GEORGIA DOT RESEARCH PROJECT 19-06

Final Report

LRFD PROCEDURE FOR PILES WITH PILOT HOLE IN ROCK

Georgia Department of Transportation

Office of Performance-based Management and Research
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16. Abstract:

A pile driven into the ground gets its bearing capacity from skin friction along the pile surface as well as from end resistance at the toe. The load transfer mechanism determines how much load is carried by the shaft and by the toe. In Georgia, when a hard/dense layer exists in the pile length or the vibration/noise during the driving causes secondary issues, a pilot hole is often adopted as a pile-driving assistance method to aid driving displacement piles through, especially if a competent hard rock layer exists in a reasonable depth. The use of a pilot hole reduces construction time and uncertainties related to driving through the problematic layers. However, it also reduces the side resistance within the drilled zone due to the disturbance and size of the hole. This process also complicates the prediction of long-term pile capacity. An objective of this study was to identify and document the current guidelines available and adopted by different states, and investigate the relationship between the load capacity of piles installed in rock and their design parameters with respect to the pilot hole, rock conditions, and installation method. Another objective was to identify a reliable design procedure that incorporates proper LRFD resistance factors, and a field verification method for quality assurance of rock. A compilation of best practice methods was necessary, which includes a literature review, a survey with state highway agencies, field tests, a review of past projects and testing data, and making final conclusions. Through these efforts, it is found that most states using a pilot hole in rock, the hole size is larger that the pile and the skin friction is ignored for the nominal resistance. In addition, geotechnical aspects control the design in soft rock, whereas structural aspects control the design in hard rock. Some states do not run a field test but use refusal/end of driving criteria for piles driven into rock. Nevertheless, PDA can be applied to the piles with a pilot hole on rock to check the internal stress to avoid the damage during striking. Moreover, it can verify the structural capacity of the pile if not the geotechnical capacity due to the higher bearing capacity on rock.

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GDOT Research Project 19-06

Final Report

LRFD PROCEDURE FOR PILES WITH PILOT HOLE IN ROCK

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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EXECUTIVE SUMMARY

A pile driven into the ground gets its bearing capacity from skin friction along the pile surface as well as from end resistance at the toe. The load transfer mechanism determines how much load is carried by the shaft and by the toe. In Georgia, when a hard/dense layer exists in the pile length or the vibration/noise during the driving causes secondary issues, a pilot hole is often adopted as a pile-driving assistance method to aid driving displacement piles through, especially if a competent hard rock layer exists in a reasonable depth. The use of a pilot hole reduces construction time and uncertainties related to driving through the problematic layers. However, the pilot hole is considered different from a pre-drilled hole in terms of construction method and design assumption. For example, driving to penetrate the ground is not necessary for the pile with a pilot hole. In addition, the side resistance is ignored with the pilot hole in most states; however, it can be considered with a predrilled hole, although the resistance would depend on the disturbance within the drilled zone and size of the hole. This process also complicates the prediction of long-term pile capacity with a predrilled hole. An objective of this study was to identify and document the current guidelines available and adopted by different states, and investigate the relationship between the load capacity of piles installed in rock and their design parameters with respect to the pilot hole, rock conditions, and installation method. Another objective was to identify a reliable design procedure that incorporates proper LRFD resistance factors, and a field verification method for quality assurance of rock. A compilation of best practice methods was necessary, which includes a literature review, a survey with state highway agencies, field tests, a review of past projects and testing data, and making final conclusions. Through these efforts, the research team was able to identify several important findings.

This study was conducted to understand how piles with a pilot hole in rock are being considered in design and construction, especially with the American Association for State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) recommendations. The research team conducted a literature review to understand how the pilot hole and bearing rock layer have been considered in the design and how they can be considered with the new design approach. In addition, a thorough review of the design approaches in the United States was completed by sending out two rounds of survey: (1) an initial survey to all 50 states, and (2) a second survey to 20 selected states. The response rate was very high and provided very useful information about how the pile is being considered and used in different states. Furthermore, field tests were added with the extension of the project to evaluate the possibility of the use of the Pile Driving Analyzer® (PDA) as a verification method.

TASKS

After thoroughly reviewing the literature regarding each task, the major findings and conclusions are summarized below.

Task 1: The Current Design Methodology for the Construction of Piles in Rock

- Various static and dynamic methods are available for driven piles.
- The static methods include the Canadian Geotechnical Method introduced in the AASHTO LRFD Specifications, the Federal Highway Administration

(FHWA) rock quality designation (RQD) toe resistance method, the Illinois Department of Transportation (IDOT) static method, and the Tomlinson and Woodward method.

- The dynamic methods include the FHWA modified Gates formula, the Engineering News formula, the Washington State Department of Transportation (WSDOT) pile driving formula, and wave analysis.

Task 2: The Effects of the Pilot Hole on the Pile Capacity and Behavior, along with the Associated Design and Construction Considerations

- From the collection of survey responses from 48 out of 50 states, it was found that most states have guidelines on how they use a pilot hole (or pile driving assisting methods in a broader term). However, there seems to be no consensus in the terminology, or the design and construction methods related to the use of the pilot hole.
- Some states were identified for a second-round survey to review further for specific information.

Task 3: The Current Specifications and Verification Methods for Pile Installation in Rock with a Pilot Hole and the Available Equipment/Methods

- The second-round survey was sent to 20 states, and 14 of those states responded. Two of the responding states were dropped due to their ambiguous statements regarding their guidelines, and 12 states were evaluated.
- Most of the responding states use one or more of the dynamic methods (e.g., WEAP, PDA, CAPWAP) for their quality control and quality assurance in the

field and AASHTO resistance factors for the design. Only a few states responded that they use empirical methods such as bearing refusals.

- Concrete is often filled to the level of the rock, while other low-cost materials are used to fill the hole all the way to the ground level.

Task 4: The Field Data on Driven and Seated Piles in Georgia

- Ten projects in Georgia were selected, of which eight were reviewed for their design and verification methods, if available.
- One of the projects includes one set of static load tests and three sets of Pile Driving Analyzer tests.
- All three piles were prepared with a pilot hole. However, due to the geological conditions faced during the driving, one of them (Bent 1 Pile 5) was sliced and driven further (about 16 ft) into the ground. The other two piles were seated on the rock layer as planned and driven for the PDA test.
- PDA with CAPWAP has been applied to some of the projects already.

Task 5: The Verification Methods for Quality Assurance of Rock

- Rock quality has been assessed by collecting core samples in the field and conducting laboratory tests.
- In situ rock quality tests are limited, especially for the pile with a pilot hole, mainly due to its size.
- The traditional rock mass classification methods and uniaxial compressive strength can provide enough information to estimate the end bearing of the pile on rock.

Task 6: Development of Appropriate LRFD Design Methodology and Resistance Factor

- The resistance factors could not be calibrated for use in Georgia due to the number of data necessary for the statistical and reliability analyses.

CONCLUSIONS

1. Geotechnical aspects control the design in soft rock, and structural aspects control the design in hard rock.
2. PDA can be applied to the piles with a pilot hole on rock to check the internal stress to avoid damage during striking. It also can verify the structural capacity of the pile, if not the geotechnical capacity, due to the higher bearing capacity on rock.
3. Many states use the resistance factor suggested in the AASHTO *LRFD Bridge Design Specifications*. However, some states also use the revised resistance factor for their current pile design on rock, whether a pilot hole is being used or not.
4. For most states using a pilot hole, the skin friction is ignored for the nominal resistance because the pilot holes are larger than the pile, which seems to be a reasonable and conservative assumption.
5. Some states do not run a field test but use refusal/end of driving criteria for piles driven into rock. Similar criteria could be applied to the pile with a pilot hole in rock as a supplement or replacement to PDA.

RECOMMENDATIONS

1. For piles with a pilot hole in rock, this study recommends the use of PDA tests and the AASHTO resistance factor for driven piles with dynamic testing.

2. Use of skin friction is not recommended unless further research is conducted to evaluate the load transfer mechanisms and the effects of the pilot hole size and filling on the piles.

3. Performing the drivability analysis and checking the hammer settings (e.g., stroke or energy) are recommended using wave equation software (e.g., GRLWEAP) to avoid overstressing of the pile.

4. Collecting the PDA test results along with the strength properties of the rock mass is also recommended.

5. If the PDA test is not available, the driving refusal criterion (e.g., 5 blows per 0.5 inch) can be used. However, it is still recommended that the correlations between the refusal guidelines and rock properties are verified with PDA.

6. An alternative option could be using the resistance factor for the drilled shaft for tip resistance in rock to the pile with a pilot hole on rock.

Even though larger resistance factors are being used by some states, it is not recommended that Georgia adopt larger factors from other states at this point. Instead, additional studies are recommended to investigate further the use of PDA and CAPWAP for hard rock to ensure the appropriate use of the technology. In addition, collecting the data for PDAs, penetration per blow, and rock properties is strongly recommended so that they can be correlated with each other eventually.

FUTURE RESEARCH RECOMMENDATIONS

This study was able to provide an overview of how piles with pilot holes are considered in state agencies in the U.S. and how those are designed and constructed. Although most states use the resistance factors recommended by AASHTO (2020), some states have improved their resistance factors for piles on rock.

Therefore, this study suggests the following future work:

- Set up a rock mass properties database from projects during the design phase and/or construction sites.
- Collect the PDA test results for the piles on rock in Georgia for future research recommendations.
- Create an integrated load test database especially for piles on rock that drive through weak rock or seat on hard rock.
- Correlate the PDA results with other properties, such as design methods, static load test results, penetration per blow, and rock properties.
- Investigate the effects of pilot hole and socket sizes and depth to the pile with a pilot hole for certain cases that skin friction is considered.
- Compare the cost and benefits for a larger resistance factor with no test versus a smaller resistance factor with PDA tests.

CHAPTER 1. INTRODUCTION

Pilot holes aid in the installation of piles through rigid layers of the ground to include rock. Pilot holes are drilled to a specific depth, allowing a reduction in driving resistance by reducing or eliminating the shaft resistance but improving the construction quality by avoiding vibration or noise. The goal of this project is to investigate the design method and the verification of the pile with a pilot hole design on rock in relation to its rock properties, hole geometry, and installation method. To identify some possible recommendations of the design approach of piles with pilot hole on rock, this study includes a literature review, surveys from most states, and field data.

In general, different types of piles are used in the civil engineering field, such as steel H-piles, concrete-filled metal shell piles, and precast concrete piles, with the piles being driven or drilled and placed. The main goal of the piles is to place a pile in the rigid ground to provide enough support for the structure from the shaft and end resistances. Figure 1 shows a conceptual diagram of typical pile installation methods, including the forces expected to be required or develop during the installation. A pile can be driven into the ground, developing resistance from both shaft and toe as shown in figure 1(a), which is a preferred method in Georgia. The other common type is a cast-in-place pile for which a hole is drilled and filled with reinforcements and concrete, as in figure 1(b). When a rock layer is at a relatively shallow depth, an alternative option can be either driving or lowering a pile into a drilled hole that is smaller, equal to, or larger than the size of the pile, as in figure 1(c), (d), and (e), respectively.

Factors that must be considered when selecting a pile installation method often include construction costs, availability of equipment, the ground conditions at the site, and previous experience and preference of the clients.

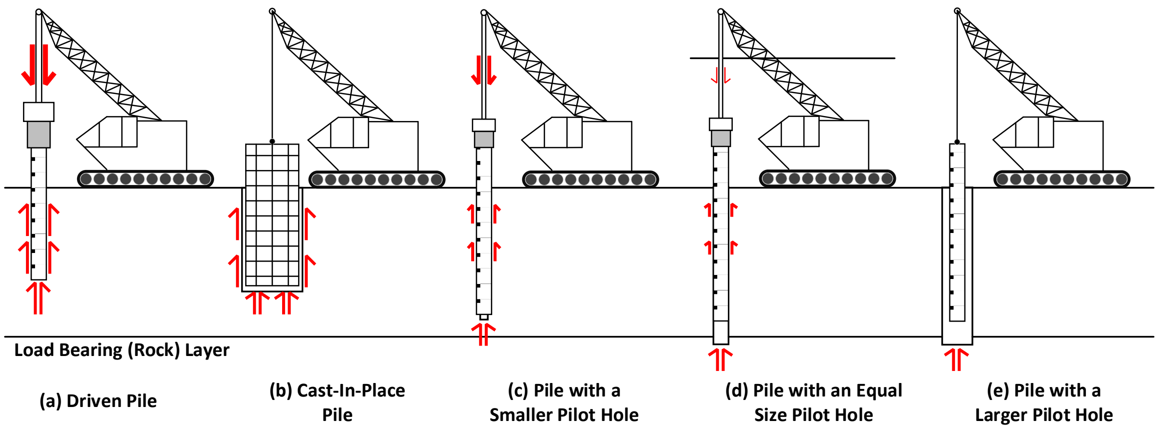

Figure 1. Diagrams. Different pile installation methods (Modified from Crowner et al. 2021).

A pile with a pilot hole method is intended to aid the pile in installation on rigid ground and make it easier to place the pile on the bearing layer. The pilot hole can be larger or smaller than the pile being installed, depending on the types of ground, and subsequently, the source of the bearing capacity can be considered differently. For example, the Georgia Department of Transportation (GDOT) recommends the pilot hole be larger than the diameter of the pile when installed on rock, whereas the Texas Department of Transportation (TxDOT) recommends a smaller pilot hole in soil. The Indiana Department of Transportation (INDOT) recommends either a smaller or larger hole, whether it is preboring or predrilling. The size of the pilot hole will have an impact on the drivability and the long-term capacity of the pile (Crowner 2021). These pilot holes can aid in the driving of the piles. Drilling a pilot hole and placing the pile in the ground significantly reduces the risk of the pile being damaged should it be driven on the hard rock layer.

This study consists of seven major tasks; six of these tasks are mainly literature reviews that were originally included, and the seventh is field tests that were added when the project was extended with no cost change. Each task is provided as a major chapter in this report with the exception that Tasks 4 and 6 were combined and presented as one chapter. Conclusions and recommendations are provided at the end. Appendices include the detailed information that was prepared or collected during the study period.

The original six tasks are as follows:

- Task 1. Review the current design methodology for the construction of piles in rock.
- Task 2. Review the effects of a pilot hole on the pile capacity and behavior, along with the associated design and construction considerations.
- Task 3. Review current specifications and verification methods for pile installation in rock with a pilot hole and the available equipment/methods.
- Task 4. Collect and review field data on driven and seated piles in Georgia.
- Task 5. Review verification methods for quality assurance of rock.
- Task 6. Develop an appropriate load and resistance factor design (LRFD) methodology and resistance factor(s).

