2020-07-17.pdf
Oregon	TM653 for typical condition and TM650 for foundations to resist larger design loads. AASHTO LRFD LTS (2015)	https://www.oregon.gov/ODOT/Engineering/BaselineReport/TM653.pdf
Texas	Standard drawings are provided.	ftp://ftp.dot.state.tx.us/pub/txdot-info/cmd/cserve/standard/traffic/ridfn11.pdf

2.5 Corrosion Considerations

This discussion is relative to metals in direct contact with the soil. In the case of directly embedded HMTs, the surrounding backfill is concrete or aggregate. For concrete, clearly a large protective layer exists and this is similar as for assets such as drilled-shaft foundation, concrete slabs on grade, abutments, and so forth. The considerations in this case are similar to protecting reinforcement steel with a coating. For aggregate backfill, the site soil is not in direct contact; however, the groundwater and the associated chemistry is likely best characterized by this soil.

It is likely that concrete will be the typical solution, and that the aggregate fill is the exception. Moreover, concrete backfill could be explicitly specified in corrosive locations. The discussion below helps to provide a means for identifying such locations. (this should be consistent with NDOT methods used for other assets as well)

2.5.1 Parameters that influence underground metal corrosion

Metals buried under soil behave completely different from metals in the atmosphere. Because the underground environment changes constantly depending on the soil condition, the electrochemical process (oxidation and oxygen reduction; anodic and cathodic reactions) can differ for the underground environment even when the locations considered are geographically close. The soil texture (silt, sand, clay contents), moisture content, oxygen concentration, redox potential (degree of aeration in soil), pH level, resistivity, ion contents (specifically, the amount of aggressive ion salts such as chloride and sulfate contents), and bacteria level of the soil are all important parameters that control the rate of corrosion of metal in soil.

It is well known no single parameter listed above that affects the rate of corrosion of buried metal but rather multiple of them interacting together through an iterative process that affects the metallic structures buried underground. For this reason, measurements of a single parameter (for example, only measuring the soil resistivity) should not be used to plan corrosion mitigation plans and design for corrosion resistance of a buried metal structure. As shown in Figure 2.6 (de Arriba-Rodriguez et al., 2018), any of the parameters listed above can affect the rate of corrosion.

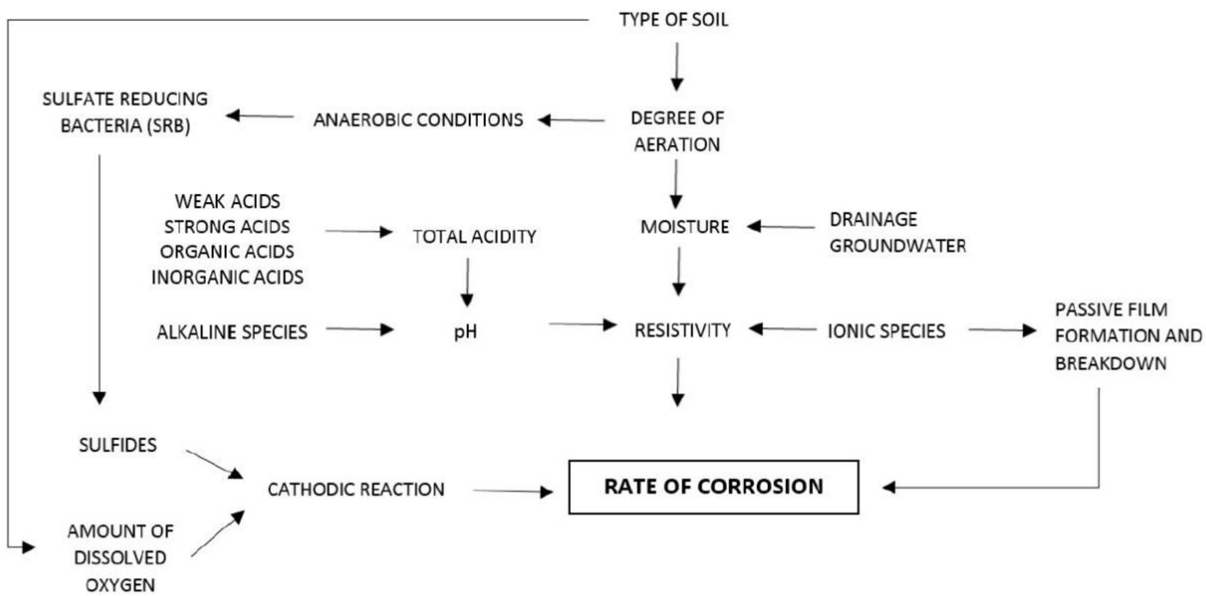

Figure 2.6: Rate of Corrosion of Metals buried under Soil (retrieved from de Arriba-Rodriguez et al., 2018)

2.5.2 Qualitative evaluation of underground corrosion

There are multiple qualitative and quantitative methods that can be used to make the decision to select which corrosion protection method will be used or determine the design values (coating thickness, metal thickness for corrosion protection, etc.) for steel structures being embedded in soil. Again, with the understanding no one single parameter but multiple parameters that affect the underground corrosion, NDOT can adapt or modify one of these existing qualitative or quantitative methods that consider various factors in evaluating the underground corrosion steel embedded poles.

For example, for a qualitative evaluation, the American Water Works Association (AWWA) uses a point system that takes account for soil resistivity, pH, Redox Potential (mV), Sulphide, and Moisture levels. The Ductile Iron Pipe Research Association (DIPRA) developed a design decision model based on the likelihood, considering various parameters that AWWA listed, and consequence (pipe diameter, construction repair cycles, depth of the embedment, and alternate water supply conditions) they selected for underground pipes. Table 2.4, Table 2.5, Table 2.6 and are a score sheet, consequence score sheet, and the corrosion protection recommendations, respectively, that DIPRA recommends for use in qualitative design of underground pipes. Based on this qualitative design decision model, the engineer can select either which corrosion protection method (standard protection, polyethylene encasement, metallized zinc-coating, or cathodic protection) to use in underground applications.

NDOT can possibly use a similar method to qualitatively determine the level of corrosion protection for the HMT foundation directly embedded using aggregate backfill. The parameters used by AWWA and DIPRA for the likelihood (Table 2.4) of metals being corroded under soil is similar for the HMT directly embedded. For example, the California Department of Transportation (CALTRANS) classifies every site as corrosive or non-corrosive site for their projects. They define a site to be corrosive if one or more of the following conditions exist for the representative soil at the site: Chloride concentration is 500 ppm or greater, sulfate concentration is 2,000 ppm or greater, or the pH is 5.5 or less. If the site is defined to be corrosive, corrosion mitigation is required.

Table 2.4: Underground Corrosion Likelihood Score Sheet (retrieved from DIPRA, 2018)

Likelihood Factor		Points	Maximum Possible Points
Soil Resistivity	< 500 ohm-cm	30	**30**
	≥ 500-1000 ohm-cm	25	
	> 1000-1500 ohm-cm	22	
	> 1500-2000 ohm-cm	19	
	> 2000-3000 ohm-cm	10	
	> 3000-5000 ohm-cm	5	
	> 5000 ohm-cm	0	
Chlorides	> 100 ppm = positive	8	**8**
	50 – 100 ppm = trace	3	
	< 50 ppm = negative	0	
Moisture Content	> 15% = wet	5	**5**
	5 – 15% = moist	2.5	
	< 5% = dry	0	
Ground Water Influence	Pipe below the water table at any time	5	**5**
pH	pH 0 – 4	4	**4**
	pH > 4 – 6	1	
	pH 6 – 8, with sulfides and low or negative redox	4	
	pH > 6	0	
Sulfide Ions	positive (≥ 1 ppm)	4	**4**
	trace (> 0 and < 1 ppm)	1.5	
	negative (0 ppm)	0	
Redox Potential	= negative	2	**2**
	= positive 0 – 100 mv	1	
	= positive > 100 mv	0	
Bi-metallic Considerations	Connected to noble metals (e.g. copper) – yes	2	**2**
	Connected to noble metals (e.g. copper) – no	0	
TOTAL POSSIBLE POINTS			**60**
Known Corrosive Environments	Cinders, Mine Waste, Peat Bog, Landfill, Fly Ash, Coal		**21**
Soils with known corrosive environment shall be assigned 21 points or the total of points for likelihood factors, whichever is greater			

Similar to Table 2.4 of DIPRA, CALTRANS would measure the soil resistivity and if the soil has a minimum resistivity less than 1,000 ohm-cm, they classify that the soil has high possibilities to transport soluble salts and is susceptible for corrosion activities to take place. When the minimum resistivity is less than 1,000 ohm-cm, chemical testing is further required to evaluate the chloride and sulfate level in the soil sample. The threshold values that CALTRANS uses are comparable with the point system used by DIPRA (Table 2.4) and indicates the cases with high points (more likely corrosion will take place). (CALTRANS, 2018)

However, the parameters listed in the consequence (Table 2.5) for pipe design should be modified for HMTs. Possible consequence factors would be the embedment depth, pole diameter below the ground-line, and access availability for potential repair work. If the embedment depth is deep, or the access availability for potential repair work will be low, or the pole diameter below the ground-line is large, the consequence scores will be high. The possible corrosion mitigation methods in Table 2.5 should also be modified for other methods that can be applied for HMTs.

Table 2.5: Underground Pipe Corrosion Consequence Score Sheet (retrieved from DIPRA, 2018 and Arriba-Rodriguez et al., 2018)

Consequence Factor		Points	Maximum Possible Points
Pipe in Service	3” to 24 “	0	**22**
	30” to 36”	8	
	42” to 48”	12	
	54” to 64”	22	
Location:	Routine (Fair to good access, minimal traffic and other utility consideration, etc.)	0	**20**
Construction-Repair Considerations	Moderate (Typical business and residential areas, some right of way limitations, etc.)	8	
	Difficult (Subaqueous crossings, downtown metropolitan business areas, multiple utilities congestion, swamps, etc.)	20	
Depth of Cover	0 to 10 feet depth	0	**5**
Considerations	> 10 to 20 feet depth	3	
	> 20 feet depth	5	
Alternate	Alternate supply available - no	0	**3**
Water Supply	Alternate supply available – yes	3	
TOTAL POSSIBLE POINTS			**50**

Table 2.6: DIPRA Design Decision Model (retrieved from DIPRA, 2018)

Likelihood (x)	Consequences (C)	Proposed Action
< 10	Any	Standard protection
10-20	< 30	Standard protection
	$1035.7x^{-1.05}$> C > $240.3x^{-0.7}$	Polyethylene Encasement (PE)
	> $1035.7x^{-1.05}$	PE with Joint Bonds
20-35	< 25	PE
	$1596.1x^{-1.08}$> C > $1035.7x^{-1.05}$	PE with Joint Bonds
	> $1596.1x^{-1.08}$	PE with Metallized Zinc Coating
35-40	< 30	PE with Joint Bonds
	$1177.8x^{-0.89}$> C > $1596.1x^{-1.08}$	PE with Metallized Zinc Coating
	> $1177.8x^{-0.89}$	PE with Cathodic Protection
40-45	< $1177.8x^{-0.89}$	PE with Metallized Zinc Coating
	> $1177.8x^{-0.89}$	PE with Cathodic Protection
45-50	Any	PE with Cathodic Protection

The most typical protection for a steel pole embedded with aggregate backfill in a corrosive environment may be achieved by providing either one of the followings or a combination of: 1) protective coatings (epoxy, bituminous coating, etc.), 2) cathodic protection, 3) increasing cross-section area of steel, or 4) reinforced concrete jacket. Again, the concrete backfill is the best solution in corrosive areas and its use parallel that of other NDOT assets in such areas.

2.5.3 Quantitative evaluation of underground corrosion

Quantitative methods exist for corrosion assessment for underground steel structures whether to decide the structural dimensions that guarantee the service life of the structure being buried in soil, field data that provides the information of soil condition (soil type, aeration level, moisture level, pH level, soil resistivity, ion contents, and any bacteria contents), the amount of steel loss in weight (due to corrosion) and the maximum penetration depth (pit depth in terms of mils/years) can be used. Fortunately, the most complete quantitative study around the world was conducted in US by the National Bureau of Standards (NBS; now National Institute of Standards and Technology) and this is the most complete database that is still used these days although the study may have limitations. NBS (now NIST) conducted a 45-year (1910-1955) study around the entire US with thousands of steel samples buried and exposed in carefully selected underground sites. The test sites and the 45-year corrosion study results are summarized by Romanoff (1957). A total number of 128 sites were selected in US which represents 95 types of soils. As shown in Figure 2.7 (Romanoff, 1957), Nebraska has four different types of soil groups: 1) Prairie Soil, 2) Chernozen Soil, 3) Nebraska Sand Hills (no other locations in US have this soil type), and 4) Dark Brown Soil.

Within the 128 sites, 10 sites can be selected to represent the Nebraska soil types. The Knox silt loam and the Wabash silt loam from Omaha, Nebraska, Lindley silt loam from Des Moines, Iowa, Marshall silt loam from Kansas City, Missouri, and the Muscatine silt loam from Davenport, Iowa may represent the Prairie soil type which can be found in most of the eastern part of Nebraska. The Fargo clay loam from Fargo, North Dakota may represent the Chernozen soil type in the center Nebraska. Unfortunately, there was no test site in the NBS study that can represent the Nebraska Sand Hills region. The unidentified silt loam from Denver, Colorado, Billings silt loam from Grand Junction, Colorado can represent the Dark Brown Soils found in the western Nebraska. The drainage, soil resistivity, pH, chemical composition, temperature, annual precipitation, moisture level, air-pore space, apparent specific gravity, and volume shrinkage of the test sites are reported for these sites. The sand, silt, clay components were also measured for some of these sites. The loss in weight and the maximum penetration depth were reported for an average of two specimens per site.

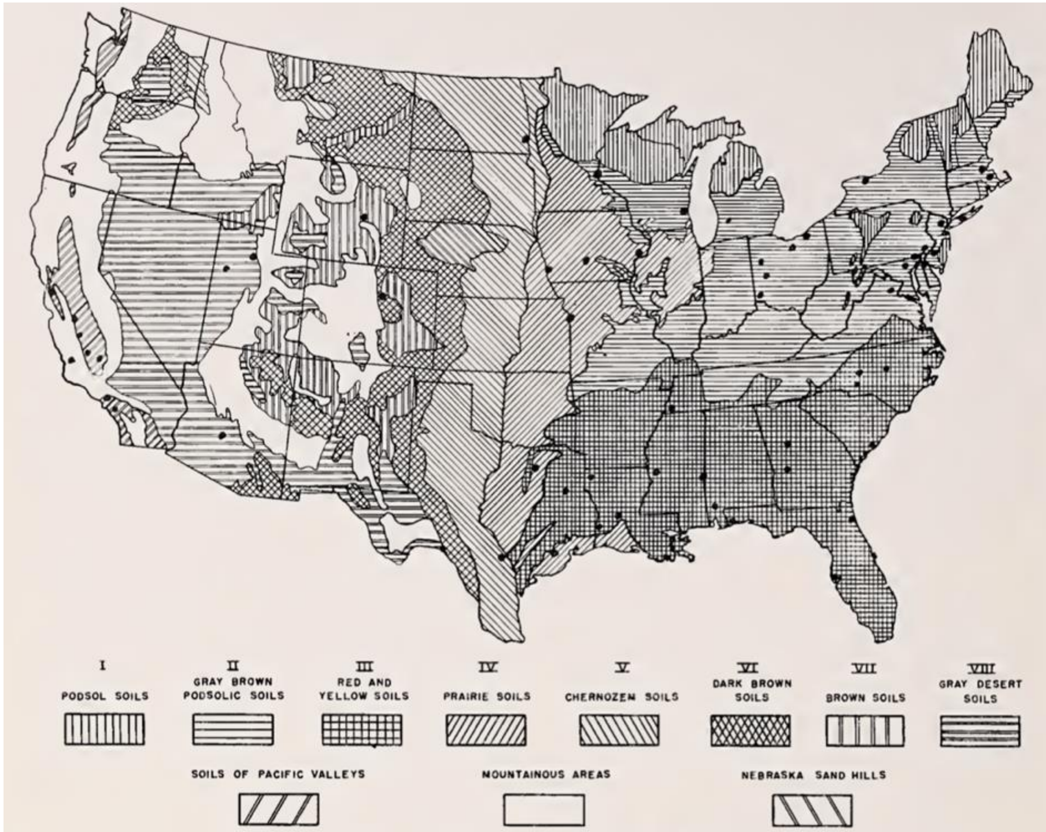

Figure 2.7: Soil Groups of US and Dots indicating the NBS Test Sites (retrieved from Romanoff, 1957)

Table 2.7 shows the chemical and physical properties of the soils at the NBS test sites. Table 2.8 shows the loss in weight and the maximum penetration depth measured during the test period. If NDOT would like to make engineering decisions for the dimensions of the High Mast Towers that will be buried under soil or coating thickness required for a desired service level Table 2.7, Table 2.8, and Table 2.9 could be a reference in providing quantitative calculations based on the extensive field observation data produced by NIST.

