Current Status of Design and Construction of Piles with a Pilot Hole on Rock

Charles C. Crowner

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CURRENT STATUS OF DESIGN AND CONSTRUCTION OF PILES WITH A PILOT HOLE ON ROCK

by

CHARLES CROWNER

(Under the Direction of Soonkie Nam)

ABSTRACT

Driven piles at project sites get their load bearing capacity from side friction along the driven lengths as well as from end resistance. Pilot holes is a pile driving assistance method used to aid driving displacement piles through hard/dense layers and rock. These pilot holes can be a size smaller or larger than the pile that is to be installed. The pilot hole is first drilled down to a specific depth. The use of a pilot hole reduces the “end bearing” and “side resistance” within the drilled zone and aids the driving of the pile. This process also complicates the prediction of long-term pile capacity. Two of the major unknowns that accompany the use of the pilot hole is the reduction of end bearings as it pertains to pile driving within the zona and the reduction of side friction. The objective of this project was to identify and document the relationship between the load capacity of piles installed with pilot holes specifically into rock and their design parameters with respect to the pilot hole geometry, rock socket geometry, geological properties, and installation method. As well to develop a reliable LRFD design procedure that incorporates proper resistance factors, and a field verification method for quality assurance of rock. To complete the objectives a compilation of best practice methods available on the subject on pilot holes was needed. This included a literature review, a survey with state highway agencies, some field testing and instrumentation, a review of past projects and testing data, and making final conclusions.

INDEX WORDS: Deep foundations, Pile foundations, Pilot hole, State status, Pile construction,
CURRENT STATUS OF DESIGN AND CONSTRUCTION OF PILES WITH A PILOT HOLE ON ROCK

by

CHARLES CROWNER

B.S., Georgia Southern University, 2019

A Thesis Submitted to the Graduate Faculty of Georgia Southern University

in Partial Fulfillment of the Requirements for the Degree

MASTER OF SCIENCE
CURRENT STATUS OF DESIGN AND CONSTRUCTION OF PILES WITH A PILOT HOLE ON ROCK

by

CHARLES CROWNER

Major Professor: Soonkie Nam
Committee: Xiaoming Yang
            Junan Shen

Electronic Version Approved:
December 2021
DEDICATION

I would like to dedicate this thesis to my family and to my friends. Their constant support, prayers, and encouragement have pushed me to go above and beyond in everything I do.
ACKNOWLEDGMENTS

I would like to thank the Georgia Department of Transportation as well as the Allen E. Paulson College of Engineering and Computing for funding this research opportunity. I would also like to thank the support of the engineers at the Georgia Department of Transportation Geotechnical Bureau. I am grateful and acknowledge the work put in by the undergraduate students Oscar Moncada, Kristin Ackerman, Donovan Clayton, Meaghan Rockmore. I also want to thank the contributions made by Dr. Xiaoming Yang and Dr. Junan Shen. Also, I would like to express my sincere appreciation to Dr. Soonkie Nam for his advisement and support in the thesis research process.
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CHAPTER 1

INTRODUCTION

Deep foundations consisting of steel H-piles, precast concrete piles, and drilled shafts have been used for nearly as long as civil engineering has been around. Piles can be categorized in different ways but two common ways of classifying them are by the installation method, which are driven or drilled. Driven piles are normally prefabricated and pounded to required depths with mechanically with the use of a hammer or vibrations. Drilled shafts are constructed by drilling a large hole, placing in reinforcements, and pouring in concrete. There are benefits of each type as well as drawbacks, and there are many factors that must be considered in the selection of pile type for a project. Some factors that affect the decision are load capacity, equipment needed, site conditions, and construction cost.

Both pile foundation types have been used in different ground conditions but in general a driven pile is known to be more economical for the smaller size, capacity, and easier quality control in construction. When driving the pile complications can arise in ground conditions that have hard or stiff layers that could damage the pile during installation. Such hard layers could be in the middle of penetrating layers or be at the end in the supporting layers. To eliminate such issues a pile driving assistant that drills a hole before driving the pile can be considered. These holes can be larger or smaller than the size of the pile and act as a guide to allow for easier pile installation as seen in Figure 1. These holes are referred to as pilot holes by the Georgia Department of Transportation (GDOT).
The focus of this research project was on steel H-piles with the use of a pilot hole on rock and dense layers. Using pile holes can make the construction process less disruptive and prevent damage to the pile, but currently there are little to no guidelines for a pile with a pilot hole. The piles with pilot holes on rock would derive their load bearing capacity from end bearing.

GDOT prefers the use of a pilot hole that is a size larger than the pile. Pilot holes are used as a method to facilitate driving displacement piles through dense soils and on rock. These pilot holes are a size larger or smaller than the size of the pile and are drilled to a specific depth. By drilling a pilot hole, the end bearing and the side friction of the pile within the predrilled zone is reduced. This allows the pile to be installed with more ease. It is expected the strength of the soil, the strength of the rock, and the diameter of the pilot hole relative to the pile will all have an impact on the pile drivability and long-term capacity. The ability to better quantify this impact will greatly aid on geotechnical design and provide a better understanding of the interactions and factors of the pilot hole on piles. The objective of this study is to identify and document the relationship between the load capacity of piles installed with pilot holes into rock and their design parameters with respect to the pilot hole geometry, rock socket geometry, geological properties, and installation method. This could be used to develop a reliable LRFD design procedure that incorporates proper resistance factors, and a field verification method for quality assurance of rock.
CHAPTER 2

REVIEW OF CURRENT DESIGN METHODOLOGY FOR THE CONSTRUCTION OF PILES IN ROCK

2.1 Introduction

The purpose of this research is to evaluate the design and verification methods for a pile with a pilot hole that is installed through softer overburden layers, weathered rock, or soft rock down to a hard rock, where the tip of the pile would be bearing on hard rock. In this chapter the current design methodology for the piles in rock is reviewed. Determining the material and geometric properties of a pile for deep foundations starts with a static design process. In general, the following is the typical process for the design and construction for a driven pile foundation as shown in Figure 2 (Xiao, 2015).
Figure 2 Driven Pile Design and Construction Process (Modified after Xiao 2015)
2.2 Static Design Methods for Driven piles on Rock

The static design and analysis are the process that establishes the pile geometry and develop the required resistance factors for a specific soil profile. Some of the needed soil parameters are particle size, specific weight, strength, location of ground water table and presence of rock. This progress is often referred to as site characterization. The three main phases of site characterization are:

1. Planning the exploration program and data collection.
2. Completing a field reconnaissance survey.
3. Performing a detailed subsurface exploration program (boring, sampling, and in-situ testing).

The subsurface exploration should provide the depth and thickness of the strata, in-situ test to determine soil design parameters, samples to determine soil and rock parameters, and groundwater levels (Federal Highway Administration, 2016).

If the subsurface investigation and soil-boing testing establish the presence of bedrock or rocklike material piles can be extended to the rock surface. The ultimate pile capacity will depend on the load bearing capacity of the underlying material, which are known as point bearing piles or end bearing piles (Das, 2007). The ultimate load of a pile constructed on the bed of hard stratum can be expressed as shown below (Das, 2007)

\[ Q_u = Q_p + Q_s \]

Where:

- \( Q_p \) = load carried by the pile point
- \( Q_s \) = load carried by skin friction developed at the side of the pile

If \( Q_s \) is very small, then

\[ Q_s \approx Q_p \]
2.2.1 Goodman's Equation

When piles are driven to these unlaying layers of rock good evaluation of the bearing capacity of the rock must be established. The Goodman expression of ultimate point resistance is approximately

\[ Q_p = q_u (N\phi + 1) \]

Where:

\[ N\phi = \tan^2\left(45 + \phi' / 2\right) \]

\[ q_u = \text{unconfined compressive strength of rock} \]

\[ \phi' = \text{drained angle of friction} \]

*Table 1 Typical Unconfined Compressive Strength (q_u) and friction angle (\(\phi'\)) of Rocks Adapted from (Das, 2007)*

<table>
<thead>
<tr>
<th>Type of rock</th>
<th>(q_u)</th>
<th>(\phi')</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MN/m²</td>
<td>lb/in²</td>
</tr>
<tr>
<td>Sandstone</td>
<td>70 - 140</td>
<td>10,000 - 20,000</td>
</tr>
<tr>
<td>Limestone</td>
<td>105 - 210</td>
<td>15,000 - 30,000</td>
</tr>
<tr>
<td>Shale</td>
<td>35 - 70</td>
<td>5,000 - 10,000</td>
</tr>
<tr>
<td>Granite</td>
<td>140 - 210</td>
<td>20,000 - 30,000</td>
</tr>
<tr>
<td>Marble</td>
<td>60 - 70</td>
<td>8,500 - 10,000</td>
</tr>
</tbody>
</table>
Chapter 10 of the American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications covers foundations including driven pile foundations. For driven piles, Table 10.5.5.2.1-1 in the specifications shows the resistance factors for driven piles and the method that was used to determine the factors as seen in Table 2.

**Table 2 Resistance Factors for Driven Piles** From (American Association of State Highway and Transportation Officials, 2020)

<table>
<thead>
<tr>
<th>Condition/Resistance Determination Method</th>
<th>Resistance Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal Bearing Resistance of Single Pile – Dynamic Analysis and Static Load Test Methods, $\Phi_{dyn}$</td>
<td></td>
</tr>
<tr>
<td>Driving criteria established by successful static load test of at least one pile per site condition and dynamic testing* of at least two piles per site condition, but no less than 2% of the production piles</td>
<td>0.80</td>
</tr>
<tr>
<td>Driving criteria established by successful static load test of at least one pile per site condition without dynamic testing</td>
<td>0.75</td>
</tr>
<tr>
<td>Driving criteria established by dynamic testing*, quality control by dynamic testing* of at least two piles per site condition, but no less than 2% of the production piles</td>
<td>0.65</td>
</tr>
<tr>
<td>Wave equation analysis, without pile dynamic measurements or load test but with field confirmation of hammer performance</td>
<td>0.50</td>
</tr>
<tr>
<td>FHWA-modified Gates dynamic pile formula (End of Drive condition only)</td>
<td>0.40</td>
</tr>
<tr>
<td>Engineering News (as defined in Article 10.7.3.8.5) dynamic pile formula (End of Drive condition only)</td>
<td>0.10</td>
</tr>
<tr>
<td>Nominal Bearing Resistance of Single Pile – Static Analysis Methods, $\Phi_{stat}$</td>
<td></td>
</tr>
<tr>
<td>Side Resistance and End Bearing: Clay and Mixed Soils</td>
<td></td>
</tr>
<tr>
<td>(\alpha)-method (Tomlinson, 1987; Skempton, 1951)</td>
<td>0.35</td>
</tr>
<tr>
<td>(\beta)-method (Esrig &amp; Kirby, 1979; Skempton, 1951)</td>
<td>0.25</td>
</tr>
<tr>
<td>(\lambda)-method (Vijayvergiya &amp; Focht, 1972; Skempton, 1951)</td>
<td>0.40</td>
</tr>
<tr>
<td>Side Resistance and End Bearing: Sand</td>
<td></td>
</tr>
<tr>
<td>Nordlund/Thurman Method (Hannigan et al., 2005)</td>
<td>0.45</td>
</tr>
<tr>
<td>SPT-method (Meyerhof)</td>
<td>0.30</td>
</tr>
<tr>
<td>CPT-method (Schmertmann) End bearing in rock (Canadian Geotech. Society, 1985)</td>
<td>0.50</td>
</tr>
<tr>
<td>Block Failure, $\Phi_{bf}$</td>
<td>Clay</td>
</tr>
<tr>
<td>Uplift Resistance of Single Piles, $\Phi_{uq}$</td>
<td>All soils</td>
</tr>
<tr>
<td>Lateral Geotechnical Resistance of Single Pile or Pile Group</td>
<td>All soils and rock</td>
</tr>
<tr>
<td>Structural Limit State</td>
<td></td>
</tr>
<tr>
<td>Steel piles</td>
<td>See the provisions of Article 6.5.4.2</td>
</tr>
<tr>
<td>Concrete piles</td>
<td>See the provisions of Article 5.5.4.2</td>
</tr>
<tr>
<td>Timber piles</td>
<td>See the provisions of Article 8.5.2.2 and 8.5.2.3</td>
</tr>
<tr>
<td>Pile Drivability Analysis, $\Phi_{dia}$</td>
<td></td>
</tr>
<tr>
<td>Steel piles</td>
<td>See the provisions of Article 6.5.4.2</td>
</tr>
<tr>
<td>Concrete piles</td>
<td>See the provisions of Article 5.5.4.2</td>
</tr>
<tr>
<td>Timber piles</td>
<td>See the provisions of Article 8.5.2.2 and 8.5.2.3</td>
</tr>
</tbody>
</table>

*In all three Articles identified above, use $\Phi$ identified as “resistance during pile driving”
2.2.2 Canadian Geotechnical Method

When a pile is installed on rock, it is often considered as an end bearing pile. As listed in Table 2, a static analysis method by the Canadian Geotechnical Society is suggested for end bearing on rock (American Association of State Highway and Transportation Officials, 2020).

The Canadian Geotechnical Society proposes the following equation to estimate the approximate capacity of a pile on rock based from rock cores (Canadian Geotechnical Society, 2006).

$$q_u = \sigma_c K_{sp} d$$

Where:

$q_u$ = allowable bearing pressure

$\sigma_c$ = average unconfined compressive strength of rock core, from ASTM D2938

$K_{sp}$ = empirical factor, including a factor of safety of 3

$d$ = depth factor $= 1 + 0.4 \frac{L_s}{B_s} \leq 3$

$L_s$ = depth (length of rock socket)

$B_s$ = diameter of rock socket

The empirical factor $K_{sp}$ which includes a factor of safety of is by the following table.

<table>
<thead>
<tr>
<th>Discontinuity Spacing</th>
<th>Distance (m)</th>
<th>$K_{sp}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moderately Close</td>
<td>0.3 to 1</td>
<td>0.1</td>
</tr>
<tr>
<td>Wide</td>
<td>1 to 3</td>
<td>0.25</td>
</tr>
<tr>
<td>Very Wide</td>
<td>&gt; 3</td>
<td>0.4</td>
</tr>
</tbody>
</table>

The bearing pressure coefficient, $K_{sp}$, takes into account the size effect and the presence of discontinuities, and includes a nominal safety factor of 3 against the lower-bound bearing capacity of the rock foundation. The factors can also be expressed graphically as shown in Figure 3. The relationship in the following graph is valid for a rock mass with spacing of discontinuities greater than 300mm, aperture
of discontinuities less than 5mm, and for foundation width greater than 300mm. The strata must also be near horizontal for sedimentary rocks.

The bearing pressure coefficient, $K_{sp}$, in Figure 3 is defined below, which is valid for $0.05 < \frac{c}{B} < 2.0$ and $0 < \frac{\delta}{c} < 0.02$.

$$K_{sp} = \frac{3 + \frac{c}{B}}{10 \sqrt{1 + 300 \frac{\delta}{c}}}$$

Where:

c = spacing of discontinuities

$\delta$ = aperture of discontinuities

B = footing width
2.2.3 Federal Highway Administration (FHWA) RQD Toe Resistance

The Federal Highway Administration (FHWA) manual provides an expression based on data from Kulhawy and Goodman (Federal Highway Administration, 2016) that showed unit toe resistance \( q_p \) can be estimated from Rock Quality Designation (RQD) of an intact rock mass and the unconfined compressive strength of the rock \( q_u \). The expression is as follows:

For RQD values 0 to 70%

\[
q_p = 0.33 q_u
\]

For RQD values 70 to 100%

\[
q_p = 0.33 q_u \text{ to } 0.80 q_u
\]

Where the nominal toe resistance can be linearly interpolated from 0.33\( q_u \) at the RQD value of 70% to 0.80\( q_u \) at the RQD value of 100% (Federal Highway Administration, 2016).

2.3 Dynamic Design Methods for Driven Piles

For nearly as long as driven piles and foundations have been used engineers have desired to find rational methods to estimate geotechnical resistance of driven piles. Some of the early methods proposed were based on pile penetration during driving. Over time it was determined that more realistic measurements could be obtained during driving and based on pile set per blow. Energy concepts were then developed to equate the potential energy of the hammer to the penetration resistance of the pile as it was driven. This could be used to estimate the geotechnical capacity or nominal pile resistance. These expressions are known as dynamic formulas (Federal Highway Administration, 2016).