The revised tasks as presented in this report are as follows:

- Task 1. Review the current design methodology for the construction of piles in rock.
- Task 2. Review the effects of a pilot hole on the pile capacity and behavior, along with the associated design and construction considerations.
- Task 3. Review current specifications and verification methods for pile installation in rock with a pilot hole and the available equipment/methods.
- Task 4. Collect and review field data on driven and seated piles in Georgia.
- Task 5. Review verification methods for quality assurance of rock.
- Task 6. Investigate the feasibility of the Pile Driving Analyzer® (PDA) test for verifying the capacity of a pile in rock.
- Task 7. Develop an appropriate LRFD design methodology and resistance factor(s).

CHAPTER 2. CURRENT DESIGN METHODOLOGY FOR THE CONSTRUCTION OF PILES IN ROCK

INTRODUCTION

The purpose of this research project is to evaluate and analyze 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. When piles are constructed in such environments, the capacity of the piles will be determined by the properties of both the bearing layer and the piles. Therefore, it is important to understand how design methods consider various parameters differently. In this chapter, the current design methodology for driven piles and drilled shaft in rock is reviewed.

DESIGN METHODS FOR DRIVEN PILES ON ROCK

Determining the material and geometric properties of a pile for deep foundations starts with a static design process. In general, the typical process for the design and construction of a driven pile foundation is as shown in figure 2 (Xiao 2015).

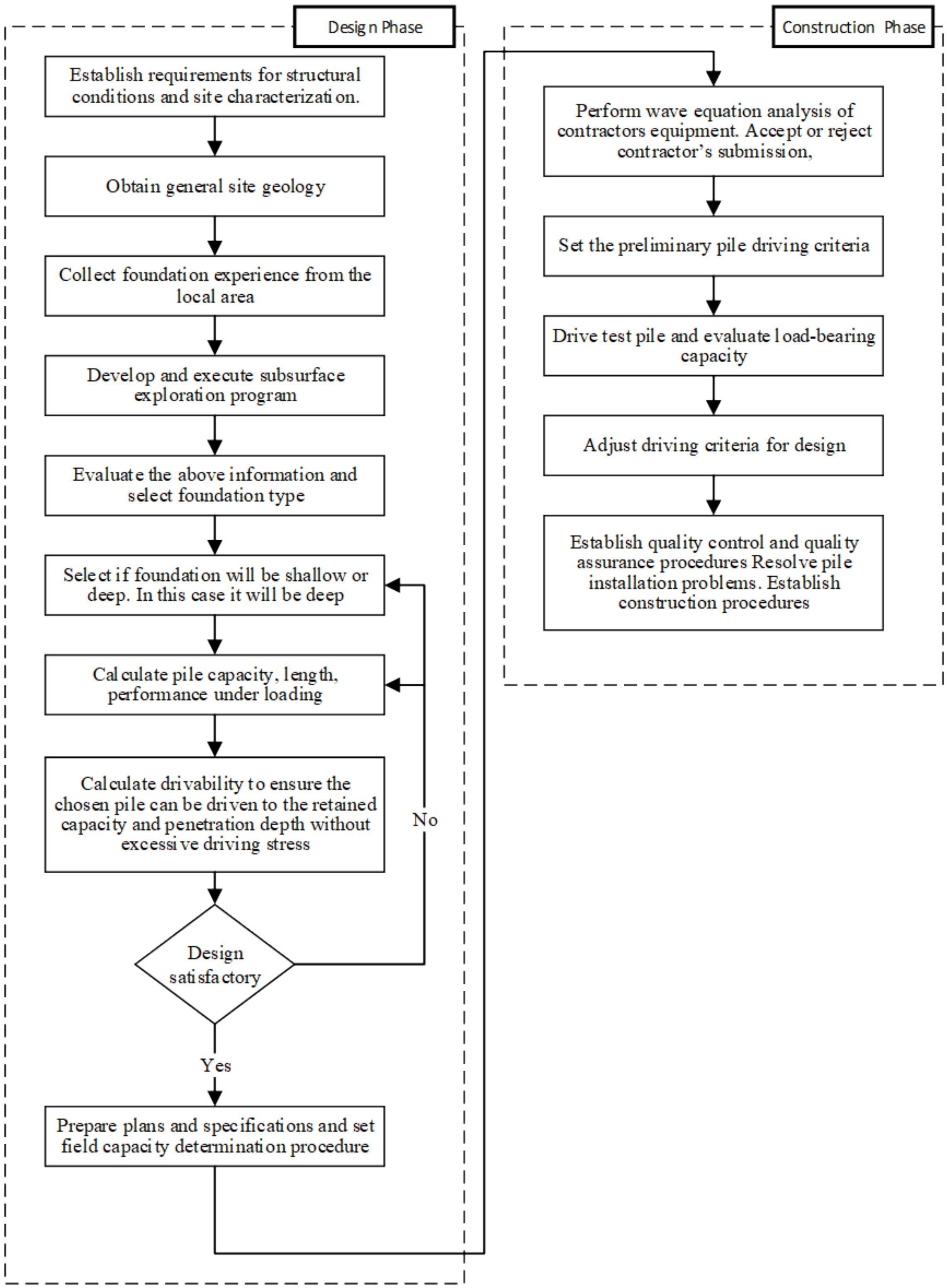

Figure 2. Flowchart. Driven pile design and construction process (Modified after Xiao 2015).

STATIC DESIGN METHODS FOR DRIVEN PILES ON ROCK

The static design and analysis process is what establishes the pile geometry and develops the required resistance factors for a specific soil profile. Some of the needed soil parameters are soil index properties, strength, location of ground water table, and presence of rock. This process 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 (i.e., boring, sampling, and in situ testing).

The subsurface exploration should provide the depth and thickness of the strata, in situ testing to determine soil design parameters, samples to determine soil and rock parameters, and groundwater levels (Hannigan et al. 2016a). If the subsurface investigation and soil-boring testing establish the presence of bedrock or rock-like material, piles can be extended to the rock surface. The ultimate pile capacity will depend on the load-bearing capacity of the underlying material, and these piles are known as "point bearing piles" or "end bearing piles." The ultimate load of a pile constructed on the bed of hard stratum can be expressed as shown in equation 1 (Das 2011).

$$Q_u = Q_p + Q_s \qquad (1)$$

Where:

Q_p = load carried by the pile point

Q_s = load carried by skin friction developed at the side of the pile

When a pile is installed with a pilot hole, the shaft resistance (Q_s) will be smaller than that of typical piles driven through soils without a pilot hole, and thus, it is often ignored in the design, as shown in equation 2.

If Q_s is very small or can be ignored, then:

$$Q_u \approx Q_p \tag{2}$$

This chapter focuses on the review of the end bearing estimating methods with rock assuming that Q_s can be negligible, as the skin friction would be reduced if not eliminated with a pilot hole. In addition, the reduced Q_s would be relatively small enough to be ignored compared to Q_p.

Canadian Foundation Engineering Manual Method

The Canadian Geotechnical Society proposes equation 3 to estimate the ultimate end bearing capacity of a pile on rock with certain properties based from rock cores (Canadian Geotechnical Society 2006; Morton 2012).

$$q_p = \mathrm{FS}\sigma_c K_{sp} d \tag{3}$$

Where:

q_p = ultimate end bearing capacity

FS = factor of safety

σ_c = average unconfined compressive strength of rock core

K_{sp} = empirical factor

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 coefficient K_{sp} is provided in table 1.

Table 1. Coefficients of discontinuity spacing (K_{sp})
(Canadian Geotechnical Society 2006).

Discontinuity Spacing		K_{sp}
Description	**Distance (m)**	
Moderately Close	0.3–1	0.10
Wide	1–3	0.25
Very Wide	>3	0.40

The bearing pressure coefficient (K_{sp}) considers 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.

When sufficient information is available, the coefficient can also be determined by equation 4, which is valid for $0.05<\frac{c}{B}<2.0$ and $0<\frac{\delta}{c}<0.02$. It also can be expressed graphically as shown in figure 3. This method relies highly on good evaluation and description of rock formation (Shao 2023).

$$K_{sp} = \frac{3+\frac{c}{B_s}}{10\sqrt{1+300\frac{\delta}{c}}} \tag{4}$$

Where:

c = spacing of discontinuities

δ = aperture of discontinuities

B_s = diameter of rock socket

The relationship in figure 3 is valid for a rock mass with a spacing of discontinuities (c) greater than 300 mm, aperture of discontinuities (δ) less than 5 mm, and foundation width (B_s) greater than 300 mm. The strata must also be near horizontal for sedimentary rocks.

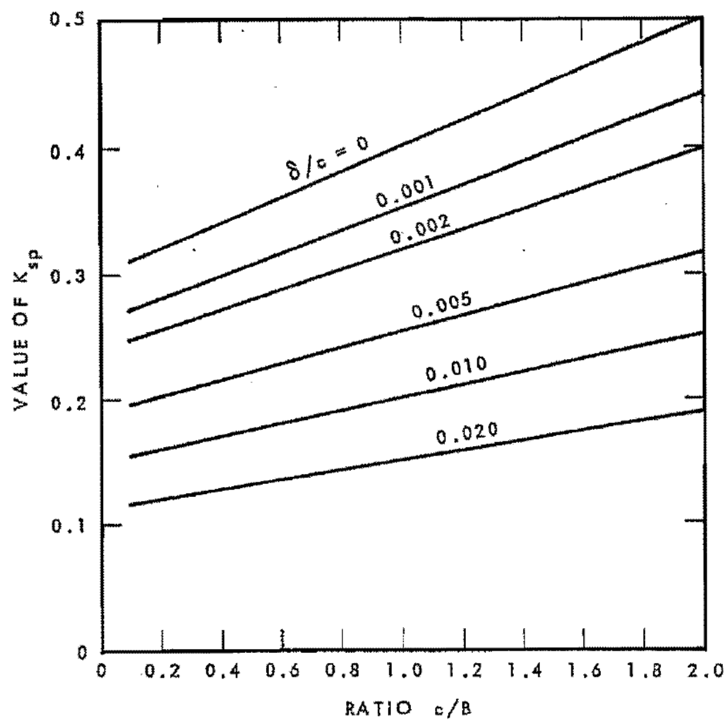

Figure 3. Graph. Bearing pressure coefficient (K_{sp}) (Canadian Geotechnical Society 2006).

Federal Highway Administration RQD Toe Resistance

The Federal Highway Administration (FHWA) manual (Hannigan et al. 2016a) provides an expression based on data from Kulhawy and Goodman's study (1980). Their study showed that 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) (see equations 5–7).

$$q_p = 0.33 q_u \qquad \text{for } 0\% \le \text{RQD} \le 70\% \qquad (5)$$

$$q_p = [0.33 + 0.0157 \times (\text{RQD}-70\%)]\ q_u \qquad \text{for } 70\% < \text{RQD} < 100\% \qquad (6)$$

$$q_p = 0.80 q_u \qquad \text{for RQD} = 100\% \qquad (7)$$

The nominal toe resistance (q_p) can be linearly interpolated from 0.33 q_u to 0.80 q_u if the RQD value is between 70 and 100 percent, as given in equation 6.

Goodman's Equation

The Goodman expression of ultimate point resistance is approximately:

$$q_p = q_u\,(N_\phi + 1) \qquad (8)$$

Where:

$$N_\phi = \tan^2\left(45 + \frac{\phi'}{2}\right)$$

ϕ' = drained angle of friction

q_u = unconfined or uniaxial compressive strength of rock

It is also recommended that q_p is reduced by one-fifth when used in the design to take into account the scale effect (Das 2011).

Typical values of q_u and ϕ' for different types of rocks are provided in table 2.

Table 2. Typical unconfined compressive strength (q_u) and friction angle (ϕ') of rocks. (Adapted from Das 2011, Hoek and Bray 1981, and Wylie and Norrish 1996).

Type of Rock	q_u		ϕ' (degrees)
	MN/m^2	lb/in^2	
Shale[a]	35–70	5,000–10,000	10–20
Sandstone[a]	70–140	10,000–20,000	27–45
Limestone[a]	105–210	15,000–30,000	30–40
Granite[a]	140–210	20,000–30,000	40–50
Marble[a]	60–70	8,500–10,000	25–30
Schist[b,c]	25–50[b]	3,500–7,500[b]	20–27[c]
Gneiss[b,c]	100–200[b]	15,000–30,000[b]	27–34[c]

[a] Das (2011); [b] Hoek and Bray (1981); [c] Wyllie and Norrish (1996).

Illinois Department of Transportation Static Method

Illinois DOT (IDOT) conducted a research project with the University of Illinois that identified the static method for the design of pile foundations considering the ground conditions, piles, and equipment commonly used in the state, and suggested the new Modified IDOT Static Method, as given in equation 9 (IDOT 2010).

$$R_N = \left(F_S q_S A_{AS} + F_p q_p A_p\right) \times I_G \qquad (9)$$

Where:

R_N = nominal pile resistance

F_S, F_p = pile type correction factor for side and toe resistances, respectively

q_S, q_p = nominal unit side and tip resistances, respectively

A_{AS}, A_p = surface and cross-sectional areas of the pile, respectively

I_G = bias factor ratio relating to the bias between the design and construction verification techniques

The bias factor ratio (I_G) is accounted to differentiate bias between construction method (Washington State DOT [WSDOT] method) used to verify the R_N and the method (IDOT method) used to estimate scour, downdrag, and resistance required to support downdrag loads. I_G values used by WSDOT and IDOT are 1.05 and 1.04, respectively.

Nominal unit pile resistances (q_s and q_p) of rock are determined by using the presumptive values for the type of rock encountered, as shown in table 3.

Table 3. Nominal pile resistance of rock (q_s and q_p) . (IDOT 2010).

Type of Rock	q_s (ksf)	q_p (ksf)
Shale	12	120
Sandstone	20	200
Limestone/Dolomite	24	240

Actual penetration of a pile into rock is affected by several factors, such as degree of weathering, rock and pile strength, and hammer energy. It is known that the IDOT Static Method represents these by rock type and nominal bearing pile size. The empirical pile resistance values in table 3 provide a conservative representation of pile penetration into rock resulting in determination of total pile length, when they are employed with the soil side resistance and rock side resistance.

Tomlinson and Woodward Method

Tomlinson and Woodward (2008) noted that very high concentrated loads can be created on the rock beneath the pile toe. Physical rock properties, such as compressive strength, frequency of fissures and joints in the rock mass, and their conditions (i.e., the discontinuities are tightly closed or are open and filled with weathered material) are known as the critical factors. As expected, very high loads can be supported if the rock is strong and has closed joints or joints on a shallow angle to horizontal, whereas the resistance may be reduced if the rock has steeply inclined and open joints.

With a pilot hole, shaft resistance will be significantly reduced or eliminated, but the load may still be acceptable for strong, intact rock. Some of the empirical values for nominal toe resistances can be useful and are available in table 4 . These values can be used for estimating purposes or as a check of values obtained from field tests. However, they are not supposed to be used as final design values unless the applicability of the underlying method or the suitability of a reported nominal resistance value to a given site or geologic formation are considered.

Table 4. Empirical values for nominal toe resistances (Hannigan et al. 2016a).