Table 2.7: Chemical and Physical Properties of the NBS Test Sites that represent Nebraska Soil Groups (retrieved from Romanoff, 1957)

Soil		Location	Inter-nal drain-age of test site	Resist-ivity at 60°F (ohm-cm)	pH	Composition of water extract, mg-eq per 100 g of soil								Mean Tem-pera-ture (°F)	An-nual preci-pita-tion (in.)	Mois-ture equiv-alent (%)	Air-pore space (%)	Appar-ent speci-fic gravit y	Vol-ume shrin k-age (%)
Test Site No.	Type					Total Acidit y	Na + K as Na	Ca	Mg	CO_3	HCO_3	CI	SO_4						
8	Fargo clay loam	Fargo, North Dakota	P	350	7.6	A	1.42	1.72	2.55	0.00	0.71	0.01	4.43	39	24	37.0	8.7	1.56	21.0
18	Knox silt loam	Omaha, Nebraska	G	1,410	7.3	1.4	0.27	0.63	0.20	0.00	0.94	0.00	0.25	50.6	27.8	28.4	16.6	4.26	1.3
19	Lindley silt loam	Des Moines, Iowa	G	1,970	4.6	10.9	0.38	0.32	0.41	0.00	0.46	0.03	0.46	49.5	32.0	28.4	3.9	4.76	11.8
21	Marshall silt loam	Kansas City, Missouri	F	2,370	6.2	9.5	-	-	-	-	-	-	-	54.4	37.1	31.2	10.8	1.66	6.5
30	Muscatine silt loam	Davenport, Iowa	P	1,300	7.0	2.6	0.32	0.65	0.40	0.00	0.71	0.09	0.24	49.9	32.4	29.4	7.2	1.81	7.5
44	Wabash silt loam	Omaha, Nebraska	G	1,000	5.8	8.8	1.05	1.08	0.66	0.00	1.97	0.82	0.41	50.6	27.8	31.2	7.2	1.55	6.0
46	Unidentified silt loam	Denver, Colorado	G	1,500	7.0	-	-	-	-	-	-	-	-	50.0	14.4	7.6	23.2	-	0
101	Billings silt loam	Grand Junction, CO	F	261	4.5	A	5.21	19.24	1.43	0.00	0.66	1.56	22.48	52.0	8.8	30.0	-	-	-
102	Billings silt loam	Grand Junction, CO	F	103	7.3	A	22.63	16.56	3.85	0.00	0.56	4.67	36.82	52.0	8.8	20.4	-	-	-
103	Billings silt loam	Grand Junction, CO	F	81	7.3	A	22.01	13.32	2.00	0.00	0.18	11.09	25.70	52.0	8.8	30.6	-	-	-

Table 2.8: Loss in weight and maximum penetration of wrought black ferrous metal (retrieved from Romanoff, 1957)

Soil				Loss in weight (oz/ft^2)								Maximum penetration (mils)							
				1.5-in. pipe				3-in. pipe				1.5-in. pipe				3-in. pipe			
Test Site No.	Type of Soil	Location	Duration of exposure (years)	Open-hearth iron	Wrought iron	Bessemer steel	Bessemer steel (scale free)	Wrought iron	Open-hearth steel	Bessemer steel	Open-hearth Steel with 0.22 percent Cu	Open-hearth iron	Wrought iron	Bessemer steel	Bessemer steel (scale free)	Wrought iron	Open-hearth steel	Bessemer steel	Open-hearth Steel with 0.22 percent Cu
8	Fargo clay loam	Fargo, North Dakota	1.1	0.7	0.7	0.7	0.8	0.7	0.6	0.8	0.7	44	30	38	30	38	43	47	62
			3.8	1.9	1.9	2.0	2.0	1.7	1.7	1.9	1.8	46	36	44	37	38	51	48	57
			5.8	3.2	3.2	2.9	2.6	2.9	3.1	3.3	3.3	62	52	52	54	68	77	56	70
			7.7	3.3	3.3	3.2	3.2	4.1	3.5	4.2	4.3	62	52	66	52	62	86	75	94
			9.9	5.1	5.1	4.6	4.6	5.2	5.3	5.6	5.6	74	66	61	67	63	93	72	75
			11.8	8.4	6.9	7.7	6.5	8.7	7.9	8.3	8.8	100	76	74	58	83	92	110	127
18	Knox silt loam	Omaha, Nebraska	1.2	0.4	0.7	0.4	0.6	0.4	0.3	0.5	0.5	27	20	19	14	24	28	16	28
			3.8	1.2	1.8	1.9	1.6	1.3	1.2	1.2	1.7	40	40	48	34	34	52	43	37
			5.8	2.3	4.0	3.2	2.6	2.9	2.7	3.2	2.6	71	72	71	67	64	66	62	80
			7.7	1.8	3.3	2.9	2.0	2.2	1.9	2.3	2.1	55	37	40	42	56	57	42	50
			9.8	2.8	3.5	3.2	3.3	3.0	3.4	3.1	2.9	46	46	51	57	54	64	50	46
			11.7	3.0	2.7	3.1	2.6	2.0	2.7	2.4	3.9	52	42	41	38	41	70	44	44
19	Lindley silt loam	Des Moines, Iowa	1.1	0.6	0.8	0.7	0.7	0.6	0.7	0.8	0.8	38	24	26	24	28	25	32	38
			3.7	2.0	1.5	2.2	2.2	2.3	2.3	1.8	2.0	38	45	50	38	36	46	59	50
			5.7	2.2	3.0	2.2	2.3	2.3	2.3	2.0	2.1	46	50	48	48	58	59	58	50
			7.6	2.5	3.0	2.7	2.8	2.4	2.4	3.0	2.5	44	40	46	40	56	58	56	66
			9.7	3.0	3.2	3.2	2.9	3.1	2.9	3.1	3.0	62	51	61	55	56	78	65	68
			11.6	2.9	3.5	3.8	3.4	3.2	3.4	3.3	3.5	56	62	71	66	66	85	60	63
21	Marshall silt loam	Kansas City, Missouri	1.5	2.0	2.1	2.1	2.1	2.2	2.0	2.0	2.1	< 10	< 10	< 10	< 10	< 10	< 10	< 10	< 10
			4.0	2.5	2.6	2.9	2.4	2.8	2.9	2.9	2.5	61	40	50	63	60	39	48	41
			6.0	4.2	4.4	5.0	4.7	4.8	5.1	4.6	4.6	71	52	60	58	60	59	66	60
30	Muscatine silt loam	Davenport, Iowa	1.1	0.9	0.9	1.1	0.9	0.9	1.1	1.1	1.0	24	< 10	12	10	12	16	16	18
			3.6	1.1	1.2	1.2	1.2	1.4	1.3	1.4	1.2	< 20	< 20	< 20	< 20	< 20	< 20	< 20	< 20
			5.7	1.8	2.3	2.6	2.0	2.5	2.7	2.5	2.4	< 20	< 20	24	16	28	28	27	28
			8.2	4.1	4.7	4.1	4.2	4.7	4.0	4.4	3.8	46	36	33	34	62	35	48	31
			11.6	5.2	5.6	4.8	5.3	6.3	5.8	5.6	5.2	54	51	58	51	53	54	63	66
			17.0	6.1	5.9	5.7	5.4	6.0	6.0	6.9	6.4	50	44	52	42	53	64	76	65
44	Wabash silt loam	Omaha, Nebraska	1.1	0.3	0.6	0.4	0.5	0.4	0.3	0.4	0.4	38	36	28	32	26	39	32	38
			3.6	1.4	1.8	2.0	1.8	1.4	1.4	1.2	1.3	78	43	54	55	46	44	44	50
			5.7	2.3	2.2	2.3	2.4	2.1	2.2	2.0	2.0	70	52	51	66	56	72	60	68
			7.6	1.7	2.3	2.2	2.0	1.9	2.1	2.0	2.1	72	49	62	50	56	50	62	88
			11.6	2.9	4.1	4.7	3.5	3.4	2.8	3.4	3.2	87	56	63	69	65	58	82	74
46	Unidentified silt loam	Denver, Colorado	1.5	0.8	1.3	1.2	0.9	1.0	1.1	1.2	1.2	57	54	55	54	50	40	56	79
			4.0	2.7	3.2	2.9	2.6	2.4	2.6	2.7	3.2	80	64	79	82	66	58	106	110
			5.1	2.9	2.8	3.1	3.0	2.8	3.0	2.6	3.2	68	65	66	54	68	46	96	96
			8.0	5.6	6.2	5.2	5.7	5.7	5.9	5.8	6.7	60	80	108+	118+	69	68	136	134
			10.2	4.0	4.7	4.8	4.1	4.4	3.6	4.3	3.9	74	95	68	83	82	66	84	80
			12.0	4.0	5.1	4.5	4.4	4.7	4.3	4.8	4.8	48	62	64	104	77	62	114	80
101	Billings silt loam (low alkali)	Grand Junction, CO	1.9					5.2	3.9	3.9						66	70	60	
			4.1	-	-	-	-	8.8	7.5	7.2	-	-	-	-	-	94	116	94	-
			9.3					9.4	10.5	9.1						95	131	86	
102	Billings silt loam (moderate alkali)	Grand Junction, CO	1.9					5.1	3.9	3.9						37	42	26	
			4.1	-	-	-	-	10.2	9.4	7.2	-	-	-	-	-	80	102	72	-
			9.3					16.1	18.3	9.1						93	124	95	
103	Billings silt loam (high alkali)	Grand Junction, CO	1.9					5.0	3.7	3.6						48	62	37	
			4.1	-	-	-	-	10.4	11.2	10.1	-	-	-	-	-	86	88	66	-
			9.3					21.3	18.8	17.8						136	190	192	

Table 2.9: Loss in weight and maximum penetration of 6-in. cast-iron pipe (retrieved from Romanoff, 1957)

Soil		Location	Duration of exposure	Loss in weight (oz/ft^2)						Maximum penetration (mils)					
				Centrifugal process			Vertical cast in sand mold			Centrifugal process			Vertical cast in sand mold		
Test Site No.	Type of Soil		(years)	deLavaud C	deLavaud CCd	Mono-Cast I	Northern Ore L	Southern Ore Z	Southern Ore A	deLavaud C	deLavaud CCd	Mono-Cast I	Northern Ore L	Southern Ore Z	Southern Ore A
8	Fargo clay loam	Fargo, North Dakota	1.1	0.8	-	-	0.7	2.1	-	< 10	-	-	59	73	-
			3.8	1.9			2.7	5.4		34			76	61	
			5.8	3.4			5.0	5.7		64			91	81	
			7.7	5.6			5.0	7.5		107			85	78	
			9.9	8.4			10.5	9.6		142			217	169	
			11.8	16.8			20.3	30.8		179			240	239	
18	Knox silt loam	Omaha, Nebraska	1.2	0.1	-	-	-	-	-	< 10	-	-	66	< 10	-
			3.8	0.6			1.1	0.9		38			99	92	
			5.8	2.3			3.3	5.1		76			107	128	
			7.7	2.0			3.2	2.7		< 20			< 20	< 20	
			9.8	3.5			4.8	4.6		69			138	142	
			11.7	5.5			2.7	4.8		85			103	147	
19	Lindley silt loam	Des Moines, Iowa	1.1	0.4	-	-	0.6	0.6	-	16	-	-	29	< 10	-
			3.7	1.4			1.7	0.8		36			70	70	
			5.7	2.1			2.3	1.6		47			104	100	
			7.6	2.6			3.0	2.8		74			159	177	
			9.7	2.6			3.0	5.0		70			118	176	
			11.6	3.1			5.5	4.6		69			207	259	
21	Marshall silt loam	Kansas City, Missouri	1.5	1.6	-	-	2.4	2.1	-	17	-	-	< 10	< 10	-
			4.0	2.5			3.2	3.2		41			71	53	
			6.0	4.4			5.0	6.3		56			101	57	
30	Muscatine silt loam	Davenport, Iowa	1.1	1.2	-	-	1.2	1.3	-	< 10	-	-	< 10	< 10	-
			3.6	1.6			1.7	2.0		< 20			< 20	< 20	
			5.7	3.0			2.0	3.6		32			25	34	
			8.2	4.8			4.7	5.5		77			49	79	
			11.6	9.3			12.5	9.8		136			143	117	
			17.0	8.2			11.2	8.9		170			140	344	
44	Wabash silt loam	Omaha, Nebraska	1.1	1.7	-	-	0.3	1.0	-	< 10	-	-	< 10	30	-
			3.6	1.0			1.4	0.9		46			81	59	
			5.7	1.0			1.2	2.2		40			50	37	
			7.6	1.6			1.4	2.5		36			44	53	
			11.6	3.8			3.3	4.0		72			65	69	
46	Unidentified silt loam	Denver, Colorado	1.5	0.5	-	-	2.5	1.8	-	15	-	-	35	36	-
			4.0	2.2			4.0	5.5		< 20			55	53	
			5.1	2.2			2.3	3.6		26			29	70	
			8.0	5.6			5.3	6.6		50			63	104	
			10.2	4.5			4.0	8.1		54			38	86	
			12.0	4.2			5.6	8.1		68			67	102	
101	Billings silt loam (low alkali)	Grand Junction, CO	1.9	-	-	5.5	7.4	-	6.4	-	-	76	49	-	41
			4.1			7.9	8.2		8.2			103	128		61
			9.3			8.0	11.0		10.2			165	203		128
102	Billings silt loam (moderate alkali)	Grand Junction, CO	1.9	-	-	4.7	4.6	-	5.5	-	-	63	54	-	57
			4.1			9.2	9.0		14.4			99	90		130
			9.3			23.1	25.6		25.7			247	293		410
103	Billings silt loam (high alkali)	Grand Junction, CO	1.9	-	-	6.3	14.1	-	13.7	-	-	85	99	-	96
			4.1			28.1	14.6		42.8			215+	132		208
			9.3			45.2	42.4		58.6			215	361		418

2.6 Summary

Two backfills, concrete and aggregate, are considered for direct embedment. Concrete encases the steel and help to protect against corrosion. This is a similar situation for other reinforced concrete applications use with NDOT assets in corrosive soils. Aggregate backfill is permeable and groundwater from the in-situ soil contacts the steel. This situation is of more concern in corrosive environments. The typical backfill candidate is concrete and this is the best choice in corrosive sites. Electric utilities use concrete encasement, galvanization, and a protective mastic with success. A similar approach could be employed for the present work.

Table 2.10 provides a matrix of typical Nebraska soils and potential embedment options. The culvert casing is not recommended due to compaction issues associated with the corrugations at the soil-culvert. This was discussed previously in the constructability section.

Table 2.10: Tentative NDOT Applications for Direct Embedment Foundations

		Foundation Type			
		Earth Form Concrete Backfill	Earth Form Aggregate Backfill	Permanent Casing Form Concrete Backfill	Permanent Casing Form Aggregate Backfill
Soil Type	Loess and Silt	x	x		
	Sand			x	x
	Silty Clay	x	x		
	Silt				

3. HIGH MAST LOAD AND RESISTANCE

3.1 Introduction

This chapter provides an example of structural load calculations and the direct embedment foundation resistance calculations based on a typical 120- and 140-ft HMTs constructed in Nebraska. The structural loads were computed using the AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (2015) and a spreadsheet that was based on the fundamental principles of structural analysis (strain compatibility, material properties, and equilibrium condition). The geotechnical foundation resistance calculations were demonstrated through checking the vertical and horizontal capacities. The horizontal load carrying capacity was calculated based on the procedures introduced in the AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (2015). This method of analysis was developed by Broms (1964a and 1964b).

3.2 Structural Loads

3.2.1 General

The general drawings and the specifications of HMTs used by NDOT (Bushnell Tower Project Plans and Specs, 2015). The pole was divided into twenty stations where static shear and moments are computed assuming a uniform tape in diameter and thickness. Based upon the load effects, the resulting curvature is integrated to calculate rotation, and the rotation is integrated to calculate translation. The resulting translation is also used to compute the second-order load effects (P-Delta effect).

The following example provides the steps to calculate the base reaction (axial, shear, moment at the base) and tip displacement at the top of the tower for a 140-ft HMT with 12 luminaires which is the maximum possible for NDOT standards. The wind load in this example is calculated following the AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (2015). The Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (2015) (allowable stress method) was also used.

3.2.2 Dimensions and input parameters

The analysis was performed on a typical NDOT HMT make of steel galvanized tube that tapers at a rate of 0.14in/ft vertically. This rate is typical and is set in the manufacturing process. The pole is modeled with typical bending assumptions as outline above. The pole height above grade is L = 140 ft, the top diameter is D_{top} = 7.76 in., the bottom diameter is D_{bot} = 26.5 in., and the thickness at the top and bottom is t_{top} = 0.1875 in. and t_{bot} = 0.4375 in. The thickness typically varies from section to section (three or four are typical). But, for numerical analysis, the thickness was assumed to be varied at a constant rate which is more conservative regarding the load increase and simple to implement. The luminaire has an effective projected area (12 luminaires) EPA = 18.72 ft^2 weighing W = 3,450 lbs. NDOT specifications state 250 lbs per luminaire, twelve luminaires, additional 15% added for arms and other assemblies.

3.2.3 Wind load

Wind load shall be based on the pressure of the wind acting horizontally on the HMT as defined in Section 3.8 of the AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (2015).

Design wind pressure

$$P_z = 0.00256\, K_z\, K_d\, G\, V^2\, C_d \text{ (psf)}$$

where:

V is the basic wind speed (mph),

K_z is the height and exposure factor defined Article 3.8.4,

K_d is the directionality factor defined in Article 3.8.5,

G is the gust effect factor defined in Article 3.8.6, and

C_d is the drag coefficient defined in Article 3.8.7.

Basic wind speed

The LRFD wind maps use an extreme wind matched with an ultimate strength approach for resistance. AASHTO LRFD LTS 3.8-1b for a 700-year MRI (Mean Recurrence Interval) Basic Wind Speed for Nebraska is 115 mph. However, the basic wind speed is 114 mph for a 700-year MRI per the latest ASCE 7-16 which is adopted within AASHTO LRFD LTS. Therefore, we used 114 mph as the basic wind speed in this example.

Standard Specifications uses a wind speed of 90 mph with an allowable stress approach for resistance. Both specifications were used to determine the load effect at the top of the foundation (bottom of the pole). A brief commentary is provided:

AASHTO LRFD LTS 3.8-2b for a 1,700-year MRI (Mean Recurrence Interval) Basic Wind Speed for Nebraska is 120 mph. The load and resistance factors are calibrated to provide a reliability index of approximately 3.0 for 300-year MRI, 3.0 to 3.5 for 700-year MRI, and 3.5-4.0 for 1,700-year for main members. NCHRP Report 796 provides the details of this calibration (Puckett, et al. 2014)

Based on the fact that the basic wind speed is squared in calculating the design wind pressure, the basic wind speed used in this example will result in approximately 60% higher wind pressure than typical design calculations made following the current NDOT standard specifications (e.g. $(114/90)^2=1.6$); however the full strength limit is used rather than only a fraction of the strength per allowable stress design (standard specifications) The results should be essentially the same as the standard specification. A secondary analysis was also performed with the Standard Specification loads.

Height and Exposure Factor

The height and exposure factor shall be determined either from Table C3.8.4-1 or calculated using the following equation.

$$K_z = 2\left(\frac{z}{z_g}\right)^{\frac{2}{\propto}}$$

where:

z is the height above the ground at which the pressure is calculated, and

or 16 ft, whichever is greater

z_g is a constant that varies with the exposure condition and based on ASCE/SEI 7-16, it should be taken to be ***900*** ft for Exposure C

α is a constant that varies with the exposure condition and based on ASCE/SEI 7-16, it should be taken to be ***9.5*** for Exposure C.

Directionality Factor

The directionality factor is defined in Table 3.8.5-1 of AASHTO LRFD LTS Article 3.8.5. The values in the table are consistent with those from ASCE 7-16 as based upon work by Ellingwood (1981) and Ellingwood et al. (1982). Because the typical drawings of the HMT used in Nebraska is a hexdecagonal (16-sides) shape, the directionality factor is K_d = 0.95.

Gust Effect Factor

The gust effect factor, ***G***, shall be taken as a minimum of ***1.14***. Information presented in ASCE/SEI 7-16 states that if the fundamental frequency of a structure is less than 1 Hz or if the ratio of the height to least horizontal dimension is greater than 4, the structures should be designed as a wind-sensitive structure. Thus, the structures being considered in AASHTO LRFD LTS are all under this category and the gust effect factor shall be taken as a minimum of 1.14.