2.3.1 FHWA Gates Formula

The AASHTO (2020) LRFD Bridge design specifications includes two dynamic formulas. The first is the FHWA Gates formula is the preferred dynamic formula to predict bearing capacity and establish driving criterion. The AASHTO manual recommends this method and the following formula. (American Association of State Highway and Transportation Officials, 2020)

\[
R_n = 1.75 \sqrt{E_d} \log_{10}(10N_b) - 100
\]
Where:

\[ R_n = \text{nominal pile driving resistance measured during pile driving (kips)} \]

\[ E_d = \text{developed hammer energy. This is the kinetic energy in the ram at impact for a given blow. If ram} \]

velocity is not measured, it may be assumed equal to the potential energy of the ram at the height of the

stroke, taken as the ram weight times the actual stroke (ft-lb)

\[ N_b = \text{Number of hammer blows for 1.0in of pile permanent set (blow/in)} \]

In 1967 the original Gates formula was modified by Roy E. Olson and Kaare S. Flaate (Bostwick, 2014) to have a better statistical fit through the predicted measured data. The FHWA introduced more

modifications which takes the average of the equations of the equations for steel and concrete piles. The

FHWA Gates equation reduced the tendency to under predict capacity and the equations. (Bostwick, 2014)

2.3.2 Engineering News Formula

Another dynamic formula modified to predict nominal bearing resistance and the Engineering

News Formula. This formula was developed by Arthur M. Welllinton in 1892 and was originally developed

for evaluating resistance or capacity of timber piles. This is another recommended formula by AASHTO.

The nominal pile resistance using this method is taken as:

\[ R_n = \frac{12E_b}{(s + 0.1)} \]

Where:

\[ R_n = \text{Nominal pile resistance measured during driving (kips)} \]

\[ E_b = \text{developed hammer energy. This is the kinetic energy in the ram at impact for a given blow. If ram} \]

velocity is not measured, it may be assumed equal to the potential energy of the ram at the height of the

stroke, taken as the ram weight times the actual stroke (ft-lb)

\[ s = \text{pile permanent set (in)} \]

(American Association of State Highway and Transportation Officials, 2020)
The Engineering News formula in its normal form has a factor of safety of 6.0, but for LRFD applications to produced nominal resistance, the factor of safety has been removed. Driving formula should only be used to determine end of driving blow count criteria.

2.3.3 Wave Equation

The wave equation is a dynamic predictive method that represents a better relationship between capacity and driving resistance. This equation was first introduced by Leo A. Pochhammer in 1876 as the analysis of a stress wave propagation through an infinitely long cylindrical bar with a circular cross section. In 1960, E.A Smith proposed an approach that used a numerical closed form solution to investigate the effects of the ram weight, ram velocity, cushion, pile properties, and the soils dynamic behavior during driving (Bostwick, 2014). In his study the pile-soil model was molded into lumped masses connected with springs. The controlling equation for one dimensional wave propagation in a rod in the form of double derivatives as follows (Morton, 2012):

$$\frac{\partial^2 u}{\partial t^2} = \frac{E}{\rho} \frac{\partial^2 u}{\partial z^2}$$

Where:

$E$ = elastic modulus of the pile

$\rho$ = the mass density of the pile

$u$ = displacement of the pile at depth $z$

$z$ = depth below the ground surface

The wave equation proves a relationship between force, stress, and strain in the first set of variables, and displacement, velocity, and acceleration in the second set of variables. Both helps determine the stress within the pile during driving. Results of the wave equation offer a reliable and realistic approach to pile capacities when compared to the values obtained from field test (Bostwick, 2014). The wave equation is normally used with static and dynamic load testing on pile foundations. If a wave equation analysis is used for the determination of the nominal bearing resistance, the driving criterion of the normal (blow count)
may be taken either at the end of driving (EOD) or at the beginning of redrive (BOR) (American Association of State Highway and Transportation Officials, 2020).

2.4 Design Considerations for Bored Piles

A traditional end bearing pile is installed through soft overburden and onto strong rock that the pile gets an end bearing capacity from. Bored piles are drilled down though soft overburden layers for a depth into weathered rock or into weak rock and is terminated within these rocks or on the rock. Piles of this nature act partly as a friction pile and partially as an end bearing pile. These piles can be socketed into the rock, but the depth of the socket can vary. Some factors that govern the bearing capacity and settlement of a bored pile or a cast in place pile are (Tomlinson., 1994).

1. The length to diameter of the socket.
2. The strength and elastic modulus of the rock around and beneath the socket.
3. Layering of the rock with seams of different strength and moduli.

2.5 Current Design Procedure adopted by Georgia Department of Transportation.

The Georgia Department of Transportation Geotechnical Bureau has its own set of guidelines for Load and Resistance Factor Design of deep foundations such as driven piles for bridges. The overview of the process is as follows: (GDOT Geotechnical Bureau, 2020)

1) Organize Drilling

When a BFI is assigned, a consultant must be arranged to do drilling, sampling, and labeling.

2) Perform Field Inspection

An engineer will visit the location where the bridge is to be built and perform a visual inspection. It is good to look at boring and foundation data from other existing bridges in the same area.

3) Examine Soil/Rock Samples and Submit Tests for Classification

Once samples are back from drilling, they are to be examined and compared to the soil descriptions on the field boring logs. If the description does not match the sample write down the boring sample numbers on a form to submit for testing. If there is rock the samples should be sent for, RQD determination, uniaxial compressive strength test, and rock mass rating determined.
4) Prepare Boring Logs

Once samples are submitted for lab testing enter borings into the gINT software. The borings will be preliminary and not include laboratory test results. Once results are obtained correct the soil classification based on the results.

5) Determine Site Class

Site class is a site rating from A to F based on the site’s stiffness. This is determined by shear wave velocity, standard penetration test blow counts, and /or undrained shear strengths in the upper 100 feet of soil samples.

6) Prepare Bridge Foundation Recommendation

It is critical in the LRFD design to make foundation and site class recommendations to the bridge designer. Bridge design loads are also requested at this stage. The foundation types are determined using the following criteria.

i. Geographical Location – North Georgia (above fall line) or South Georgia (below fall line),

ii. Bent Location – Intermediate or End Bents,

iii. Bridge Location/Purpose – Stream/water Crossing, Grade Separation, or Railroad Crossing,

iv. Scour – foundation type must provide adequate penetration below scour,

v. Span Length – Short Span (up to 55 ft. for H-Piles and up to 80 ft. for PSC or MS piles) or Long Spans (greater than 55 ft. for H-Piles and greater than 80 ft. for PSC or MS piles),

vi. Vertical Clearance/Column Height – Short Column (up to 20 ft.) or Long Column (greater than 20 ft.),

vii. Foundation Depth – Shallow (up to 15 ft.), Average (up to 70 ft.), Deep (greater than 70 ft.),

viii. Piling Characteristics – Normal/Uniform vs. Erratic piling,

ix. Geology/Sub-Surface Conditions – Presence of Boulders, Rock Formation, Karst Topography (landforms such as bowl-shaped lime sinks, underground caves and channels), blow counts, presence of compressible clay layers in soil profile, etc.,

x. Other Structures – embedment below walls/wall abutments at end bents, etc.,
xi. Historic Information – the type of foundation previously used for other bridges in the county/vicinity,

xii. Pilot Holes - if your selected foundation type will require pilot holes, discuss alternate foundation type (such as drilled shaft) and most suitable PDA locations with a senior engineer or supervisor.

7) Analysis and Selecting Pile minimum Tips

Steps 7, 8, and 9 cover analysis or bents on driven/drilled pile foundations. Step 10 for analysis of drilled shaft foundations, Step 11 for analysis of spread footing foundations, and Step 12 for analysis for micro pile foundations.

For step 7 the minimum tip elevation is the minimum depth of embedment the pile is to have. Several factors such as theoretical scour, soil density/blow count, and minimum pile length affect where to set the minimum tip elevations. The following are some quick guidelines to use when selecting minimum tip elevations:

a. Set minimum tips in double digit blow count material, preferably 15 blow count soil or denser.

b. At end bents/abutments, set tips a minimum of 5 feet into natural ground and try to have minimum pile lengths of 10 to 15 feet.

c. At intermediate bents, set minimum tips 15 feet below theoretical scour.

8) Analysis with APILE

Once loads are received from the Office of Bridge and Structures the APILE analysis is now ready to be performed. This static analysis is used to determine the required pile depth based on the calculated driving resistance.

9) Analysis- GRL WEAP

This program used the wave equation to model pile driving. It can be used for drivability and bearing analysis.
10) Analysis- Shaft

For bridges which have a drilled shaft as a designed foundation type, analysis must be completed using the SHAFT software.

11) Analysis- Spread Footings

Spread footings are shallow foundation designs where the supporting soil or rock provides adequate bearing resistance without transferring the load deep into the ground with the use of piles or drilled shafts. Procedures for spread footings on soil and spread footings on rock are provided.

12) Micropiles

Micropile foundations are used when difficult ground conditions (such as large boulders in the subsurface or potential for erosion of footings) or construction issues (such as low overhead clearance) are present. For projects where micropiles are used, one axial tension and one axial compression load test should be performed. Capacity for micropiles can be calculated in the same manner as a skin friction drilled shaft.

13) BFI Report

At this step use the BFI report template to make official reporting of findings and analysis easy and consistent.

14) Project location Map and Pictures

Create a project location map of the project and include it as an attachment for the report. The map should have a title at the top that states the P.I. number and the description of the project. The map should show a call-out of the project location and show nearest city, urban areas, major landmarks, or interstates.

15) BFI Checklist

A BFI has been created to help the engineer ensure all options have been considered and standard practices practiced. Fill the checklist.
CHAPTER 3

REVIEW OF ROCK CLASSIFICATION METHODS AND ROCK PARAMETERS FOR QUALITY OF ROCK.

3.1 Introduction

When upper soil layers are very compressible or weak to support loads that are applied by superstructures, deep foundations are used. Piles are a commonly used deep foundation that transmits the superstructure load to the underlying bedrock or a stronger soil layer. The interest of this project is on piles that are to rock layers and bedrock. In cases of piles to bedrock, the ultimate capacity of the pile depends on the load bearing capacity of the underlaying material. Thus, there is a great importance to be able to properly classify, test, and confirm the strength of rock material. Over the decades there have been methods developed to classify rock masses for their strength.

3.2 Rock Parameters

For piles driven into either hard rocks or soft rocks design parameters must be determined. Rock cores are most often collected and from these cores rock weathering, fracturing, strength, and other parameters can be gathered from the rock cores and their classification. In many rock classification systems the transition between hard soils and soft rock happens at an unconfined compression strength, $q_u$, around 20 ksf. The transition between soft and hard rock usually occurs between unconfined compressive strength of 200 and 100 ksf (Federal Highway Administration, 2016). Rock shear strength is typically measured in the laboratory through uniaxial compression testing where recovered core samples are prepared and subjected to loading. As load is applied axial strain is measured and plotted to determine the elastic modulus. The peak load is divided by the specimen’s cross-sectional area to provide an unconfined compressive strength $q_u$. AASHTO and other methods for determining the nominal resistance of end bearing piles on rock utilizes the rock unconfined compressive strength. For both hard rocks and soft rock the FHWA recommends rock classification, core recovery, RQD, unconfined compression strength and density parameter should be quantified for pile designs. (Federal Highway Administration, 2016)
3.2.1 Rock Quality Designation (RQD)

In 1964 D.U. Deere introduced Rock Quality Designation as an index of assessing rock quality quantitatively. The method is a modified per cent core-recovery that uses on sound pieces of the core that are 4in (100mm) or greater in length on the core axis.

\[
RQD = \frac{\text{Sum of Core Pieces } \geq 10\text{cm}}{\text{Total Drill Run}} \times 100\%
\]

The Direct Method is the main method for determining RQD. The International Society for Rock Mechanics for this method recommends a core size of at least 54.7mm be drilled with a double-tube core barrel using a diamond bit. Rock drilling and coring are normally performed at the end of soil boring once bedrock is encountered. The rock core is classified by rock type, core recovery length and given a rock quality designation (RQD) (Federal Highway Administration, 2016). A slow rate of drilling gives a better RQD, and the relationship between RQD and the engineering quality of the rock mass is given in Table 4 (Singh & Goel, 1999).

*Table 4 Correlation between RQD and Rock Mass Quality* from (Singh & Goel, 1999)

<table>
<thead>
<tr>
<th>No.</th>
<th>RDQ (%)</th>
<th>Rock Quality</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>&lt;25</td>
<td>Very Poor</td>
</tr>
<tr>
<td>2</td>
<td>25-50</td>
<td>Poor</td>
</tr>
<tr>
<td>3</td>
<td>50-75</td>
<td>Fair</td>
</tr>
<tr>
<td>4</td>
<td>75-90</td>
<td>Good</td>
</tr>
<tr>
<td>5</td>
<td>90-100</td>
<td>Excellent</td>
</tr>
</tbody>
</table>

RQD is a simple and generally inexpensive index. However, when considered alone it is often not sufficient to provide adequate description of a rock mass because it does not take into account joint orientation, joint condition, and stress condition. However, RQD values can be indicative of the pile penetration into need that would be needed to satisfy resistance requirements when they are combined with additional test results. (Federal Highway Administration, 2016)
3.2.2 Rock Mass Rating

The Rock Mass Rating (RMR) system was developed at the South African Council of Scientific and Industrial Research by Z. T. Bieniawski in 1973. Since its development, the system has been modified several times. Each change altered how RMR was calculated so there is an importance to make note of which version is used for official purposes. In general, to apply this system a given site should be divided by into several geological structural units in a way that each type of rock mass is represented by a separate geological structural unit (Singh & Goel, 1999).

Rock Mass Rating can be determined by five parameters: uniaxial compressive strength of intact rock material, rock quality designation (RQD), joint or discontinuity spacing, condition of discontinuities, and ground water condition.

Each of the five parameters are found using a table. The strength of the intact rock material parameter is obtained from rock cores in accordance with the site conditions. The ratings are based on uniaxial compressive strength and point load strength in Table 5. Rock Quality Designation is the second parameter, and a rating are given in Table 6.

Table 5 Strength of Intact Rock Material from (Singh & Goel, 1999)

<table>
<thead>
<tr>
<th>Qualitative Description</th>
<th>Compressive Strength (MPa)</th>
<th>Point Load Strength (Mpa)</th>
<th>Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exceptionally Strong</td>
<td>&gt;250</td>
<td>8</td>
<td>15</td>
</tr>
<tr>
<td>Very strong</td>
<td>100 – 250</td>
<td>4 – 8</td>
<td>12</td>
</tr>
<tr>
<td>Strong</td>
<td>50 – 100</td>
<td>2 – 4</td>
<td>7</td>
</tr>
<tr>
<td>Average</td>
<td>25 – 50</td>
<td>1 – 2</td>
<td>4</td>
</tr>
<tr>
<td>Weak</td>
<td>10 – 25</td>
<td>use of uniaxial compressive strength</td>
<td>2</td>
</tr>
<tr>
<td>Very weak</td>
<td>2 – 10</td>
<td>–</td>
<td>1</td>
</tr>
<tr>
<td>Extremely weak</td>
<td>1 – 2</td>
<td>–</td>
<td>0</td>
</tr>
</tbody>
</table>

Note: At compressive strength less than 0.6 MPa, many rock material would be regarded as soil.
Table 6 Rock Quality Designation (RQD) with Rating from (Singh & Goel, 1999)

<table>
<thead>
<tr>
<th>Qualitative Description</th>
<th>RQD</th>
<th>Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excellent</td>
<td>90 – 100</td>
<td>20</td>
</tr>
<tr>
<td>Good</td>
<td>75 – 90</td>
<td>17</td>
</tr>
<tr>
<td>Fair</td>
<td>50 – 75</td>
<td>13</td>
</tr>
<tr>
<td>Poor</td>
<td>25 – 50</td>
<td>8</td>
</tr>
<tr>
<td>Very Poor</td>
<td>&lt; 25</td>
<td>3</td>
</tr>
</tbody>
</table>

The Spacing of Discontinuities would be the third parameter and covers joint foliations, minor faults, shear zones, and other surfaces of weaknesses. The distance between two adjacent discontinuities should be measured for all sets of discontinuities and the rating obtained from Table 7.

Table 7 Spacing of Discontinuities from (Singh & Goel, 1999)

<table>
<thead>
<tr>
<th>Description</th>
<th>Spacing (m)</th>
<th>Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Wide</td>
<td>&gt;2</td>
<td>20</td>
</tr>
<tr>
<td>Wide</td>
<td>0.6 - 2</td>
<td>15</td>
</tr>
<tr>
<td>Moderate</td>
<td>0.2 – 0.6</td>
<td>10</td>
</tr>
<tr>
<td>Close</td>
<td>0.06 – 0.2</td>
<td>8</td>
</tr>
<tr>
<td>Very Close</td>
<td>&lt; 0.06</td>
<td>5</td>
</tr>
</tbody>
</table>

The fourth parameter, Condition of Discontinuities, includes roughness of discontinuity, the separation length, weathering of the wall rock or places of weakness. This rating is given by Table 8.
Table 8 Conditions of Discontinuities from (Singh & Goel, 1999)

<table>
<thead>
<tr>
<th>Description</th>
<th>Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very rough and unweathered, wall rock tight and discontinuous, no separation</td>
<td>30</td>
</tr>
<tr>
<td>Rough and slightly weathered, wall rock surface separation &lt;1mm</td>
<td>25</td>
</tr>
<tr>
<td>Slightly rough and moderately to highly weathered, wall rock surface separation &lt;1mm</td>
<td>20</td>
</tr>
<tr>
<td>Slickensided wall rock surface or 1-5mm thick gouge or 1-5mm wide continuous discontinuity</td>
<td>10</td>
</tr>
<tr>
<td>5mm thick, soft gouge, 5mm wide continuous discontinuity</td>
<td>0</td>
</tr>
</tbody>
</table>

Ground water presence, the fifth parameter, is mostly for the case of tunnels or conditions where groundwater is near the surface or near where piling is to occur. The groundwater flow is determined, or the general condition is described as completely dry, damp, wet, dripping and flowing as seen in Table 9.