Rock Description	Pile Type	Nominal Unit Toe Resistance (ksf)
Weak Carbonate Siltstone/Sandstone (coral detrital limestone)	N/A	106.7
Limestone	Steel H-Pile	240
Weak Calcareous Sandstone	Steel Pipe (Metal Shell) Pile	62.6
Sandstone	Steel H-Pile	200
Shale	Steel H-Pile	120

For piles driven to hard rock, the nominal resistance is usually controlled by the structural limit state. In hard rock designs, the nominal structural resistance of pile will generally be less than the nominal geotechnical resistance of hard rock. The nominal unit toe resistance (q_p) in ksf can be calculated using equation 10:

$$q_p = P_s s_u N_c + \gamma D N_q + P_t \gamma \left(\frac{bN_\gamma}{2}\right) \tag{10}$$

Where:

s_u = undrained shear resistance of rock (ksf)

γ = effective density of the rock mass (kcf)

D = pile penetration below the rock surface (ft)

b = pile width diameter (ft)

P_s = pile toe shape factor of 1.25 for square piles or 1.2 for circular piles

P_t = pile base factor of 0.8 for a square pile or 0.7 for a circular pile

Figure 4 shows the bearing capacity factors (N_c, N_q, and N_γ) for equation 10.

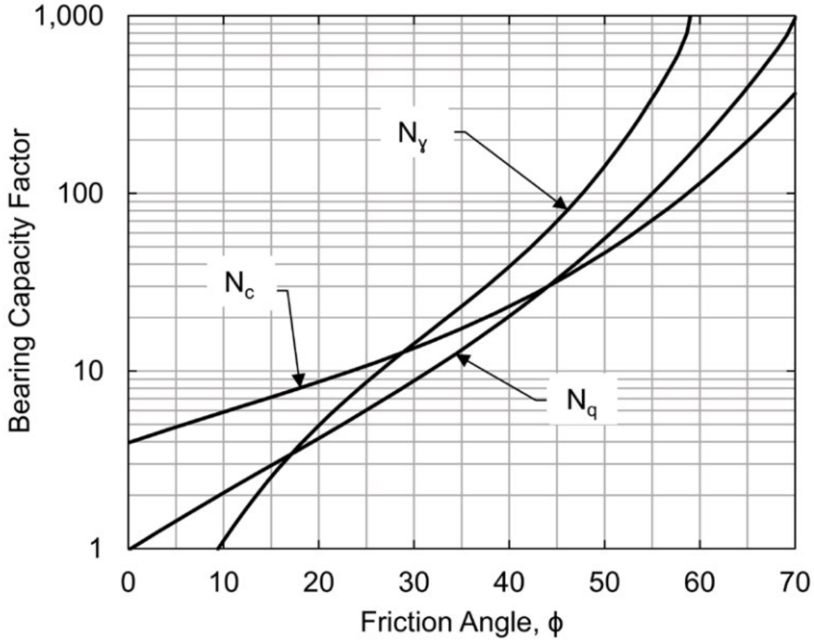

Figure 4. Graph. Bearing capacity factor for foundations on rock (Hannigan et al. 2016a).

DYNAMIC DESIGN METHODS FOR DRIVEN PILES

Engineers have sought to find rational methods to estimate geotechnical capacity/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 (Hannigan et al. 2016a).

FHWA Modified Gates Formula

In 1967 the original Gates formula was modified by Roy E. Olson and Kaare S. Flaate to have a better statistical fit through the predicted measured data. The FHWA introduced more modifications, which take the average of the equations for steel and concrete piles. The FHWA Gates equation reduced the tendency to underpredict capacity and has demonstrated improved accuracy compared to the Engineering News Equation (Bostwick 2014).

The American Association of State Highway and Transportation Officials (AASHTO) *LRFD Bridge Design Specifications* include two dynamic formulas (AASHTO 2020). One of those is the modified FHWA Gates formula, which is preferred by AASHTO to predict bearing capacity and establish driving criterion because it is known to correlate better with static load test results (Hannigan et al. 2016b).

Equation 11 includes the 80 percent efficiency factor on the rated hammer energy recommended by Gates.

$$R_n = 1.75\sqrt{E_d}\ \log_{10}(10N_b) - 100 \qquad (11)$$

Where:

R_n = nominal pile driving resistance measured during pile driving (kips)

E_d = developed hammer energy (ft-lb)[1]

[1] 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, which is the ram weight times stroke height (ft-lb).

N_b = Number of hammer blows for 1.0 inch of pile permanent set (blow/inch)

AASHTO (2020) recommends a resistance factor (φ_{dyn}) of 0.40 for the FHWA Modified Gates formula, and that the formula be used only for end-of-drive conditions.

Engineering News Formula

Another dynamic formula introduced in the AASHTO *LRFD Bridge Design Specifications* is the Engineering News Formula. This formula was originally developed by Arthur M. Wellington in 1892 for evaluating resistance or capacity of timber piles. It is modified to predict nominal bearing resistance of a driven pile as shown in equation 12:

$$R_n = \frac{12E_b}{(s+0.1)} \tag{12}$$

Where:

R_n = Nominal pile resistance measured during driving (kips)

E_b = developed hammer energy[1]

s = pile permanent set (inch)

The Engineering News Formula in its normal form has a factor of safety of 6.0, but for LRFD applications to produce nominal resistance, the factor of safety has been removed. Driving formula should only be used to determine end-of-driving blow count criteria.

Washington State Department of Transportation Pile Driving Formula

The WSDOT pile driving formula was developed empirically to maintain the low prediction variability of the Gates Formula but at the same time to minimize its tendency to under- or over-predict the pile nominal resistance (Allen 2005).

$$R_n = 6.6 \times F_{eff} \times E \times L_n(10N) \tag{13}$$

Where:

R_n = nominal bearing resistance (kips)

F_{eff} = hammer efficiency factor

E = developed energy, equal to W times H (ft-kips)

W = weight of ram (kips)

H = vertical drop of hammer or stroke of ram (ft)

N = average penetration resistance in blows per inch for the last 4 inches of driving

L_n = the natural logarithm (base "e")

In the WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction* (WSDOT 2020), Section 6-05.3(12), equation 14 has been simplified to:

$$R_n = F \times E \times L_n (10N) \tag{14}$$

Where:

R_n = nominal bearing resistance (tons)

F = a constant that varies with hammer and pile type

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 soil's 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 is as follows (Morton 2012):

$$\frac{\partial^2 u}{\partial t^2} = \frac{E\,\partial^2 u}{\rho \partial z^2} \tag{15}$$

Where:

E = elastic modulus of the pile

ρ = 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 relationships help 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 the 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 to determine the nominal bearing resistance, the driving criterion (blow count) may be the value either at the end of driving (EOD) or at the beginning of redrive (BOR) (AASHTO 2020).

DESIGN METHODS FOR DRILLED SHAFTS ON ROCK

Drilled shaft is a broad term that describes a kind of deep foundation where a hole is drilled or excavated to the bottom of the foundation level and filled with concrete. It is often used when larger capacity is necessary, as it can be as large as it is excavated because of casting in place in the field. The concept of the design estimating the skin friction and toe bearing capacity is the same, but due to the different behavior because of the size and construction methods, the design methods are different compared to the driven piles. This study does not review the design methods associated with the socketing and load transfer mechanism and construction methods in general for drilled shaft. Instead, a general procedure for design of drilled shafts under axial loading is provided in LRFD format.

The design procedures are summarized as follows:

1. At each foundation position, a finite number of geomaterial (rock) layers can be evaluated by division of subsurface strata.
2. Review the strength and service limit states to be satisfied, and establish the load factors and corresponding axial load combinations.
3. For each limit state/load case and geomaterial layer, assign the suitable geomaterial properties required for evaluation of axial resistances.

4. Select diameters and trial lengths for initial analyses. The minimum trial shaft diameter may be governed by structural requirements, lateral load considerations, or scour requirements.
5. Determine values of nominal unit side friction for all geomaterial layers through which the trial shaft extends, and the nominal unit base resistance at the trial tip elevation. GDOT does not consider both side and base resistance in the drill shaft design. Only one—either side resistance or base resistance—is used.
6. Iterating from step 4, adjust the trial design as necessary to satisfy the LRFD requirement for each strength limit state.
7. For each trial design, execute load-deformation analysis and iterate from step 4 to satisfy the LRFD requirement as necessary for each service limit state. Service limit state evaluation for axial loading requires analysis of base resistances and side that are mobilized at axial displacement corresponding to the specified deformation estimated for the structure being designed.

Current Design Procedure Adopted by Georgia Department of Transportation.

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

1. **Organize Drilling**: When a Bridge Foundation Investigation (BFI) is assigned, a geotechnical drilling crew 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, the boring sample numbers are written on a form to submit for testing. If there is rock, the samples should be sent for, RQD determined, uniaxial compressive strength tested, and rock mass rating determined.
4. **Prepare Boring Logs**: Once samples are submitted for lab testing, borings are entered into the digital asset management system. The borings will be preliminary and not include laboratory test results. Once results are obtained, the soil classification is corrected based on the results.
5. **Determine Seismic 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 ft of soil samples.
6. **Prepare Bridge Foundation Recommendation**: It is critical in the LRFD 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.
    a. Geographical Location – North Georgia (above fall line) or South Georgia (below fall line).

b. Bent Location – Intermediate or End Bents.

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

d. Scour – Foundation type must provide adequate penetration below scour.

e. Span Length – Short Span (up to 55 ft for H-Piles and up to 80 ft for prestressed concrete [PSC] or metal shell [MS] piles) or Long Spans (greater than 55 ft for H-Piles and greater than 80 ft for PSC or MS piles).

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

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

h. Piling Characteristics – Normal/Uniform vs. Erratic piling.

i. 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.

j. Other Structures – Embedment below walls/wall abutments at end bents, etc.

k. Historic Information – The type of foundation previously used for other bridges in the county/vicinity.

l. 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 Select Pile Minimum Tips**: Steps 7, 8, and 9 cover analysis of bents on driven/drilled pile foundations; step 10 for analysis of drilled shaft foundations.
    a. 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:
        i. Set minimum tips in double digit blow count material, preferably 15 blow count soil or denser.
        ii. At end bents/abutments, set tips a minimum of 5 ft into natural ground and try to have minimum pile lengths of 10 to 15 ft.
        iii. At intermediate bents, set minimum tips 15 ft below theoretical scour.
8. **Analysis with APILE**: Once loads are received from the Office of Bridge and Structures, the APILE analysis is ready to be performed. This static analysis is used to determine the required pile depth based on the calculated driving resistance.
9. **Analysis – GRLWEAP**: This program uses the wave equation to model pile driving. It can be used for drivability and bearing analysis.
10. **Analysis – SHAFT**: For bridges that have a drilled shaft as a designed foundation type, analysis is typically completed using the SHAFT software.

CHAPTER 3. EFFECTS OF THE PILOT HOLE ON THE PILE CAPACITY AND BEHAVIOR, ALONG WITH THE ASSOCIATED DESIGN AND CONSTRUCTION CONSIDERATIONS

INTRODUCTION

Using a driving assistant method such as a pilot hole or predrilling is a common technique when driving a pile confronts high resistance that may damage the pile. However, when a driving assistant method is applied, it is generally expected that the resistance to the piles will decrease, and thus the capacity of the piles will be eventually reduced. Therefore, it is critical to understand how the pile with a pilot hole is considered in the design and construction states.

This task was completed by reviewing approaches currently followed by other state departments of transportation (DOTs). Documents such as state design/construction manuals and standard specifications on bridge foundations were reviewed from each state. In addition, a survey was also sent out to all 50 state DOT agencies asking specifically about their use of a pile with a pilot hole (see appendix B). The survey was a way to ascertain the current way states handle the case of a pile with a pilot hole, and the results are summarized in this chapter.

REVIEW OF OTHER STATES' DOCUMENTS

The research team collected over 100 published documents by the state departments of transportation that are available to the public online. The focus of the review was the use of any driving assistant method and its guidelines, specifically for the use of pilot hole, hole size, back fill, and casing, and any remarks regarding construction.

The literature review showed that only seven states did not disclose any information regarding the use of a pilot hole or similar pile driving assistant methods, as the research team could find some statements indicating the use of the pile driving assistant methods for each of the other states. However, this does not mean that those states do not use the pile driving assistant methods. It is possible that the research team could not review all their documents because of the team's inability to locate the references online as well as the limited accessibility of documents. Furthermore, even if information is found, it does not necessarily mean that the states with the information included in their documents are consistently using the construction method. In addition, the information was not specific enough to conclude the design and construction methods of the pile with a pilot hole in many cases. Therefore, direct contact with the state agencies was followed as in the research plan.

Nevertheless, it is noted that, in general, the main reasons for using a pilot hole are to reduce the vibration or to reach a certain tip elevation. Specifically in soil, it is often considered to reduce the negative skin friction on piles due to settlement of the ground around the pile, whereas in rock, it is to reach to the bearing layer without damaging the pile, especially when the rock layer is at a relatively shallow depth.

The list of the documents reviewed are provided in appendix A.

SURVEY TO STATE TRANSPORTATION AGENCIES

Since the publicly available documents may not provide the actual use of the driving assistant methods, the most up-to-date status of the guidelines, the internal guidelines, or regional/local adoption of the methods, the research team also contacted each state DOT by survey.

A five-question survey (see appendix B) was created to understand how the pilot hole or other driving assistant methods are being considered in the design and construction of piles (i.e., use of driving assistant methods, applying ground conditions, size of the hole, use of skin friction, and pile capacity estimation).

SUMMARY OF THE RESPONSES FROM STATES

The short survey was sent to all 50 state DOT agencies, and a summary of the responses is presented in this chapter. The answers to the questions gave a picture of how terminology, conditions of use, and hole size are viewed across all 50 DOTs.

Out of the 50 state DOTs, 48 replied to the survey. For those 2 states which did not respond, information on the first 3 survey questions was found in their standard specifications available online. Of the 48 that replied to the survey, 3 states were removed from the analysis because they responded that they only use drilled shafts, have removed the pilot hole from the recent specifications because of not using it, or just have not used it for a few decades although it is still in the specifications. That leaves 45 states with information from the survey, plus 2 states with partial information from the specifications. However, it is noted that 8 states among the survey-responded states also specifically indicated that they rarely use a pilot hole.

The following section summarizes the current practice by state agencies, their responses to the survey, and a summary of the survey responses.

Terminology

A variety of terms were used to refer to a pile with a driving assistant method. Some states were noted to use multiple terms, which indicated the necessity of careful review of their responses and guidelines. The terms that were the most used across the DOTs are "predrilling/predrilled hole," "preboring/prebored hole," "pilot hole," and "preauguring/preaugured hole." Other terms used by a few states were "precoring/precored hole," "augured hole," and "pile excavation." These varying terms all refer to the similar situation of a pile being installed to elevation with the assistance of a hole to guide it. Some states use more than one term to define the hole specifically for different construction methods for different grounds. The variety in terms used is shown in figure 5.

Hereafter, for consistency, in this report a hole drilled before driving that is used as a pile driving assistant method that does not require driving to penetrate is called a "pilot hole."