Drag Coefficient

The wind drag coefficient shall be determined from Table 3.8.7-1. According to Table 3.8.7-1, poles that have hexdecagonal (16-sides) shape with design wind speed higher than 78 mph, the drag coefficient, $\boldsymbol{C_d}$, is ***0.55***.

3.2.4 Flexural response

The pole is divided into twenty sections assuming a uniform taper rate in diameter. The calculated pole taper rate is $(D_{bot} - D_{top})/L$ = 0.134 in./ft which is slightly lower than the typical Valmont design with a 0.14 in./ft taper rate. Based on the input parameters and the pressure calculated following the wind load parameters of the AASHTO LRFD LTS (2015), lateral load,

P, is calculated at twenty stations. This lateral load accumulated towards the base to calculate the shear force, ***V***. Next, shear force at each location multiplied by the length of each station (total pole length divided by the number of stations) provides the moment, ***M***, at each station. Dividing the moment by stiffness, ***EI***, will provide curvature, $\emptyset$ at each station. This is possible based on the Bernoulli-Euler beam theory 'plane section remains plane', which gives the connection between curvature and strain produced at each station. With the material properties of steel, and the moment calculated at each station, strain can be calculated at each station and dividing the maximum strain at each station by the 'distance from the neutral axis to the location strain is being calculated' will provide curvature. In summary, using the statics (equilibrium conditions; arithmetic), material properties (constitutive behavior; stress-strain relationship), and the theory that plane section remains plane, the lateral load at each station was translated into curvature. Finally, from geometry (compatibility), multiplying curvature with the length of each station will provide rotation, θ, at each station. By adding up all the rotations, slope from the groundline can be calculated. In addition, rotation at each station multiplied by the distance between each station to the groundline will provide translation (displacement), Δ ,at each station. By summing all translations, the tip translation at the top of the tower can be calculated.

As a final step, the axial load including the self-weight of the pole at each station and the weight of the luminaire was multiplied with the translation at each station to take account the second-order effects that increases the moment at each station. The load factor for dead load was 1.25 and for wind load 1.0. Using statics, material properties, and geometry, the three fundamental principles for flexural analysis, the curvature, rotation, and translation was all re-calculated considering the P-Δ effects. The total moment at the base increased by 12% considering the second-order analysis. As a check, the moment magnifier calculated using the simplified approach in AASHTO LRFD LTS Article 4.8.1 is 1.273 which is a higher value and more conservative at the tower base. A typical moment diagram is illustrated in Figure 3.1 including the initial moment, second-order moment considering the P-Δ effects, and the AASHTO LTS magnified moments using the magnifier calculated by the simplified approach. The simplified approach provides higher moment below 60 ft but less moment above 60 ft than the second-order moment calculated from the structural analysis.

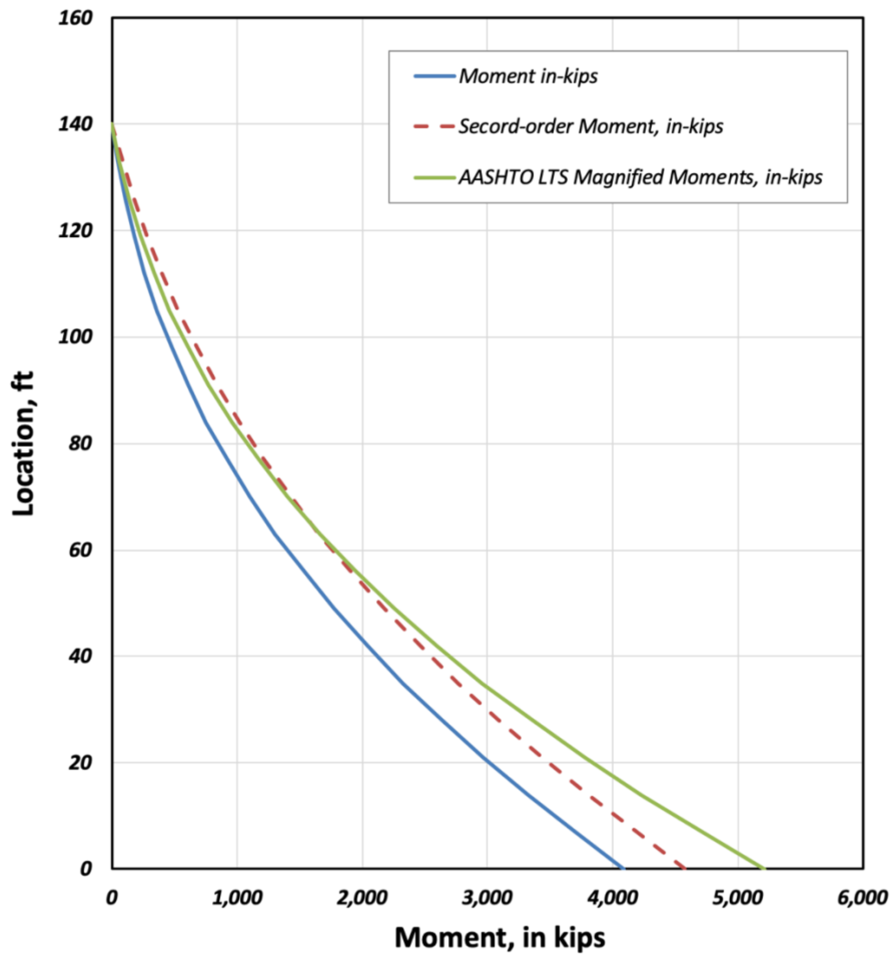

Figure 3.1: Typical Moment Diagram from Analysis

3.2.5 Base reactions from structural analysis

The following

Table 3.1 summarizes the base reactions including the axial load, shear force, moment at the base, and tip translation at the top of the 140-ft HMT. A typical moment diagram is illustrated in Figure 3.1.

Table 3.1: Calculated Base Reactions for a typical Nebraska 140-ft HMT

AASHTO LRFD LTS Base Reactions (these are LRFD values)		
Axial, kips	14.9	Adjust to ASD depending upon foundation design method
Shear, kips	**4.8**	
Moment, ft-kips	**382**	
Tip Translation, in	102	
Base stresses, ksi	20.4	Assumes circular section
Base moment magnification factor	1.12	

3.3 Foundation Resistance for Nebraska High Mast Towers

3.3.1 General

Foundation resistance calculations were conducted to check the capacity of the pile foundation for luminaire poles based on the combination of available information (e.g., structural information) and unavailable (e.g., geotechnical information). For the unavailable information, conservative engineering judgement was applied. In addition, the ultimate vertical and lateral resistance of the pile foundation were computed and compared with the factored axial force, shear, and moment.

3.3.2 Loads on foundation

Loading condition for this foundation is shown in Table 3.2. Primarily, the axial force and moment from the structural analysis was used for checking the vertical capacity of the foundation while shear and moment was used to check the horizontal capacity of the foundation in this example.

Table 3.2: Calculated Base Reactions for a typical Nebraska 120-ft and 140-ft HMT

NDOT Foundation Load Effects			
AASHTO LRFD LTS Base Reactions (these are LRFD values)			
Pole Height, ft	140	120	
Axial, kips	14.9	13.4	Adjust to ASD depending upon foundation design method
Shear, kips	4.8	4.0	
Moment, ft-kips	382	273	
Tip Translation, in	102	53	
Base stresses, ksi	20.4	14.6	Assumes circular section
Base moment magnification factor	1.12	1.09	
Natural Period, T_1, sect	5.0	4.0	

3.3.3 Load carrying capacity

The vertical load carrying capacity of the HMT foundation was calculated based on the following assumptions.

Foundation dimensions

Foundation diameter

D = 4 ft (assuming that the pole is embedded inside a concrete backfill, this diameter will allow 1 ft of working space additional to the typical diameter of steel poles used in Nebraska)

Foundation depth

L = 14 ft (10% of the height of the luminaire, a conservative assumption)

Net Load Carrying Capacity of a (single) pile:

$Q_{u,net} = A_p\, c_u\, N_c^*$

where,

A_p is the cross-sectional area of the shaft. $A_p = \frac{\pi D^2}{4}$= 12.57 (ft^2)

c_u is the undrained cohesion (=200 psf, assumed highly saturated cohesive soil for a conservative condition, Song et al., 2019) (ultimate strength)

N_c^* is the bearing capacity factor (=6.5 when c_u / P_a = 0.25, this magnitude of N_c^* is practically a low limit from Das, 2014) (ultimate strength)

P_a is the atmospheric pressure cross-sectional area of a pile

Therefore, $Q_{u,net}$ = (12.57)(200)(6.5)/1000 = 16.3 (kips)

The factored axial load from the structural analysis was 14.9 (kips)

< Net load carrying capacity = 16.3 (kips)

This calculation is based on the assumption of using a closed end pipe pile. In addition, the side friction of the embedded pole and the buoyance force are reviewed as follows:

Friction Capacity of a (single) pile:

$Q_{f,net} = A_s\, \alpha\, c_u$

where,

A_s is the surface area of the embedded pole, $A_s = \pi DL$ = 175.9 (ft^2)

α is the reduction factor = 0.9 (could be 1.0 but used 0.9 to be conservative)

c_u is the undrained cohesion (=200 psf, assumed highly saturated cohesive soil for a conservative condition, Song et al., 2019) (ultimate strength)

Therefore, $Q_{f,net}$ =(175.9) (0.9) (200/1000) = 31.6 (kips). The total vertical resistance becomes 47.9 kips with the sum of bearing capacity and friction capacity.

Buoyancy Check:

$B = A_p \gamma_w L$

where,

A_p is the cross-sectional area of the shaft. $A_p = \frac{\pi D^2}{4}$= 12.57 (ft^2)

γ_w is unit weight of water (62.4 lb/ft^3)

L is the embedded depth (14 ft in this example)

Therefore, B = (12.57) (62.4/1000) (14) = 11.0 (kips). Therefore, the buoyancy force is lower than the summation of the net load bearing capacity (16.3 kips) and the side friction force (31.6 kips). Since, there is sufficient side friction force that exceeds the factored axial load capacity and buoyancy, respectively, a 2 in. hole at the base of the pole can be drilled on the bearing plate to relieve the buoyancy force.

3.3.4 Horizontal load carrying capacity

The horizontal load carrying capacity was checked following the procedures introduced on the AASHTO LRFD LTS Article 13.6.1 Commentary. The procedure outline a procedure to compute the required shaft depth that provides sufficient lateral resistance for the given factored moment and shear.

<u>AASHTO LRFD LTS (2015) procedure</u>

Required embedment length

The AASHTO LRFD LTS (2015) Article 13.6.1 provides the approximate procedures for the estimation of embedment length in commentary. The method of analysis is based on procedures developed by Broms (1964a and 1964b). For cohesive soil, the required embedment length *L* can be determined as follows:

$$L = 1.5D + q\left[1 + \sqrt{2 + \frac{(4H+6D)}{q}}\right]$$

in which,

$H = \frac{M_F}{V_F}$, and

$q = \frac{V_F}{9cD}$ where,

D is the shaft diameter (ft)

c is the ultimate shear strength of cohesive soil (ksf)

M_F is the factored moment at ground-line (kip-ft)

V_F is the factored shear at ground-line (kip)

Based on the factored moment and factored shear provided from the structural analysis and the assumed shaft diameter of 4 ft,

H = (382/4.8) = 79.6 (eccentricity or equivalent height)

q = (4.8)/(9 x 0.2 x 4) = 0.67 (ksf)

Therefore, L = 1.5(4) + (0.67) x [1 + {2 + (4 x 79.6 + 6 x 4)/0.67}$^{1/2}$] = 21.85 -> Use 22 ft

Maximum moment in the shaft

$M_u = V_F(H + 1.5D + 0.5q)$ = (4.8) x (79.6 + 1.5 x 4 + 0.5 x 0.67) = 412.5 (k-ft)

Location of maximum moment below ground-line

$(1.5D + q)$ = (1.5 x 4 + 0.67) = 6.67 (ft) below ground-line

3.4 Summary

This foundation resistance calculation example demonstrates that the 4-ft diameter foundation with an embedment of 14 ft (10% of the total height of the HMT) which was the initial design values chosen may not satisfy the lateral resistance requirement for undrained cohesive soil (200 psf) based on Brom's graphical solution. Both the graphical solution and the AASHTO LRFD LTS procedures demonstrated that the embedment length should be increase approximately up to 17% of the total height (24 ft embedment length for the 140-ft pole). However, this example is based on the conservative soil condition (reduced capacity) where cohesive soils are used and with different soil conditions, the required embedment length can change. Further numerical analysis with commercial software was conducted to perform parametric study with varying soil conditions. The diameter can also be increased to satisfy the strength required.

4. FOUNDATION SYSTEMS

4.1 Introduction

This chapter presents the results of the parametric study that was conducted using LPILE and COMSOL with varying conditions. Based on the numerical analysis, various foundation systems with different soil conditions were suggested for direct embedment options that NDOT may choose for future projects. For the parametric studies, service level base moment and shear from the HMT were used.

4.2 LPILE Analysis

4.2.1 Input parameters

A round concrete shaft with permanent casing and core/insert as shown in Figure 4.1 was used to model the steel HMT directly embedded in soil with concrete backfill. The embedment length of 24 ft was selected from the load and resistance calculations in the previous chapter. The steel section, casing, and core/insert material properties can be selected to represent the pole directly embedded in soil with concrete backfill. The casing outside diameter and casing wall thickness were selected based on the outside diameter of the concrete backfill and thickness of the concrete backfill (subtracting outside diameter with the average steel pole diameter), respectively. The core diameter and core wall thickness values were selected based on the average diameter of the steel pole, and average thickness of the steel pole between the diameter and thickness at the ground-line and bottom of the pole at the 24 ft embedment base. The outside diameter of the concrete backfill was 48 in., the thickness of the concrete backfill was 23 in., the average diameter of the steel pole used in this analysis was 24.9 in., and the average thickness of the steel pole used was 0.4161 in. based on the properties received from the typical cross sections.

The material properties of the steel section, casing, and core/insert is shown in Table 4.1. The model was also prepared to not have concrete filled in the core for numerical analysis for the second run. The first run of simulation was to test out the conditions of the actual site conditions while the second run of simulation was conducted to check whether the stiffness of pole or backfill material affects the behavior of different foundation systems.

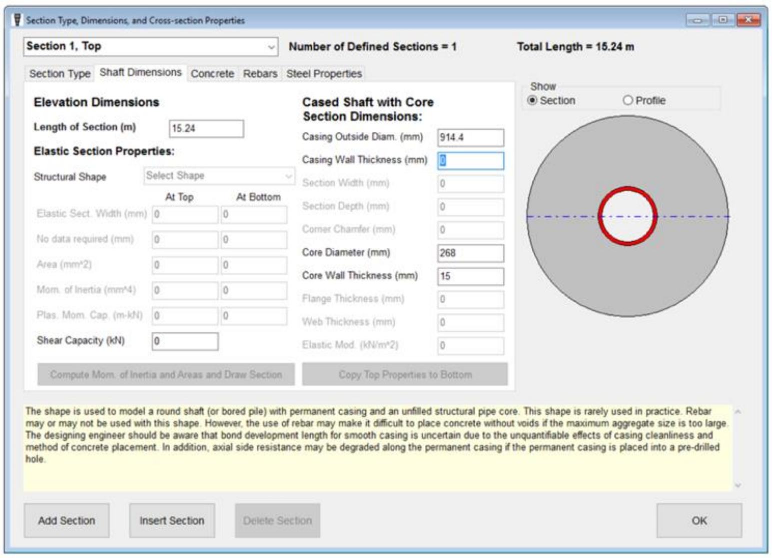

Figure 4.1: Screen Capture of Figure 3.19 from LPILE Manual (Tab Sheet for Shaft Dimensions of Drilled Shaft with Casing and Core to represent the Direct Embedment Foundation)

Table 4.1: Input Material Properties for LPILE Analysis

<table>
<tr><th>Material Properties</th><th>First Run
(Round Concrete Shaft with Steel Core)</th><th>Second Run
(Elastic Body: To check how the soil LPILE system behaves for different modulus, embedded depth, and pile diameter)</th></tr>
<tr><td>Modulus of Steel Pole (ksi)</td><td>29,000</td><td rowspan="2">First Trial: 36,000
Second Trial: 3,600</td></tr>
<tr><td>Modulus of Concrete Backfill (ksi)</td><td>3,600</td></tr>
<tr><td>Cohesion of Soil (psf)</td><td>200</td><td>200</td></tr>
<tr><td>Yield Stress of Steel Pole (ksi)</td><td>60</td><td>Elastic</td></tr>
<tr><td>Yield Stress of Concrete Backfill (ksi)</td><td>4</td><td>Elastic</td></tr>
</table>

4.2.2 Behavior of lateral piles with round concrete shaft and steel casing

The service level unfactored load conditions applied in this analysis was 245 ft-kips moment and 3.1 kips shear force at the pile head. For the 140-ft tall HMT with a 28 ft embedment depth and 4 ft diameter clayey soil with 200 psf cohesive, the deflection computed at the ground-line of the embedded foundation was 0.6 in. The deformed shape is linear as shown in Figure 4.2 indicating that the pile behaved close to a rigid body. This also indicates that the behavior of lateral pile is governed by the surrounding soil (soft clay with 200 psf cohesive). Such behavior is known as the condition of “short pile”.

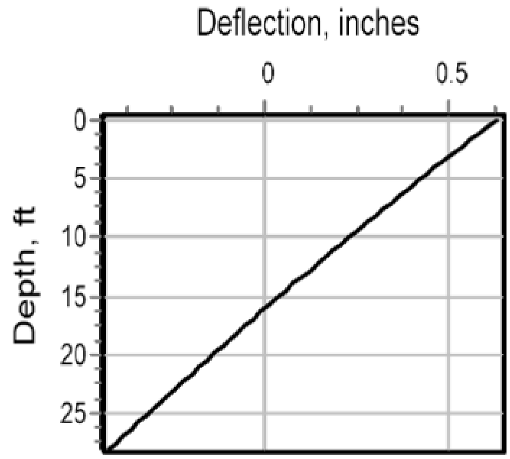

Figure 4.2: Deflection of the embedded steel pole based on LPILE analysis

The shear force and bending moment diagram from the LPILE analysis is shown in Figure 4.3. The service level shear force and the bending moment applied at the pole head at the ground-line can be observed in both plots. The change in shear force direction occurs at 16 ft below the ground-line. The maximum bending moment is observed at 4 ft depth from the pile head. There is approximately 3 ft difference from the maximum bending moment location calculated using the AASHTO LRFD LTS procedures in the previous chapter. But the calculations in the previous chapter were based on factored loads and the loads in the LPILE analysis was based on service loads which will cause the difference. The bending moment diagram on Figure 4.3 demonstrates the applied moment at the pile head and a gradual decrease due to soil resistance, and zero bending moment at the pile base as expected for a “short pile”.