Table 9 Ground Water Condition from (Singh & Goel, 1999)

<table>
<thead>
<tr>
<th>Inflow per 10m tunnel length (liter/min.)</th>
<th>None</th>
<th>&lt;10</th>
<th>10 – 25</th>
<th>25 – 125</th>
<th>&gt;125</th>
</tr>
</thead>
<tbody>
<tr>
<td>Joint water pressure / Major principal stress</td>
<td>0</td>
<td>0 – 0.1</td>
<td>0.1 – 0.2</td>
<td>0.2 – 0.5</td>
<td>&gt;0.5</td>
</tr>
<tr>
<td>General description</td>
<td>Completely dry</td>
<td>Damp</td>
<td>Wet</td>
<td>Dripping</td>
<td>Flowing</td>
</tr>
<tr>
<td>Rating</td>
<td>15</td>
<td>10</td>
<td>7</td>
<td>4</td>
<td>0</td>
</tr>
</tbody>
</table>

The addition of the rated values of the above five ratings parameters will give the basic rock mass rating between 0 and 100 and additional rating adjustments for discontinuity orientations are available for different applications.

Table 10 was introduced by (Hoek, 2007) based upon the 1989 version of the RMR classification by (Bieniawski, 1989).
### Table 10 Rock Mass Rating System From (Hoek, 2007)

| A. CLASSIFICATION PARAMETERS AND THEIR RATINGS | \( \begin{array}{|c|c|c|c|c|c|c|} \hline \text{Parameter} & \text{Range of values} & \text{Class} & \text{Rating} & \text{Condition of discontinuities} & \text{Strike and dip orientations} & \text{Strike perpendicular to tunnel axis} \\ \hline \text{Strength of intact rock material} & \text{Point-load strength index} & >10 \text{ MPa} & 4 - 10 \text{ MPa} & 2 - 10 \text{ MPa} & 1 - 2 \text{ MPa} & \text{Unweathered} \\ \hline \text{Uniaxial comp. strength} & >250 \text{ MPa} & 100 - 250 \text{ MPa} & 50 - 100 \text{ MPa} & 25 - 50 \text{ MPa} & 5 - 25 \text{ MPa} & 1 - 5 \text{ MPa} & < 1 \text{ MPa} \\ \hline \text{Ratings} & & 15 & 12 & 7 & 4 & 2 & 1 & 0 \\ \hline \text{Drill core Quality RQD} & 90\% - 100\% & 75\% - 90\% & 50\% - 75\% & 25\% - 50\% & < 25\% \\ \hline \text{Rating} & 20 & 17 & 13 & 8 & 3 \\ \hline \text{Spacing of} & > 2 \text{ m} & 0.6 - 2 \text{ m} & 200 - 600 \text{ mm} & 60 - 200 \text{ mm} & < 60 \text{ mm} \\ \hline \text{Rating} & 20 & 15 & 10 & 8 & 5 \\ \hline \text{Condition of discontinuities} & \text{Joint water pressure} & \text{Inflow per 10 m tunnel length (lim)} & \text{None} & < 10 & 10 - 25 & 25 - 125 & > 125 \\ \hline \text{Rating} & 30 & 25 & 20 & 10 & 0 \\ \hline \text{Groundwater} & \text{Inflow per 10 m tunnel length (lim)} & \text{Major principal e} & 0 & < 0.1 & 0.1 - 0.2 & 0.2 - 0.5 & > 0.5 \\ \hline \text{General conditions} & \text{Completely dry} & \text{Damp} & \text{Wet} & \text{Dripping} & \text{Flowing} \\ \hline \text{Rating} & 15 & 10 & 7 & 4 & 0 \\ \hline B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F) | | \text{Strike and dip orientations} & \text{Very favorable} & \text{Favorable} & \text{Fair} & \text{Unfavorable} & \text{Very Unfavorable} \\ \hline \text{Ratings} & \text{Tunnels & mines} & 0 & -2 & -5 & -10 & -12 \\ \hline \text{Foundations} & 0 & -2 & -7 & -15 & -25 \\ \hline \text{Slopes} & 0 & -5 & -25 & -50 \\ \hline C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS | | \text{Class number} & I & II & III & IV & V \\ \hline \text{Description} & \text{Very good rock} & \text{Good rock} & \text{Fair rock} & \text{Poor rock} & \text{Very poor rock} \\ \hline D. MEANING OF ROCK CLASSES | | \text{Class number} & I & II & III & IV & V \\ \hline \text{Average stand-up time} & 20 yrs for 15 m span & 1 year for 10 m span & 1 week for 5 m span & 10 hrs for 2.5 m span & 30 min for 1 m span \\ \hline \text{Cohesion of rock mass (kPa)} & > 400 & 300 - 400 & 200 - 300 & 100 - 200 & < 100 \\ \hline \text{Friction angle of rock mass (deg)} & > 45 & 35 - 45 & 25 - 35 & 15 - 25 & < 15 \\ \hline E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions | | \text{Discontinuity length (persistence)} & \text{Rating} & \text{< 1 m} & 6 & 1 - 3 \text{ m} & 4 & 3 - 10 \text{ m} & 2 & 10 - 20 \text{ m} & > 20 \text{ m} \\ \hline \text{Separation (aperture)} & \text{Rating} & \text{None} & 6 & < 0.1 \text{ mm} & 5 & 0.1 - 1.0 \text{ mm} & 4 & 1 - 5 \text{ mm} & > 5 \text{ mm} \\ \hline \text{Roughness Rating} & \text{Very rough} & 6 & \text{Rough} & 5 & \text{Slightly rough} & 3 & \text{Smooth} & 1 & \text{Slickensided} \\ \hline \text{Infilling (gouge)} & \text{Rating} & \text{None} & 6 & \text{Hard filling < 5 mm} & 5 & \text{Hard filling > 5 mm} & 4 & \text{Soft filling < 5 mm} & 3 & \text{Soft filling > 5 mm} & 2 & \text{Decomposed} \\ \hline \text{Weathering} & \text{Unweathered} & 6 & \text{Slightly weathered} & 5 & \text{Moderately weathered} & 4 & \text{Highly weathered} & 3 & \text{Decomposed} & 2 & \text{Decomposed} & 1 & \text{Decomposed} \\ \hline F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING** | | \text{Strike perpendicular to tunnel axis} & \text{Drive with dip – Dip 45 – 90\textdegree} & \text{Drive with dip – Dip 20 – 45\textdegree} & \text{Dip 45 – 90\textdegree} & \text{Dip 20 – 45\textdegree} \\ \hline \text{Very favorable} & \text{Favorable} & \text{Very unfavorable} & \text{Fair} \\ \hline \text{Drive against dip – Dip 45-90\textdegree} & \text{Drive against dip – Dip 20-45\textdegree} & \text{Dip 0-20 – Irrespective of strike\textdegree} \\ \hline \text{Fair} & \text{Unfavorable} & \text{Fair} \\ \hline | | | | | | * Some conditions are mutually exclusive. For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly. ** Modified after Wickham et al (1972).
3.2.3 Rock Mass Index (Rmi)

The Rock Mass Index (Rmi) is used to characterize rock mass strength as a construction material and is based on selected well defined geological parameters. The system was proposed by Palmstrom in 1995 (Palmstrom, 1996). Rock masses have various discontinuities that tend to reduce the inherent strength of the rock mass index express as

\[ RMi = q_c \times J_p \]

Where:

\( q_c \) = the uniaxial compressive strength of the intact rock material in MPa.

\( J_p \) = the jointing parameter that is composed of four jointing characteristics of block volume or density of joints, joint roughness, joint alteration, and joint size.

The value of \( J_p \) varies from almost 0 for crushed rock masses to 1 for intact rocks. RMi is the rock mass index denoting the uniaxial compressive strength of the rock mass in Mpa (Singh & Goel, 1999).

The parameters selected to be used in RMi are recommended to represent the average condition of the rock mass. However, it is still important to retain the names for different rock types present and their parameters. From the study of 15 different classification systems that have been used by Palmstrom (1995) in the selection of these input parameters to RMi.

1. The size of the blocks delineated by joints- measured as block volume, \( V_b \)
2. Strength of block material- measured as uniaxial compressive strength \( q_c \)
3. The shear strength of the block faces- characterized by factors for the joint characteristics, \( j_R \) and \( j_A \) (Tables 11 and 13).
4. The size and termination of the joints – given as their length and continuity factor, \( j_L \)

The expression to find the value of \( J_p \) is as follows.

\[ J_p = 0.2(jC)^{0.5} (Vb)^D \]
Where:

\( j_C \) = is the joint condition factor

\( V_b \) = the block volume which can be found from field measurements. \( V_b \) is given in \( m^3 \)

\[ D = 0.37 \cdot j_C^{-0.2} \]

Joint condition factor \( j_C \) is correlated with \( j_R \), \( j_A \), and \( j_L \) as follows.

\[ j_C = j_L \left( \frac{j_R}{j_A} \right) \]

Various parameters of RMi and their combinations in the Rock Mass Index are shown in the following Tables 11, 12, and 13. It shows a graphical combination of block volume (\( V_b \)), joint condition factor (\( j_C \))

Table 11 The Joint Roughness Ratings \( j_R \) Found from Smoothness and Waviness from (Palmstrom, 1996)

<table>
<thead>
<tr>
<th>Small Scale Smoothness* Of Joint Surface</th>
<th>Large Scale Waviness of Joint Plane</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Planar</td>
</tr>
<tr>
<td>Very Rough</td>
<td>3</td>
</tr>
<tr>
<td>Rough</td>
<td>2</td>
</tr>
<tr>
<td>Slightly Rough</td>
<td>1.5</td>
</tr>
<tr>
<td>Smooth</td>
<td>1</td>
</tr>
<tr>
<td>Polished</td>
<td>0.75</td>
</tr>
<tr>
<td>Slickensided**</td>
<td>0.6-1.5</td>
</tr>
</tbody>
</table>

* For filled joints: \( j_R = 1 \).

** For slickensided joints the values of \( R \) depends on the presence and outlook of the striations; the highest value is used for marked striations.
Table 12 The Joint Length and Continuity Rating $jL$ from (Palmstrom, 1996)

<table>
<thead>
<tr>
<th>Joint Length (m)</th>
<th>Term</th>
<th>Type</th>
<th>$jL$</th>
<th>Continuous Joints</th>
<th>Discontinuous Joints</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>&lt; 0.5</td>
<td>Very Short</td>
<td>Bedding/foliation parting</td>
<td>3</td>
<td>6</td>
<td></td>
</tr>
<tr>
<td>0.1 – 1</td>
<td>Short/small</td>
<td>Joint</td>
<td>2</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>1 – 10</td>
<td>Medium</td>
<td>Joint</td>
<td>1</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>10 – 30</td>
<td>Long/large</td>
<td>Joint</td>
<td>0.75</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>&gt; 30</td>
<td>Very long/large</td>
<td>Filled joint scam* Or shear</td>
<td>0.5</td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>

* Often a singularity, and should in these cases be treated separately
Table 13 Characterization and Rating of the Joint Alteration Factor $j_A$ from (Palmstrøm, 1996)

<table>
<thead>
<tr>
<th>Term</th>
<th>Description</th>
<th>$j_A$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>A. Contact between rock wall surfaces</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clean joints</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Healed or welded joints</td>
<td>Softening, impermeable filling (quartz, epidote, etc.)</td>
<td>0.75</td>
</tr>
<tr>
<td>Fresh rock walls</td>
<td>No coating or filling on joint surface, except of staining</td>
<td>1</td>
</tr>
<tr>
<td>Alteration of joint wall</td>
<td></td>
<td></td>
</tr>
<tr>
<td>i. 1 grade more altered</td>
<td>The joint surface exhibits one class higher alteration than the rock</td>
<td>2</td>
</tr>
<tr>
<td>ii. 2 grade more altered</td>
<td>The joint surface shows two classes higher alteration than the rock</td>
<td>4</td>
</tr>
<tr>
<td>Coating or thin filling</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand, silt calcite, etc.</td>
<td>Coating of friction material without clay</td>
<td>3</td>
</tr>
<tr>
<td>Clay, chlorite, talc, etc.</td>
<td>Coating of softening and cohesive minerals</td>
<td>4</td>
</tr>
<tr>
<td><strong>B. Filled joints partly or no contact between the rock wall surfaces</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type of Filling Material</td>
<td>Description</td>
<td>Partly Wall Contact (thin filing &lt;5mm*)</td>
</tr>
<tr>
<td>Sand, silt, calcite, etc.</td>
<td>Filing of friction material without clay</td>
<td>4</td>
</tr>
<tr>
<td>Compacted clay materials</td>
<td>“Hard” filing of softening and cohesive materials</td>
<td>6</td>
</tr>
<tr>
<td>Soft clay materials</td>
<td>Medium to low over-consolidation of filing</td>
<td>8</td>
</tr>
<tr>
<td>Swelling clay materials</td>
<td>Filing material exhibits clear swelling properties</td>
<td>8-12</td>
</tr>
</tbody>
</table>

* Based on joint thickness division in RMR system (Bieniawski, 1973)
Figure 4 shows how the combination of parameters are used in the calculation of in the RMI.

**Figure 4 The Main Parameters in the Rock Mass that are Applied in RMI** from (Palmstrom, 1996)

It is common for the joint condition factor $jC$ and the jointing parameter $J_p$ to be given as

$$jC = 0.2Vb^{0.38} \quad \text{and} \quad J_p = 0.28Vb^{0.32}$$

For $jC=1.75$ and the jointing parameter can be expressed as

$$J_p = 0.25(Vb)^{0.33}$$

For $jC=1$ the jointing parameter is expressed as

$$J_p = 0.2Vb^{0.37}$$

(Singh & Goel, 1999)

When a sample size of a rock mass is enlarged from laboratory size to field size a significant scaling effect is involved. For very large rock masses where the jointing parameter $J_p \approx 1$ the scale effect for uniaxial compressive strength $q_c$ should be accounted for as $\sigma_c$ is related to a sample a size of 50mm. Data presented by Hoek and Brown (1980) and Wagner (1987) that the actual compressive strength for large field samples with diameter $d$ (in mm). Figure 5 shows the graphical representation of Hoek and Brown and Figure 6 the jointing parameter value for joint conditions (Palmstrom, 1996).

$$\sigma_c f = \sigma_{c50} \left(\frac{0.05}{D_b}\right)^{0.2} = \sigma_{c50} \times f_\sigma$$
Where:

\( \sigma_{c\,50} = \) is the uniaxial compressive strength for 50mm sample size.

\( D_b = \) block diameter measured in meter

\( f_\sigma = \left( \frac{0.05}{D_b} \right)^{0.2} \) is the scale factor for compressive strength.

Figure 5 Empirical Equations for the Scale Effect of the Uniaxial Compressive Strength based on data from Hoek and Brown from (Palmstrom, 1996)
Figure 6 The Jointing Parameter Jp Found from the Joint Condition Factor jC and Various Measurements of Jointing Intensity (Vb, Jv, RDQ) from (Palmstrom, 1996)
The classification of RMI is presented in the following Table 14. Alone numerical values are not sufficient for proper characterizing of complex materials such as rock masses. RMi parameters are accompanied by supplementary descriptions.

*Table 14 Classification of RMi from (Singh & Goel, 1999)*

<table>
<thead>
<tr>
<th>TERM for RMi</th>
<th>Related to Rock Mass Strength</th>
<th>RMi VALUE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extremely low</td>
<td>Extremely weak</td>
<td>&lt;0.001</td>
</tr>
<tr>
<td>Very low</td>
<td>Very weak</td>
<td>0.001-0.01</td>
</tr>
<tr>
<td>Low</td>
<td>Weak</td>
<td>0.01-0.1</td>
</tr>
<tr>
<td>Moderate</td>
<td>Medium</td>
<td>0.1-1.0</td>
</tr>
<tr>
<td>High</td>
<td>Strong</td>
<td>1.0-10.0</td>
</tr>
<tr>
<td>Very high</td>
<td>Very strong</td>
<td>10-100</td>
</tr>
<tr>
<td>Extremely high</td>
<td>Extremely strong</td>
<td>&gt; 100</td>
</tr>
</tbody>
</table>

Some of the advantages of using the rock mass index (RMi) are that its systematic approach of rock mass characteristics will enhance accuracy of the input data needed. RMi can be used for rough estimates when limited ground condition information is available. RMi offers a stepwise judgement suitable for engineering judgement. The RMi covers a wide variety of rock masses and has a wide application. Some limitations of this system however are that it can only express compressive rock strength of masses. It is not possible to characterize all the variations of a rock mass in a single number with this system but may characterize a wide range of materials. RMi may best be considered as a relative index in its characterization of rock mass strength (Palmstrom, 1996).