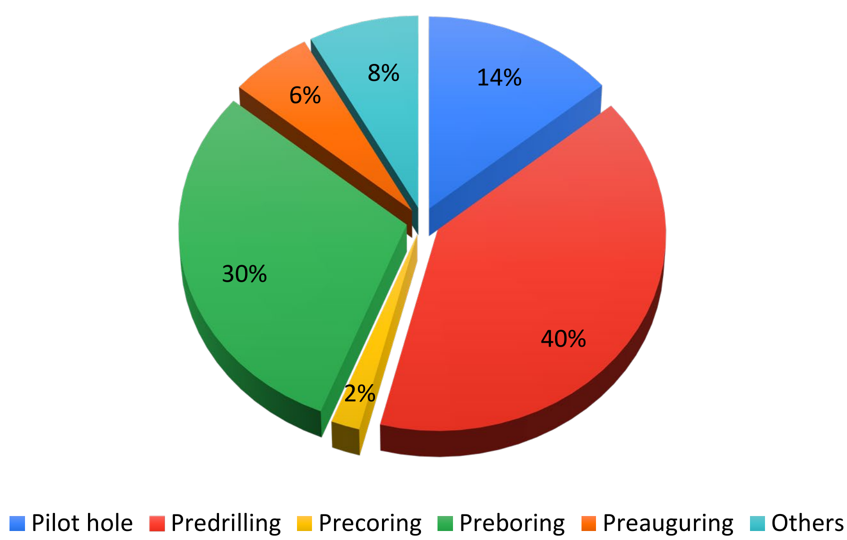

Figure 5. Chart. Terminology used for pilot holes.

Ground Conditions

The current study is specifically interested in the use of a pilot hole for a pile on rock layer. However, the preliminary literature review indicated that some documents stated that a pilot hole is being used in soils as well.

Based on the survey, 33 states indicated the use of a pile with a pilot hole in both soils and on rock layers, 7 states indicated that they have used a pile with pilot hole only in soils, and 5 states indicated that they have used a pile with pilot hole only in rock. Figure 6 displays the use of pilot holes in soils and rocks.

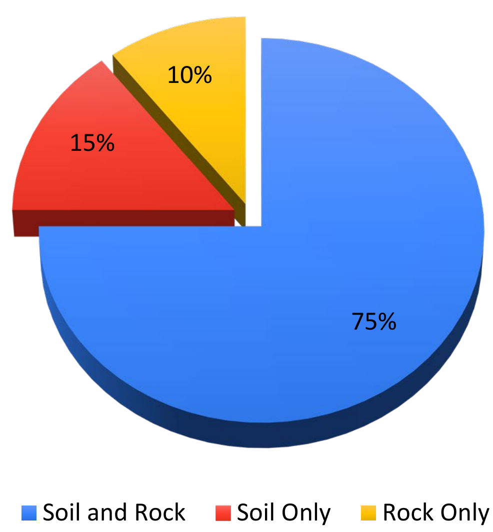

Figure 6. Chart. Ground conditions of pilot hole use.

Pilot Hole Size

The next survey question was regarding the size of the hole. If a pilot hole is created before driving a pile, it is expected to disturb the ground around the hole and may even require a casing to maintain the hole so that a pile can be installed with less or no driving.

However, the size of the hole can play an important role directly in terms of development of the skin resistance. It also can affect the hammer selection and other equipment for the pile installation, and thus, the status of the pilot hole size was collected.

The responses to the survey question on the size of the hole are presented in figure 7. A larger hole is used for more than half of the cases. However, this result includes multiple answers that a state may use a larger hole in rock and a smaller hole in soil. In addition, twenty states did not specifically indicate if it is for rock or soil.

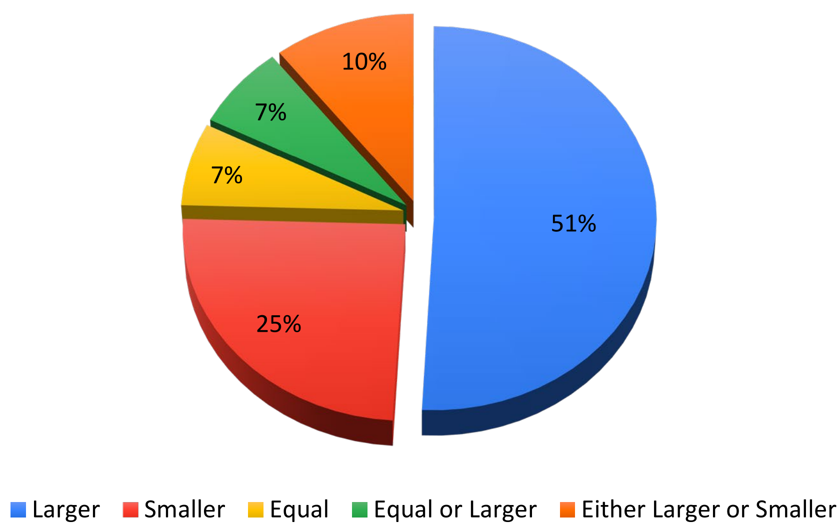

Figure 7. Chart. Size of pilot hole used.

Figure 8 shows responses from 24 states that provided the preferred hole size specifically in rock. Seven of them indicated they do not use a pilot hole in rock, and the rest of the states responded that they use either equal or larger size of the pilot holes. The next section addresses that the size of the hole also affects the consideration of the skin friction.

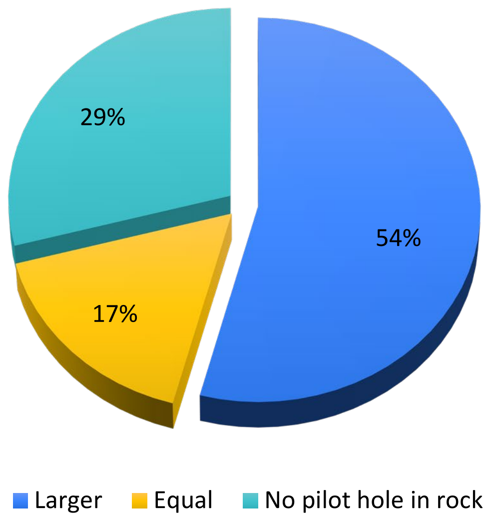

Figure 8. Size of pilot hole used in rock.

Consideration of Skin Friction

A deep foundation usually develops its resistance from tip and shaft. However, if a pilot hole is adopted, it is expected that the shaft resistance will be reduced. It is often ignored for a conservative design due to uncertainties involved during the construction of a pilot hole. Especially when a pile is sitting on a hard rock layer, it is reasonable to ignore the skin friction, as most of the resistance is expected to develop from the tip.

Therefore, the last survey question was to study how skin friction is considered in the design of a pile with a pilot hole.

Among 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 “Partial or Varies” answers include different skin friction considerations by hole sizes, ground types, and pilot hole sections. The distribution of use of skin friction is shown in figure 9.

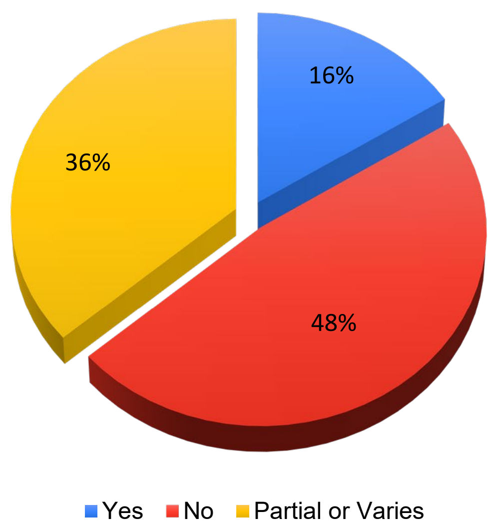

Figure 9. Chart. Use of skin friction for piles with a pilot hole.

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 10, 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, a larger hole size negates skin friction and a smaller hole size accounts for skin friction.

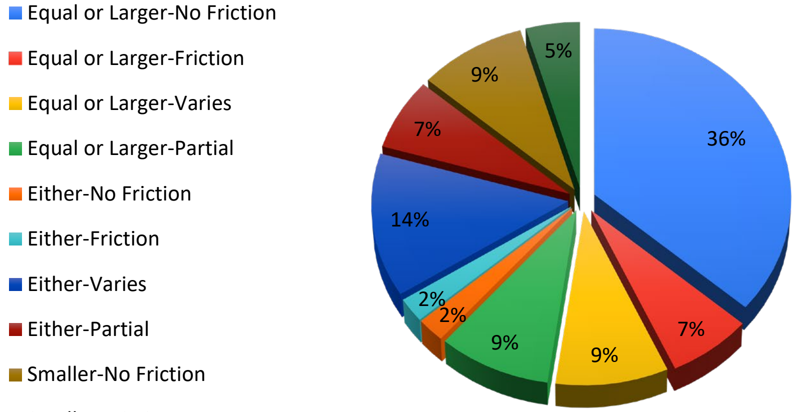

Figure 10. Chart. Hole size and skin friction of piles with a pilot hole.

Design of Pile with a Pilot Hole

The last question in the survey was to find out how the capacity of the pile is determined when a pilot hole is installed. The survey results originally included the responses for the piles in soil.. Therefore, 13 states were excluded due to their reasons such as not using the pilot hole, using it only in soils, or not providing the specific applied ground condition. Of the 37 eligible states using a pilot hole for the piles in rock, more than half of the states indicated that the pile is designed with a larger pilot hole and end bearing, whereas 6 states indicated that the structural capacity of the pile governs or the pile is design as column. They use the structural capacity of the pile as its capacity. It is noteworthy that three states design the pile as a small drilled shaft in a socket. The overall responses and their percentages with respect to the pilot hole size and design methods are presented in figure 11.

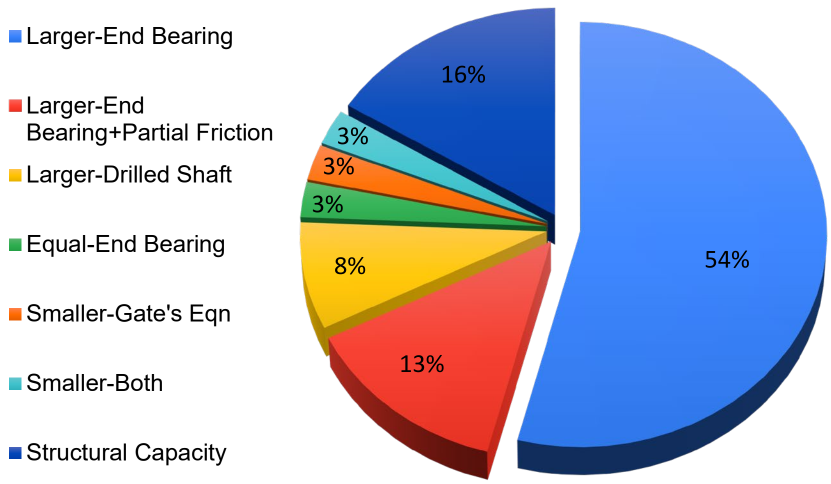

Figure 11. Chart. Hole size and capacity estimation in rock.

SUMMARY

From the initial survey, it was found that there are no common standards of the pile with a pilot hole, from terminology to construction method. Almost every state has different terminologies, requirements related to the diameter and filling of the pilot hole made to assist the pile driving, and the use of skin friction in the design. Depending on the ground conditions, some states recommended the use of a larger pilot hole and others recommended the use of a smaller hole. Furthermore, some states recommending a smaller pilot hole still do not consider the skin friction, and some other states consider the skin friction only in the socketed depth. Additionally, most states indicated they use pilot holes in both soil and rock, whereas eight states use it in rock only. From the seven states that responded as using the pilot hole in soil only, four states specifically indicated that their geological conditions rarely include bearing rock layer.

CHAPTER 4. CURRENT SPECIFICATIONS AND VERIFICATION METHODS FOR PILE INSTALLATION IN ROCK WITH A PILOT HOLE AND THE AVAILABLE EQUIPMENT/METHODS

INTRODUCTION

After analyzing the first survey received from 48 states and reviewing the published documents for 50 states that are available online from, it was noticed that the use of a driving assistant method such as a pilot hole is not uncommon regardless of the ground conditions. However, there were still some ambiguous responses or not enough explanations on how to consider the pilot hole in the design and construction. Therefore, this task was completed by sending out the 2nd survey to the selected states based on their first survey responses. In addition, reviewing published documents, such as specifications and manuals, from departments of transportation from other states.

REVIEW OF CURRENT DESIGN AND CONSTRUCTION CONSIDERATIONS

A second survey was sent out to a selected group of states for more detailed information regarding the case of a pile with a pilot hole in 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 in comparison to Georgia. The questions included in the survey are provided in appendix B. This second survey was sent to 20 DOTs. From the contacted states, 14 states replied to the second survey, and 2 of those states' responses were dropped as they seemed to have provided the information for pile in soil.

Summary of Survey

A summary of the state responses to the second survey is presented in table 5.

Table 5. Summary of second survey responses.

State	Capacity Verification, QA/QC Method	Design Capacity / Resistance Factors	Pilot Hole Backfill
AL	• Field verification	• Static analysis programs • AASHTO LRFD	• Top of the rock with concrete • Entire hole with either sand or concrete
AK	• Strong rock: shaft with no verification • Weak rock: driven with PDA/ CAPWAP or presumptive wave equation w/o signal matching • No test for low expected driving stress	• Expected driving resistance charts based on previous PDA data/ experience • AASHTO LRFD	• Socket (when too shallow): with grout to top of the socket • Entire hole with sand
CO	• PDA/CAPWAP	• Program that treats a pile like a column in the predrilled hole • AASHTO LRFD	• Entire hole with the most cost-effective material at the site
FL	• PDA or embedded data collectors	• Program that uses Davisson capacity • AASHTO code and local calibrated factors (0.65–0.85 per test quantity and method)	• Entire hole with the most cost-effective material at the site
IA	• WEAP • Require hitting for seating the pile	• State modified Engineering News Record (ENR) formula • 0.7 for rock end bearing	• Concrete with a minimum of 3 ft into sound rock • Sand or bentonite above it
KY	• Practical refusal: pile driver with a certain number of blows and movement that depend on the strength of rock	• Governed by structural design (full yield strength of pile) • (AASHTO) LRFD code	• Entire hole with sand, gravel (when axial strength only), or concrete (when lateral strength required)

State	Capacity Verification, QA/QC Method	Design Capacity / Resistance Factors	Pilot Hole Backfill
NC	• Crystalline rock (N ≥ 60 blows in 0.10 ft): No verification • Weather rock: WEAP or PDA/CAPWAP	• SPT borings and APile/DRIVEN • H-piles in coastal plain ➔ static analysis: 0.7 • Other pile types ➔ static analysis: AASHTO LRFD • WEAP or 1 PDA: 0.60 • 2 or more PDAs: 0.75	• Entire hole with concrete, grout, flowable fill
OH	• CAPWAP for friction pile • Practical refusal for point bearing driven on rock: 20 bl/inch • Pilot on rock: visual observation	• Governed by structural design with $\varphi_c = 0.95$	• 4000 psi concrete until the top of rock; granular fill above it • Filled to bottom of pile cap elevation
OK	• Pile driver for Gates equation or practical refusal	• Governed by structural design and factors from AASHTO LRFD	• Entire hole with sand
PA	• WEAP	• Driven to refusal (20 bl/inch or ¼ inch or less for 5 consecutive blows) • AASHTO LRFD	• Granular material but concrete/grout can be used • Backfilled to tip elevation
SC	• Strong rock: shaft with no verification • Soft rock: WEAP/PDA	• Strong rock: shaft (0.5–0.6) • Soft rock: piles driven to weak rock (0.55–0.70)	• Concrete for shafts • Filled to minimum length
WY	• Driving refusal: 10 bl/inch • WEAP and hammer stroke	• Structural strength / AASHTO	• Entire hole with pea sand or gravel

Regarding the verification method, all responding states except one use dynamic methods (e.g., WEAP, PDA, CAPWAP, or a combination of these) to verify the capacity. However, some of these states do not verify it when it is on hard rock. Moreover, one state did not provide specific methods.