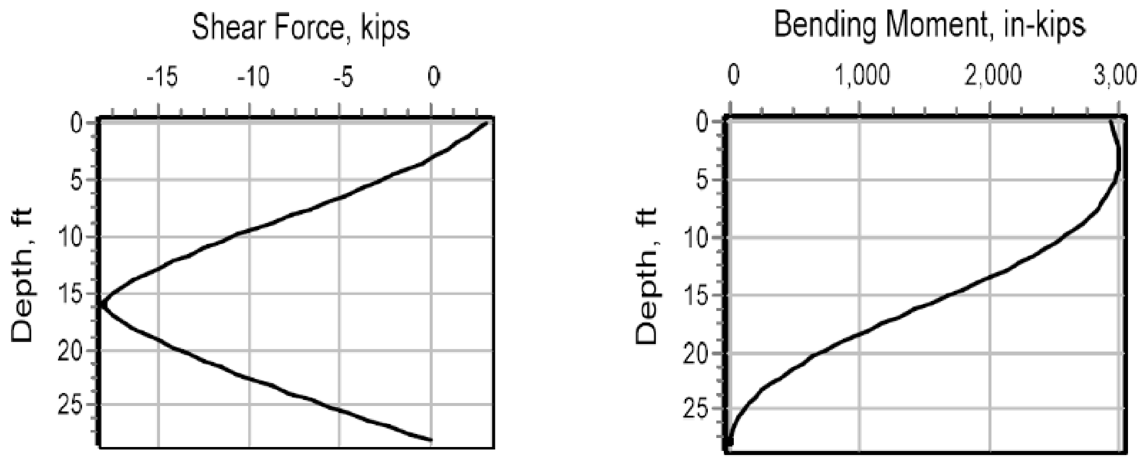

Figure 4.3: Shear Force and Bending Moment Profile for the embedded pole

The deflection of the pile head for different embedment length is plotted in Figure 4.4. As shown in Figure 4.4, the top head deflection at the ground-line converges to less than 1 inch when the pile length reaches 24 ft embedment length. This indicates that the selected 28 ft embedment length will provide ground-line deflections less than 1 in.

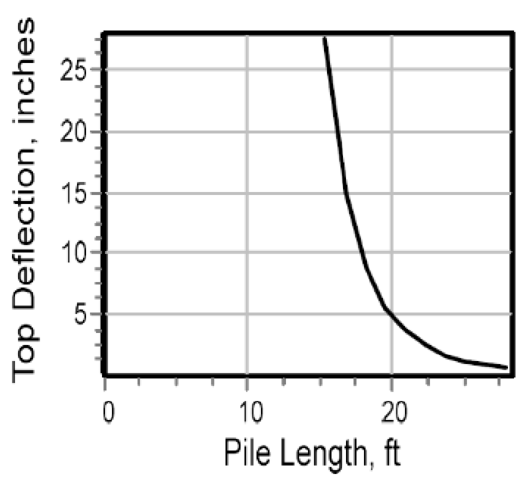

Figure 4.4: Deflection of the embedded steel pole based on LPILE analysis

4.2.3 Effect of modulus of piles

As discussed in Table 4.1, in order to check the soil-pile system behavior for different modulus, embedded depth, and pile diameter, a second set of numerical analyses was conducted with material properties shown in Table 4.1. The input value varied from the modulus of 3,600 ksi to the 36,000 ksi. The left figure on Figure 4.5 is showing the deflection profile for the given service load conditions for the lower modulus while the right figure on Figure 4.5 shows the deflection profile for the higher modulus. As shown in Figure 4.5, there are negligible effect of modulus on the behavior of these lateral piles for the 200 psf cohesive soil. Therefore, a parametric study with a solid column for various diameters and embedment depth were conducted.

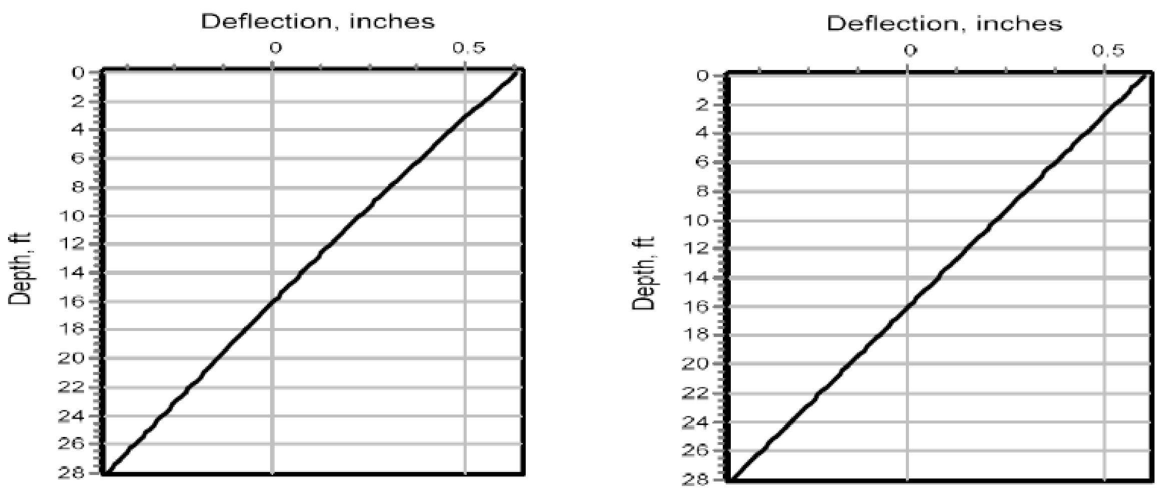

Figure 4.5: Shear Force and Bending Moment Profile for the embedded pole

4.2.4 Parametric study for various foundation systems

For the parametric studies, the service loads (moment and shear) for the two typical HMTs (140 ft and 120 ft) were considered. The horizontal displacement at the ground-line for eight different embedment depth 28 ft, 26 ft, 24 ft, 22 ft, 20 ft, 18 ft, 16 ft, and 14 ft were analyzed. Concrete backfill was used with five different types of natural soil surrounding the backfill was studied: 200 psf cohesive clay, 400 psf cohesive clay, 600 psf cohesive clay, 30-degree (friction angle) sand, and 35-degree sand. Two different backfill diameter was considered: 4 ft and 3 ft. This backfill diameter was chosen because the typical diameter of the 140-ft HMT constructed in Nebraska is approximately 2 ft at the base and 6 in. to 1 ft working space for concrete placement surrounding the 2 ft pole will result in 3 ft or 4 ft backfill. Table 4.2 summarizes the various foundation systems analyzed for the 140-ft HMT. The parametric study started with a 28-ft embedment length which was the required length calculated based on the AASHTO LRFD LTS procedures introduced in Chapter 3. For the worst condition with 200 psf cohesive clay surrounding the backfill, the ground-line deflection at the surface (at the head of the pile) was 0.6 in. for 245 ft-kips moment and 3.1 kips shear (both surface level loads) applied at the pile head. As the embedment length decreased to 24 ft, the ground-line deflection was larger than twice the displacement shown for the 28-ft case (1.5 in. displacement for the 24 ft, and 0.6 in. for the 28 ft case). The analysis was conducted again for the case with 400 psf cohesive clay. There the embedment length can be decreased to 22 ft from 28 ft with a relatively better cohesion clay surrounding the backfill. Comparable ground-line deflection (0.55 in.) was observed for the 22-ft embedment length pile surrounded by the 400 psf cohesive soil with the case with 28-ft embedment when 200 psf clay is surrounding the backfill. The parametric study was further extended to a 600 psf cohesive clay and the analysis demonstrates that the direct embedment length can be decreased down to 16 ft to have comparable level of ground-line deflection (0.7 in.) with the worst-case scenario (0.6 in. with 200 psf cohesive clay) as the cohesion increased three times the worst case.

The parametric study was conducted for the 30-degree friction angle sand that showed much less ground-line deflection for the 28-ft embedment length with the identical loading conditions (approximately 1.33% of the deflection calculated for the 200 psf cohesive clay). This is expected because sand typically have higher modulus than clay. With the 30-degree sand, the embedment length can be decreased to 16 ft while still having less than half of the ground-line deflection (0.24 in.) for the 28 ft embedment length with 200 psf cohesive clay (0.6 in.). The friction angle was increased up to 35-degree sand (stiffer sand) which demonstrated that the deflection decreases to less than half of what was calculated for the identical embedment of 16 ft with a 4-ft diameter backfill to 0.09 in. With such a small displacement, a smaller diameter backfill was included to check the feasibility of having a smaller pile diameter. The backfill was decreased from 4 ft to 3 ft. With the 16 ft embedment (this is 10% of the tower height + 2 ft), the backfill diameter of 3 ft with a 35-degree sand had 0.14 in. ground-line displacement. With the 3 ft backfill diameter, the study further investigated the ground-line displacement with 30-degree sand and 200 psf cohesive clay. With the 30-degree sand, the 3-ft backfill diameter pile with 16-ft embedment length resulted in 0.22 in. ground-line displacement (approximately 1/3 of the worst-case scenario with 200 psf-clay, 4 ft backfill, and 28 ft embedment length). One additional analysis was conducted for the 3-ft backfill with the 200 psf clay with 24-ft embedment length, and the computed ground-line displacement was 2.2 in. (approximately 50% more than the identical case with 4-ft diameter backfill).

The parametric study for a 120-ft tower was additionally conducted with the service level moment of 177 ft-kips and 2.6 kips shear. The ground-line horizontal displacements calculated from the analysis is shown in Table 4.3. With the loads decreased with a shorter tower height, only the 3-ft backfill option was considered. With the worst case of 200 psf cohesive soil, 28-ft embedment length was the only case that provided ground-line displacement less than 0.6 in. However, as the cohesion was increased to 400 psf, the embedment was decreased to 20 ft providing 0.55 in. ground-line displacement at the pile head.

Table 4.2: Parametric Study for Various Foundation Systems (140-ft Tower)

Case (Pole Height – Embedment Length – Pile Diameter - Shear Strength – Soil Type)	Ground-Line Displacement (in.)
140’-28’-4’-200 psf-clay	0.60
140’-26’-4’-200 psf-clay	1.10
140’-24’-4’-200 psf-clay	1.50
140’-28’-4’-400 psf-clay	0.18
140’-24’-4’-400 psf-clay	0.34
140’-22’-4’-400 psf-clay	0.55
140’-22’-4’-600 psf-clay	0.11
140’-20’-4’-600 psf-clay	0.20
140’-18’-4’-600 psf-clay	0.28
140’-16’-4’-600 psf-clay	0.70
140’-28’-4’-30 degree-sand	0.08
140’-24’-4’-30 degree-sand	0.11
140’-22’-4’-30 degree-sand	0.12
140’-20’-4’-30 degree-sand	0.15
140’-18’-4’-30 degree-sand	0.18
140’-16’-4’-30 degree-sand	0.24
140’-16’-4’-35 degree-sand	0.09
140’-16’-3’-35 degree-sand	0.14
140’-28’-3’-30 degree-sand	0.16
140’-24’-3’-30 degree-sand	0.16
140’-20’-3’-30 degree-sand	0.20
140’-16’-3’-30 degree-sand	0.22
140’-24’-3’-200 psf-clay	2.20

Table 4.3: Parametric Study for Various Foundation Systems (120-ft Tower)

Case (Pole Height – Embedment Length – Pile Diameter - Shear Strength – Soil Type)	Ground-Line Displacement (in.)
120’-28’-3’-200 psf-clay	0.45
120’-24’-3’-200 psf-clay	1.00
120’-22’-3’-200 psf-clay	2.50
120’-28’-3’-400 psf-clay	0.13
120’-24’-3’-400 psf-clay	0.21
120’-22’-3’-400 psf-clay	0.26
120’-20’-3’-400 psf-clay	0.55
120’-20’-3’-600 psf-clay	0.14
120’-18’-3’-600 psf-clay	0.20
120’-16’-3’-600 psf-clay	0.41
120’-14’-3’-600 psf-clay	0.90
120’-24’-3’-30 degree-sand	0.09
120’-20’-3’-30 degree-sand	0.13
120’-16’-3’-30 degree-sand	0.16
120’-14’-3’-30 degree-sand	0.22
120’-14’-3’-35 degree-sand	< 0.22

With the 600 psf cohesive soil, it was found that 10% of the tower height + 2 ft with a 3 ft diameter backfill will still provide ground-line displacement less than 0.5 in. With 30-degree or 35-degree sand the 120-ft HMT with 3-ft diameter backfill with an embedment length of 10% of the tower height will only produce approximately half of the displacement (0.22 in. or less) observed in the worst-case scenario with 200 psf cohesive clay as the surrounding soil.

4.3 COMSOL Analysis

4.3.1 Input parameters

Geometry and Loads

Because of the sensitive nature of soil-structure interaction and the possibility of encountering weak clay soil in eastern Nebraska, a finite element model was developed as an independent check on the LPILE and Brom's analysis.

In this analysis, the steel pole was modeled as tube that has the height above grade equal to the eccentricity associated with the service load effects, M_{base} = 245 ft-kips and V_{base} = 3.1 kips. The equivalent load effects are P_{tip} = 3.1 kips applied at L_{beam} = 245/3.1 = 79 ft.

The pole is surrounded by a "backfill" that is either concrete or large aggregate, see the green domain in Figure 4.6, which is surrounded by soil that is assumed to be either a soft cohesive clay or sand (cohesionless).

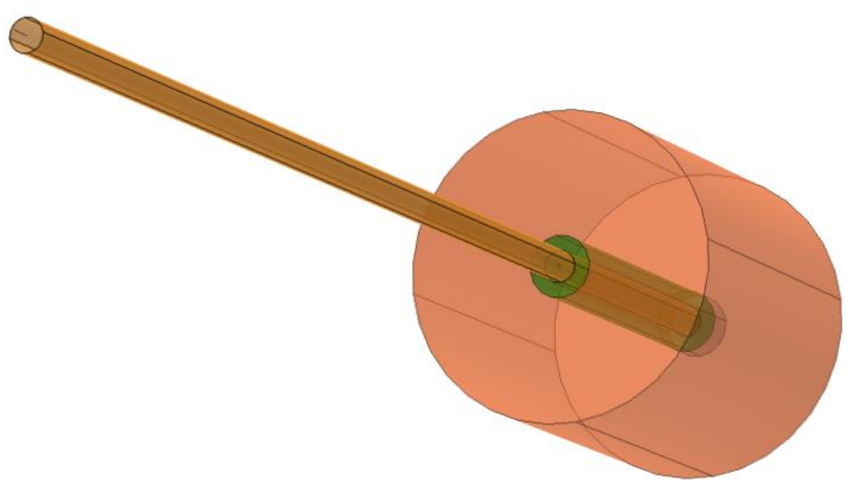

Figure 4.6: Domain Geometry

The base geometry properties are provided in Table 4.4. Figure 4.7 provides a close up of the tube, backfill, and soil interfaces. Figure 4.8 provide the detail of the bottom of the domains. Note that the pole and the backfill do not extend to the bottom of the soil domain thereby permitting movement. The soil domain base boundary condition is considered fixed; however, the depth below the pole/fill is sufficiently large to obviate boundary effects associated with stiffening the pole's base boundary. This is also illustrated in elevation in Figure 4.9. The pole was modeled with solid elements, this was a matter of convenience as its role is to: provide proper load effect to the fill and provide proper stiffness within the fill. Note that the fill, whether modeled as aggregate or concrete is significantly stiffer than the surrounding soil. This is illustrated below in the analysis results that show near rigid body behavior.

Table 4.4: Geometric Properties for COMSOL model

Geometric Dimension	**Value**	**Comment**
r1	12[in]	inside tube radius
thickness	0.5 [in]	tube thickness
r2	r1+thickness	outside tube radius
r3	24[in]	fill radius (concrete or aggregate)
r4	5*r3	soil radius, large to avoid boundary effects
PoleBottom	2[ft]	distance of bottom to model boundary, large to avoid boundary effects
h2	24[ft]	Foundation depth (model depth)
h1	h2-PoleBottom	Pole depth
r5	r4+1[ft]	Outer radius for infinite model
h3	Lbeam	Pole ht above fill

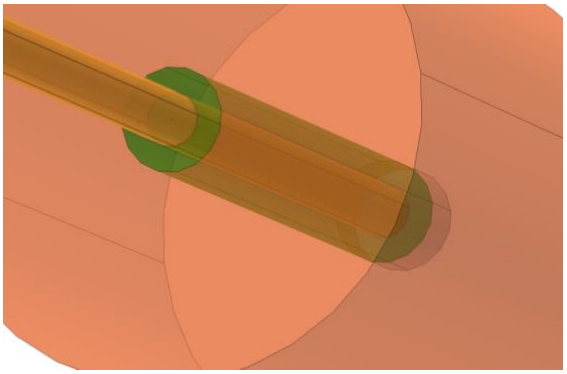

Figure 4.7: Close Up of Tube, Fill, and Soil Domains

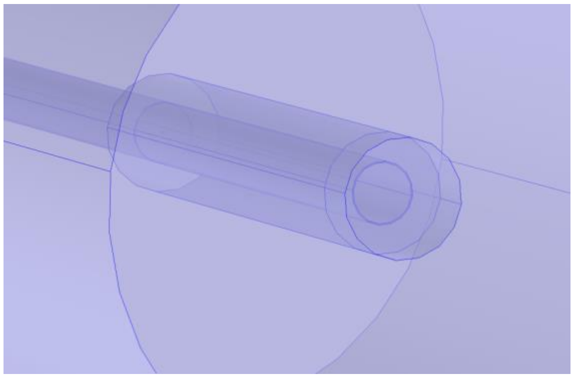

Figure 4.8: Bottom of Domain

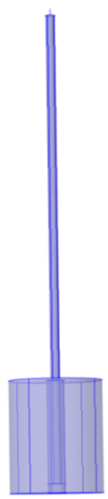

Figure 4.9: Elevation

Material Properties

The material properties are provided in Table 4.5. The clay properties are listed first and the *EsoilSoftClay* parameter was used in the analysis. Sand, concrete, and steel properties are listed, respectively.

Table 4.5: COMSOL Material Properties

Property (variable)	**Value**	**Typical Range**
Clay		
CohensionClay	(200/144) [psi]	12-384 kPa
EsoilClay	20[MPa]	20-200 MPa
InternalFrictionClay	10[deg]	
DilitationAngleClay	SmallNumberDeg[deg]	make small number
UniaxialCompressionClay	CohensionClay	
UniaxialTensionClay	CohensionClay	
EsoilSoftClay	3 [MPa]	1-3MPa
Sand		
EsoilSand	10[MPa]	10-50 MPa
InternalFrictionSand	30[deg]	30-40 deg
CohensionSand	1 [psi]	
TensionMaxSand	CohensionSand	
Concrete		
ConcreteCompressionStrength	4000[psi]	
ConcreteTensileStrength	ConcreteCompressionStrength/10	
ConcreteGamma	145[lbf/ft^3]/g_const	
Econcrete	3600000[psi]	
Steel		
SteelGamma	490[lbf/ft^3]/g_const	
Esteel	29000000[psi]	

4.3.2 Analysis (Clay Soil)

First, the results from the finite element model for a soft clay is presented. This is likely the most critical application for the embedment design. The sandy soil is presented thereafter.