### 3.2.4 Geological Strength Index (GSI)

The strength of intact rock material is determined often by using the results of unconfined compressive test on intact rock cores. The strength of the rock mass should first be classified by using its geological strength index (GSI) then assessed using the Hoek-Brown failure criterion (American
The geological strength index was introduced by Hoek and Brown for both hard and weak rock masses. It is generally liked for its simple, fast, and reliable classification based on visual inspection of the geological conditions (Singh & Goel, 1999). As computer modeling and testing became more prevalent Hoek and Brown developed charts for estimating GSI based on the following correlations.

\[
GSI = \begin{cases} 
RMR - 5 & \text{for } GSI \geq 18 \text{ or } RMR \geq 23 \\
9 \ln Q' + 44 & \text{for } GSI < 18 
\end{cases}
\]

Where:

- \(Q'\) = the modified rock mass quality index
- \(RMR\) = the Rock Mass Rating

Evert Hoek and E. T. Brown (1997) proposed a chart for GSI so experts can classify a rock mass by visual inspection alone. In this classification the four main qualitative classifications are: 1. Blocky, 2. Very Blocky, 3. Blocky/Folded, 4. Crushed. These are adopted from the Terzaghi classification. Furthermore, discontinuities are classified into 5 surface conditions of 1. Very Good, 2. Good, 3. Fair, 4. Poor, and 5. Very Poor (Singh & Goel, 1999). The Hoek and Brown chart can be found in the AASHTO LRFD Bridge design manual and can be seen in Figure 7.
Figure 7 Geological Strength Index for Jointed from (American Association of State Highway and Transportation Officials, 2020)
GSI assumes that the rock mass is isotropic, and therefore only rock cores without weak planes should be tested in triaxial cell to determine \( q_c \) and \( m_r \) as GSI downgrades strength according to schistosity. Hoek-Brown in 1994 suggested the following modified strength criterion for a jointed rock mass

\[
\sigma_1 = \sigma_3 + q_u \left[ m_b \times \frac{\sigma_3}{q_u} + s \right]^a
\]

Where:

\[
s = e^{\left( \frac{GSI-100}{9-3D} \right)}
\]

\[
a = \frac{1}{2} + \frac{1}{6} \left( e^{-\frac{GSI}{15}} - e^{-\frac{20}{3}} \right)
\]

\( \sigma_1 \) = the maximum principal stress,

\( \sigma_3 \) = the effective principal stress,

\( q_u \) = the average unconfined compressive strength of the rock core,

\( D \) = the disturbance factor (dim) that ranges from 0.0 to 1.0,

\( m_b, s, \) and \( a \) are empirically determined parameters.

\[
m_b = m_i e^{\left( \frac{GSI-100}{28-14D} \right)}
\]

The constant \( m_i \) can be found from Table 10.4.6.4.1 in the AASHTO LRFD Bridge Design Specifications (American Association of State Highway and Transportation Officials, 2020). The values are given in Table 15.
Table 15 Values of the Constant $m_i$ by Rock Group from (American Association of State Highway and Transportation Officials, 2020)

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Class</th>
<th>Group</th>
<th>Texture</th>
<th>Coarse</th>
<th>Medium</th>
<th>Fine</th>
<th>Very fine</th>
</tr>
</thead>
<tbody>
<tr>
<td>SEDIMENTARY</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Clastic</td>
<td>Non-Clastic</td>
<td>Sedimentary</td>
<td>Carbonates</td>
<td>Crystalline Limestone (12 ± 3)</td>
<td>Sandstone 17 ± 4</td>
<td>Siltstone 7 ± 2 Greywacke (18±3)</td>
<td>Claystone 4 ± 2 Shale (6 ± 2) Marl (7 ± 2)</td>
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<tr>
<td>Evaporites</td>
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<td></td>
<td></td>
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<tr>
<td>Organic</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>METAMORPHIC</td>
<td>Non Foliated</td>
<td>Metamorphic</td>
<td>Marble 9 ± 3</td>
<td>Hornfels (19 ± 4) Metasandstone (19±3)</td>
<td>Quartzite 20 ± 3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slightly Foliated</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
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<tr>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Foliated*</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>IGNEOUS</td>
<td>Plutonic</td>
<td>Igneous</td>
<td>Light</td>
<td>Granite 32 ± 3 Granodiorite (29 ± 3)</td>
<td>Diorite 25 ± 5 Granodiorite (29 ± 3)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
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<td></td>
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</tr>
<tr>
<td></td>
<td>Dark</td>
<td></td>
<td></td>
<td>Gabbro 27 ± 3 Norite 20 ± 5</td>
<td>Dolerite (16 ± 5) Norite 20 ± 5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hypabyssal</td>
<td></td>
<td></td>
<td></td>
<td>Porphyries (20 ± 5)</td>
<td>Porphyries (20 ± 5)</td>
<td>Diabase (15 ± 5)</td>
<td>Peridotite (25 ± 5)</td>
</tr>
<tr>
<td>Volcanic</td>
<td>Lava</td>
<td></td>
<td></td>
<td>Rhyolite (25 ± 5) Andesite 25 ± 5</td>
<td>Dacite (25 ± 3) Basalt (25 ± 5)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pyroclastic</td>
<td></td>
<td></td>
<td></td>
<td>Agglomerate (19 ± 3)</td>
<td>Volcanic Breccia (19 ± 5)</td>
<td>Tuff (13 ± 5)</td>
<td></td>
</tr>
</tbody>
</table>

These values are for intact rock specimens tested normal to bedding or foliation. The value of $m_i$ will be significantly different if failure occurs along a weakness plane.
CHAPTER 4

REVIEW CURRENT SPECIFICATIONS AND VERIFICATION METHODS FOR PILE INSTALLATION IN ROCK WITH A PILOT HOLE AND THE EQUIPMENT/METHODS BY OTHER STATES.

4.1 Introduction

Using a driving assistant method such as a pilot hole or pre-drilling is a common technique when driving a pile confronts high resistance that may damage the pile. However, it is common sense that when predrilling is applied, the pile will lose resistance and thus the capacity of the piles will be eventually reduced. Therefore, it is critical to understand how much reduction in capacity is expected and how the actual capacity is verified. Unfortunately, there is no consensus on such activities, especially when the pile is sitting on rock layer, because the piles in such cases usually have less chance of having capacity failure and the structural failure could be more possible than the failure of the bearing layer.

This task was completed by reviewing existing pilot hole specifications followed by other State Departments of Transportation (DOTs). The documents including the Standard Specifications for Roads and Bridges or similar design specifications on bridge foundations were reviewed from each state. In addition, a survey was sent out to all 50 State DOT agencies asking specifically about their use of a pile with a pilot hole. The survey was way to get an idea of the current way states handle the case of a pile with a pilot hole was the survey. The five-question survey was sent to all 50 state DOT agencies, and summary of the answers to questions 1, 2 and 3 will be presented in this chapter. The answers to the questions gave a picture to terminology, conditions of use, and hole size are viewed across all 50 DOT agencies.
The survey questions are as follows:

Q1) Do you use a pilot hole (or other driving aid methods such as predrilled, pre-boring, pre-augured etc.) for the piles? If yes, how do you call it, pilot hole, predrilled, pre-boring, others?

Q2) Do you use this for soil, rock or both?

Q3) Do you use a larger or smaller hole than the size of the pile? How much larger or smaller, if suggested?

Q4) Do you take into account the skin friction when estimating the pile capacity?

Q5) When designing, how do you estimate the capacity of this pile type? That being a pile that will be constructed with a driving aid.

Based on the review, it was found that there are no common standards of the pile with a pilot hole, from terminology to construction. Every state has different terminologies and requirements related to pilot hole diameter and the use of skin friction. Depending on the soil conditions some states recommended the use of a larger pilot hole and others recommended the use of a smaller one. The same went for the account of skin friction or not. Additionally, some states indicated they use pilot holes in soil layers only while others use them in soils and rock layers, and a few only use them in rock. The following section summarizes the current practice by state highway agencies, their responses to the survey, and a summary of the first three questions of the survey.

4.2 Responses from States

The survey results are summarized in the alphabetical order of the states. The state-by-state summary includes information from the first survey summary with additional information from the state standard specifications.

1) Alabama: From the survey Alabama refers to the hole has a pilot hole. Pilot holes are to be a size smaller than the diameter or diagonal of the pile cross section that is sufficient to allow penetration of the pile to a specific depth. When subsurface obstructions are encountered such as rocks or boulders the hole diameter may be increased to the least dimension of the pile. These pilot holes are used to
advance piles in soil and rock. Additional information from the specifications provided that augering, wet rotary drilling, and other methods are used to make the pilot hole. Pilot holes are only used when it has been approved by the engineer or is shown on the plans. Where the piles are to be end bearing on rock the pilot hole may be carried to the surface of the rock. After the pile is placed in the pilot hole the voids around the pile are backfilled with clean sand. Pilot holes that terminate in rock are backfilled to the top of the rock with substructure concrete after seating the pile. In the design process skin friction is not taken into account when estimating the capacity of the pile. The pile is often driven to refusal and the pile is limited to the structural capacity of the pile (Alabama Department of Transportation, 2018).

2) Alaska: From the survey Alaska uses the terms “predrilled” and “pre-bore” for pilot holes. The hole sized varies per situation but are typically larger. The pilot holes are typically used for very dense soils, or highly weathered rock, where pile damage will not occur when driving through the pre-drilled hole. As well from the survey response the hole size is no larger than 75% of the pile diameter. When drilling in a rock socket the diameter will be always greater than the pile. After driving is complete the space around the pile is backfilled with sand. The pilot holes are used when expected driving stresses are higher than the pile can withstand using a reasonable hammer. In the design process of a pile with a pilot hole all of the resistance that the driven pile can provide is included, regardless if it is end bearing or not. The resistance is verified during installation with the use of the wave equation without signal matching or with dynamic testing via PDA. When predrilling is incorporated in contracts, it is because it has been assessed that without, the pile would not reach refusal when using a typical hammer. The pre-bore must extend to the minimum pile penetration specified that will achieve the required lateral resistance. It is expected that the pile will then quickly achieve refusal (or at least the required resistance) once it is driven past the pre-drilled depth and into the native material. Additional information from the specifications provides that piles placed at abutment embankments that are more than 5 feet in depth require pre-drilling. The size of the pre-drilled hole is 2 inches larger than the diameter or largest dimension of the pile. In the specifications when driving piles through new embankment and the depth of the embankment at the pile location is in excess of 5 feet, the pile is
driving in a hole made through the embankment, and the hole diameter is 6 inches greater than the pile (Alaska Department of Transportation Public Facilities, 2017).

3) **Arizona**: From the survey Arizona uses the term “predrilled” for pilot holes. The hole size shall have a diameter of not less than the greatest dimension of the pile cross section plus six inches. After driving the pile, the space around the pile is backfilled with sand or pea gravel. In response to the survey, it was mentioned that Arizona rarely uses driven piles and prefers the use of drilled shafts. It has been more the 20 years since Arizona constructed with driven piles. So, the state was counted as not using pilot holes.

3) **Arkansas**: Arkansas uses the term “prebored holes” for pilot holes. The size of the prebore hole varies depending on the situation. Information from the survey adds that a larger hole is used for integral end bents and smaller holes are used to achieve pile penetration. When a larger hole is used it is specified that the holes will be six inches greater than the pile diameter or diagonal, and a smaller hole size is to be determined in the field. In the design process skin friction is not taken into account for prebored holes that are larger than the pile. Skin friction is accounted for holes that are smaller than the pile. Information from the survey also adds that if preboring is specified for only a portion of the length of the pile below the ground, we account for both end bearing and skin friction below the prebored hole depth. If preboring is the full length of the pile below the ground, we only account for end bearing. According to the specifications prebored holes will be smaller than the diameter or diagonal of the pile cross section and allow penetration of the pile to the specified depth. If subsurface obstructions, such as boulders or rock layers, are encountered the hole diameter may be increased to the least dimension that is adequate for pile installation. Any void space is backfilled with sand, sand grout mixture, or other approved materials (Arkansas State Highway Transportation Department, 2014).

4) **California**: From the survey California uses the terms “drilling” and “predrilled holes” for pilot holes. The size of the holes varies and are used in both soils and rock. For drilled holes the hole diameter is no greater than the least dimension of the pile. Information from the survey also provided that skin friction is typically ignored, and the design side resistance starts at the bottom the hole. From the
foundation manual drilled holes are used if it is necessary to attain the specified tip elevation. For predrilled holes driven through embankments constructed under the contract, piles are driven through predrilled holes where the depth of the new embankment at the pile location is in excess of 5 feet. The hole diameter must be at least 6 inches larger than the greatest dimension of the pile cross section. The larger holes are backfilled to the ground surface with dry sand or pea gravel (California Department of Transportation Division of Engineering Services, 2015).

5) **Colorado**: From the survey Colorado uses the term “predrilled” for pilot holes and they are used in both soil and rock layers. Additional information from the survey provided that skin friction is accounted for in the design when the supporting strata is well defined. When piles are driven to bedrock the use of skin friction is unnecessary. From the survey the engineer provided that the size of the predrilled hole would be larger than the pile being driven. This would ensure the pile reached the supporting strata without being damaged by objects in the way. According to the state specifications “drilled” holes are to be two inches smaller the diameter or diagonal of the pile cross section, and the hole size is increased to the least dimension of the pile if boulders or rock layers are encountered. Where piles are to be end-bearing on rock or very dense cobbles and gravels (hardpan), drilling may be carried to the surface of the rock or the hardpan. Any void space around the pile us backfilled with sand, pea gravel, concrete, or other materials (Colorado Department of Transportation, 2021).

6) **Connecticut**: From the survey Connecticut uses the term “pre-augering” for pilot holes and they can be used in both soil and rock. In cases that larger holes are used, they are not used for non-bearing portions of friction piles. In the design process skin friction is not taken into account for piles within a pre-augered section of the hole. Standard pile design methods per AASHTO are used for end bearing and portions of frictions piles not in the augered portion. According to the specifications pre-augered holes are used when stated in the contract and are a size smaller that the diameter or diagonal of the pile cross section. If subsurface obstructions, such as boulders or rock layers, are encountered, the hole diameter may be increased to the least dimension which is adequate for pile installation. Any remaining
void space is backfilled with sand or approved material (Connecticut Department of Transportation, 2020).

7) Delaware: From the survey Delaware uses the term “augering” for pilot holes and these holes are used in both soil and rock. Additional information from the survey added that augered holes are rarely used in the state. There are no recommendations for how much larger or smaller a hole should be for a pile. It is determined by the designer on a case-by-case basis. Skin friction is generally only accounted for in the augered layers and for holes that are smaller than the pile. No design methods are used for piles with an augered hole as they are very rarely used in the state. According to the specifications a smaller hole size is used for the pile, but when in rock layers or boulders are encountered a larger hole is used. The void space around the pile is to be filled with approved material (Delaware Department of Transportation, 2016).

8) Florida: From the survey Florida uses the terms of “predrilled” and “preforming” for pilot holes and they are a size larger than the size of a pile. Additional information from the survey provides there is a distinct difference between a predrilled hole and preforming. A predrilled hole is allowed by a contractor to drill up to 10ft or 20% of the piles length in order to set the pile tip below ground surface prior to driving. Preforming is prescribed by the engineer in the plans. Performed holes are typically adopted when a strong layer is present above the minimum tip elevation. Padrilled holes are used in both soil and rock whole preforming is more commonly used when a strong layer is present above minimum tip elevation. Skin friction would be marginally accounted for or not at all since this is evaluated on a project-by-project basis. In the design phase prior to construction, when using static methods of analysis, there is either no resistance or nominal amount within the predrilled or preformed zone. The resistance factor used in design corresponds with the dynamic method of testing in the field where all the effects of the construction can be captured. According to the specifications the predrilled holes are at least two inches larger than the size of the pile. Annular space around the pile is backfilled with an approved A-3 material or grout after driving (Florida Department of Transportation, 2022).
9) **Georgia**: This research is being done for use of the Georgia Department of Transportation. Currently Georgia uses the terms pilot holes and pre-drilling. Pilot holes are installed by placing the pile in a hole and hitting it with a hammer to make sure its set and then the hole is backfilled. Pre-drilling involves loosening in the soil at the location the pile will be driven and then driving it normally. Currently pilot holes are used in both soil and rock layers and are a size larger than the pile.