Seven out of 12 states indicated the use of the resistance factors in the AASHTO *LRFD Bridge Manual Specifications* (AASHTO 2020). The other 5 states used local values, and one of them was using the modified resistance factor for the structural capacity. All 4 states using the modified version use resistance factors larger than the one suggested by AASHTO, varying from 0.50 to 0.85 depending on the rock types, regions, or number and type of tests performed. Two states specifically indicated that they design the pile on hard rock as a drilled shaft and do not verify the capacity with testing based on their previous experience.

A summary of each state's response to the second survey is provided in appendix C.

Implementation Status in Other Countries

In South Korea, a new regulation for the vibration and noise during construction was enforced in 1994. Therefore, a new pile type, "prebored and precast pile (PPP)," has been widely used instead of typical driven piles, especially in urban areas (see figure 12). Even though installing the pile with no or minimum driving with a prebored hole can be viewed similar to the piling with a pilot hole in Georgia, there are two major differences between the two methods, as listed below.

1. Hole filling: The prebored hole can be filled with cement paste before or after the pile is inserted before the paste sets.

2. Shaft resistance: The shaft resistance is considered in the design.

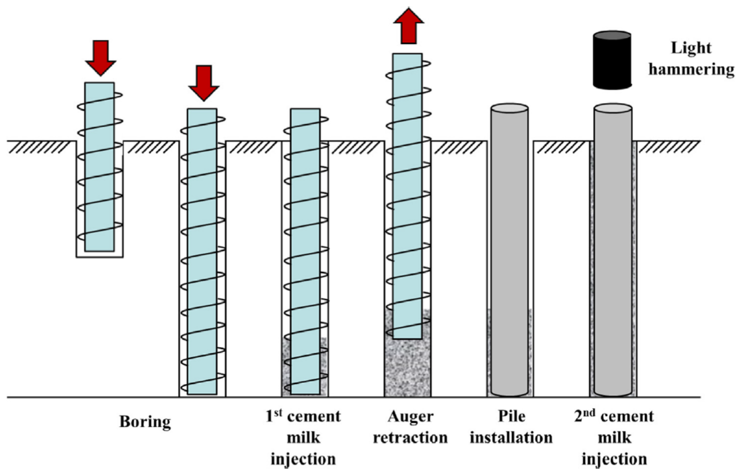

Figure 12. Drawing. Prebored and precast pile installation process (Kim et al. 2020).

Depending on the ground condition, a different variant can be applied to install a PPP, as shown in table 6. A summary of the construction methods for the PPP in South Korea is provided in table 6.

Table 6. Summary of prebored pile installation methods in South Korea (Jeong et al. 2017).

Term	Types	Use of Filler
Prebored and precast pile (PPP)	Preboring and driving	After auguring, a pile is lowered, and (soil) cement paste is filled. Then, the pile is driven into rock layer as a socket depth, which is 3 times the pile diameter.
	Preboring and light striking	After auguring, (soil) cement paste is filled at the bottom. A pile is lowered, and the hole is filled again with the cement paste.
	Preboring and cement paste	After preboring, cement paste is filled and the pile is inserted. Driving/striking is not necessary.
	Inner boring and light striking	After preboring, cement paste is filled and the pile is inserted. Driving/striking is not necessary.

Jeong et al. (2017) conducted a study to revise the design method for the PPP. The empirical equation based on the allowable stress design needed to be updated with the LRFD with new resistance factors for the design. They first conducted a series of numerical analyses and then reduced-scale field tests with 15 test metal shell piles (D = 2.64 inch and L = 44.5–47 inch) by varying the prebored hole sizes (D = 2.91–6.90 inch) and water-cement ratio (w/c = 60,70, and 90%). In order to develop the local LRFD resistance factor for the piles, they conducted one static load test and two PDA tests (at EOD and restriking) for each test pile for a total of 20 metal shell piles at 4 different construction sites to determine the resistance factors. As a conclusion, they proposed the shaft resistance and toe bearing equations using the Standard Penetration Test (SPT) blow counts, and they proposed the resistance factors for them. The calibrated resistance factors for toe bearing capacity on rock were suggested to be 0.63–0.65 for the strength limit and 0.41–0.42 for the service limit.

SUMMARY

Among 12 states in the U.S. identified as using a pile with a pilot hole on rock, the resistance factors for the design seem to be mostly 0.45 or 0.50, as a pile or drilled shaft, respectively, from the AASHTO *LRFD Bridge Design Specifications* (AASHTO 2020). Furthermore, it is also commonly assumed that the structural capacity governs the capacity, which might be the case for most piles on rock unless it is driven through the rock. The local resistance factors are larger than the AASHTO values varying from 0.50 to 0.85. From a study conducted in Korea, they suggested higher resistance factors (0.63-0.65) than the AASHTO LRFD Specifications.

CHAPTER 5. FIELD DATA ON DRIVEN AND SEATED PILES TESTED IN GEORGIA AND INVESTIGATE FEASIBILITY OF THE PDA TEST FOR VERIFYING CAPACITY OF A PILE IN ROCK.

INTRODUCTION

The research team coordinated with the GDOT Geotechnical Branch and collected the Bridge Foundation Investigation (BFI) Reports for projects conducted in Georgia. A total of 10 projects with different project identification numbers (PIs) at different counties were selected, and their BFI reports were reviewed to identify any piles with a pilot hole on rock. In addition, in order to review the feasibility of Pile Driving Analyzer (PDA) tests on hard rock, the PDA results and rock properties at those project sites were also reviewed. As the pile with a pilot hole on rock is not the type of foundations that are frequently considered, there were not many cases available meeting the needs of the research. In addition, due to COVID-19 and the pursuant remote working environment, the activities for this task were limited as well.

REVIEWED PROJECTS IN GEORGIA

Ten projects found from the GDOT Geotechnical Branch repository were reviewed. The original plan was to specifically find projects that include construction projects employing piles with a pilot hole in rock. It was also planned to select projects at different regions in Georgia. However, it was soon apparent that projects meeting all of the specific preferences are very rare; thus, the selection of the project site locations was somewhat random as well. The locations of the reviewed projects are provided in figure 13.

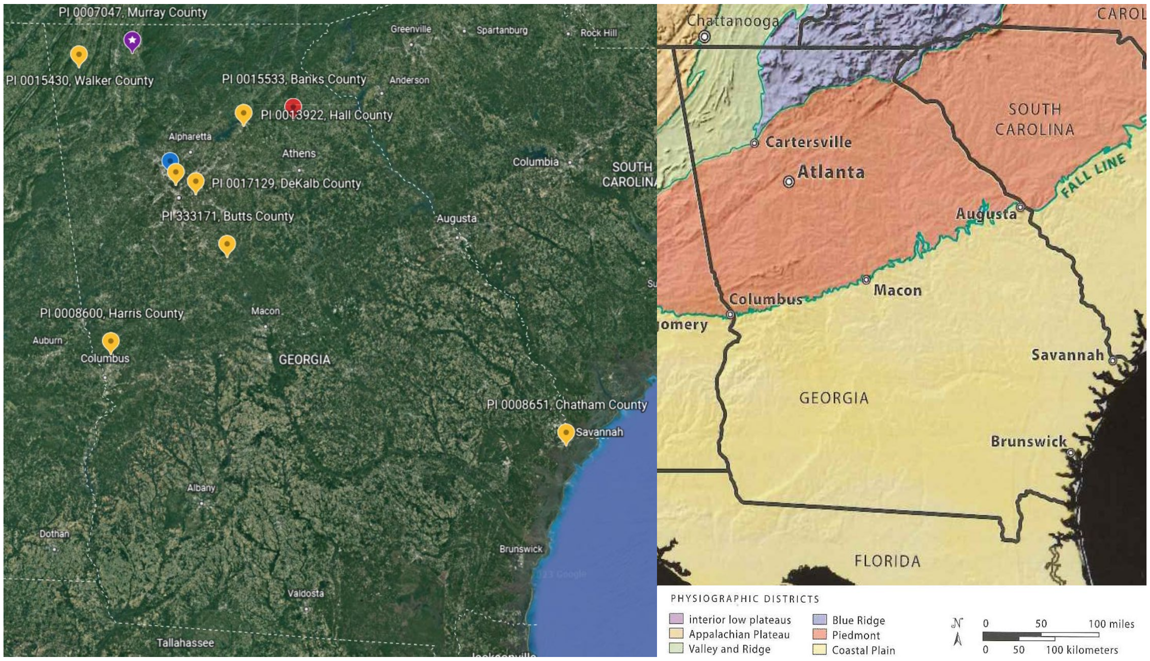

Figure 13. Maps. Locations of the reviewed projects and geological map of Georgia (Gore and Witherspoon 2013).

As shown in figure 13, only one site is in Southeast Georgia and the other 9 sites are located in the northwestern part of the state, which is somewhat expected due to the geological condition of the state. Besides, two projects (No. 9 and No. 10 in table 7) did not meet the requirements, as the projects had different types of foundations. Unfortunately, the project in Southeast Georgia is one of those two, and thus, all the reviewed projects were located in Northwestern Georgia. This was expected considering the locations and preferred methods based on the geological conditions of the state. The foundation types and some of the pile and design properties are provided in table 7. It is noted that different resistance factors have been applied depending on pile types and applied verification method per AASHTO Specifications (2020). The rock properties obtained from the projects are summarized in table table 8.

Table 7. Summary of design information for selected projects in Georgia.

No.	PI No. & Year	Location (County & District)	Foundation Type & Size	Max. Factored Str. Resist (kips)	Driving/ Geotechnical Resistance (kips)	Design Load (Strength) (kips)	Pilot Hole	Resistance Factor	Verification
1	0010009, 2015	Cobb County, District 7	HP 12×53	384	71	45.6	Yes	0.65	PDA*
			HP 14×73	520	151	97.8	Yes	0.65	PDA
2	0008600, 2016	Harris County, District 3	HP 12×53	384	273–456	205–334	Yes	0.75[a]	SLT**
			HP 14×89	653	456	334	Yes	0.75[a]	SLT
3	0007047, 2017	Murray County, District 6	HP 14×117 (36 ksi)	619	240–367	105–175	Yes	0.75[a]	SLT
4	0015430, 2017–2018	Walker County[b], District 6	HP 14×89 (50 ksi)	653	391	159	No	0.65[c]	PDA
			HP 14×117 (50 ksi)	860	564	159	Yes	0.45[d]	—
5	333171, 2018	Butts County, District 3	HP 14×89 (50 ksi)	653	559	381	Yes	0.75[a]	SLT
			HP 14×89 (50 ksi)	653	559	381	Yes	0.75[a]	SLT
6	0013922, 2021	Hall County, District 1	HP 14×89 (45 ksi)	653	585	380	Yes	0.65[c]	PDA[e]
			Drilled shaft	N/A	3451	2480	N/A	0.50[f]	—
7	0017129, 2021	DeKalb County, District 7	HP 14×102	750	657	427	Yes	0.65[c]	PDA
			HP 14×89	653	652	407	Yes	0.65[c]	PDA
			HP 14×102	750	702	456	No	—	PDA
8	001533, 2022	Banks County, District 1	HP 14×73 (45 ksi)	520	491–482	210–220	Yes	0.65[c]	PDA SLT

No.	PI No. & Year	Location (County & District)	Foundation Type & Size	Max. Factored Str. Resist (kips)	Driving/ Geotechnical Resistance (kips)	Design Load (Strength) (kips)	Pilot Hole	Resistance Factor	Verification
9	0008651, 2010	Chatham County, District 5	PSC 24″ square	—	—	310–370	Pre-drilling	—	Statnamic &PDA
			PSC 36″ Square	—	—	540		—	
10	0007174, 2018	Fulton County, District 7	Spread footing	277	225	232	—	0.45	—
			Micropiles	—	—	190–232	—	0.55	—

*PDA = Pile Dynamic Analyzer; **SLT = Static Load Test
[a] Driving criteria established by successful static load test of at least one pile per site condition without dynamic testing.
[b] Originally design with a smaller pile (HP 14×89) but revised with a larger pile with a pilot hole.
[c] Driving criteria established by dynamic testing of at least two piles per site condition, but no less than 2% of the production piles.
[d] End bearing in rock (Canadian Geotechnical Society 2006).
[e] The nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. The Nominal Driving Resistance should not exceed the Factored Structural Resistance. Dynamic pile measurements should be used to monitor pile damage.
[f] Tip resistance in rock by Canadian Geotechnical Society (2006).

Table 8. Summary of rock properties for selected projects in Georgia.