Figure 4.10 illustrates the tip-loaded pole and the von Mises stresses in the pole. These stresses are of little consequence in this analysis as the pole is designed and checked using the AASHTO Specification. However, the bending behavior certainly is as expected. Additionally, statics checks were performed to ensure that the applied load at the tip is appropriately balanced by the reactions of the soil domain; this included horizontal forces and overturning moment. Note the load factor illustrated in the following plots is *Lfactor* = 1.8 times the stresses due to the expected service loads, e.g., Figure 4.10.

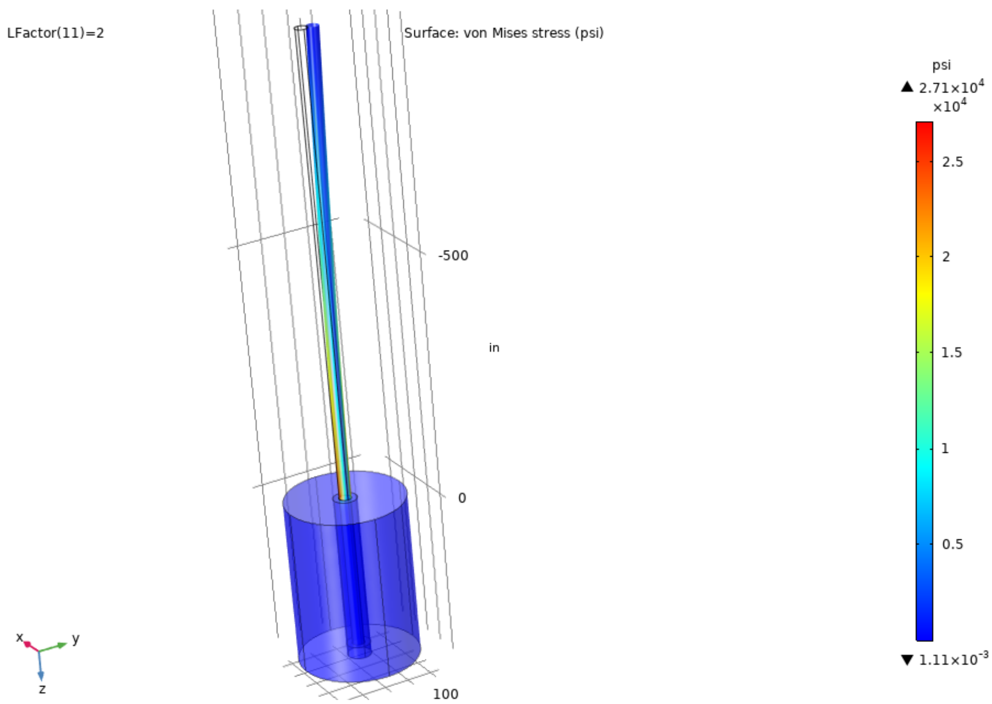

Figure 4.10: Pole Von Mises Stresses (Clay)

The displacement field is shown in Figure 4.11. Note the top of the pole moves in the direction of the load and the bottom of the pole moves in the opposite direction. Both translations are distributed vertically along the pole and decrease to a center of rotation. See Figure 4.12. Note that the translation line is linear, therefore the pole and fill act as a nearly rigid body as compared to the deformation of the soil. This is as expected and consistent with LPILE and Brom's analyses. The typical stress field for normal stresses is shown in Figure 4.13.

Figure 4.14 provides a typical load vs. translation plot. Note that some nonlinearity exists indicating that the soil is experiencing plastic behavior. Figure 4.15 illustrates a Boolean plot where red indicates locations plastic strains exist which are primarily near the top of the fill and near the bottom of the file. These areas create a force couple that resist the overturning moment. The magnitudes, however, are small as shown in Figure 4.16.

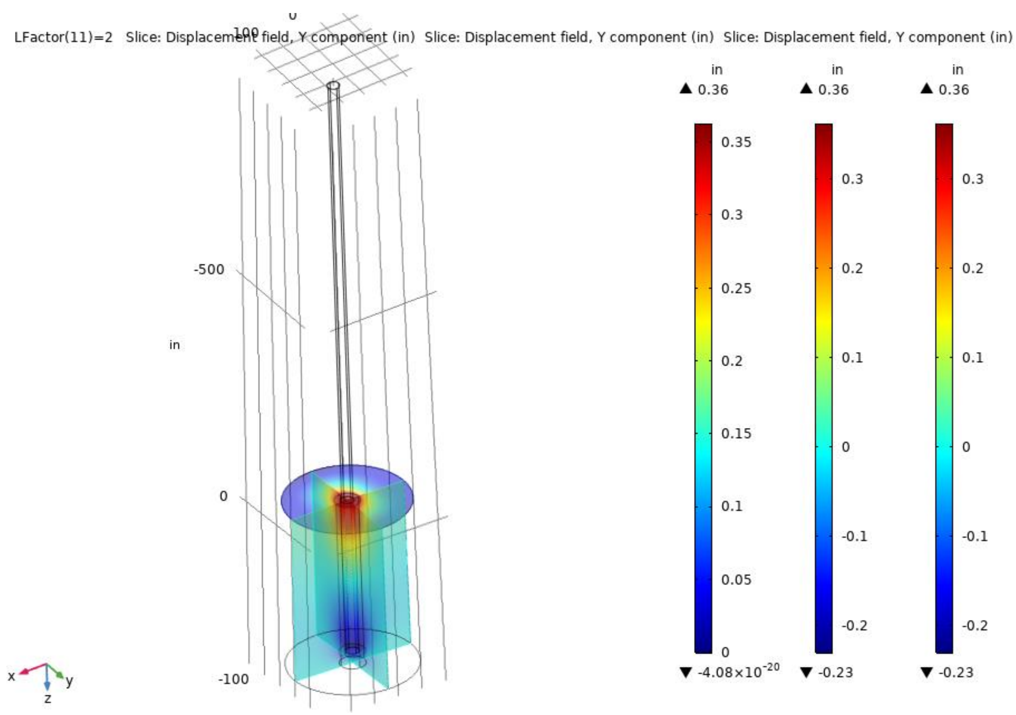

Figure 4.11: Translation in the direction of load

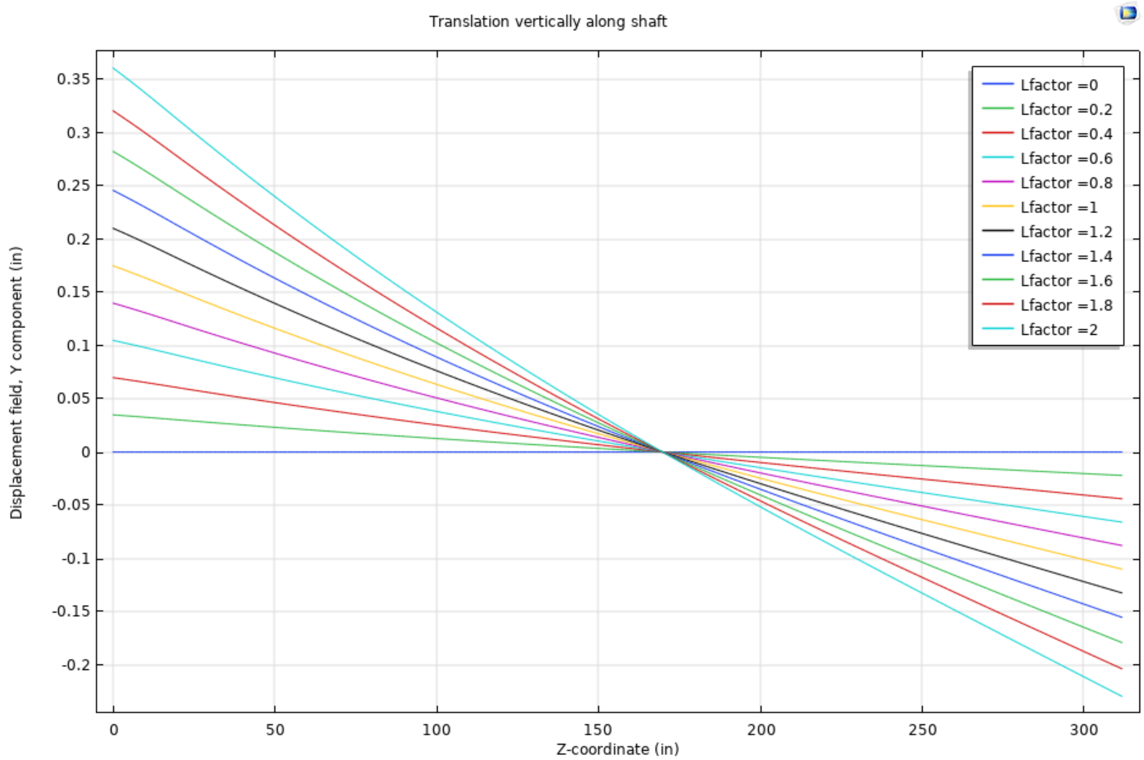

Figure 4.12: Translation along the fill edge (Clay)

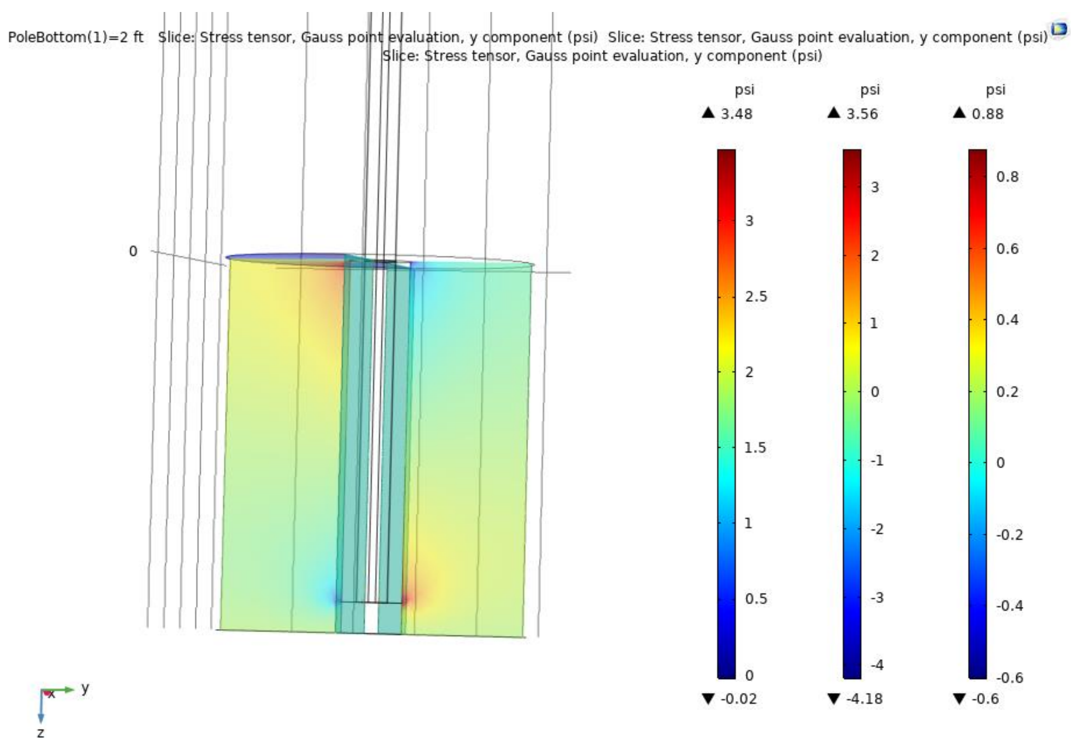

Figure 4.13: Normal Stresses in the direction of loading (Lfactor = 1, Clay)

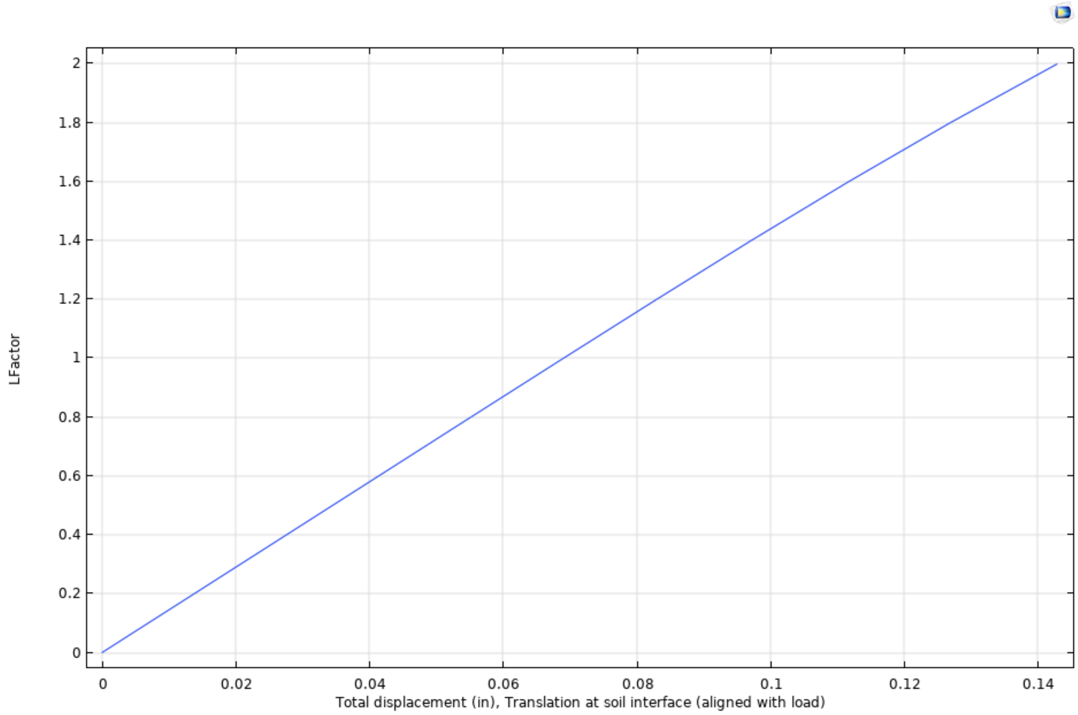

Figure 4.14: Load-translation plot (Clay)

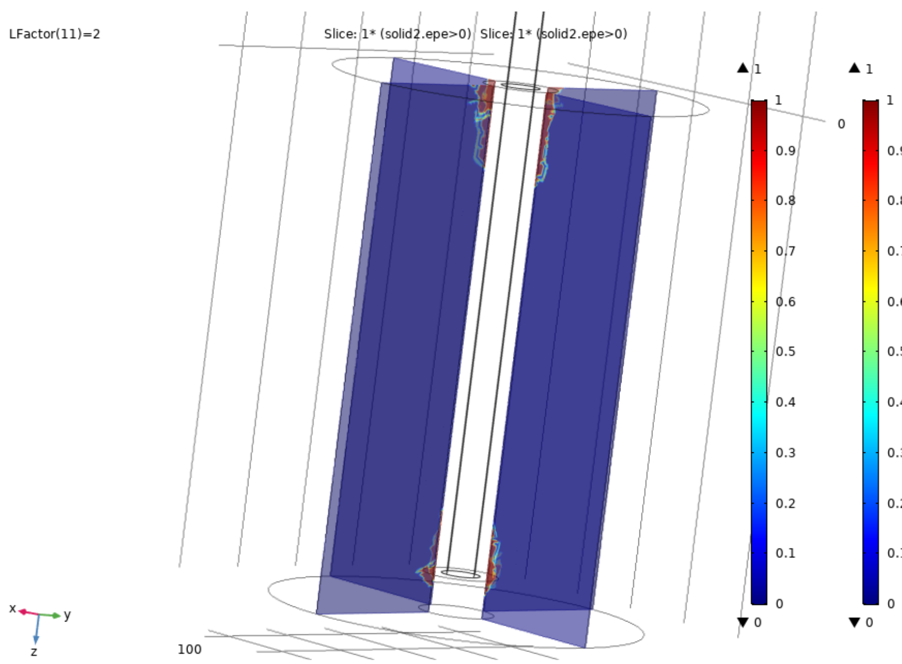

Figure 4.15: Boolean plot where plastic strains exist (red = yes, blue = no)

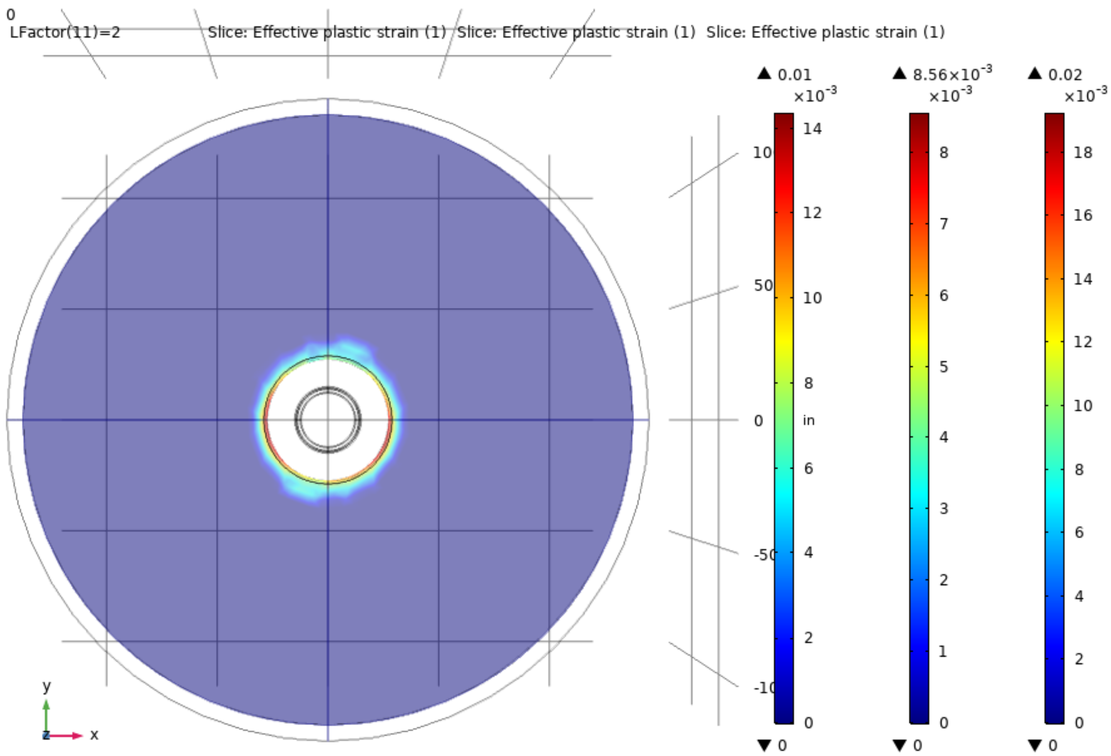

Figure 4.16: Effective Plastic Strain

The simulation was rerun at *Lfactor* = 1 several times, while changing the soil modulus of elasticity, *Esoil* over a range to values. The translation profile is plotted for each analysis. As expected higher *Esoil* the smaller the displacements. Note that as the soil become stiffer relative to the tube and fill, a slight nonlinear translation, i.e., flexing of the pole/fill is illustrated. In all cases, the center of rotation is approximately 150 in = 12.5 ft from the top of grade, or about 0.625 of the shaft depth.