10) **Hawaii**: Hawaii uses the terms “pre-drilled” and “pilot holes” and they are used for both soil and rock. Additional information from the survey added that depending on the case larger and smaller holes have been used before. If the pile just needs to get through a rock layer or bad ground a larger hole will be used and if a friction value is needed a smaller hole is used. As well a large hole may be made and filled with a self-consolidating material to establish friction. In the design standard methods are used. Due to factors such as backfill and size of the hole, any value in this area may be disregarded since there are issues that negate utilizing any bearing capacity in this area. According to the specifications the hole diameter should be equal to the pile diameter plus 6 inches. The pilot hole should also stop 5ft above pile tip elevation and be driven the rest of the way. After the pile is driven all voids around the pile shall be filled with B borrow (Hawaii Department of Transportation, 2013).

11) **Idaho**: Idaho uses the terms “predrilled” and “borehole” for pilot holes and are used in both soils and in rock. Additional information from the survey provided that predrilling is used when boulders and large cobbles are expected and when placing a pile in a rock socket. Predrilled hole size can be adjusted to the geo-technical report. Skin friction is typically not taken into account as when predrilling is used its mostly for end bearing piles on rock. Typically, friction piles are not used with predrilling. Pile capacity is verified with dynamic pile testing and CAPWAP. According to the Structures Book the hole size diameter is slightly larger than the pile diameter or diagonal dimension for H piles. In the specifications when predrilling for piling the holes are drilled and backfilled in both soil and rock to allow piles to be driven to the highest pile tip elevations. The holes are to be 4” larger than the diagonal of the H-pile or circular shell pile (Idaho Transportation Department, 2018).
12) **Illinois**: Illinois uses the term “precore” for pilot holes and they are used in both soils and rock. Additional information from the survey provided more detailed information of the use of precored holes for soils and for rock. Precoring in soils commonly used to reduce downdrag and has been used to get through hard layers for metal shell piles. The top 10 feet for soils are precored beneath the cap at integral abutments to allow for sufficient movement of the abutments. Precoring is used for piles set in rocks to provide a rock socket. The piles are not driven to a particular bearing but are merely set in a rock socket that is filled with concrete. A larger hole size is used. For steel H-piles, HP 8s and HP 10s use an 18 in. hole, for HP 12s and 14s 24 in. holes are used. HP 16s and HP 18s are rarely used but based on their geometry a 30 in. hole is used. For metal shell piles typically an 18 in hole is used. Timber piles are rarely used but would likely use an 18 in. hole. For driven piles skin friction is accounted for, and for piles set in rock capacity is determined by treating the socket like a drilled shaft. The capacity of the pile will be attributed to the socket only with no contribution of resistance from soil layers above the socket. For driven piles capacity will be calculated based on the LRFD factored axial resistance equation that is outlined the IDOT Bridge Manual. For piles set in rock capacity is calculated based on AASHTO LRFD Chapter 10 for drilled shaft axial resistance in rock. For rock socketed piles the entire coil column is precored. According to the specifications precoring through embankments or dense soils is done when shown on the plans. If the holes are oversized the void space outside the pile shall be filled with dry loose sand (Illinois Department of Transportation, 2016).

14) **Indiana**: From the survey Indiana uses the terms “prebored, predrilled”, and “cored” for pilot holes. Predrilled and precored holes are used for soils only and cored holes are used for rock only. Additional information from the survey provided that skin friction for these holes is disregarded except when the cored hole in rock is filled with the concrete. According to the specifications prebored holes are to be 2in smaller than the pile, and predrilled holes are to be 4in larger. All voids around the pile are to be filled with “B” borrow. Piles that are end bearing on rock or a hardpan the hole may be carried to the surface of the rock or hardpan (Indiana Department of Transportation, 2020).
15) **Iowa**: Iowa uses the term “prebored” and “rock cored hole” for pilot holes and they are used in both soils and rock. Prebored holes are used for soils and rock cored holes for rock. Additional information from the survey provided that most steel H-piles are driven through soil and sometimes end bearing on rock. A rock core hole may be used if rock is less than 10ft below the surface. Rock core holes are 2in larger than the diameter of the greatest pile dimension to allow a concrete flow around the pile. Skin friction is not taken into account. Piles are seated on rock with a pile driving hammer and concrete at least 3ft into rock. They are designed as end bearing only piles. According to the specifications the holes are to be 4in larger than the maximum cross-sectional area of the pile (Iowa Department of Transportation Highway Division, 2015).

16) **Kansas**: Kansas uses the term “predrill pile” for pilot holes and these are used in both soils and rock. The information from the survey provided that a hard rock layer may damage piles, so predrilling is done below the rock layer before driving begins. Another method is to predrill though the hard layer to the required length, set the pile, and then backfill to top of rock with concrete. When estimating the capacity, the predrilled area is not counted for skin friction. Capacity is additional confirmed in the field by using PDA. Regarding the capacity of the predrilled pile field data collected during the foundation investigation would be input into the DRIVEN program that is FHWA recommended. LRFD Specifications for driven piles are also followed. Additional information from the specifications added the hole size is required to allow 6in of annual space around the pile (Kansas Department of Transportation, 2015).

17) **Kentucky**: From the survey Kentucky uses the term “pre-drilling” for pilot holes and they are used in both soils and rock. Additional information from the survey provided that KDOT cares only about rock. A good amount of room is left around the pile, and for a 12” pile a 24” hole is used. For a 14” pile and 30” hole would be used. Skin friction is not taken into account for piles on rock. Piles are on are assumed to have full code structural capacity of 50% yield. The specifications provided the size of the predrilled holes are to have a maximum diameter equal to the least cross-sectional dimension of the
pile. Voids that occur around the pile are to be backfilled with free-flowing sand (Kentucky Department of Transportation, 2019).

18) **Louisiana**: From the survey Louisiana uses the term “predrilling” or “preboring” for pilot holes. Additional information from the survey provided that predrilling is used to help stand up the pile, and the hole depth is usually 5 feet. Deeper predrilling is only allowed when difficult pile driving is expected. These holes are used only in soils as the state of Louisiana has very little to no rock. Above the scour elevation a hole larger than the pile is used and below a hole 80% of the pile diameter is used. Skin friction is taken into account only below the scout zone. If predrilling is allowed below PDA is used to test resistance of the pile. In the design of a pile that will have a predrilled hole/zone skin friction in the zone of predrilling is reduced. An alternative is that a lower resistance factor can be used. The specifications provided that voids around the pile are to be backfilled with a granular material and saturated with water. The size and depth of the hole shall in included in the plans. The depth of the prebored hole will not be below the scout elevation (Louisiana Department of Transportation, 2016).

19) **Maine**: From the survey Maine uses the terms “preaugering” and “pre-drilling” for pilot holes. Additional information from the survey provided that for pre augered holes the size is at the discretion of the contractor. For predrilled holes on bedrock the hole diameter is generally 6” larger than the pile measured diagonally. No side resistance or shaft resistance is considered for any part of a pile that is installed with a preaugered or pre-drilled hole. These piles are designed for axial loads and are pure end bearing. According to the Specifications preaugering is used to clear obstructions or otherwise obtain the specified pile tip elevation and pre drilling is used to obtain pile tip elevation for rock socketed piles within bedrock (Maine Department of Transportation, 2020).

20) **Maryland**: From the survey Maryland mentions the term “pilot hole” but on sites uses “test holes”. The “test holes” are used in the sub foundation investigation prior to foundation construction. Additionally, the survey provided that investigations with “test holes” are done on a case-by-case basis. The test holes are larger than the pile if the rock is neither too shallow for spread footing nor too deep for driven piles. These foundations would have a steel pile placed on a rock socket. The void space
around the pile would be backfilled with concrete. If piles are driven to rock, they are normally considered end nearing and the same for piles placed in a socket. In the Specifications Maryland has contractors perform a sub foundation investigation prior to construction of the site has a variable rock profile. Test holes will be drilled as per AASHTO T 206 and T225 at locations specified by the engineer (Maryland Department of Transportation State Highway Administration, 2020).

21) **Massachusetts**: Massachusetts uses the term “predrilling” and “preaugering” for pilot holes. Additional information from the survey provided that the use of a hole is a special provision. The practice is used to advance the pile if obstructions are anticipated that prevent driving from reaching tip elevation. If skin friction is needed a hole of a size smaller than the pile is used. Typically, the hole size is 6 inches less or more than the size of the H-Pile or diameter of the pipe piles. If the pile is being driven to bedrock, then the structural capacity of the pile governs the vertical capacity. Skin friction is neglected or included over the portion of the predrilled hole based on the hole size. Lateral capacity would be neglected if a larger predrilled hole was used or modified based on soil conditions. According to the specifications preaugering shall only be permitted if approved in writing by the Engineer or when stated in the contract documents. Size of the hole will have a diameter not less than the diameter of the pile (Massachusetts Department of Transportation, 2020).

22) **Michigan**: Michigan uses the term “pre-bored” holes for pilot holes, and they are used in soils. From the survey piles are typically H or Pipe piles less than 16 inches in diameter. The hole can be equal to the pile size but no greater than 6 inches. On the very rare occasions preboring will be used on rock and special provisions would be put into the contract. Skin friction would be neglected only in the prebore limits. In the estimating of the capacity, the conventional pile static analysis methods are used and the weight of the fill soil overburden is used. Additional information from the specifications provided the prebored hole is to be 6in greater than the diameter of the pile (Michigan Department of Transportation, 2020).
23) **Minnesota**: Minnesota uses the term “prebore” for pilot holes and they are used only in rock. Additional information from the survey provided that nearly all piles are installed without a driving aid. Preboring is done on shallow rock for rock sockets and around utilities to avoid damage. The hole size tends to be slightly larger than the pile. Skin friction is taken into account and there are two parts in estimating the pile capacity. The first part is the geotechnical capacity of the soil/rock to support the load and the second part is the structural capacity of the pile. The specifications provided that prebored holes must have a diameter that will admit the largest cross sectional diameter of the pile without creating friction between the pile and the prebored hole (Minnesota Department of Transportation, 2020).

24) **Mississippi**: Mississippi uses the term “pre-formed pile holes” for pilot holes and they are used in both soils in on rock. Additional information from the survey provided that the size of the hole is usually the diagonal measurement of the pile. No skin friction capacity is expected for the length of the preformed pile hole, and pile capacity curves are adjusted to take the performed hole into account. The specifications provided that a geotechnical investigation will determine if preformed pile holes are needed. The contractor will show the location, size, and the bottom elevation of each hole. If the preformed pile hole is not specified in the plans the Bridge Engineer and the Construction Engineer will determine during construction if the subsurface conditions will require the hole at certain locations (Mississippi Department of Transportation, 2017).

25) **Missouri**: Missouri uses the term “preboring” and “pilot holes” for pilot holes and they are used in both soils and rock. Additional information from the survey provided that most often these holes are used for soils. Skin friction is the primary capacity for piles due to the conditions in Missouri. MoDOT has design tables that are utilized for capacity, and there are different values for standard H piles and pipe piles. The specifications provided the holes shall have a diameter no less than that of the pile and large enough to avoid damage to the pile driven through hard material. Pilot holes are of a lesser diameter and shall not extend below the pile tip. Holes not prebored into rock will be backfilled with sand or other approved material after pile placement. Holes prebored into hard rock the hole shall be
filled with sand or other approved material prior to the pile placement (Missouri Department of Transportation, 2021).

26) **Montana**: Montana uses the terms of “pile drill, socket, and pile pre-bore” for pilot holes and they are used in both soils and rock. Additional information from the survey provided that the pre-bore method is typically used in intermediate geomaterials. Drill and socket is less common and is used to reach required elevation through resistant strata. Generally, the pre-bore holes are 1 inch less than the size of the pile. For pre-bore piles skin friction is usually taken into account and it is not for drilled and socket piles. Dynamic Load Testing is used to confirm capacity during construction. Drilled and socketed piles are usually set with an impact hammer once installed to tip elevation at the bottom of the borehole and resistance measured either by PDA or capacity curves. In the design estimation of capacity is typically done using software such as APile as well as Wave Equation analysis with GRLWeap. The specifications provided that pile pre-bore uses an auger, wet rotary drill, or other method. The hole is pre bored to a specified depth and the pile driven in the hole. Pile drill and socket the holes are to be a maximum of 1in in diameter less than the outside diameter of a round pile and 4in less than the diagonal cross-sectional measurement of a square or H-pile. The pile is to be driven into a pre drilled pilot hole to the bottom of the hole (Montana Department of Transportation, 2020).

27) **Nebraska**: From the survey Nevada uses the term “predrill” for pilot holes and they are used in both soils and rock. Predrilling is only allowed for 30% of the pile length except in the case of integral abutments. Additional information for the survey provided that predrilled holes shall not be backfilled until all abutment piles are driven. Skin friction is considered on the pile but would not be considered in the predrilled area of the pile. The area where the pile is predrilled would not be considered in the capacity. According to the Geotech Manual all abutments, excluding wing pile, shall be started in holes predrilled to elevation. The minimum diameter of the holes for the HP pile shall not be 2 inches larger. Piles placed in drilled holes and driven to design bearing and the void between the hole wall and the pile shall be backfilled with dry, clean, sand (Nebraska Department of Transportation, 2017).
28) **Nevada**: Nevada uses the term “prebored, predrill, and pilot hole” for pilot holes and they are used in both soils and rock. The holes aid in keeping the pile straight and not curve during driving. Driven piles are usually used for soils, but if there is rock at depth the tip of the pile may be embedded in the rock. Since the driven piles mainly use side friction that holes are smaller in size, and skin friction is used to calculate the capacity of the pile. In the design methods and equations from AASHTO Standard Specifications for Highway Bridges are utilized. When at the site during pile driving, the blows per foot will be counted until it reaches the expected blow count and if the pile is at the minimum allowed depth, the driving is stopped. The measure of the actual capacity is done using a pile driving analyzer (PDA). The specifications provided the holes are to be a size smaller than the diameter or the diagonal cross section of the pile. If obstructions such as boulders, caliche, or rock layers are encountered the hole diameter may be increased to the least dimension adequate for pile installation. Any void space around the pile are to be backfilled to the bottom of the hole after positioning and aligning the pile (Nevada Department of Transportation, 2014).

29) **New Hampshire**: New Hampshire uses the term “prebore” and “predrilling” for pilot holes, and they are used in both soils and in rock. Additional information from the survey provided that skin friction would be ignored in these situations. In the design the structural capacity of the pile would be used. The specifications provided preboreing is to be done when specified in contract documents and the holes are to be made where shown on the plans. Prebored holes shall be a size smaller than the diameter of diagonal of the pile cross section that is sufficient to allow penetration of the pile. If obstructions such as boulders or rock layers are encounter the hole diameter may be increased to the least dimension of the pile (New Hampshire Department of Transportation, 2016).

30) **New Jersey**: New Jersey uses the term “prebored hole” for pilot holes and they are used mostly in soils. Additional information from the survey provided that the holes are mostly used in soils. If rock is close to the surface a spread footing would be used. If drilling in rock for a pile a hole is drilled to depth and pile is placed in concrete is filled in the hole. Skin friction is ignored for the pile design resistance. According to the specifications when preboring bores for round piles an auger is used with
a diameter 2 inches smaller than the diameter of the pile. When preboring for H-Piles an auger that is 4 to 6 inches smaller than the diagonal cross section is used. Void space is backfilled with granular material (New Jersey Department of Transportation, 2019).

31) **New York**: from the survey New York uses the term “predrilled holes” for pilot holes, and they are used only in soil layers. They are most often used if there are vibration issues and when driving near a utility. The size of the hole would be a larger than the diameter of the pipe pile or H-pile diagonal plus six inches. Skin friction would be neglected in the predrilled portion of the pile. In the design of the pile skin friction in the predrilled section would be disregarded. It would be followed by WEAP analysis and a dynamic pile test in construction. (New York Department of Transportation, 2020).

32) **North Carolina**: North Carolina uses the term “pile excavation” for pilot holes in areas where rock is expected. Spudding and predrilling would be used for soils. Additional information from the survey provided that the hole size for predrilling in soils is not specified in the specifications. It is at the designer’s discretion for the hole size. For piles in rock with the pile excavation skin friction is not taken into account. The piles are to be considered end bearing and the required driving resistance with end bearing is not a concern. For predrilling and spudding skin friction is typically considered. In the design process best engineering estimates on soil strata, foundation type, and installation measurements are used. Dynamic pile testing is recommended to confirm design assumptions and requirements. According to the specifications pile excavations shall be made at locations with diameters that will result in at least 3” of clearance all around the pile. Before filling the holes, piles are to be supported and centered then drive the pile to the required driving resistance. The hole shall be filled with concrete, grout, or flowable fills (North Carolina Department of Transportation, 2018).

33) **North Dakota**: From the survey North Dakota typically uses a driven H-pile. According to the survey, rock is not frequently encountered; therefore, a hole is only necessary when driving piles through existing embankments. The term used is “pre-bore”. The hole is typically required to be within one inch greater in diameter than the pile. The response to the survey states skin friction is not considered in the length of the pre-bore, and the pile capacity is estimated using A-Pile software for static analysis
and then selecting the FHWA computational method. According to the design manual prebored holes should be a minimum of twenty-four inches for a HP14 pile, twenty-one inches for a HP12 pile, and eighteen inches for a HP10 pile (North Dakota Department of Transportation, 2020).