No	PI No. & Year	Location (County & District)	Bent No. & Boring Log No.	Depth (ft)	Elev. (ft)	Description	RQD (%)	Rec. (%)	Uniaxial Comp. Strength (ksf)
1	0010009, 2015	Cobb County, District 7	—	—			—	—	—
2	0008600, 2016	Harris County, District 3	1/B-01A	9–13.5	473–469.5	Gneiss	52	90	—
				13.5–18	469.5–464.5	Weathered Gneiss	8	48	—
				18–23	464.5–459.5	Gneiss	53	78	3315@22.6ft
			2/B-02	15–20	447.7–442.7	Gneiss	99	99	3524@19.3ft
				20–25	442.7–397.7	Gneiss	88	97	—
			3/B-03	11–17.5	451.6–446.6	Gneiss	27	32	—
				17.5–23	444.7–439.7	Gneiss	48	96	—
				23–26	439.7–434.7	Gneiss	32	88	—
3	0007047, 2017	Murray County, District 6	2/B2	17–27	665–655	Rock	95	79	—
			3/B3	22–32	659–649	Rock	97	93	—
			4/B4	14–18	667.5–663.5	Rock	86	93	—
				18–28	663.5–353.5	Rock	64	67	—
4	0015430, 2017–2018	Walker County, District 6	1/B1	25–35	746.7–736.7	Rock	83	93	4135@25.5ft 2574@34.8ft
			2/B2	39–50	731.5–721.5	Rock	86	100	2747@39.2ft 2389@45.7ft
5	333171, 2018	Butts County, District 3	1/B1	13.5–23.5	591–604	Rock	100	100	2286@13.8ft 1769@14.5ft
			2/B2	11–21	593–603	Rock	95	95	2342@12.1ft 2976@15.1ft
6	0013922, 2021	Hall County, District 1	1/B1	30–40	1162–1152	Granitic Gneiss	53	75	—
				40–45	1152–1147	Granitic Gneiss	60	100	—
			2/B2	30–40	1132–1122	Biotitic Gneiss	88	97	1019@36.5ft

No	PI No. & Year	Location (County & District)	Bent No. & Boring Log No.	Depth (ft)	Elev. (ft)	Description	RQD (%)	Rec. (%)	Uniaxial Comp. Strength (ksf)
			2/B2A	26–36	1136–1126	Biotitic Gneiss	20	67	505@44.5ft
				36–46	1126–1116	Biotitic Gneiss	76	87	791@45.5ft
			3/B3	67–77	1139–1129	Biotitic Gneiss	43	63	—
				77–87	1129–1119	Biotitic Gneiss	79	93	—
7	0017129, 2021	DeKalb County, District 7	1/B1-B	29.8–34.8	945.6–940.6	Mica Schist	45	98	—
				34.8–39.8	940.6–935.6	Mica Schist	85	100	1886@37.3ft
				39.8–44.8	935.6–930.6	Mica Schist	68	100	—
				44.8–49.8	930.6–925.6	Mica Schist	75	100	—
			2/B2-A	27.4–29.9	927.5–925.2	Mica Schist	39	80	—
				29.9–34.9	925.2–920.2	Mica Schist	60	85	687@30.9ft
				34.9–39.9	920.2–915.2	Mica Schist	82	98	—
				39.9–44.9	915.2–910.2	Mica Schist	88	100	—
			2/B2-B	33.6–35.3	922.7–921	Mica Schist	45	90	—
				35.3–40.3	915–921	Mica Schist	90	100	1768@39.4ft
				40.3–45.3	916–911	Mica Schist	92	96	—
				45.3–50.3	911–906	Mica Schist	82	98	—
			3/B3-B	48.7–50.1	927.3–925.9	Mica Schist	62	100	—
				50.1–55.1	925.9–920.9	Mica Schist	90	96	1467@53.8ft
				55.1–60.1	920.9–915.9	Mica Schist	90	100	—
				60.1–65.1	915.9–910.9	Mica Schist	97	100	—
				65.1–70.1	910.9–905.9	Mica Schist	84	100	—
8	001533, 2022	Banks County, District 1	1/B-01	49.5	625–622.5	Gneiss	100	100	—
				49.5–55	622.5–617	Gneiss	95	100	—
				55–57	617–615	Gneiss	100	100	—

No	PI No. & Year	Location (County & District)	Bent No. & Boring Log No.	Depth (ft)	Elev. (ft)	Description	RQD (%)	Rec. (%)	Uniaxial Comp. Strength (ksf)
			2/B-02	19.5–22.5	636.5–633.5	Gneiss	100	100	—
				22.5–29.5	633.5–626.5	Gneiss	55	87	—
				29.5–32.5	626.5–623.5	Gneiss	60	86	—
				32.5–39.5	623.5–616.5	Gneiss	80	100	1021@34ft 1234@37ft
			3/B-03	19.5–22.5	638.5–635.5	Gneiss	100	100	—
				22.5–25	635.5–633	Gneiss	98	100	—
				25–28.5	633–629.5	Gneiss	90	100	—
			4/B-04	25–30	646.2–641.2	Gneiss	100	100	—
				30–35	641.2–636.2	Gneiss	100	100	—

PILE DRIVING ANALYZER TEST

The primary purpose of the Pile Driving Analyzer is to monitor the stress development in the pile (structural integrity) and the ultimate resistance of the pile (pile capacity). The test typically uses a hammer to provide a heavy impact on the pile from a pre-determined height. In addition, the sensors (strain transducers and accelerometers) attached to the side of the pile above the ground measure the generated strains and accelerations. The PDA converts strain to force, while acceleration records are converted to velocities. Using a program based on closed form Case-Goble solutions, this helps to estimate static pile capacity from pile top force and velocity data (Mhaiskar et al. 2010). This pile response is typically subsequently checked with the more rigorous signal matching technique computer program CAPWAP (Case Pile Wave Analysis Program) to confirm the static pile capacity obtained at the site.

The PDA has gained popularity in recent years due to its simplicity and economic benefits. It does not require reaction pile or dead weights to perform the test. Additionally, the number of tests can be increased considerably because it requires less time, space, and cost compared to the static load test. However, it still "estimates" the capacity, unlike the static load test that "measures" the capacity. It is possible that the PDA results may not fully represent the actual capacity of the pile. According to the resistance factors recommended in the *LRFD Bridge Design Specifications* (AASHTO 2020), different resistance factors are recommended for PDA only, and PDA and SLT. Moreover, these should have been affected by the reliability of each testing method.

The other practical reason that the PDA has not been used much on piles on rock is because pile testing and resistance factor calibration have not been available, mostly due to the cost and time required. Piles on rock usually have much higher geotechnical capacity and, thus, the structural capacity dominates the design capacity. In addition, displacement is necessary in the wave theory in pile dynamic analysis. However, when a pile is sitting on hard rock layer, the displacement due to striking is very small; thus, it has been believed that the PDA may not be applicable unless it is driven through soft rock / Intermediate Geomaterial (IGM).

Banks County, GA

A bridge replacement construction site in Banks County was selected for this study, considering the pile type and construction method as well as the construction schedule. Figure 14 shows the aerial view of the site in Banks County on GA State Route 59 over the Hudson River, and figure 15 shows the preliminary layout of the project. Three PDA tests were conducted at Bent 1, Bent 3, and Bent 4. All piles for the PDA tests were designed to be constructed with a pilot hole. However, the pile in Bent 1 needed to be spliced and driven deeper as it did not hit the bearing layer as expected. In addition, one static load test was conducted at Bent 3 where the first PDA test was conducted.

Figure 14. Photo. Overview of the construction site in Banks County, GA.

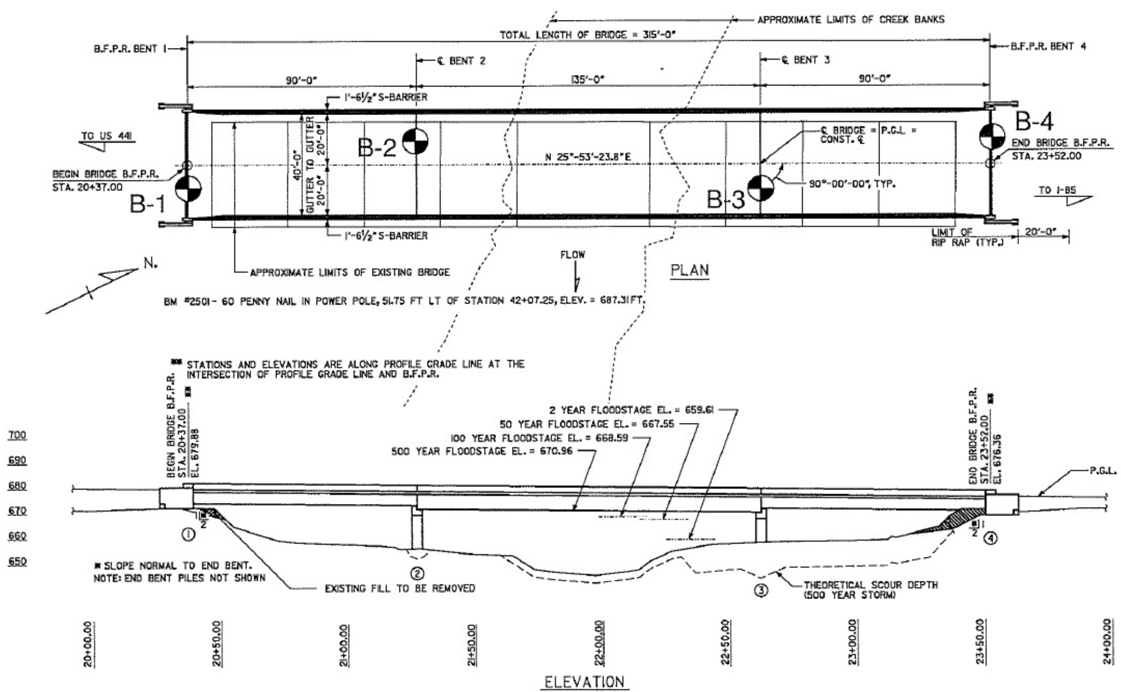

Figure 15. Drawing. Preliminary layout of the project site (GDOT Office of Materials and Testing 2021).

According to the BFI report, the geologic formation of the soil is Hornblende Gneiss/Amphibolite formation of the Georgia Piedmont Region, and hard rock was encountered at the elevations of 646 to 626 ft.

The parameters determined during the design process are provided in table 9. An LRFD resistance factor of 0.65 from the AASHTO LRFD Specifications was considered in the design. However, in order to prevent damage or potential damage to the pile, the stress level of the pile during driving was monitored not to exceed 90 percent of the yield strength of steel.

Table 9. Summary of pile design in Banks County.

Pile Type	**Pile Size (inch)**	**Nominal Compression and Tension Stress (ksi)**	**Max Factored Structural Resistance (kips)**
HP	14 × 73	45	520
Bents	**Maximum Factored Strength Limit State Load (kips)**	**Maximum Factored Service State Load (kips)**	**Factored Extreme Event I Limit State Load (kips)**
1 and 4	319	221	210
2 and 3	313	290	223

Table 10 shows the uniaxial compressive strength of the rock specimens collected at Bents 2 and 3. These properties are used to estimate the capacity of the pile.

Table 10. Uniaxial compressive strength of the rock core samples.

Core #	Bent	Sample	Distance (ft)	Diameter (inch)	Peak Load (lb)	Peak Stress (psi)
1	2	1	33–35	1.98	21,845	7,090
2	2	2	37	1.98	26,378	8,570
3	3	1	26	1.98	26,458	8,590
4	3	2	28	1.98	44,578	14,480

In addition, a static load test was conducted in the field at Bent 3. Unfortunately, the static load test was halted at 414 kips due to failure in one of the reaction beams. However, it was confirmed that 80 percent of the maximum test load was achieved, and thus, the test was not executed again.

PDA Results

The PDA tests were conducted as planned except the pile in Bent 1. The piles in Bents 2 and 3 were verified that the estimated pile capacity by the PDA with CAPWAP exceeded both the required driving resistance and maximum factored structural capacity, but the maximum compressive stress was still below the limit as shown in table 11.

Table 11. Summary of the PDA test results.

Pile HP 14 × 73	Hammer Pileco Dl9-42	Unit	Bent 1 Pile 5	Bent 3 Pile 5	Bent 4 Pile 1
Minimum tip elevation		ft	644	633	648
Estimated tip elevation		ft	623.88	633.09	649.75
Pile set at EOID[a]			½ inch in 5 blows	¼ inch in 2 blows	⅜ inch in 5 blows
Stroke height at EOID		ft	8.6+	11.6+	9 ft
Maximum Factored Structural Capacity		kips	520	520	520
Required Driving Resistance		kips	491	482	491
PDA with CAPWAP Capacity (Max.)		kips	598	816	696
Maximum Compression Stress		ksi	42.4	44.6	45.8

[a] EOID: End of Initial Driving

The estimated capacities by the PDA with CAPWAP and empirical methods based on the uniaxial compressive strength are provided for Bent 3, where most of the necessary information for the calculation was available.

From the results in table 11 and table 12, it can be concluded that the PDA tests provide useful information for the piles in rock.

Table 12. Comparisons of the estimated bearing capacity.

	Rock Core Peak Stress (psi)	Avg. q_u (psi)	FHWA Equation (kips)	CFEM[a] w/ K_{sp}=0.10[b] (kips)	Goodman (kips)	PDA w/ CAPWAP (kips)
Bent 3	8,590	11,535	1,470	1,375	1,630	822
	14,480					

Used gross area of the pile end (198.6 in^2).
[a] CFEM = *Canadian Foundation Engineering Manual* (Canadian Geotechnical Society 2006).
[b] K_{sp} was assumed 0.10.

CHAPTER 6. VERIFICATION METHODS FOR QUALITY ASSURANCE OF ROCK

INTRODUCTION

Deep foundations transmit the superstructure load to the underlying bedrock or a stronger soil layer. Specifically in this study, the main research interest is how the bearing capacity of a pile with a pilot hole on a rock layer can be estimated. In cases of the pile with a pilot hole, the nominal capacity typically depends on the end bearing capacity of the pile, which depends on the underlying material. Thus, properly classifying and verifying the strength of rock material is of great importance. Additionally, methods have been developed to classify rock masses for their strength over the decades.

ROCK PARAMETERS

For piles driven into soft rock or driven into hard rock, design parameters must be determined. Rock cores are often collected during the site investigation. Weathering, fracturing, strength, and other physical parameters can be gathered from these rock cores, and the rock can be classified. In many rock classification systems, the transition between hard soils and soft rock is considered at an unconfined compression strength (q_u) around 20 ksf. On the other hand, the transition between soft and hard rock usually occurs between unconfined compressive strength of 200 and 1,000 ksf (Hannigan et al. 2016a).

Typical values of the uniaxial compressive strength of different rocks are provided in table 13.

Table 13. Typical range of uniaxial compressive strength of rocks (Pennsylvania DOT 2019).

General Description	Rock Type	q_u (ksf)
Carbonate rocks with well-developed crystal cleavage	Dolostone	600–6400
	Limestone	600–6000
	Carbonatite	800–1400
	Marble	800–5000
	Tactite-Skarn	2800–6800
Lithified argillaceous rock	Argillite	600–3000
	Claystone	30–170
	Marlstone	1000–4000
	Phyllite	600–5000
	Siltstone	200–2400
	Shale	150–740
	Slate	3000–4400
Arenaceous rocks with strong crystals and poor cleavage	Conglomerate	600–4600
	Sandstone	1400–3600
	Quartzite	1200–8000
Fine-grained igneous crystalline rock	Andesite	2000–3800
	Diabase	450–12000
Coarse-grained igneous and metamorphic crystalline rock	Amphibolite	2400–5800
	Gabbro	2600–6400
	Gneiss	500–6400
	Granite	300–6800
	Quartzdiorite	200–2000
	Quartzmonzonite	2800–3400
	Schist	200–3000
	Syenite	3800–9000

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 utilize the rock unconfined compressive strength. For both hard rock and soft rock, the FHWA recommends rock classification, core recovery, RQD, unconfined compression strength, and density parameter be quantified for pile designs (Hannigan et al. 2016a).