The simulation was rerun by changing the distance from the bottom of the domain to the bottom of the fill, i.e., this decreases that shaft depth. The results are shown in Figure 4.17 and Figure 4.18. As expected the load effects in the soil increase with a shorter shaft; however, results are promising that both shaft depth and diameter could be optimized in the future based upon a refined analysis in Comsol and/or LPILE analysis.

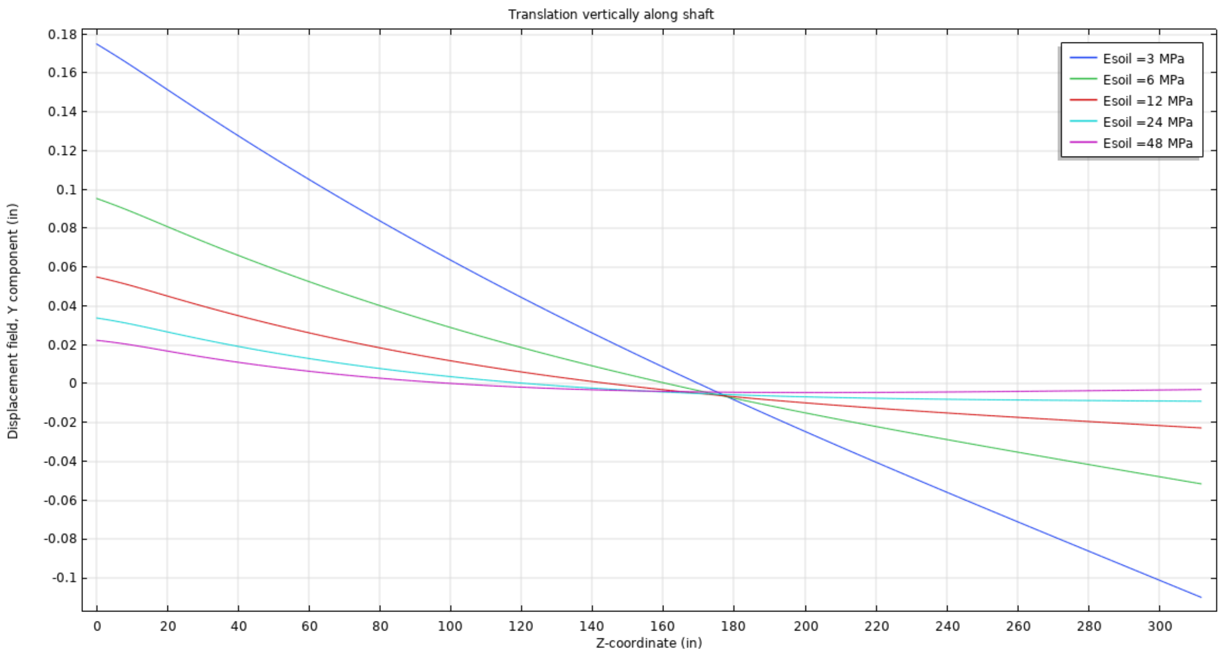

Figure 4.17: Parametric Study Esoil (Lfactor = 1, Clay)

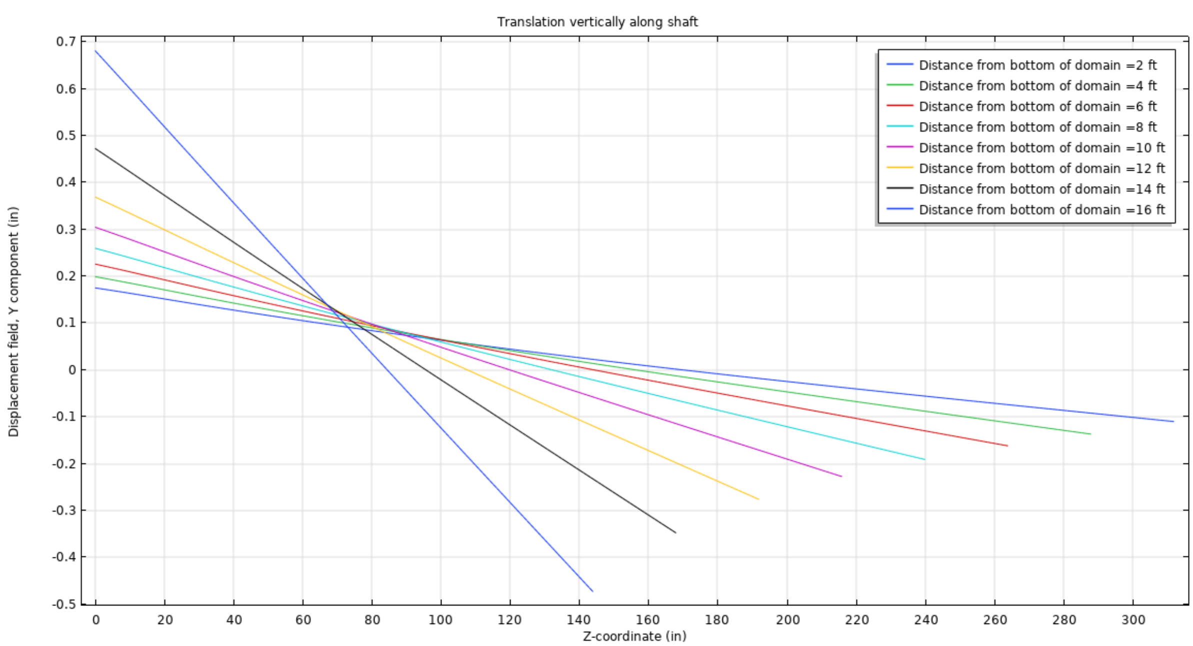

Figure 4.18: Parametric Variation Shaft Depth = 28-ft distance from bottom (Lfactor = 1, Clay)

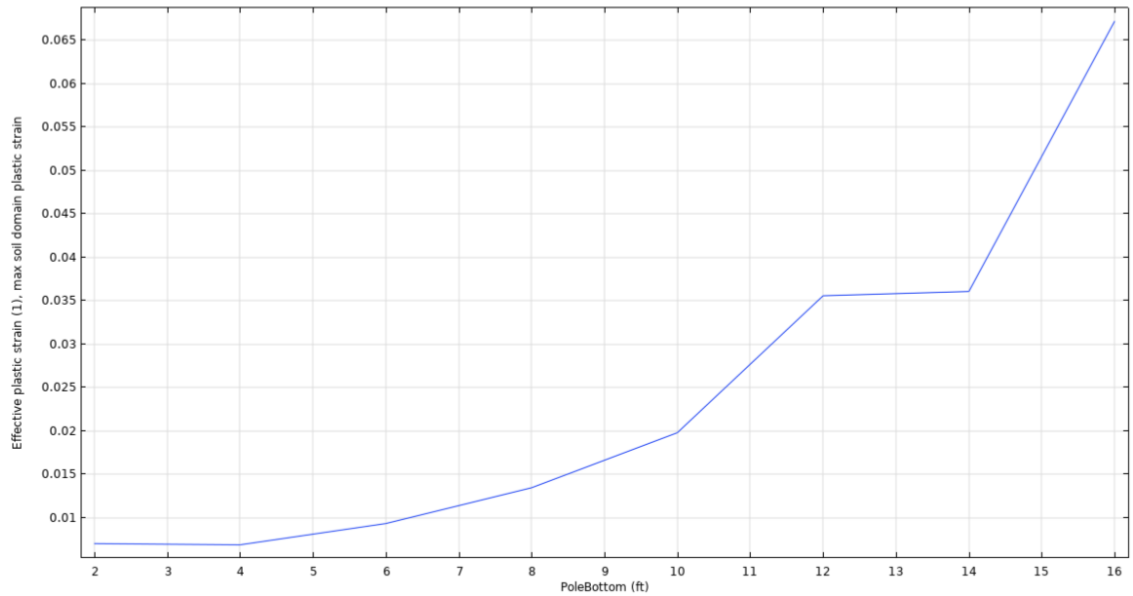

Figure 4.19: Pole Bottom vs Effective Plastic Strain (Shaft Depth = 24 ft – Pole Bottom, Clay)

4.3.3 Analysis (Sandy soil)

The model was changed to a sandy soil with other properties remaining the same. Figure 4.20, Figure 4.21, Figure 4.22 illustrate the results. There were small and localize plastic strains.

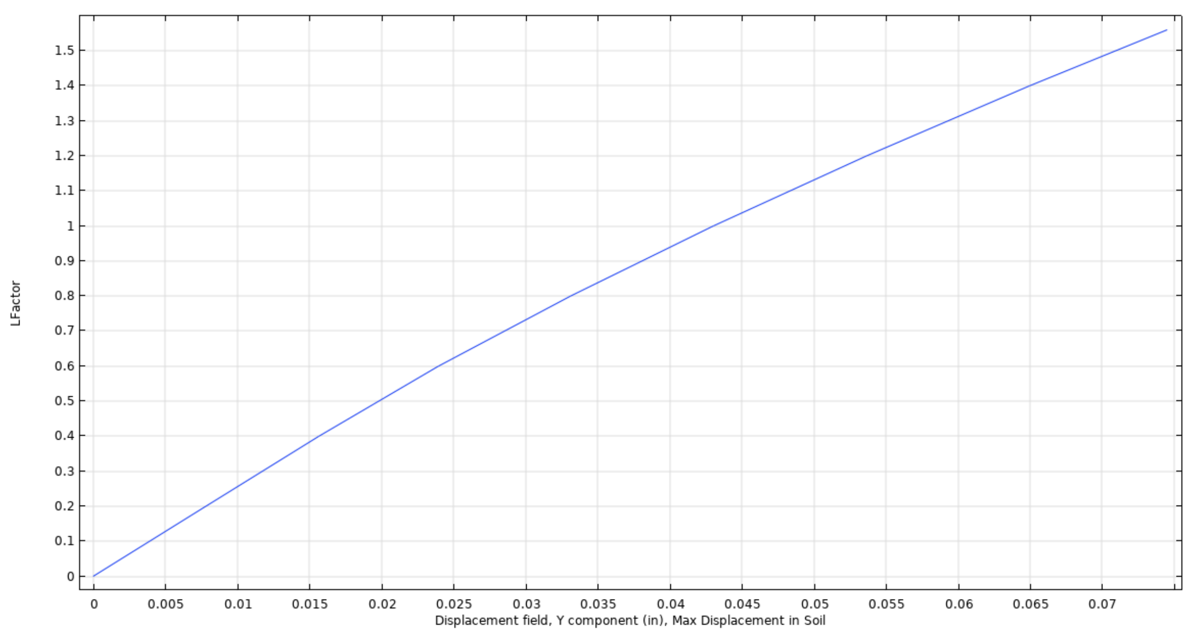

Figure 4.20: Load vs Max Translation (sandy soil)

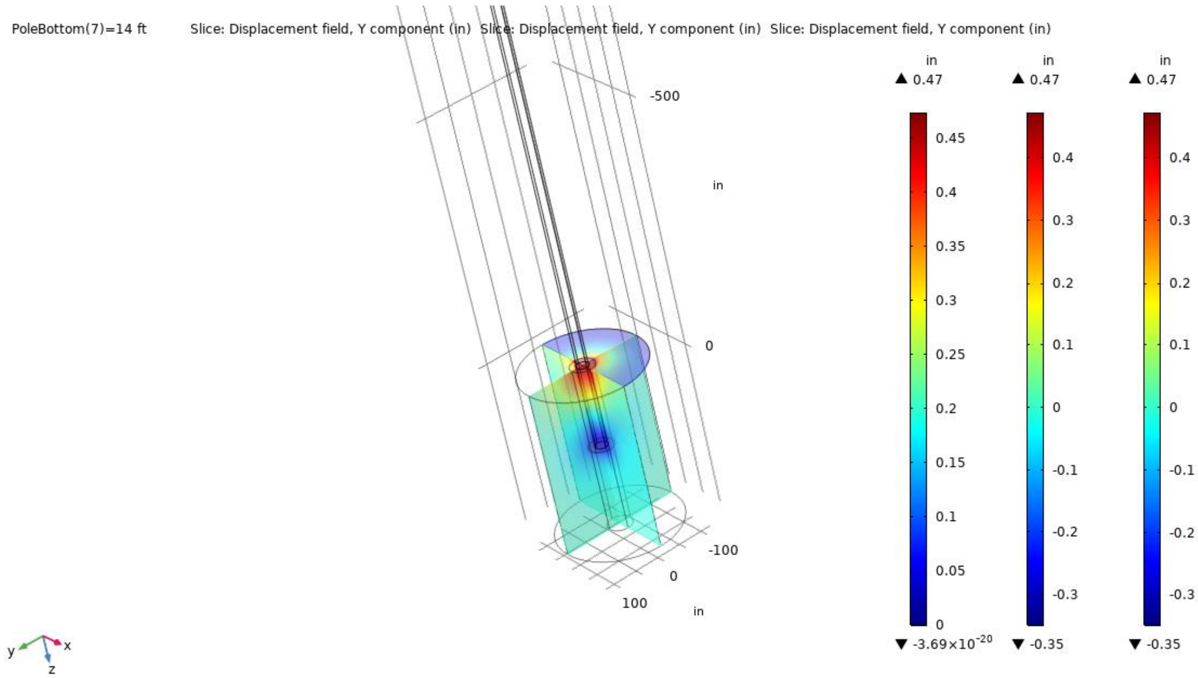

Figure 4.21: Translation in sandy soil

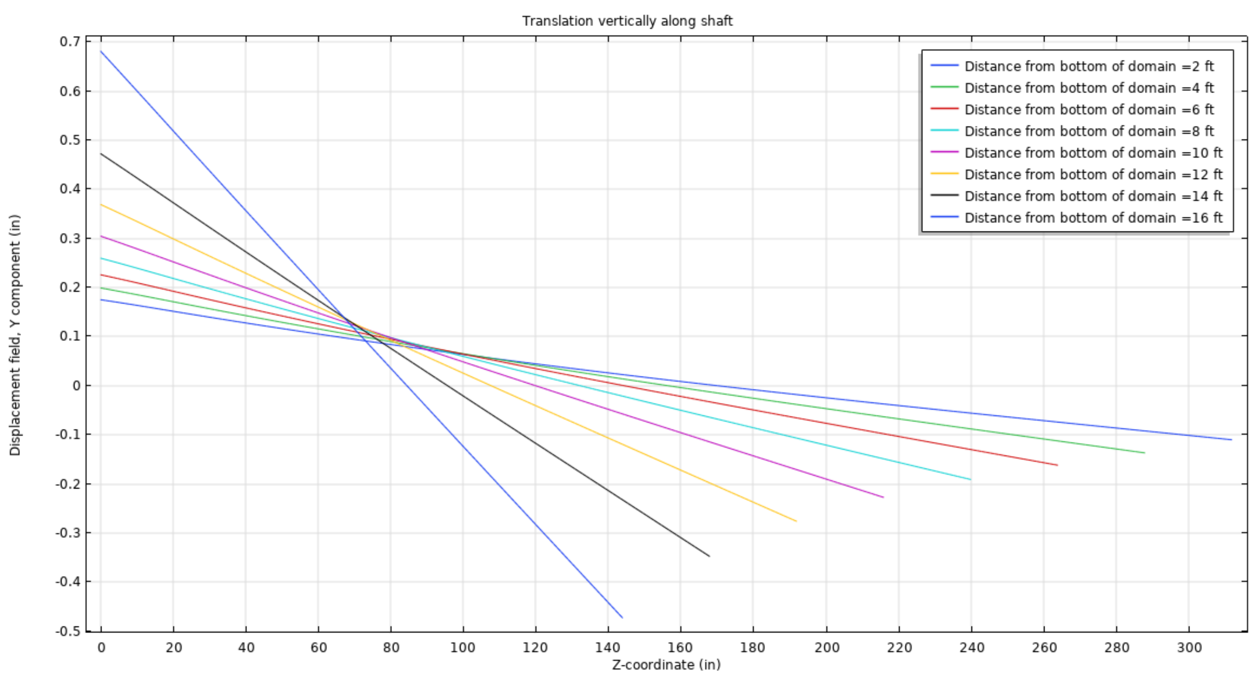

Figure 4.22: Translation along fill for sandy soil

An independent finite element model of the direct-embedment foundation was developed and documented. Clay and sandy soil were modeled. The results indicate reasonable performance with load effect less than the results from the LPILE model. This COMSOL simulation may be used in the future for more detailed soil model and optimized design, perhaps to decrease the shaft diameter and/or embedment depths. Field testing should be performed to validate this level of refined modeling. The LPILE results are larger and are used below.

4.4 Selection Matrix based on Numerical Analysis

The recommended direct embedment length for NDOT was selected based on the criteria of having horizontal ground-line displacement less than 0.6 in. for clayey soils and 0.3 in. for sandy soils. The final selection matrix for various direct embedment foundations are provided in Table 4.6.

Table 4.6: Selection Matrix for Various Direct Embedment Foundations

Pole Height (ft)	Direct Embedment Length (ft)	Diameter of Backfill (ft)	Soil Type	Expected Ground-line Displacement (in.)	Service Loads
140	28	4	200 psf clay	0.6	M = 245 ft-kips V = 3.1 kips
	22	4	400 psf clay	0.55	
	18	4	600 psf clay	0.28	
	16	4	30-degree sand	0.24	
	16	3	35-degree sand	0.14	
	16	3	30-degree sand	0.22	
120	28	3	200 psf clay	0.45	M = 177 ft-kips V = 2.6 kips
	20	3	400 psf clay	0.55	
	16	3	600 psf clay	0.41	
	14	3	30-degree sand	0.22	
	14	3	35-degree sand	< 0.22	

5. SITE CONSIDERATIONS AND CONSTRUCABILITY

5.1 Introduction

This chapter provides information related to possible issues NDOT may encounter during the construction of direct embedment foundations for High Mast Towers. Based on the construction issues listed in this chapter, a draft construction specification is listed in Appendix A as a reference to potentially mitigate the construction issues introduced in this chapter. In addition to the constructability, cost comparison of selected foundation systems, local soil conditions in Nebraska, and site considerations and steps for corrosion protection strategies that can be implemented in Nebraska is discussed in this chapter.