34) **Ohio**: Ohio occasionally uses a hole referred to as a “prebored” hole for pilot holes. The prebore hole is used in both soil and rock in order to reach minimum embedment length and reduce vibrations. According to both the survey and the construction and material specifications, for round piles, the diameter of the hole can range from two inches less than to four inches greater than the diameter of the pile, and for steel H-piles, the range is from six inches less than to two inches greater. Holes are backfilled with a granular material. Within the prebored portion, skin friction is discarded. Capacity is considered the same as without the hole, except for the discarded skin friction. When the pile is prebored into bedrock and backfilled with concrete, a resistance factor of 0.9 is used (Ohio Department of Transportation, 2019).

35) **Oklahoma**: Oklahoma calls the hole a pilot hole. Based on the survey, Oklahoma uses pilot holes through soil and into rock until the hole reaches the required depth. Typically, the diameter of the pilot hole is smaller than the pile. In addition, the standard specifications state the diameter of the pilot hole can be increased if there are surface obstructions. Skin friction is not considered when a pilot hole is used. Oklahoma places pile drivers and checks with Gates Equation or practical refusal to determine the bearing capacity. Because rock is not deep in Oklahoma, pile designs are governed by structural design. Resistance factors are from AASHTO LRFD or additional ODOT resistance factors. Sand is typically used as backfill for the full depth according to the questionnaire. According to the standard specifications, holes can be backfilled with Class A concrete followed by more concrete, sand, or other materials (Oklahoma Department of Transportation, 2019).

36) **Oregon**: Based on the response to the survey, Oregon uses the term “prebored hole” for pilot holes. The engineer may call them cast in place piles as well. Most of these piles are installed in rock to ensure bearing, but they have been installed in soil. Most cases use a hole larger in diameter than the pile, however, there have been cases of smaller holes used. Determining capacity is up to the geotechnical
engineer using state guidelines as well as AASHTO standards. Additionally, information from the specifications provided that for end bearing piles preboring may be carried to the surface of the end-bearing material. For other piles the hole may be a size smaller (Oregon Department of Transportation, 2021).

37) **Pennsylvania:** According to the response from the survey the terms, “predrilling” and “pre-augreing” are used for pilot holes. Predrilling is generally specified through soil into rock in order to achieve a specified penetration and bearing on competent rock material. Occasionally, predrilling is specified for friction piles to ensure piles penetrate to sufficient depth in dense soils, or to ensure friction piles are founded in competent soils below a soft material. A larger hole and diameter than the pile diagonal dimension is used for piles that are predrilled to bear on the rock or for integral abutments. Additional information from the survey response provided that for predrilled integral abutments through soil, a 2ft hole or pile diameter plus 10ft is used (whichever is larger). For predrilled piles point bearing on the rock, 6in would typically be added to the diagonal dimension of the pile to allow the 3in of additional coverage on the pile. As well, skin friction is not considered for the predrilled portion of the pile. Piles predrilled to rock are “point bearing” and only consider the end bearing capacity of the pile (no skin friction). When pre-augering is specified for integral abutments, the portion of the soil that is pre-augered to facilitate pile flexibility is not considered for skin friction capacity as the backfill material in the augered portion is placed after the pile is placed in the hole. Wave equation analysis is performed with graphs available to the structure control engineer for use in determining if the pile is seated on a rock. Dynamic pile monitoring may be specified if the rock is soft/weathered or if uplift of the pile is anticipated and would also be used to determine if the pile has achieved the required capacity. Additional information from the specifications provided that predrilling is the terminology that is typically used when drilling through the rock to ensure piles are bearing on a competent rock. Pre-augering is typically used when working on soils. Predrilling is used for obstructions and to ensure that the piles are founded on the competent rock (not above voids, soil seams, or on weak materials).
Predrilling and pre-augering are also specified to ensure flexibility at the top of piles for integral abutments (Pennsylvania Department of Transportation, 2020).

38) **Rhode Island:** According to the information provided by the survey response, Rhode Island does use pilot holes and refer to it as “augered holes.” Augered holes are used to guide and make pile driving easier; they are relatively shallow. However, augered piles are only used in the soil (due to its low capacity). The augered hole is made up to the diameter of the bit just surpassing the greatest dimension of the H-Pile. Skin friction is the only factor considered upon for augered piles. The piles are driven through the gravel with which the augered holes are filled. The allowable pile capacity is estimated based on AASHTO guidelines, RI-DOT specifications, and NHI manuals. Additional information from the specifications provided that any void space is to be filled with sand or other approved material. The specifications also mention the term “prebored” hole (Rhode Island Department of Transportation, 2004 (Amended 2018)).

39) **South Carolina:** Based on the information provided by the survey response, if the engineer recommends the use of pilot holes, the process is referred to as “predrilling”. However, most of the time predrilling is not recommended nor required. Sometimes, the field inspectors will allow the contractor to predrill a shallow hole to place the pile to help assist driving and installation. Typically, the diameter used is about 1.25 times the nominal pile size up to 2in larger than the largest dimension for soil, depending on the design intent. For rock, typically a diameter of 2in larger than the largest dimension of the pile is used. Depending on the soil type and design intent, some engineers use anywhere between 0 and 75% of skin friction, but it typically lies in the 50-75% range. Normal design methods for capacity calculations as if no driving aid are used, and then modified based on the range of skin friction. Additional information from the construction manual provided that pre-drilling for piles will not be permitted except where specifically noted in the contract plans or approved in writing by the bridge construction engineer. The pre-drilled hole shall be backfilled with sand or pea gravel (South Carolina Department of Transportation Engineering Publications Office, 2004).
40) **South Dakota:** From survey South Dakota uses “pre-bore” holes for all piles supporting integral abutments. Based on the survey response, in rare instances, “bore pilot holes” are set up for piling in rocks. For driven piling thru pre-bore holes, the pile capacity is developed below the pre-bore and is dependent upon subsurface conditions. Bearing may be developed by the skin friction, the end bearing, or a combination of both. Generally speaking, piling used for rocks through a pilot hole are end bearing only. Finally, pre-boring piling at integral abutments is not done as a driving aid, but rather to allow the integral abutment to translate. Additional information from the specifications provided that when pre-boring for steel piles they should not be larger than a specified diameter. After being driven the piles are to be backfilled with coarse dry sand (South Dakota Department of Transportation, 2015).

41) **Tennessee:** Tennessee uses the term “performed pile holes” for pilot holes and they are used in both soils and rock layers. Additional information from the survey provided that typically the contractor would drill a hole a little bit larger than the steel H pile. Skin friction is not taken into account for these pile types, and they are designed as end bearing. In the estimation of capacity, the pile driver is used to set the pile and the capacity is 55 tons per pile (nominal) for H-piles. According to the specifications performed pile holes are oversized so that the sides or corners of the pile are not in contact with the soil. The void space is filled with approved clean sand. The Geotechnical manual refers to pre-drilled holes and that they are larger than the pile and are surrounded by lean concrete, gravel, or sand. Steel piles that are a minimum of 10ft in length from the bottom of the abutment base are pre-drilled into the underlying soils and rock (Tennessee Department of Transportation, 2021).

42) **Texas:** Texas uses the term pilot hole, and they are used typically used in soils. Additional information from the survey provided that in general skin friction is ignored when pilot hole is used. According to the specifications the maximum hole diameter will be 4in less than the diagonal of the square piling or steel H-pile and 1in less than the diagonal of round piling. The engineer may vary the hole size and depth in order to achieve penetration and bearing resistance. Pilot holes are to extend no more than 5ft below the bottom of footings for foundation piling or 10ft below finished ground line (Texas Department of Transportation, 2014).
43) **Utah:** From the survey Utah uses the terms “predrill” and “pre-auger” for pilot holes and they are used in both soils and rock. From the survey the hole size should not be larger than the maximum pile dimension and skin friction is taken into the account in the design process. According to the specifications the holes should of a diameter smaller or equal to that of the maximum pile dimension. The holes are used when pile tip elevation cannot be reached by the pile driver. All piles are driven to required resistance and there is no distinguish if it has been pre drilled or pre-augered (Utah Department of Transportation, 2017).

44) **Virginia:** From the survey Virginia uses the term “pre-boring” for pilot holes and are used in both soil and rock layers. In rock layers a socket is established, and the size of the socket is 6in greater than the pile diagonal dimension. According to the specifications the diameter of each pre-bored hole will be approximately 75% of the pile diagonal but not more than 100% of the diagonal. When boring through rock the diameter of each hole shall not be less than 6” greater than the pile diagonal. In the design reduced skin friction is accounted for in the prebored section of the hole (Virginia Department of Transportation, 2020).

45) **Vermont:** Information from the survey provided that Vermont uses the term “pre-drilled” for pilot holes and are used in both soils and rock. Pre-drilled holes are used when shallow rock, cobbles, or boulders are encountered in the subsurface investigation and steel H-piles are sometimes designed to be placed in the hole. Additionally, from the survey the size of the hole is approximately 6in greater than the piles diagonal dimension. The holes are filled, and the pile driven through or the pile is placed (if it is in rock) and backfilled around. In the estimation of capacity skin friction is not taken into account as most of these piles are into rock. If the pile is into soil only the skin friction is only taken into account along the length of the pile driven below the pre-drilled portion of the hole. The Nordlund-Thurman method is usually used with estimating. If the pile is through a predrilled hole in soil, then a dynamic load test is required (Vermont Agency of Transportation, 2018).
46) **Washington:** From the survey Washington uses the term prebored holes for pilot holes, and they are used primarily in soils. Additional information from the survey provided that predrilled holes are not often used, and they are used for glacially over consolidated soils primarily in marine environments. The pile capacity was determined with the aid of PDA and CAPWAP. The axial resistance in the very dense materials that require pilot holes rarely govern the design. According to the specifications prebored holes have a diameter no larger than the least outside dimension of the pile. After the pile is driven the contractor shall fill all open spaces between the pile and the soil caused by the preboring with dry sand or pea gravel (Washington Department of Transportation, 2020).

47) **Wisconsin:** Wisconsin uses the term “pre-boring” for pilot holes, and are used in both soil and rock layers. For pre-bored piles in soil the hole size is approximately the pile diameter. For pre-boring in rock, the hole is at least one inch larger than the pile outside diameter. Additional information from the survey provided that skin friction is not taken into account in the prebored zones. When driving to rock or pre-boring into rock an end bearing value is used to show pile capacity can be achieved. Design software is also used. According to the specifications when pre-boring in unconsolidated materials the hole size should have a diameter approximately equal to the greatest diagonal pile cross section dimension. For round piles the hole size shall be approximately the pile diameter. An open hole for pile installation must be maintained. After driving, the area around the pile is backfilled with sand or other engineer approved material. When pre-boring in rock or consolidated materials for round piles the hole size must be one inch larger than the pile outside diameter. For other shapes pre-bore holes at least one inch greater than the greatest diagonal pile section dimension. Piles are firmly seat after pre-boring and backfill within the rock with a cement grout. (Wisconsin Department of Transportation, 2022).

48) **Wyoming:** Wyoming uses the term “predrilled” and “prebored” holes for pilot hoes and they are typically used on rock layers and dense layers of cobbles or boulders. Additional information from the survey provided that the holes are mostly used when there is a shallow depth to bedrock and the material above the bedrock does not provide adequate lateral resistance for the structural design. Skin friction is ignored in the pre-bored section when determining pile capacity. The capacity is estimated using end
bearing and driven to refusal. According to the specifications predrilled holes are to extend to the elevation specified and obtain the remaining penetration with the pile. The hole diameter shall not exceed the pile width. The pile is placed in the hole and driven to set the point firmly into bearing material. The space around the pile is filled with dry sand, pea gravel, or flowable fill (Wyoming Department of Transportation, 2021).

4.3 Summary of the findings

Based on the survey 33 states indicated the use of a pile with pilot hole in both soils and on rock layers; those states were Alabama, Alaska, Arkansas, California, Colorado, Connecticut, Georgia, Florida, Hawaii, Idaho, Indiana, Illinois, Maine, Mississippi, Massachusetts, Missouri, Montana, Nebraska, Nevada, New Hampshire, North Carolina, Ohio, Oklahoma, Oregon, Pennsylvania, South Carolina, South Dakota, Tennessee, Utah, Vermont, Virginia, Wisconsin, and Kansas. Based on the survey 7 states indicated that they have used a pile with pilot hole only in soils; those states were Louisiana, Michigan, New Jersey, New York, North Dakota, Rhode Island, Texas. Based on the survey 5 states indicated that they have used a pile with pilot hole only in rock; those states were Iowa, Kentucky, Maryland, Minnesota, and Wyoming. Figure 8 displays the use of pilot holes in soils and rocks.

![Figure 8 Ground Conditions of Pilot Hole Use](image-url)
A variety of terms were used to refer to a pile with a pilot hole. Of the DOT agencies that do use a pilot hole there are common terms used and varying ones. Some states were noted to use multiple terms. The terms that were observed to be used across the DOT agencies are predrilling and pre-drilled hole, preboring/ pre-bored hole, pilot hole, and preauguring/pre-augured hole. These were the most used terms. Other terms used by a few states were precoring/pre-cored hole, cored hole, augured hole, spudding, and pile excavation. All the varying terms were referring to the same situation of a pile being installed to elevation with the assistance of a hole to guide it. The term variety used is seen in Figure 9.

Figure 9 Terminology Used for Pilot Holes
All 45 states that responded to the survey indicated the hole size used as seen on Figure 10. Twenty-two States recommend a hole larger than the size of the pile, seven states indicated only smaller holes are used, five states specified holes equal to or larger than the pile are used, and eleven states use a larger or smaller hole depending on the site conditions.

*Figure 10 Size of Pilot Hole Used*
CHAPTER 5

REVIEW THE EFFECTS OF THE PILOT HOLE ON THE PILE CAPACITY AND BEHAVIOR, ALONG WITH THE ASSOCIATED DESIGN AND CONSTRUCTION CONSIDERATIONS.

5.1 Introduction

The five-question survey was sent to all 50 state DOT agencies. The previous chapter covered the state responses to the survey, information from state standard specifications, and summary of the answers to questions 1, 2 and 3. This chapter will go over the answers to question 4 and 5.

Q4) Do you take into account the skin friction when estimating the capacity of this type of pile?

Q5) When designing, how do you estimate the capacity of this pile type. That being a pile that will be constructed with a driving aid/pilot hole.

5.2 Survey Response

A second survey was sent out to a smaller pool of states for more detailed information. The answers to the questions gave a picture of how pile capacity and behavior were viewed across all 50 DOT agencies. Out of the 50 state DOT agencies 48 replied to the survey. The two states with no response to the survey were New Mexico and West Virginia. Of the 48 that replied to the survey, 3 states (Arizona, Delaware, and Washington) indicated that pilot holes are not used. That leaves 45 states with information from the survey.

For question 4 of the first survey 45 states that replied to the survey and use a pilot hole, 22 indicated that skin friction is not accounted for in the design of the pile with the pilot hole, 8 states indicated that skin friction is accounted for in the design, and 15 indicated that skin friction is partial or varies in the design depending on conditions. The states where no skin is used were Alabama, California, Connecticut, Georgia, Florida, Idaho, Indiana, Iowa, Kentucky, Maine, Maryland, Mississippi, New Hampshire, New Jersey, North Dakota, Ohio, Oklahoma, Oregon, Pennsylvania, Texas, Tennessee, Vermont. The states that do use skin friction were Alaska, Minnesota, Missouri, Nevada, Rhode Island, South Carolina, Utah, and Virginia. The states that indicated skin friction varied depending on conditions or hole size were Arizona, Colorado, Hawaii, Illinois, Kansas, Louisiana, Massachusetts, Michigan, Montana, Nebraska, New York, North
Carolina, South Dakota, Wisconsin, and Wyoming. The distribution of use of skin friction can be seen in Figure 11.

![Figure 11 Use of Skin Friction for Piles with a Pilot Hole](image)

Although additional analyses are necessary to understand specific considerations of the skin friction in the design and how it is related to the hole size such as shown in Figure 12, the general consensus is that the size of the hole played a role when determining if skin friction was used for piles with a pilot hole. In general, larger hole sizes negate skin friction and smaller hole sizes account for skin friction.
Some states specified that the size of the hole would be equal to the size of the pile or larger than the size of the pile. As presented in Figure 13, some states specified that the size of the hole would be equal to or larger than the size of the pile diameter.
Question 5 asked of the design and capacity estimation for a pile with a pilot hole on rock. Of the 45 states that replied to the first survey 42 supplied an answer to the how the pile with a pilot hole is designed and estimated for capacity, displayed in Figure 14. Of the 42 states 16 indicted that the pile is designed as end bearing using AASHTO LRFD methods or other methods. Those states were Arkansas, California, Connecticut, Idaho, Illinois, Iowa, Maine, Maryland, Nevada, New Jersey, Pennsylvania, South Dakota, Tennessee, Texas, Vermont, and Virginia. Of the states, 6 states (Colorado, Massachusetts, Minnesota, New Hampshire, Ohio, and Wyoming) indicated that the structural capacity of the pile governs or the pile is design as column, while 4 states (Alabama, Florida, Michigan, and North Dakota) indicated that static analysis methods are used for capacity estimation. The remaining 15 states provided various methods on capacity estimation and design some of which varied on the ground or site conditions.