A summary of the different methods that are commonly used to quantitatively analyze and classify the rock are introduced in this chapter.

Rock Quality Designation

In 1967, Deere and his colleagues first published the rock quality designation concept of rock quality logging with some correlations with velocity indices, fracture frequency, and in situ modulus values (Deere and Deere 1988). The method is related to the modified percent core-recovery that uses sound pieces of the core that are 4 inches (100 mm) or greater in length on the core axis.

$$RQD = \frac{\text{Sum of Core Pieces} \geq 10\text{cm}}{\text{Total Drill Run}} \times 100\% \tag{16}$$

The International Society for Rock Mechanics (ISRM) recommends a core size of at least 2.15 inches (54.7 mm) to be drilled with a double-tube core barrel using a diamond bit for RQD determination. The rock core is classified by rock type and core recovery length and given a rock quality designation (Hannigan et al. 2016a). Indirect methods such as seismic survey or volumetric joint count can be used to estimate RQD (Singh and Goel 1999). In addition, the relationship between RQD and the engineering quality of the rock mass is given in table 14.

Table 14. RQD and rock mass quality(Hannigan et al. 2016a).

RQD (%)	Rock Mass Description
<25	Very Poor
25–50	Poor
50–75	Fair
75–90	Good
90–100	Excellent

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 consider joint orientation, joint condition, and stress condition. Nevertheless, RQD values have been used widely and can be indicative of the pile penetration that would be needed to satisfy resistance requirements when they are combined with additional test results (Hannigan et al. 2016a).

Rock Mass Rating

The rock mass rating (RMR) system was developed at the South African Council of Scientific and Industrial Research by Bieniawski. Since its development, the system has been modified several times. Throughout the years, each change altered how RMR was calculated, so it is important to make note of which version is used for official purposes.

RMR can be determined by five parameters: uniaxial compressive strength of intact rock material, rock quality designation, joint or discontinuity spacing, condition of discontinuities, and ground water condition.

The sum of the rated values of the five ratings parameters will give the basic RMR between 0 and 100, and additional rating adjustments for discontinuity orientations are available for different applications.

Table 15 was introduced by Hoek based upon the 1989 version of the RMR classification by Bieniawski (Hoek 2007, Bieniawski 1989).

Table 15. Rock mass rating system (Bieniawski 1989).

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	Parameter		Range of values				
1	Strength of intact rock material	Point-load strength index	>10 MPa	4 – 10 MPa	2 – 4 MPa	1 – 2 MPa	For this low range – uniaxial compressive test is preferred
		Uniaxial comp. strength	>250 MPa	100 – 250 MPa	50 – 100 MPa	25 – 50 MPa	5 – 25 MPa / 1 – 5 MPa / < 1 MPa
	Rating		15	12	7	4	2 / 1 / 0
2	Drill core Quality RQD		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%
	Rating		20	17	13	8	3
3	Spacing of discontinuity		> 2 m	0.6 – 2 m	200 – 600 mm	60 – 200 mm	< 60 mm
	Rating		20	15	10	8	5
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous
	Rating		30	25	20	10	0
5	Ground water	Inflow per 10 m tunnel length (l/m)	None	< 10	10 – 25	25 – 125	> 125
		(Joint water press)/ (Major principal σ)	0	< 0.1	0.1, - 0.2	0.2 – 0.5	> 0.5
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing
	Rating		15	10	7	4	0

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)

Strike and dip orientations		Very favorable	Favorable	Fair	Unfavorable	Very Unfavorable
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES

Class number	I	II	III	IV	V
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
Cohesion of rock mass (kPa)	> 400	300 – 400	200 – 300	100 – 200	< 100
Friction angle of rock mass (deg)	> 45	35 – 45	25 – 35	15 – 25	< 15

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions

Discontinuity length (persistence) Rating	< 1 m 6	1 – 3 m 4	3 – 10 m 2	10 – 20 m 1	> 20 m 0
Separation (aperture) Rating	None 6	< 0.1 mm 5	0.1 – 1.0 mm 4	1 – 5 mm 1	> 5 mm 0
Roughness Rating	Very rough 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0
Infilling (gouge) Rating	None 6	Hard filling < 5 mm 4	Hard filling > 5 mm 2	Soft filling < 5 mm 2	Soft filling > 5 mm 0
Weathering Ratings	Unweathered 6	Slightly weathered 5	Moderately weathered 3	Highly weathered 1	Decomposed 0

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**

Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip – Dip 45° – 90°	Drive with dip – Dip 20° – 45°	Dip 45° – 90°	Dip 20° – 45°
Very favorable	Favorable	Very unfavorable	Fair
Drive against dip – Dip 45° – 90°	Drive against dip – Dip 20° – 45°	Dip 0-20 – Irrespective of strike°	
Fair	Unfavorable	Fair	

* Some conditions are mutually exclusive.
** Modified after Wickham et al. (1972).

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 Palmstrøm (1996). Rock masses have various discontinuities that tend to reduce the inherent strength of the rock. Rock mass index is expressed as in the following equations:

$$\text{RMi} = q_c \times JP \tag{17}$$

$$JP = 0.2(jC)^{0.5}(V_b)^D \tag{18}$$

$$jC = jL\frac{jR}{jA} \tag{19}$$

$$\text{D} = 0.37 \cdot j\text{C}^{-0.2} \tag{20}$$

Where:

q_c = the uniaxial compressive strength of the intact rock measured on 50 mm samples (MPa)

JP = the jointing parameter that is composed of four jointing characteristics of block volume or density of joints, joint roughness, joint alteration, and joint size, varying from almost 0 for crushed rock masses to 1 for intact rocks

jC = the joint condition factor

V_b = the block volume, which can be found from field measurements (m^3)

jR, jA, and jL = joint roughness, joint alteration, and joint length, respectively

Figure 16 shows the required parameters from the rock mass to determine the RMi.

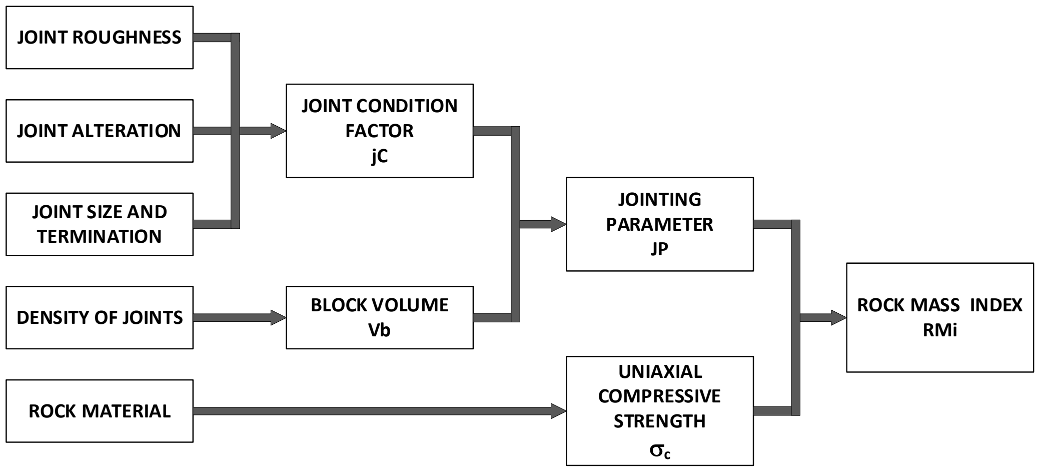

Figure 16. Flowchart. The main parameters in the rock mass applied in the RMi Palmstrøm (1996).

As expected, the parameters selected to be used in RMi are recommended to represent the average condition of the rock mass. Palmstrøm (1996) suggested these following input parameters to RMi from the study of 15 different classification systems and his own experience.

1. 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. Shear strength of the block faces, characterized by factors for the joint characteristics, jR and jA (table 16 and table 18).
4. Size and termination of the joints, given as their length and continuity factor, jL (table 17).

Various parameters of RMi and their combinations in the rock mass index are shown in table 16, table 17, and table 18. It shows a graphical combination of block volume (V_b) and joint condition factor (jC).

Table 16. The joint roughness ratings (*jR*) from smoothness and waviness (Palmstrøm 1996).

Small Scale Smoothness* of Joint Surface	Large Scale Waviness of Joint Plane				
	Planar	**Slightly undulating**	**Strongly undulating**	**Stepped**	**Interlocking**
Very Rough	3	4	6	7.5	9
Rough	2	3	4	5	6
Slightly Rough	1.5	2	3	4	4.5
Smooth	1	1.5	2	2.5	3
Polished	0.75	1	1.5	2	2.5
Slickensided**	0.6–1.5	1–2	1.5–3	2–4	2.5–5
	For irregular joints, a rating of jR = 5 is suggested				

* For filled joints: jR = 1.

** For slickensided joints, the values of R depend on the presence and outlook of the striations; the highest value is used for marked striations.

Table 17. The joint length and continuity rating (jL) (Palmstrøm 1996).

Joint Length (m)	Term	Type	*jL*	
			Continuous Joints	**Discontinuous Joints**
<0.5	Very short	Bedding/foliation parting	3	6
0.1–1	Short/small	Joint	2	4
1–10	Medium	Joint	1	2
10–30	Long/large	Joint	0.75	1.5
>30	Very long/large	Filled joint scam* or shear	0.5	1

* Often a singularity and should in these cases be treated separately.

Table 18. Characterization and rating of the joint alteration factor (*jA*) (Palmstrøm 1996).

<table>
<tr><th>Term</th><th colspan="2">Description</th><th>jA</th></tr>
<tr><td colspan="4">A. Contact between rock wall surfaces</td></tr>
<tr><td>Clean joints</td><td colspan="2"></td><td></td></tr>
<tr><td>Healed or welded joints</td><td colspan="2">Softening, impermeable filling (quartz, epidote, etc.)</td><td>0.75</td></tr>
<tr><td>Fresh rock walls</td><td colspan="2">No coating or filling on joint surface, except of staining</td><td>1</td></tr>
<tr><td>Alteration of joint wall</td><td colspan="2"></td><td></td></tr>
<tr><td>i. 1 grade more altered</td><td colspan="2">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 colspan="2">The joint surface shows two classes higher alteration than the rock</td><td>4</td></tr>
<tr><td>Coating or thin filling</td><td colspan="2"></td><td></td></tr>
<tr><td>Sand, silt calcite, etc.</td><td colspan="2">Coating of friction material without clay</td><td>3</td></tr>
<tr><td>Clay, chlorite, talc, etc.</td><td colspan="2">Coating of softening and cohesive minerals</td><td>4</td></tr>
<tr><td colspan="4">B. Filled joints party or no contact between the rock wall surfaces</td></tr>
<tr><td>Type of Filling Material</td><td>Description</td><td>Partly Wall Contact (thin filling <5mm*)</td><td>No Wall Contact (thick filling or gouge)</td></tr>
<tr><td>Sand, silt, calcite, etc.</td><td>Filling of friction material without clay</td><td>4</td><td>8</td></tr>
<tr><td>Compacted clay materials</td><td>“Hard” filling of softening and cohesive materials</td><td>6</td><td>10</td></tr>
<tr><td>Soft clay materials</td><td>Medium to low over-consolidation of filling</td><td>8</td><td>12</td></tr>
<tr><td>Swelling clay materials</td><td>Filling material exhibits clear swelling properties</td><td>8–12</td><td>12–20</td></tr>
</table>

* Based on joint thickness division in RMR system (Bieniawski 1973).

Typically, jC stays between 1 and 2 when calculated with the above parameters, which results in $JP = 0.2(V_b)^{0.37} \sim 0.28(V_b)^{0.38}$.

When a sample size of a rock mass is enlarged from laboratory size to field size, a significant scaling effect is known to be involved. For very large rock masses where the jointing parameter $JP \approx 1$, the scale effect for uniaxial compressive strength (q_c) should be accounted for since it is related to the sample size of 50 mm.

The sample scale effect to the uniaxial compressive strength is presented in figure 17. Figure 18 also presents the jointing parameter value for joint conditions (Palmstrøm 1996).

$$\sigma_c f = \sigma_{c50} \left(\frac{0.05}{D_b}\right)^{0.2} = \sigma_{c50}\, f_\sigma \qquad (21)$$

Where:

σ_{c50} = the uniaxial compressive strength for 50 mm sample size

Db = block diameter measured (m)

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

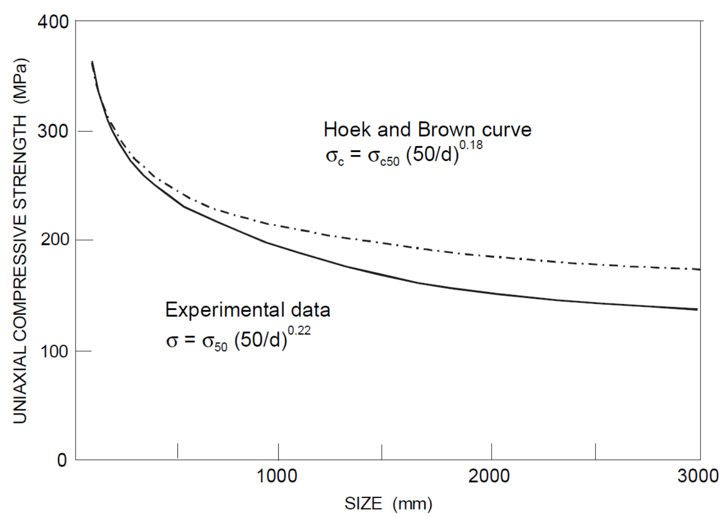

Figure 17. Graph. Empirical equations for the scale effect of the uniaxial compressive strength based on data from Hoek and Brown and Wagner (Palmstrøm 1996).