5.2 Construction Issues

5.2.1 Overview

As outlined previously, drilled shafts are common foundations for high-mast poles and other ancillary structures such as signs, traffic signals, and so forth. Local contractors likely have this experience and are familiar with the geotechnical challenges. Hole stability is paramount for both standard foundations and direct embedment. Stability is a concern for both safety and the proper consolidation of backfill; stability can be addressed in several different ways depending upon the types of soils, layering, and the moisture condition.

Geotechnical report attempts to define the local conditions of the soils to inform both the designer about the size and foundation type; but most importantly, to advise constructor about the site conditions regarding the construction method. However, geotechnical information may be limited for a particular site, and in fact, reports might be based upon previous bores located away from the site.

The excavation must be closely observed and the contractor must be ready to react if the soil is not representative of the report. The typical methods are outlined next. This discussion is general and specific requirements are outlined in the draft specification provided in Appendix A.

5.2.2 Earth formed

The soil is stable and supports the integrity of the shaft diameter. This is the most desirable situation. Unfortunately, it is so desirable that many contractors are overly optimistic and push this type of construction in poor soil conditions. Difficulties arise when the soil starts to slough and create voids in the walls. These voids are very difficult to fill and compact properly, more so if the backfill material is aggregate. Another concern is that sloughing occurs as the pole is being set and the additional soil in the bottom of the shaft affects the embedment depth. In addition, the pole may bear at the bottom of the shaft against the sloughed soil rather than the competent backfill material creating possible structural integrity concern.

5.2.3 Polymer slurry

Slurries fill the excavation with dense fluid-like material that can help to maintain the shaft integrity. This method can be used for poor soils with the exception of gravel. Polymer slurries are used frequently in the electrical utility industry, but placement can be tricky:

The shaft drill cannot be observed, so no way to directly detect a problem.

Indirect observations become important such as observing depth constantly to ensure no sloughing is occurring.

Backfill placement is critical and typically concrete is used. The concrete displaces the slurry as the concrete is placed from the bottom of the shaft upward. A tremie or concrete pump is required. Aggregates are not recommended because of separation of fines and difficulty with compaction.

5.2.4 Culvert

Using pipe culvert is a common method for hole stability because the material is relatively cheap and readily available. However, corrugated ribs in the culvert provide areas where the interface between the culvert and soil cannot be properly compacted creating voids. These voids should be filled with grout to insure the integrity of the bearing surface, which is important, especially for high-mast applications where the overturning moments are high relatively to the downward thrusts. Grouting can be expensive.

5.2.5 Permanent casing

Casings are commonplace and offer one of the best methods to insure shaft stability. Casings use, however, is the most expensive method of all outlined here. With this method the contractor advances the casing down the hole as drilling progresses. There is little chance of sloughing as the drill picks up any materials between the hole bottom and the bottom of the casings. Once the casing has reached required shaft depth, the pole can be set.

5.2.6 Setting the pole

Setting the pole for direct embedment is a critical operation. The lateral support of the pole must be of sufficient control and stiffness to maintain the pole's position during backfill that will include concrete pumping and potential wind challenges. The design will provide a section of pole above grade and this section must be checked for adequacy and consistent with the planned equipment. Adjustment after backfill placement is difficult, and certainly impossible, after concrete hardening. The specifications call for a maximum deviation of plumbness. Unlike the anchor-bolt methods, there are no leveling nuts to adjust for plumbness. Finally, poles has protective coating of galvanizing and/or mastic. Proper handling is necessary to not scrape or otherwise damage these coatings. Damage must be repaired by hand by application of coatings prior to backfill. Small uncoated areas can lead to corrosion “hot spots” that can affect the integrity of the system over time.

5.2.7 Backfill

The backfill process is critical. The design must allow for enough space between the pole and the soil for the backfill operation to occur. This process must be monitored to make sure that the consolidation or compaction (depending on the backfill) is to specifications. If the hole is earth formed the contractor needs to take the necessary precautions to not cause sloughing into the placed backfill material. The pole also needs monitored so that there is minimal displacement during the backfill placement and the plumbness tolerances are maintained. On site owner inspection should occur during the backfill operation, once fill is placed there is no way to discern that critical problems did, or did not occur.

5.3 Locality of Soil Conditions in Nebraska

The highly-populated area of Nebraska – where Lincoln and Omaha is located (the area within the red boundary in Figure 5.1), the shales are bed rocks covered by glacial tills and intermittent Loess layers/pockets as summarized in Pabian (1970). These layers are the local soil conditions for major structures such as cut/fill slopes, structural foundations, subgrade for highways, dams and levees.

The weathered shales contain approximately 50% fine contents. Swelling pressure of these weathered shales is in the range of 10 kPa to 24 kPa according to Song et al. (2019). The 24 kPa swelling pressure is high enough to nullify the effect of overburden thickness up to 40 ft. This is high enough to open cracks in these soils. In addition, the temperature condition in this area is frequently below negative 4 degrees Fahrenheit (-20 °C) during winter days, but it is frequently above 104 degrees Fahrenheit (40 °C) during the summer days providing a temperature fluctuation of 108 degrees Fahrenheit. Freeze-thaw cycles combined with wet-dry cycles may cause substantial strength reduction for these soils.

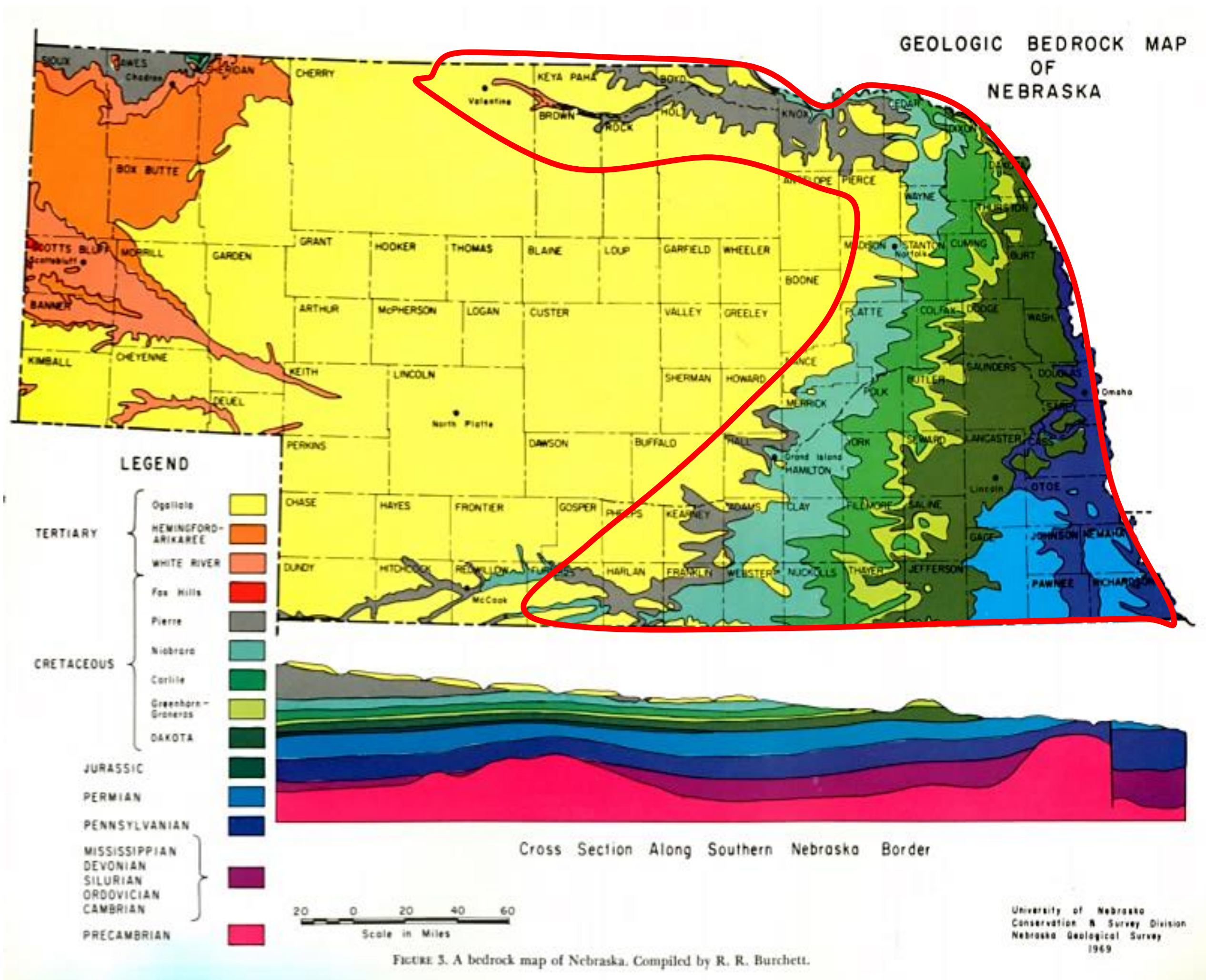

Figure 5.1: Geologic Bed Rock Map of Nebraska (retrieved from Pabian, 1970)

Glacial tills typically contain 10% to 25% fines with occasional inclusion of core stones, and their strength characteristic is significantly affected by the water content. The glacial tills at the surface during the rainy season show immeasurably low vane shear strength while it is in the range of 100 t/m^2 during dry seasons (Song et al. 2019).

Back calculated strength of failed slopes in Nebraska by Song et al. (2019) showed that the field strength of these soils at failure could be in the range of 70% to 10% of the initial strength. Extensive laboratory test results by Stark and Eid (1994), Stark et al. (2005), and Wright et al. (2007) also demonstrated that substantial strength degradation for highly plastic soils are observed through ring shear tests and drained triaxial compression tests. Findings of these previous studies indicate that weathering induced strength degradation of surface soils in this area could be an underlying source of troubles for many different geotechnical structures.

Therefore, in the eastern part of Nebraska, a critical shear strength (200 psf for clayey soils, internal friction angle of 30 degrees for sandy soils) is recommended to be used in designing direct embedment foundations for HMT. In other areas, field testing such as field vane or CPT tests are recommended to obtain the correct soils parameters, and corresponding foundation embedment depth similar to what is listed in Table 4.6. For the case with no available direct field test results of clay cohesion or internal friction angle for sand, NDOT engineers are recommended to resort to SPT vs. cohesion, and/or SPT vs. internal friction angle relationship available in Appendix B.

When there is absolutely no information regarding the soil conditions, particularly areas outside Eastern Nebraska, it is recommended to use a cohesion of 200 psf for clayey soils, internal friction angle of 20 degrees for sandy soils and use the Broms' method to compute the required embedded depth conservatively. When LPILE is used, special care must be taken because the stiffness calculation module in LPILE may not properly accommodate this internal friction angle lower than 29 degrees, though it may conduct the calculation and present an output file.

5.5 Site Considerations and Steps for Corrosion Protection Strategies

Although the NIST field data introduced in Chapter 2 is an extensive field data that can be used for quantitative design, it may be less practical to conduct corrosion design based on this data. It may make more sense to conduct the preliminary design with a qualitative method similar to the DIPRA design decision tables. The likelihood, consequences, and the possible protection method will determine the initial design for corrosion resistance. The field data produced by NIST can be used as a reference to what possible pit depths could be estimated and the service life of the structure that will be buried under soil. The followings are suggested procedures for corrosion protection strategies in Nebraska based on the literature review:

1. Check with the NDOT M&R Division whether the site is in a corrosive area. The definition of corrosion area could be similar to what CALTRANS is using for their judgement. For example, the site is in a corrosive area if chloride concentration is 500 ppm or greater, sulfate concentration is 2,000 ppm or greater, or the pH is 5.5 or less.

2. If the site is in the corrosive area, and the soil resistivity is equal or less than 1,000 ohm-cm (both AWAA and CALTRANS classify this to be a highly corrosive area), the degree of aeration (oxygen concentration), amount of moisture, pH level (hydrogen activity of the solution) and ion contents or organic contents should be additionally measured by the corrosion consultant or expert hired by the NDOT M&R division to collect additional information that will affect the corrosion rate of the embedded HMT.

3. Depending on the HMT embedment length, pole diameter, and access availability for repair work (if necessary), and the likelihood of underground corrosion (based on the parameters measured in step 2), the NDOT M&R division may make decisions which corrosion protection method to choose. Available methods include protective coatings with epoxy or bituminous material, cathodic protection, increasing the pole thickness below the ground-line, or use reinforced concrete jackets around the steel pole.
4. If both the likelihood and the consequence scores are calculated to be high, a combination of the corrosion protection method listed in step 3 can be applied.
5. If necessary, the NIST tables provided (old reference but the most extensive field data up to date) can be used to compute the potential maximum pit depth and weight loss of the embedded pole. This will provide the expected service life of the embedded HMT structure. Based on the expected life calculations, the design can change as needed.

5.6 Cost Comparisons

This section looks into the cost comparisons made for the conventional High Mast Tower with the bolted base plate versus the case with direct embedment. The goal of the project is to eliminate the fatigue-prone pole to base detailing by directly embedding the High Mast Tower and improving the performance not necessarily to save the cost. However, it is anticipated that there will be some cost difference due to the change. For example, since the baseplate to tube connection is eliminated, and there is no need for anchor bolts, the fabrication cost will most likely decrease with the change in detailing. The handhold does not change for both cases and the slip connection for the remaining sections are based on standard practice for both cases which will not likely to change the cost. Table 5.1 summarizes the estimated cost difference including the material and fabrication cost of the pole, concrete and rebar required in both construction methods, and anchor bolts needed in the conventional construction. It is anticipated that there will be approximately $9,278 cost savings per pole.

Table 5.1: Cost Comparisons between Conventional vs. Direct Embedment (USD)

	Bolted with Base Plate	Direct Embedment	Savings with Direct Embedment
Pole Difference	$2,458	$0	**$2,458**
Concrete and Rebar	$2,300	$480	**$1,820**
Anchor Bolts	$5,000	$0	**$5,000**
Summary	$9,758	$480	**$9,278**

6. SUMMARY AND CONCLUSION

6.1 Summary

High-Mast Tower (HMT) foundations have been traditionally designed and constructed using cast-in-place foundation with anchor bolts that are used to secure the tower to the foundation. This type of design requires a base plate that is welded to the tower shaft. The Nebraska Department of Transportation (NDOT) has recently experienced issues with stresses that this type of design presents at the anchor bolt/foundation or base plate/tower shaft interface. This research project objective was to develop an alternative design for HMT foundations with direct embedment of the HMT which can eliminate fatigue-prone details associated with the pole-to-base plate connection which is the primary location of failure.

First, the literature that includes research from academia and industry, current and proposed state of practice from industry, examples of design specifications and guidelines, and corrosion for buried structures were reviewed. Secondly, structural loads for the typical 120- and 140-ft HMTs constructed in Nebraska and the soil resistance for them were calculated. The structural loads were computed using the AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals (2015), with a spreadsheet based on the fundamental principles of structural analysis. The geotechnical foundation resistance calculations were made to check the vertical and horizontal soil capacity for the typical HMTs used in Nebraska. Further parametric study was conducted using two numerical software: LPILE and COMSOL for varying soil conditions and foundation systems with different embedment length and backfill diameter for the service level base moment and shear. Required embedment length and backfill diameter is provided as a matrix using the LPILE analysis results which provided higher values than the finite element model created with COMSOL. Finally, based on the site considerations and construability, a draft design specification and soil parameters that can be used for Nebraska soil conditions are provided with details listed in the Appendix of this report.

6.2 Conclusion

Based on the literature review, concrete and aggregate are recommended as backfill for direct embedment of HMT. The typical backfill candidate is concrete since this is the best choice in corrosive sites. Earth form or permanent casing form with concrete or aggregate backfill is recommended for Nebraska soil. Culvert casings are not recommended due to compaction issues associated with the corrugations at the soil-culvert interface.

From the load and resistance calculations, the shear force at the base (ground-line) for the 120-ft and 140-ft HMT were 4.0 kips, and 4.8 kips, respectively. The moment at the base for the 120-ft and 140-ft HMT were 273 ft-kips, and 382 ft-kips, respectively. The axial loads at the base were 13.4 kips for the 120-ft HMT and 14.9 kips for the 140-ft HMT. Without considering the side friction of the embedded pole, the net load carrying capacity of the HMT is sufficient to resist the axial loads subjected at the ground-line. The required embedment length for the worst-case soil conditions (200 psf cohesive clay) based on the AASHTO LRFD LTS procedures was 24 ft for the 140-ft pole.

Several foundation systems were recommended as a result of the parametric study conducted by LPILE for different soil conditions. In order to maintain a ground-line displacement of 0.6-inch, 28 ft embedment with a 4 ft diameter backfill will be required for a 140-ft HMT embedded in 200 psf cohesive clay. This embedment length can be decreased down to 18 ft (10% pole height plus 4 ft) as the cohesion increases up to 600 psf. If the natural soil surrounding the directly embedded HMT is sand with an internal friction angle of 30 degrees, the embedment length can further be decreased to 16 ft which is 10% of the pole height plus 2 ft. The backfill diameter can further be decreased to 3 ft from 4 ft if the internal friction angle is increased 5 degrees. The 120-ft HMT embedded in 200 psf cohesive clay may require 28 ft embedment length with a 3 ft diameter backfill to maintain ground-line displacement below 0.6 in. The embedment length can be decreased to 16 ft (10% pole height plus 4 ft) if the cohesion is increased three times the worst-case scenario. For the 120-ft HMT, with 30-degree internal friction angle sand, the required embedment can decrease down to 14 ft which is 10% of the tower height plus 2 ft when, 0.3 in. is the target ground-line displacement.