![Figure 14 Capacity Estimation Methods for a Pile with a Pile Hole](image)

The use capacity estimation of a pile with a pilot hole can as well be broken down into more detail with the addition of hole size. The size of the hole did play a role in how the capacity was estimated in the state as seen by Figure 15.
5.3 Second Survey

A second survey was sent out to a select group of states for more detailed information on the case of a pile with a pilot hole on rock. These states were selected based on the response to the first survey, proximity to Georgia, and how similar their handling of a pile with a pilot hole on rock were to Georgia. The second survey was sent to 20 states Alabama, Alaska, Arkansas, Colorado, Florida, Illinois, Iowa, Kentucky, Maryland, Minnesota, Nevada, New Jersey, North Carolina, Ohio, Oklahoma, Pennsylvania, South Carolina, Tennessee, Utah, and Wyoming. Of the states contacted 14 replied to the second survey these states were Alabama, Alaska, Colorado, Florida, Iowa, Kentucky, North Carolina, Ohio, Oklahoma, Pennsylvania, South Carolina, Tennessee, Utah, and Wyoming. As summary of the state responses can be seen in Table 16.
The questions asked in the second survey are as follows:

**Q1) Verification of the pile capacity**

1) During or after the construction, how do you verify the pile capacity? What QA/QC practices do you have in place to review/confirm and approve field verification of capacity?

**Q2) Design of the pile**

1) How is the pile capacity determined in the design?

2) In addition, what are the resistance factors that are used, and how are the resistance factors determined or where are the resistance factors sourced from?

**Q3) Hole size and finishing**

1) It was indicated in the response the hole size can be larger than the pile if very hard bedrock is encountered. For the backfill for the hole which methods are preferred in your state among these: sand, concrete, or grout?

2) Is the hole backfilled to the top of rock only or is the entire depth backfilled with the aforementioned material?

**5.4 Second Survey Response Summary**

1) **Alabama:** For the second survey Alabama provided that field verification is used to verify the pile capacity in answer to the first question. In reply to the second question static analysis programs are used to determine pile capacity in the design process. Additionally, resistance factors developed by AASHTO are used. In reply to question 3, parts one and two, after a pile is placed in a pilot hole the voids around the pile are filled with a clean sand before the pile is driven. After driving additional sand is added to fill any additional voids. Pilot holes that terminate in rock shall be backfilled to the top of the rock with substructure concrete after seating the pile and the remainder of the hole filled with concrete or sand.

2) **Alaska:** For the second survey Alaska provided for question 1 that verification depends on if the foundation is considered a shaft or a pile. If the foundation is on competent, strong rock, that it will be likely be considered a shaft and not verification would be needed. Instead for this case empirical
methods used in the design phase would determine the capacity. If the pile is driven into weak rock, then PDA/CAPWAP dynamic testing or presumptive wave equation without signal matching is used to verify capacity. The reason this method is used is primarily based on the expected driving stresses. A higher resistance factor with dynamic testing can be used, so there would be need to verify as much capacity, and therefore have a higher strength load can be had. If expected driving stresses are low, then there is no need for dynamic testing. In response to both parts of question 2, pile capacity is predicted based on past experience. It was found that standard predictive methods are not reliable, so expected driving resistance is based on previous PDA data is available. Additionally standard resistance factors published by AASHTO LRFD are utilized for design.

In response to question 3; if hard bedrock is encountered where the piles are too shallow to develop adequate lateral resistance soils then the pile is socketed into place. The preferred method used is to grout the annular space between the oversized socket and the pile. Lateral strength of the rock is not relied on and in the past large diameter pilot holes have been drilled, filled with aggregate, and the pile driven through the aggregate. This is not the preferred method to use. Any grout is only pumped to the top of the socket and any additional annular space around the pile is filled to the top with sand.

3) Colorado: For the second survey Colorado provided for question 1 that the Pile Driving Analyzer (PDA) or CAPWAP are used to verify the capacity. Any QC/QC practices are added to contractor responsibilities. Colorado receives the reports. The design of a pile with a predrilled hole on rock is the same method used as other piles regardless of predrilling. In response to question 2; there isn’t a preferred method of backfilling but is mostly dependent of cost effectiveness for the project. The fill is often recommended by the contractor and agreed upon with the project Engineer. When the hole is backfilled, it is done so to ground level.

4) Florida: For the second survey Florida provided for question 1 that dynamic testing methods are used to verify capacity such are pile driving analyzer or embedded data collectors. For question two it was provided that pile capacity is determined using software called FBDEEP. Resistance factors used are
sourced from AASHTO code and local research. The factors are found in the Structures Manual-
Volume I “Structures Design Guidelines” Chapter 3. Table 3.5.6-1 provides the resistance factors used.

In response to question 3, there is not a preferred method of backfilling a hole and depends on the project, and the hole is backfilled the entire performed depth.

5) Iowa: For the second survey Iowa provided for question one that Wave Equation Analysis is performed on all driven piles to check for driving stresses and determine capacity in the field. The contractor to hit the pile with an approved hammer to “seat” the pile on rock to confirm the rock is solid. For question 2, capacity is determined using LRFD methods considering structural resistance and geotechnical resistance. A resistance factor of 0.7 is used based on local research. Predrilled holes backfilled with concrete. The pile is a minimum of 3’ into sound rock and above the concrete can be sand.

6) Kentucky: For the second survey Kentucky specified that pilot holes are smaller than the piles and predrilled holes are larger. Predrilled holes are only used to get through boulders, obtain pile embedment, or drilled into solid rock to obtain lateral length. Additionally, the line between predrilled pile and drilled shaft reinforced with a pile can be blurry. For the second survey question 1, the contractor is required to use the pile driver on predrilled piles set in the hole, and they are required to obtain a certain number of blows with less than a certain movement to achieve practical refusal. The number of blows depends on the strength of rock. In response to question 2, full yield strength of piling and resistance factors given in the LRFD code are used in the design and capacity estimation. What is called out in the geotechnical report is also taken into consideration on anticipated driving difficulty. For question 3, predrilled holes are backfilled with sand, gravel, and concrete depending on what is needed. If axial strength is needed only sand and gravel is used and if lateral strength is needed concrete is used. The hole is backfilled the entire length.

7) North Carolina: For the second survey in response to the first question North Carolina provided that if the rock is crystalline rock with \(N \geq 60\) blows in 0.10ft) the capacity is not verified. If the rock is weathered the pile would be driven and verified based on WEAP or PDA/CAPWAP. Driving would be minimal and a few sets in 10 blows would be measured to prove capacity. For the question 2 response,
the design capacity is determined geotechnically. The ultimate resistance is estimated/predicted versus
depth/elevation using SPT borings for driven piles. Software such as Apile or DRIVEN have been used
before as well. For H-piles in the coastal plain region that South Carolina resides static analysis is used
for estimated lengths with a 0.70 factor. The resistance factor for driving resistance is 0.060 (WEAP or
one PDA) or 0.75 if two or more PDA are needed. For non-integral end bents/abutments the hole us
filled with concrete, grout, or flowable fill. For integral end bents/abutments the hole is generally filled
to the natural ground ± 3 ft.

8) Ohio: For the second survey Ohio provided for question one that, for a pile constructed with a pre-
bored hole into bedrock the pile would be placed directly into the hole with no pile driving. The hole
would be backfilled at least to the top of rock with (4000psi) concrete. Ohio provides a special as-per-
plan note in the project plans for this case and does not perform field verification of the pile bearing
capacity. The pile is assumed to be essentially identical to a pile driven to refusal on top of rock. In the
case of a driven pile there no verifications of firm contact with rock other than counting pile driving
hammer blows. In response to question 2, in the design for a pile pre-bored into rock a 0.95 resistance
factor is used as the pile is considered a continuously- braced steel column. Ohio sources this from
AASHTO LRFD Article 6.5.4.2 for Axial Compression, Steel Only. In response to question 3, the pre-
bored hole is always larger than the pile and is backfilled with class “QC Misc” (4000psi) concrete to
the top of rock. Above the rock pre-bored hole is either backfilled with more 4000 psi concrete to the
bottom of the pile cap elevation or the hole is backfilled with a granular material to the bottom of the
pile cap elevation depending on the abutment design.

9) Oklahoma: For the second survey Oklahoma provided that a pile driver is placed, and Gates Equation
is checked or practical refusal is used to verify pile capacity. In response to question two, rock in
Oklahoma is not deep and pile designs are governed by structural design. Additionally, AASHTO
LRFD resistance factors are used along with local ODOT factors. For the question 3 response sand is
typically used as backfill material and is filled the full depth.
10) **Pennsylvania:** In response to question 1 of the second survey Pennsylvania indicated that wave equation analysis or dynamic testing are used to approve capacity in the field. Additionally, piles are driven to absolute refusal (20 blows per inch prior to placement of concrete). Capacity of a pile with a predrill hole is determined the same way as typical piles driven to rock as only the end bearing portion of the pile is used for bearing capacity. Side friction is ignored. Typically, granular material is used to backfill the piles, but concrete and grout have been used before as well.

11) **South Carolina:** For the second survey South Carolina provided for question 1 and 2, if the rock is hard and strong enough the pile is design uses drilled shaft methodology without field capacity verification. Otherwise, resistance factors are used for piles driven in weak rock. The shaft excavation is visually verified to match the soil/rock assumptions. If the rock is soft and weathered enough then WEAP and PDA are used during driving to verify capacity. AASHTO factors are used and the South Carolina Geotech Manual with a typical factor of 0.6 for piles driven to rock. For question 3 concrete is used as a backfill for shaft design. How much of the hole is filled is dependent on the structural capacity, but a minimum length is specified the shaft design.

12) **Tennessee:** In response to the first question of the second survey Tennessee provided that most often concrete piles are used unless bedrock is likely then H piles are selected. H piles are driven to practical refusal in accordance with the contract and a nominal bearing of 55 tons used. For question 2 response capacity is often based on site characterization that involves rotary drilling with SPT test. If steel piles are on bedrock then elevation is determined, and the design methodology is typically selected by the structural engineer. The entire depth of the hole is filled used a #57 stone that is preferred.

13) **Utah:** Utah began the second survey response with that the state has only predrilled for driven piles when near surface conditions have not allowed driving piles in soil, for instance near surface boulders or obstructions. There may not be cases for predrilled driven piles into rock. In response to the first question when piles are driven with a predrilled hole the initial pile is monitored with PDA and resistance verified with CAPWAP. Subsequent piles are monitored for blow count and hammer stroke. The size of the hole is smaller unless very hard bedrock is encountered. As there have been no known
cases of a pile predrilled to bedrock a likely backfill used for such a situation would be flowable fill. The backfill would be to the top of the hole.

14.) Wyoming: For the second survey Wyoming indicted for the first question that a pile (end bearing) in bedrock is driven with refusal criteria for a properly operating and sized hammer of a maximum of 10 blows per inch. WEAP analysis is used, and stroke of the hammer is monitored to prevent pile overstressing. Pile refusal is assumed achieved at less than 10 blows per inch. For piles end bearing in hard bedrock capacity is determined by strength of the pile. It is assumed that in an end bearing driving refusal condition that the resistance of the bedrock is greater than the allowable design strength of the pile. Used resistance factors are from the AASHTO LRFD code. Annular space around the pile is backfilled with pea gravel or sand, and the entire length of the pile is backfilled up to the cutoff elevation.
<table>
<thead>
<tr>
<th>State</th>
<th>How Pile Capacity is Verified</th>
<th>Design Capacity/ Resistance Factors</th>
<th>Pilot Hole Backfill</th>
<th>How Much Fill</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alabama</td>
<td>Field Verification</td>
<td>Static Analysis Programs</td>
<td>Clean sand before pile is driven</td>
<td>More sand to fill voids after driving</td>
</tr>
<tr>
<td>Alaska</td>
<td>PDA/CAPWAP or Wave Equation</td>
<td>Based on Past Experience</td>
<td>Granular Grout to top of socket</td>
<td>Entire Pile Length</td>
</tr>
<tr>
<td>Colorado</td>
<td>PDA/ CAPWAP</td>
<td>All Factors from AASHTO LRFD Bridge Design Manual</td>
<td>Most Cost-effective method used</td>
<td>Entire length to ground level</td>
</tr>
<tr>
<td>Florida</td>
<td>PDA or Embedded Data Collectors</td>
<td>AASHTO Code and Factors from Local Research</td>
<td>No single preferred. Depends on Project</td>
<td>Entire performed Depth</td>
</tr>
<tr>
<td>Iowa</td>
<td>Wave Equation Analysis</td>
<td>Modified Iowa ENR formula, combination of WEAP, and PDA, with subsequent pile signal matching analysis using CAPWAP. 0.7 factor</td>
<td>Concrete</td>
<td>A minimum of 3’ is backfilled.</td>
</tr>
<tr>
<td>Kentucky</td>
<td>Pile driver with certain number of blows.</td>
<td>Full Yield Strength of piling and LRFD code</td>
<td>Sand, Gravel, or Concrete</td>
<td>Entire Pile Length</td>
</tr>
<tr>
<td>North Carolina</td>
<td>WEAP or PDA/CAPWAP or pile driver with number of blows</td>
<td>SPT borings and Apile software H-piles 0.60 factor or 0.75</td>
<td>Concrete, grout, or flowable fill</td>
<td>Hole filled to up natural ground</td>
</tr>
<tr>
<td>Ohio</td>
<td>CAPWAP</td>
<td>0.60 or .50 Factors AASHTO LRFD Article 6.5.4.2</td>
<td>Concrete or granular fill</td>
<td>Filled to bottom of pile cap Elevation</td>
</tr>
<tr>
<td>Oklahoma</td>
<td>Gates Equation or Practical Refusal</td>
<td>Structural Design and factors from, AASHTO LRFD and ODOT</td>
<td>Sand</td>
<td>Entire Pile Length</td>
</tr>
<tr>
<td>Pennsylvania</td>
<td>Wave Equation Analysis</td>
<td>Driven to Refusal</td>
<td>Granular Material</td>
<td>Backfilled to Tip Elevation</td>
</tr>
<tr>
<td>South Carolina</td>
<td>WEAP /PDA</td>
<td>0.6 Factor used for piles in weak rock</td>
<td>Concrete for shafts</td>
<td>Filled to Minimum Length</td>
</tr>
<tr>
<td>Tennessee</td>
<td>Practical Refusal</td>
<td>SPT test and Design Values Selected by Engineer</td>
<td>#57 Stone</td>
<td>Entire Depth</td>
</tr>
<tr>
<td>Utah</td>
<td>PDA/CAPWAP for initial pile</td>
<td>0.65 Factor from AASHTO LRFD</td>
<td>Flowable Fill</td>
<td>Entire Pile Length</td>
</tr>
<tr>
<td>Wyoming</td>
<td>WEAP</td>
<td>Allowable design strength of pile</td>
<td>Pea sand or Gravel</td>
<td>Entire Pile Length</td>
</tr>
</tbody>
</table>
Multiple methods were mentioned for capacity verification of a pile with a pilot hole. The methods of capacity estimation used were dynamic methods such as, PDA/CAPWAP, Wave Equation, and WEAP, practical refusal, and other methods seen in Figure 16.

![Figure 16 Capacity Verification From 2nd Survey](image)

From the second survey more detailed information was provided on the backfill used for pilot holes as seen in Figure 17. The mentioned fills were concrete, sand, granular fills, grout, and a few others. Concrete was used most frequently by 5 states, followed by sand, granular fills, and grout by 4, 3 and 2 states respectively, while 2 states had no preferred material and 3 states mentioned other various materials.
Figure 17 Backfill Material Used for Pilot Holes
CHAPTER 6

COLLECTION AND REVIEW OF DATA FOR DOT PROJECTS WITH SEATED PILES WITH A PILOT HOLE ON ROCK.