Finally, based on the construction issues listed for earth formed, polymer slurry, culverts, and permanent casings, earth formed and permanent casing were the two options recommended for Nebraska HMTs. Appendix A provides specifications based on the two critical operation in construction which is setting the pole and installing backfill. Based on the locality of soil conditions in Nebraska, if Standard Penetration Testing data are available, methods introduced in Appendix B can be used to calculate the soil parameters required in the design process. If soil conditions are not available, it is recommended to use a cohesion of 200 psf for clayey soils and an internal friction angle of 20 degrees for sandy soils and use the Brom's method to compute the required embedded depth conservatively. Since, the corrosion activity for buried structures can be affected by many different scenarios, a five-step procedure for corrosion protection strategy is provided in Section 5.5 of this report. The first step includes evaluating the site if it is a corrosive area or not based on the threshold values of chloride concentration with 500 ppm or greater, sulfate concentration of 2,000 ppm or greater, and the pH being 5.5 or less. If the site is in the corrosive area, and the soil resistivity is equal or less than 1,000 ohm-cm, further site measurements should be taken including the amount of moisture, pH level, ion contents, and organic contents to characterize the site soil conditions. Based on these measurements the NDOT Materials and Research division should choose either to use protective coatings with epoxy or bituminous material, cathodic protection, or increase the pole thickness (yet, with the concrete backfill, this may not be required). If the likelihood and the consequence of corrosion is specifically high in the site where the HMT will be constructed, a combination of the corrosion protection methods listed above can be applied. If the corrosion rate (pit depth per years) is required to calculate the expected life, or the required increase in thickness of the pole buried underground is needed, the NIST tables provided in Chapter 2 can be utilized as a reference. It is anticipated that there will be a cost saving of approximately $9,300 per pole with the reduced baseplate detailing with 6 to 8 anchor bolts required in conventional construction additional to the reduction in reinforced concrete required for the foundation.

6.3 Further Research

An independent finite element model of the direct-embedment foundation using COMSOL was developed and documented in Chapter 4. The results indicate reasonable performance with load effects less than the results from the LPILE model. This COMSOL simulation can be used in the future for more detailed soil model and optimized design to decrease the shaft diameter and/or embedment depths. In order to have this level of refined modeling, additional field testing should be conducted.

In addition, since the calculations provided in this report is based on assumptions for the soil parameters, the research team can further engage with the NDOT Materials and Research division to conduct field demonstration including design and construction of a full-scale HMT with different site conditions.

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APPENDIX A

NDOT Sample Construction Specification for Direct Embedded Poles

Qualifications and Submittals

Submit the following for review at least 10 working days before constructing drilled piers. NDOT review of the Contractor's personnel qualifications and installation plan does not relieve the Contractor of the responsibility for obtaining the required results in the completed work.

Personnel Qualifications

Construction Personnel. Use a supervisor with at least three years of experience in the construction of direct embedded poles. Supervisor must remain on-site during all direct embedment installation activities. Upon request provide a resume of job experience, project description, the owning agency's name, email address, and phone number.

Submittals

Furnish the following in the installation plan:

A. Details of proposed pier drilling methods; methods for removing materials from the piers; procedures for maintaining correct horizontal and vertical alignment of the excavation; and a disposal plan for the excavated material.

B. A description, including capacities, of the proposed equipment to be used including cranes, drills, drilling unit, augers, bailing buckets, and final cleaning equipment.

C. Demonstrate an understanding of the subsurface conditions at the site. Reference the available geotechnical report and/or any other subsurface data provided by the Company.

D. Details of methods to be used to ensure drilled pier hole stability during excavation and concrete placement. Include a review of the chosen method's suitability for the anticipated site and subsurface conditions. If permanent casings are proposed or required, provide casing dimensions and detailed procedures for permanent casing installation.

E. As applicable, details of bracing, centering centralizers, and lifting and support methods.

F. Details of Aggregate placement including compaction methods.

G. Details of concrete placement including proposed operations procedures for free fall, tremie, or pumping methods. Provide summary of proposed actions to be taken when concrete does not meet minimum specifications or when unforeseen delays occur during the concreting process.

Other Required Submittals

1. Concrete Mix Design
2. Aggregate gradation
3. Direct Embedment Installation Record

Execution

Drilling Operations

A. Excavate holes according to the installation plan. Report all deviations from the plan to the onsite inspector.

B. When required, casings shall be installed as the drilling proceeds or immediately after the equipment is withdrawn to prevent sloughing and caving of the excavation walls. Casing shall be advanced ahead of the drilling operation in order to maintain a soil plug capable of producing a positive seal at the bottom that prevents piping of water or other material into or out of the hole.

C. Slurry may be used to stabilize the excavation; however, a specific plan, including the material to be used, must be submitted to NDOT for review prior to use. Refer to FHWA Standard Specifications for Construction of Road and Bridges, Section 565 “Drilled Shaft Installation” for all slurry use requirements.

D. Steel casings of ample strength to withstand handling and installation stresses shall be used. Use casing with the outside diameter equal to or greater than the specified diameter of the pole and the inside diameter not exceeding the specified diameter of the pole by more than 6 inches.

E. Each drilled shaft shall be accurately located, sized and plumbed. The maximum deviation of the drilled pier from its designated location shall not be more than 2 inches at its top elevation. The drilled shaft shall not be out of plumb more than 1 inch in 5 feet of height.

F. Each drilled excavation shall be made to the approximate depth indicated on the drawings. All weathered and loose material shall be removed from the excavations. NDOT shall verify the final tip elevation before concrete or aggregate placement. Classification of the excavated materials will not be made except for identification purposes. Drilled excavation shall include the removal and handling of all excavated materials from the site.

G. All drilled excavations will be inspected by NDOT before the placement of concrete or aggregate. All drilled excavations that cannot be visually inspected shall be treated as a wet hole. Refer to wet method for concrete placement.

Aggregate Placement

A. Backfill: Holes shall be backfilled with crushed aggregate backfill as specified on the Drawings.

B. Backfill shall be compacted until the material has reached the required compaction

C. Native and engineered backfill shall be banked and tamped twelve (12) inches above the natural ground surface.

D. Surplus excavated material shall be evenly spread along the right-of-way or hauled to an offsite location for dumping, according to the permissions and requirements of each individual landowner.

E. Lifts of backfill material shall not exceed six inches in depth. Any extremely dry materials shall be dampened during the backfill operation to obtain the desired density

Concrete Placement

A. Dry Method

Use the dry construction method at sites where the groundwater level and soil conditions are suitable to permit construction in relatively dry conditions and where the sides and bottom of the excavation may be visually inspected before placing concrete.

i. Unless otherwise accepted by the NDOT, concrete shall be placed in drilled holes within 24 hours of completing excavation.

ii. All water and loose materials shall be removed from the holes and reinforcement shall be thoroughly cleaned before concrete is placed.

iii. Concrete shall be placed with a tremie or funnel to prevent segregation. Use free-fall placement only in dry holes with a maximum 6-foot free-fall height or NDOT approved height. The concrete shall fall directly to the pier base without contacting either the pole or hole sidewall. If concrete placement causes the excavation to cave or slough or if the concrete strikes the pole or sidewall, reduce the height of free-fall and reduce the rate of concrete flow into excavation. If placement cannot be satisfactorily accomplished by free-fall, use tremie or pumping to place concrete.

iv. The top 6 feet of concrete shall be rodded or vibrated to provide a dense mass free of voids. As placed, the concrete shall have a slump between 6 to 8 inches. When scum or laitance accumulates on the top of the concrete, it shall be removed and replaced with fresh concrete to the proper elevation.

v. If approved casings are left in place, the void areas between the form and the excavation walls shall be filled with lean concrete mix. The lean concrete or grout mix shall be placed and tamped to fill the annular space.

vi. The volume of each drilled excavation shall be documented and compared to the concrete volume of each drilled pier. If the concrete volume placed is less than the calculated (theoretical) volume, the NDOT shall be notified immediately.

vii. Concrete shall maintain a minimum 6-inch slump for the duration of the pour.

viii. Self-consolidated concrete may be used meeting NDOT specifications. In this case, rodding or vibrating per iv above is not necessary.

B. Wet Method

Use the wet construction method or the casing construction method for shafts that do not meet the above requirements for the dry construction method.

i. Concrete shall not be deposited under water except with NDOT permission. The proportions for underwater concrete mix shall be adjusted to provide 7 to 9 inches of slump and the cement factor shall be increased by one sack per cubic yard.

ii. Underwater concrete shall be placed through a tremie equipped with a seal at the lower end and a hopper at the upper end. The tremie shall be watertight and a minimum diameter of 6 times the maximum concrete aggregate size to allow a free flow of concrete. After the flow of concrete is started, the lower end of the tremie shall be kept below the surface of the deposited concrete. The entire mass of concrete shall be placed as quickly as possible and shall flow into place without shifting horizontally under the water.

iii. Fluid within the excavation shall be stable when concrete is deposited, and shall be maintained at a height necessary to ensure hydrostatic equilibrium during concrete placement, but not less than 5 ft above the water table. After placing, the ground water level in the area adjacent to the drilled shaft shall be kept static (no pumping) until the concrete has taken its initial set.

iv. Concrete shall maintain a minimum 7-inch slump for the duration of the pour.

Direct Embedment Installation Record

An accurate record of each pier installation and concrete placement shall be completed that contains, as a minimum, the information listed below. The Contractor shall submit the installation record to the Company Field Representative at the end of each day. Submitted records will not become official until the Company Field Representative agrees with the accuracy and completeness and signs the document.

The drilled shaft installation record shall contain the following information:

A. Contractor's name
B. Drilled shaft number and location
C. Overall depth of excavation
D. Depth to water
E. Final depth if different from design drawings
F. Note any caving, sloughing of excavation and drilling difficulties
G. Casing insertion, size and length, and whether or not removed
H. Date and time of start and finish excavation
I. Date and time concrete placed
J. Calculated volume of excavation based on diameter of shaft
K. Total actual quantity of concrete or aggregate placed within shaft excavation
L. Concrete Yield Plot (volume versus shaft depth)
M. Concrete batch plant ticket numbers

APPENDIX B

Soil Parameters for Nebraska

SPT Blow Counts

SPT has been used for a long time in geotechnical investigation. The method is conducted by dropping a 140 lb (63.5kgf) hammer from 30 in (762mm) height and hitting the SPT shoe (also called as a sampler) of a 2 in (50.8 mm) outside diameter and 1.5 in (35 mm) inside diameter into the ground, and measuring the number of blows required to penetrate the SPT shoe 12 in (30 mm) into the ground.

The SPT does not utilize any stress and/or strain measurement mechanism, so it does not provide any direct measurement of strength, deformation, or modulus of soils. However, it is anticipated that stronger/harder soils may show higher penetration resistance resulting a higher SPT blow count. Engineers, therefore, have used the SPT blow counts to indirectly obtain the engineering properties of soils based on empirical charts/correlations for more than two generations.

In this study, SPT blow counts were practically the only engineering test results available, therefore, the best engineering judgement was used to obtain required engineering parameters introduced in Section 3.3.

Correction Factors of SPT Blow Count (N_{60})

The SPT blow count technique was widely used in the geotechnical area due to its simplicity and ease of fabrication (USBR, 2020). This simple mechanism and easy to fabricate feature made the creation of several different versions of SPT testing devices and practices. At some point, it was realized that the SPT test results obtained in one area may not be compatible to SPT test results from another area. Similarly, SPT test results obtained from style A equipment may not be compatible to SPT test results obtained from style B equipment. As a solution to secure an interchangeable SPT blow counts, the standardization of SPT has developed. This involves in equalization of SPT blow count by applying several correction factors. Following is the summary of correction factors based on Das (2014) for soil samples from Juneau, Arkansas. This detailed correction method is originally proposed by Skempton (1986) and Seed, et al. (1985).

Overall form of correction scheme

The N_{60} can be determined using the following equation:

$$N_{60} = \frac{N\eta_H\eta_B\eta_S\eta_R}{60\%}$$

where,

N_{60} = Standard penetration number for 60% energy ratio

N = Measured penetration number

η_H = Hammer efficiency (Table B-1)

η_B = Borehole diameter correction (Table B-1)

η_S = Sampler correction (Table B-1)

η_R = Rod length correction (Table B-1)

Table B-1: Variation of correction factors (retrieved from Das, 2014)

1. Variation of η_H

Country	Hammer type	Hammer release	η_H (%)
Japan	Donut	Free fall	78
	Donut	Rope and pulley	67
United States	Safety	Rope and pulley	60
	Donut	Rope and pulley	45
Argentina	Donut	Rope and pulley	45
China	Donut	Free fall	60
	Donut	Rope and pulley	50

2. Variation of η_B

Diameter		
mm	in.	η_B
60–120	2.4–4.7	1
150	6	1.05
200	8	1.15

3. Variation of η_S

Variable	η_S
Standard sampler	1.0
With liner for dense sand and clay	0.8
With liner for loose sand	0.9

4. Variation of η_R

Rod length		
m	ft	η_R
>10	>30	1.0
6–10	20–30	0.95
4–6	12–20	0.85
0–4	0–12	0.75

The correction factors in Table B-1 are computed for representative soil conditions as follows:

a) Hammer efficiency:

For United states, safety, rope and pully hammer, $\eta_H = 60\% = 0.6$

b) Borehole diameter correction:

For diameter (2.4 –4.7 in.), η_B=1

c) Sampler correction:

For no liner, $\eta_S = 1$

d) Rod length correction:

For (12 – 20 ft), $\eta_R = 0.85$ (average)

Additional Correction Factors for Cohesionless Soils

The strength and stiffness of cohesionless soils are affected by the confining pressure that which varies by depth. This is the same with the SPT blow counts. Therefore, additional correction is applied as follows (Das, 2014, p98):

For sand:

$$(N_1)_{60} = C_N N_{60}$$

where,

$(N_1)_{60}$ = value of N_{60} corrected by confining pressure with respect to a standard value

$\sigma'_a = P_a$ [≈ 100 kN/m^2 (2000 lb/ft^2)].

C_N = Correction factor found in Table 3.3.2.

N_{60} = value of N obtained from the previous relations.

At approximately (3– 15 ft) depth, $C_N \approx 1.0$ to 1.33 (Normally consolidated coarse sand)

Combined Correction Factor for this Research

From the correction factors introduced previously, the representative correction factors for the SPT blow counts for this research are as follows.

$$N_{60} = \frac{N(0.6\text{x}1\text{x}1\text{x}0.85\text{x}\ (1.0 \text{ to } 1.33))}{0.6}$$

= (0.85 – 1.13) N for cohesionless soils

= 0.85 N for cohesive soils (Correction for the confining pressure not applied.)

≈ 1.0 N for both cohesionless soils and cohesive soils because this much difference in SPT blow count does not make substantial difference in predicted material constant of soils.

However, it should be noted that some correlations are based on the uncorrected SPT blow counts and in such cases, uncorrected SPT blow counts should be used.

Internal Friction Angle

The following Figure B-1 shows the typical trend lines of the internal friction angle and the SPT blow count based on different research (Wolff, 1989; Peck et al. 1974; Hatanaka and Uchida, 1996; Mayne et al., 2001 and JRA, 1996). From Figure B-1, the reseults from Wolff (1989), Peck et al. (1974) and JRA (1996) predicted consistent and slightly conservative internal friction angle.

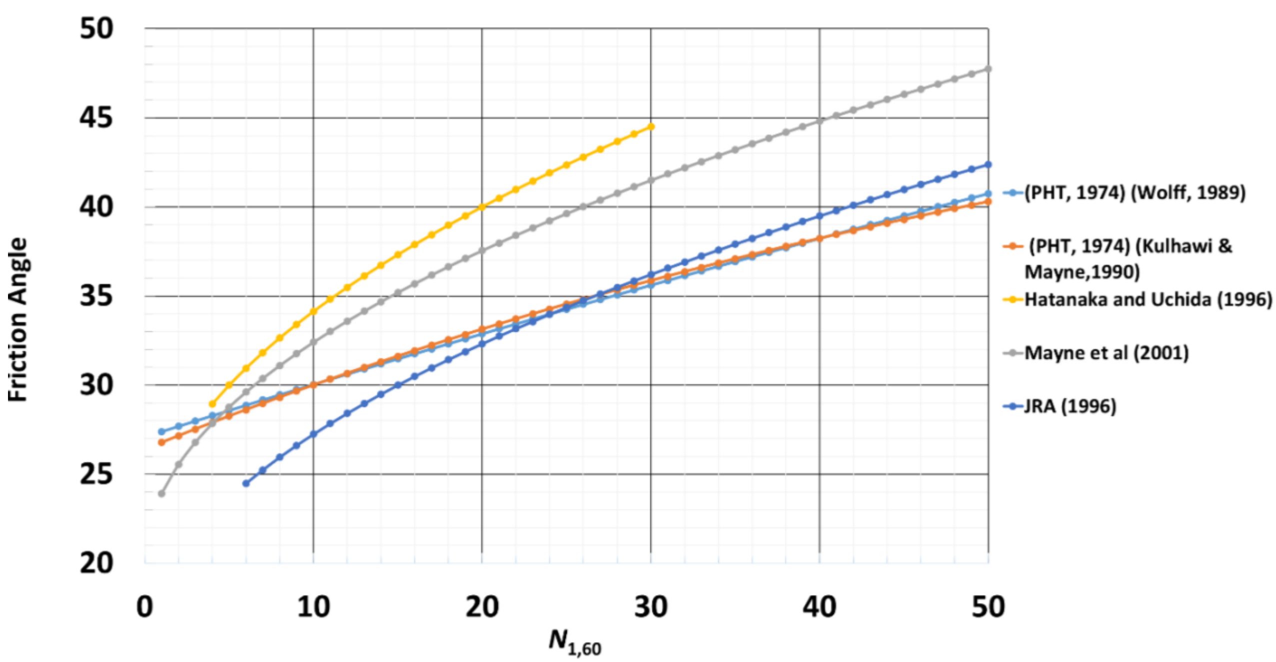

Figure B-1: Prediction of internal friction angle based on SPT blow count

Therefore, these three methods were used for the prediction of internal friction angles based on the SPT blow counts. For SPT blow counts ranging between 10 to 25, the graphical solutions provided an internal friction angle in the range of 27˚ to 35˚.

Cohesion

Cohesion for cohesive soils were also predicted from the SPT blow counts. This study compared the proposed equations or plots based on corrected SPT blow count and uncorrected SPT blow count as shown in Tables B-2 and B-3.

Table B-2: Predicted cohesion by corrected SPT blow count

N_{60}-Value	< 2	2–4	4–8	8–15	15–30	> 30
Terzaghi and Peck (1976)	< 12	12–24	24–48	48–91	91–182	> 182
Hara et al. (1974)	< 48	48–79	79–131	131–206	206–340	> 340

Table B-3: Predicted cohesion by uncorrected SPT blow count

N_{60}-Value	< 2	2–4	4–8	8–15	15–30	> 30
Terzaghi and Peck (1976)	<12.5	12.5–25	25–50	50–100	100–200	> 200
Parcher and Means (1968)	<12	12–25	25–50	50–100	100–200	> 200
Tschebotarioff (1973)	<15	15–30	30–60	60–120	120–225	> 225
Karol (1960)	12	12–24	24–48	48–96	96–192	> 192
Sowers (1970) (CH)	< 24	24–49	49–99	99–186	186–373	> 373
Sowers (1970) (CL)	< 14	14–29	29–58	58–109	109–218	> 218
Sowers (1970) (SC–ML)	< 7	7–14	14–29	29–54	54–109	> 109