6.1 Introduction

The goal of this task was to collect and review field data on driven piles that were constructed with pilot holes on rock in the United States. The project data that had piles with a pilot hole in Georgia was collected with the help of the GDOT Geotechnical Branch. In addition, data was gathered from the states of North Carolina and Kansas thanks to their DOT agencies. As discussed earlier, there are currently no standards set for verifying the capacity of the pile with a pilot hole on rock. It was found from the surveys that 15 states responded the use of PDA with CAPWAP analysis to the confirm capacity for the case of a pile with a pilot hole on rock. Each collected project includes PDA data that was used to confirm the pile capacity in the field. In the review of these projects with a pilot hole, the design, construction methods, test data, and site conditions were all reviewed to have a better understanding of the use of pilot holes in the state. The research team collected and reviewed four projects. Due to the limited number of applications studying this pile type with load test results, a very limited number of projects were collected even from the 15 states that indicated the use of a pile with a pilot hole on rock. Four projects between 2020 and 2021 in GA, KS, and NC that had piles with pilot holes were collected and reviewed in this study.
6.2 Pile Driving Analyzer (PDA)

The Pile Driving Analyzer (PDA) system is one of the most widely used dynamic load test systems in the United States and other nations. The system can assess the capacity of driven piles, drilled shafts, cast-in-place piles, bored piles, and other pile types (Pile Dynamics Inc, 2017). As noted from the response to the second survey PDA used for a bored pile, also known as a pile with a pilot hole, has been utilized by several DOT agencies. PDA test, also known as high strain dynamic Test, uses a drop weight to impact a pile and the top of the foundation and is cushioned by a few thin plywood sheets when the weight hits the foundation. PDA sensors such as accelerometers and strain transducers obtain data that can calculate foundation capacity (Pile Dynamics Inc, 2017).

When the drop weight impacts the pile, a wave propagates through the pile at the speed of sound until it arrives at the pile toe. Once at the toe the wave is reflected back up the pile and the displacement of the pile as a result of the wave propagation can be described by the following equation (Morton, 2012):

\[
\frac{\delta^2 u}{\delta t^2} = \frac{E \delta^2 u}{\rho \delta z^2}
\]

Where:

\(z\) = depth below ground surface (L)

\(t\) = time

\(u\) = displacement of the pile at depth \(z\) (L)

\(\rho\) = density of the pile material

\(E\) = elastic modulus of the pile

\(L\) = movement of the pile
Setup and running a PDA test takes little time when compared to the other classical pile load test methods. The gauges and sensors that are attached to the pile are connected to a computer on site to record all the data. Both force and velocity wave propagation are recorded through the pile with each blow of the hammer (Morton, 2012). This relationship can be seen in the equation as follows:

\[ \sigma = \frac{E}{c_v} v \]

Where:

\( v \) = particle velocity in the pile (L/t)

\( \sigma \) = stress in the pile (F/L^2)

\( c_v \) = velocity of the compressive wave in the pile (L/t)

The velocity of the compressive wave \( c_v \) is defined as

\[ c_v = \sqrt{\frac{E}{\rho}} \]

The force and velocity waves related to the pile material can be shown as:

\[ F = \left( \frac{EA}{c_v} \right) v \]

Where:

\( F \) = force in pile tip

\( E \) = elastic modulus of the pile

\( A \) = cross sectional area of pile

The measured velocity and force wave versus the time can be plotted for each hammer blow in the driving process. These plots are a good aid in illustrating resistances in the pile.
6.3 Case Pile Wave Analysis Program (CAPWAP)

Case Pile Wave Analysis Program (CAPWAP) is a signal matching software that estimates the bearing capacity of a pile and resistance distribution along the pile shaft and toe. The program utilizes data of force and velocity from the PDA (Inc, 2019). The use of the PDA with CAPWAP can break down a more precise estimation of ultimate pile resistance. Where PDA can be performed in the field CAPWAP analysis needs to be performed in office after the PDA data has been obtained. PDA presents a plot of the measured waves travelling through the pile for each hammer strike, and CAPWAP presents the best match of a theoretical wave plot to the measured wave plot. Achieving the best plot match in CAPWAP is an iterative process (Morton, 2012).

6.4 Project Literature Review

Case I - Georgia

According to the design report (Georgia Department of Transportation, 2018) a 157ft long three span bridge that consisted of four bents was constructed for this project. The project is geologically sited in a biotite gneiss/amphibolite formation of the Georgia Piedmont physiographic province. The subsurface borings encountered partially weathered rock and rock among a variety of soils. The rock type encountered at the site was a metamorphic rock, Gneiss. Auger refusal was also encountered at depths ranging from 9.5 to 27.8 feet from the surface (approximate elevation 448 to 473 feet). Ground water was encountered as well. From the investigation pilot holes would be set up for the potential of hard driving. Pilot holes would be set up for bents 1, 2, 3, and 4. The maximum pilot hole diameter of 24in were used and the holes filled with concrete to the top rock after the piles were driven. Pilot holes would be drilled at least five feet into the rock. The pilot holes should be in accordance with Special Provision 520 Pilot Holes.

PDA with CAPWAP testing was set up near bent 4 for the project and the test pile had and tip elevation of 350. Minimum and Estimated Pile Tips are established with 5 feet of pilot hole from top of bedrock. Additional information analyzed came from the boring log of the project with values of RQD and $q_u$. 

$q_u$. 
Case II- North Carolina

The testing report for the project C204196 had high strain dynamic pile test (PDA) and CAPWAP analysis was performed on one 12×53 pile at end bent 2 on this project. The pile was predrilled to an elevation of +350.9 ft and was monitored during the entire initial drive. The total drive of the tested pile consisted of 27 hammer blows. Signal matching was performed for a representative blow (Blow 23) near the end of drive. The bearing rock consisted of sedimentary rock Triassic Conglomerate. PDA with CAPWAP was set up at end bent two on pile number 1. The project subsurface investigation including bore logs, $q_u$ test results, and RDQ values was also reviewed.

Case III - Kansas

The bridge foundation report provided that the project consisted of the replacement of a bridge. (Kansas Department of Transportation, 2018) The site consisted of fill and residual soils overlaying the bedrock of the Bader Limestone, Steans Shale, and Beattie Limestone Formations. Bedrock is made of alternating layers of shale and limestone. Piles were designed to bear approximately 6 to 7 feet into the bedrock. Steel H piles were driven to bear within the limestone. The recommended piles were HP 12×42 and HP 12×53. The nominal geotechnical resistance were 310 kips for HP 12×42 and 387 kips for HP 12×53 respectively.

Pre-drilling was recommended to be used at pile locations to assist driving to adequate depth. The piles were centered and leveled in the pre-drilled holes before driving. The predrilled hole was then backfilled with clean loose sand after driving the pile. Two test piles with PDA test were set up for the project. The purpose of the PDA was to give a more accurate prediction for production pile performance for the structure. For the test piles an additional 10’ of pile length was added to account for any unforeseen penetration deeper then plan tip elevation. The test piles were set up at Abutment 1 KS(1) and Abutment 2 KS(2) on the project.

6.5 Comparison of bearing capacity

From each project PDA with CAPWAP was used to the measure the pile capacity at the end of driving. The following Table 17 shows the measured total capacity and the portion that was toe capacities.
Table 17 PDA Measured Capacity and Toe Capacity by PDA with CAPWAP

<table>
<thead>
<tr>
<th>State</th>
<th>Pile Type</th>
<th>Measured Ultimate Capacity (kips)</th>
<th>Measured Capacity at the Toe (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GA</td>
<td>HP12×53</td>
<td>383</td>
<td>259</td>
</tr>
<tr>
<td>NC</td>
<td>HP12×53</td>
<td>731</td>
<td>699</td>
</tr>
<tr>
<td>KS(1)</td>
<td>HP12×53</td>
<td>454.9</td>
<td>418.2</td>
</tr>
<tr>
<td>KS(2)</td>
<td>HP12×53</td>
<td>472.9</td>
<td>362.1</td>
</tr>
</tbody>
</table>

These measured values were then compared to calculated capacity estimations. The estimation methods used were Goodman’s Equation, The Canadian Geotechnical Society (CGS) Equation, and the FHWA equation with RQD. All three of the method were discussed previously in Chapter 2. The following is Table 18 with the values used for the calculations.

Table 18 Values for Calculations

<table>
<thead>
<tr>
<th>State</th>
<th>Pile Type</th>
<th>Pile Gross Area (in²)</th>
<th>$q_u$ (psi)</th>
<th>RQD (%)</th>
<th>Bearing Layer</th>
<th>Angle of friction of Rock ($\phi$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GA</td>
<td>HP12×53</td>
<td>141.6</td>
<td>24472</td>
<td>27</td>
<td>Rock: Gneiss</td>
<td>30</td>
</tr>
<tr>
<td>NC</td>
<td>HP12×53</td>
<td>141.6</td>
<td>5366</td>
<td>85</td>
<td>Non-Crystalline Rock Triassic Conglomerate</td>
<td>25</td>
</tr>
<tr>
<td>KS(1)</td>
<td>HP12×53</td>
<td>141.6</td>
<td>2513</td>
<td>63</td>
<td>limey shale bedrock</td>
<td>30</td>
</tr>
<tr>
<td>KS(2)</td>
<td>HP12×53</td>
<td>141.6</td>
<td>2513</td>
<td>45</td>
<td>limey shale bedrock</td>
<td>30</td>
</tr>
</tbody>
</table>

Assumptions were made for inputting parameters for the calculations. For Goodman’s equation the angle of friction was assumed based on if the rock was a sandstone, limestone, shale, granite, or marble.
Table 19 Typical Values of Angle of Friction from (Das, 2007)

<table>
<thead>
<tr>
<th>Type of rock</th>
<th>Angle of friction, $\phi'$ (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandstone</td>
<td>27-45</td>
</tr>
<tr>
<td>Limestone</td>
<td>30-40</td>
</tr>
<tr>
<td>Shale</td>
<td>10-20</td>
</tr>
<tr>
<td>Granite</td>
<td>40-50</td>
</tr>
<tr>
<td>Marble</td>
<td>25-30</td>
</tr>
</tbody>
</table>

As explained in Chapter 2.2.2 the input parameters of depth of rock socket ($L_o$), spacing and aperture of discontinuities (c and $\delta$), and footing width (B) are necessary for the CGS Equation, and these were assumed for the calculation for each project. For the FHWA RDQ equation $0.33q_u$ was used for RQD values less than 70% and $0.80q_u$ was used for RQD values greater than 70%. The calculations were made, they calculated values were then compared to the measured PDA with CAPWAP values and results are shown on Figure 18.

![Figure 18 Measured vs Calculated Ultimate Pile Capacity](image)
The first observation made from the comparison of the values are the high numbers from the Georgia DOT project. Here all three calculated values from Goodman’s, CGS, and FHWA-RQD equations result higher than the measured PDA total capacity and the measured toe capacity. Both Goodmans and Canadian Geotechnical Society equations rely on the unconfined compressive strength of the rock, $q_u$, to estimate capacity. In general, the higher the $q_u$ value the higher the capacity estimated. The rock that was tested on the for the site has very high $q_u$ values. On this project five boreholes were obtained in the subsurface exploration with identifiers B-01, B-01A, B-02, B-03, and B-04. B-04 was the borehole closest to where the test pile was set up. However, it was noted that the borehole closest to the where the test pile was done had no $q_u$ testing at the pile tip elevation. This testing was done at boreholes B-01 and B-01A located on the south side of the bridge. From the values of $q_u$ provided at these boreholes it was assumed the borehole closest to the test pile B-04 would carry similar values. In addition, it was noted that RQD values for B-04 were not presented on the boring log, and thus, values from B-03 were used. At the pile tip elevation, the RQD value from B-03 was used and the $q_u$ from B-02 was used. The value of RQD of 27% was much lower and did correspondent to a very strong rock as $q_u$ of 24,472psi indicated. The RQD value for rock this strong would generally be in the 90 percentiles.

As seen in Figure 16, the estimated values from Goodman’s, FHWA, and Canadian Geotechnical Society equations were compared well to the measured values PDA values for the North Carolina project. This was in part to this project had the most detailed subsurface investigation information. The PDA test was conducted at end bent 2 however borings with $q_u$ testing and RDQ values were done at bent 1. Three borings were made at this location and the three $q_u$ values along with three RQD were averaged to get the value used in the calculations.

The two test piles on the Kansas project did not compare to well for Goodman’s and the FHWA RQD equations but did compare well for the CGS Equation. While Goodman’s equation is reliant on the value $q_u$ and the friction angle of the rock, the Kansas test piles did have the lowest $q_u$ values of the group. In addition, there were no tested $q_u$ values for the rock at the pile tip elevation for abutment 2 according
to the boring logs. Abutment 1 did have tested values for $q_u$ of rock and these rock values were assumed for abutment 2 as according to the boring log the same rock types were present at both pile tip elevations. The angle of friction of the rock was assumed based off the values in Table 1.
CHAPTER 7

CONCLUSIONS

7.1 First Survey

It was found that the majority of states have used the pile installation method with the assistance of a hole to guide it, but a variety of terms are used to refer to the hole. The three most common terms used were pre-drilled hole, pre-bored hole, and pilot hole. It was found that these holes (hereinafter referred to as pilot holes) are used in both soil and in rock layers for the majority of state DOT agencies including Georgia, while only a quarter of the states only use it in soil or rock. Regarding the size of the pilot hole, nearly half of all the states indicated that a larger hole size is used, and the rest indicated different options such as a smaller hole, hole equal or larger than the pile size, or the hole sizes varied depending on the site and project conditions. The consideration of the skin resistance is the one of the critical aspects in the pile design particularly when the pile with a pilot is on rock. Half the state agencies indicated skin friction is neglected in the design process for a pile of this type. A small number indicated the used of skin friction is partial or varies along the pile length. A smaller number indicated that skin friction accounted for in the design. It was found that the size of the pilot hole plays a significant role if skin friction was accounted for or neglected. For states who use a larger pilot hole mostly did not account for skin friction. Ones with a smaller hole size accounted for skin friction. Many states seemed to indicate that skin friction was dependent on the size of the hole.

The last question of the first survey was how pile capacity was estimated for a pile with a pilot hole particularly when they are installed to rock layers. A large number of states responded that the pile is designed as an end bearing pile using the AASHTO LRFD end bearing pile design methods with a smaller number using static analysis methods to estimate pile capacity. A few states had their own state specific design procedures. Some were state specific design tables or capacity charts, and others used a mixture of AASHTO, static analysis, and state researched methods to estimate pile capacity. Another portion limited the capacity with the structural capacity of the pile.
The largest observation made in the first survey responses was the lack of consensus on nearly all aspects of a pile with a pilot hole to rock. From all the various terms, this study recommends the use of the term “Pilot Hole” as it clarifies the role of the hole as a pile driving assistant meant to offer as an aid to achieve pile tip elevation. This term also prevents confusions with similar terms being used for other deep foundation types. When such a pile with a pilot hole is in rock layer, it was most common to design the pile as an end bearing pile or limited to the structural capacity of the pile. The first survey was able to provide a good picture of the current status of how piles with a pilot hole on rock and soil are designed and construed.

7.2 Second Survey

The second survey was sent out to a selected pool of states based on their answers to the first survey. The selection was made on proximity to Georgia and how similar the handling of a pile with a pilot hole on rock was to Georgia.

The first question was in regard to how the capacity of the pile with a pilot hole was verified in the field. The majority of states indicated the use of dynamic methods such as the Pile Dynamic Analyzer (PDA), or wave equation analysis to verify capacity. The next method in a good number of responses was to drive the pile to practical refusal. In addition, more details in the design of a pile with a pilot hole were requested including the determination of resistance factors and pile capacity. As a result, the most commonly used capacity estimation and resistance factors were the ones recommended by the AASHTO LRFD Bridge Design Manual. The final question of the second survey was in regard to the materials for filling the holes when the hole size is larger than the pile. The reported materials were primarily sand or concrete and are typically filled to the surface. A few other states did not have a specified backfill material as these states indicated the use of the most cost effective or readily available material for the project.

7.3 Measured PDA to Calculated Comparison

The comparison of pile capacity that was measured with PDA testing with CAPWAP to what was calculate using equations yielded various results. The project that compared the best was the one in North
Carolina and the project in Georgia showed the largest disparity. The disparity between the measured and calculated values could be attributed to the assumptions made for some of the RQD, unconfined compressive strength ($q_u$) and/or friction angle of the rock. For the project in Georgia, the estimated capacity by PDA was significantly higher than that by the static method. This seems to be due to the discontinuities monitored in the boring log and samples, while the unconfined compressive strength test must have been conducted with an intact section of the Gneiss rock.

The two tested piles from Kansas project both had a decent compression with what was measured with the dynamic testing. Some assumptions had to be made as well in the calculations for the pile capacity because of spatial variability or lack of design parameters. For example, the value of ($q_u$) from abutment 1 was used for abutment 2, and the friction angle of the rock was assumed as well using best judgments.

7.4 Improvements for Future Study

The study was able to provide a good picture of how piles with pilot holes are considered in the state agencies in the U.S. through the two rounds of surveys and additional follow up communications, and the goal to establish the current design and construction methods used for piles with pilot holes was accomplished. The next improvement for the study would be to get more information from states about how the pile capacity is verified in the field and designed. This would mean the collection of more design and PDA data with CAPWAP to be able to have a better comparison to see if PDA testing could be adequate to giving an estimation of pile capacity in the field. Getting responses from DOT agencies is time consuming and at times took multiple attempts. From those attempts only three states provided project data to compare where there are at least 15 that use similar methods to verify pile capacity in the field. If the study could have obtained more project data to review, then a better comparison and conclusion could be made for the best way to verify the pile with a pilot hole in the field.
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