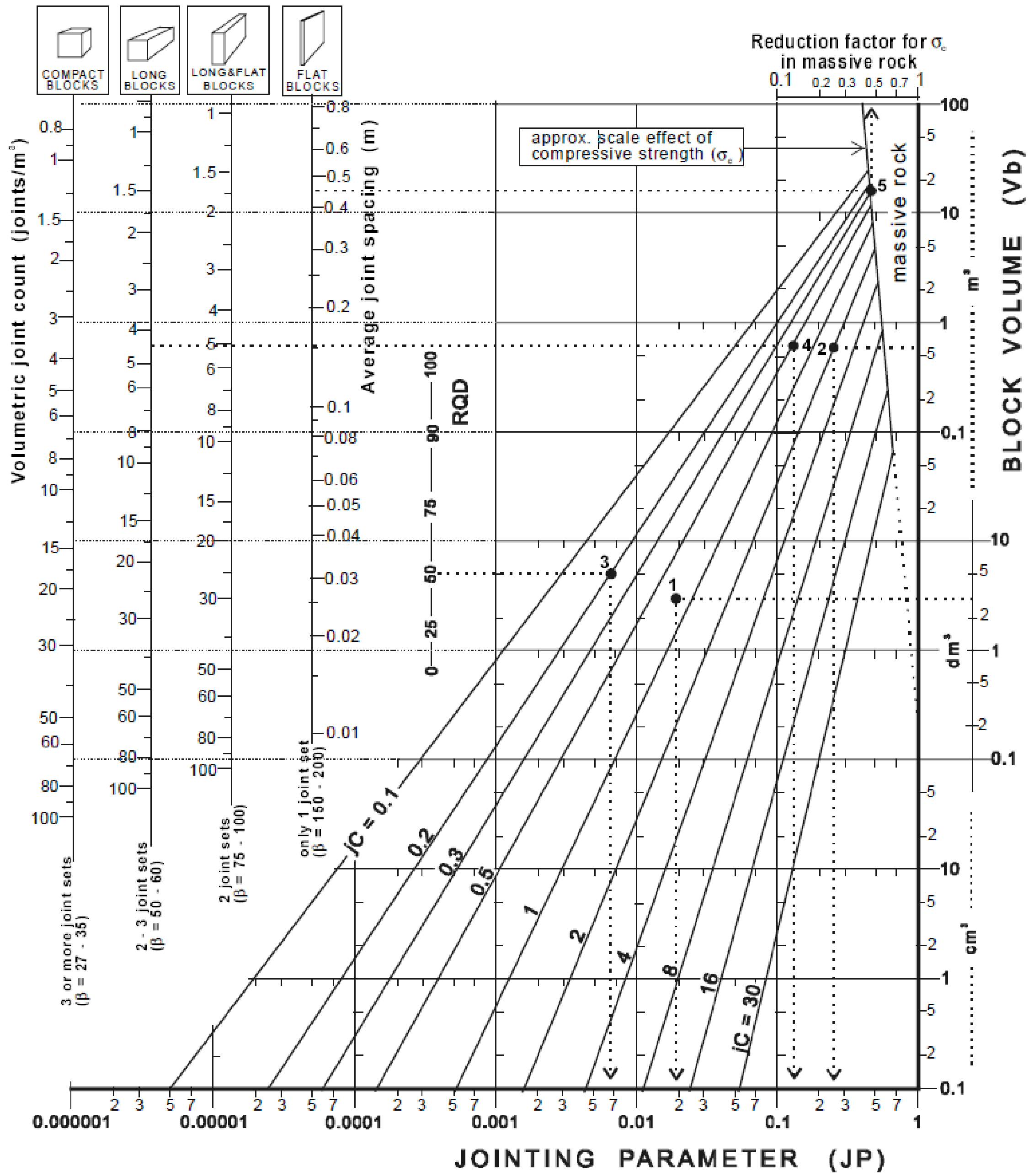

Figure 18. Graph. The jointing parameter (*J*P) from the joint condition factor (*jC*) and various measurements of jointing intensity (V_b, *jA*, RQD) (Palmstrøm 1996).

The classification of RMI is presented in table 19. The numerical values are not sufficient for proper characterization of complex materials such as rock masses. RMi parameters are accompanied by supplementary descriptions.

Table 19. Classification of RMi (Palmstrøm 1996).

Term		RMi Value
For RMi	**Related to Rock Mass Strength**	
Extremely low	Extremely weak	<0.001
Very low	Very weak	0.001–0.01
Low	Weak	0.01–0.1
Moderate	Medium	0.1–1.0
High	Strong	1.0–10.0
Very high	Very strong	10–100
Extremely high	Extremely strong	>100

Some of the advantages of using the rock mass index are that its systematic approach to rock mass characteristics will enhance the accuracy of the input data needed. RMi can be used for rough estimates when limited ground condition information is available. RMi offers a stepwise judgment suitable for engineering judgment. Furthermore, it covers a wide variety of rock masses and has a wide application. However, some limitations of this system include that it can only express compressive rock strength of masses and 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. In addition, RMi is best considered as a relative index in its characterization of rock mass strength (Palmstrøm 1996).

Geological Strength Index

The strength of intact rock material is determined 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), and then assessed using the

Hoek–Brown failure criterion (AASHTO 2020). The geological strength index was first introduced by Hoek in 1994 to overcome the fact that RMR values did not work for very poor rock masses to get materials constants for the Hoek–Brown equation (Hoek 1994). It has been well-known for its simple, fast, and reliable classification based on visual inspection of geological conditions that can be used for both hard and weak rock masses (Singh and Goel 1999). As computer modeling and testing became more prevalent, Hoek and Brown developed charts for estimating GSI based on the following correlations:

$$\mathrm{GSI} = \mathrm{RMR}'_{89} - 5 \text{ for GSI} \geq 18 \text{ or RMR} \geq 23 \tag{22}$$

$$\mathrm{GSI} = 9 \ln Q' + 44 \text{ for GSI} < 18 \tag{23}$$

Where:

RMR'_{89} = rock mass rating according to Bieniawski (1989)

Q' = modified rock mass quality index = $\left[\frac{RQD}{J_n}\right] \cdot \left[\frac{J_r}{J_a}\right]$

Hoek and Brown (1997) proposed a chart for GSI for experts to use to 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, and (4) Crushed. These are adopted from the Terzaghi classification. Furthermore, discontinuities are classified into five surface conditions of: (1) Very Good, (2) Good, (3) Fair, (4) Poor, and (5) Very Poor (Singh and Goel 1999).

The Hoek and Brown chart can be found in the AASHTO *LRFD Bridge Design Specifications* and are also provided in figure 19 and figure 20.

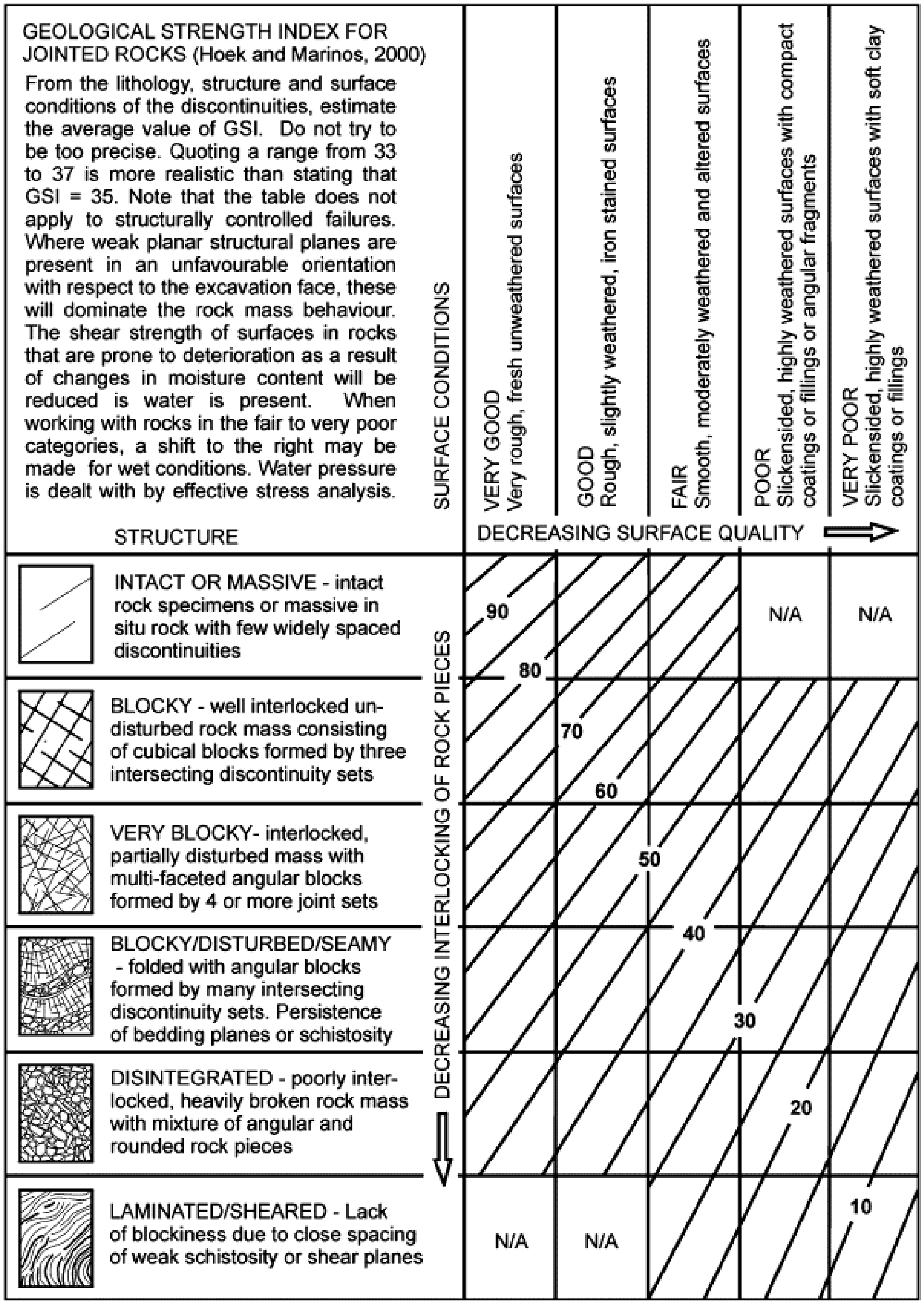

Figure 19. Chart. Geological strength index estimating chart from the geological observations (Marinos et al. 2005).

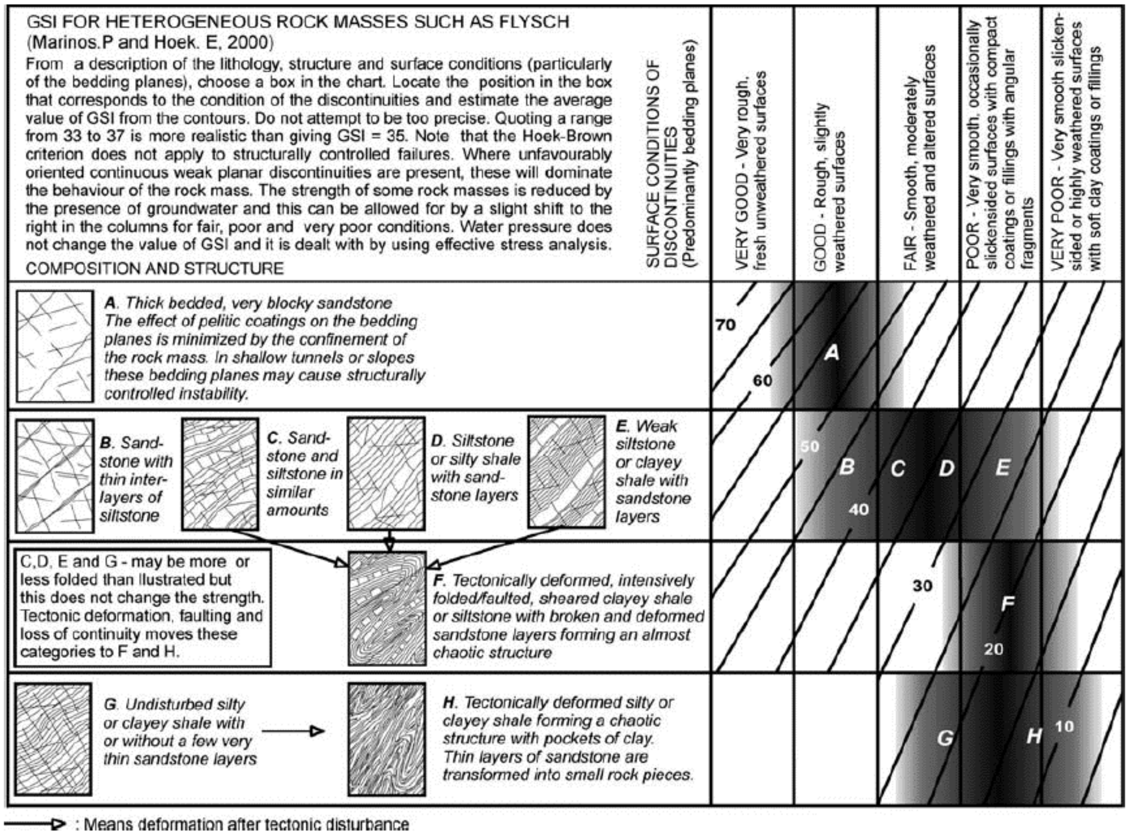

Figure 20. Chart. Geological strength index estimates for heterogeneous rock masses (Marinos et al. 2005).

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 (2019) suggested the following modified strength criterion for a jointed rock mass.

$$\sigma'_1 = \sigma'_3 + q_u \left(m_b \frac{\sigma'_3}{q_u} + s\right)^a \tag{24}$$

Where:

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

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

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

e = 2.718 (natural log base)

σ'_1 and σ'_3 = the principal effective stresses (ksf)

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

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

m_b, s, and a = empirically determined parameters

The constant m_i values are provided in table 20.

SUMMARY

Several rock mass classification methods explained in this chapter were proposed and revised for decades to estimate values on shear strength and deformation characteristics of rock. In the 1960s, RQD was first suggested and soon became widely accepted internationally. Since then, new and revised rock mass classification systems are primarily upgrading the RQD by considering additional parameters such as strength, discontinuity, groundwater, etc. These parameters have been introduced and their applications have been expanded to broader areas.

Table 20. Values of Hoek–Brown constant m_i by rock group (AASHTO 2020).

Rock Type	Class	Group	Texture: Coarse	Texture: Medium	Texture: Fine	Texture: Very fine
SEDIMENTARY	Clastic		Conglomerate 21 ± 3 Breccia 19 ± 5	Sandstone 17 ± 4	Siltstone 7 ± 2 Greywacke 18 ± 3	Claystone 4 ± 2 Shale 6 ± 2 Marl 7 ± 2
SEDIMENTARY	Non-Clastic	Carbonates	Crystalline Limestone 12 ± 3	Sparitic Limestone 10 ± 5	Micritic Limestone 8 ± 3	Dolomite 9 ± 3
SEDIMENTARY	Non-Clastic	Evaporites		Gypsum 10 ± 2	Anhydrite 12 ± 2	
SEDIMENTARY	Non-Clastic	Organic				Chalk 7 ± 2
METAMORPHIC	Non-Foliated		Marble 9 ± 3	Hornfels 19 ± 4 Metasandstone 19±3	Quartzite 20 ± 3	
METAMORPHIC	Slightly Foliated		Migmatite 29 ± 3	Amphibolite 26 ± 6	Gneiss 28 ± 5	
METAMORPHIC	Foliated*			Schist 10 ± 3	Phyllite 7 ± 3	Slate 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3	Diorite 25 ± 5		
IGNEOUS	Plutonic	Light	Granodiorite 29 ± 3 (Coarse–Medium)			
IGNEOUS	Plutonic	Dark	Gabbro 27 ± 3	Dolerite 16 ± 5		
IGNEOUS	Plutonic	Dark	Norite 20 ± 5 (Coarse–Medium)			
IGNEOUS	Hypabyssal		Porphyries 20 ± 5 (Coarse–Medium)		Diabase 15 ± 5	Peridotite 25 ± 5
IGNEOUS	Volcanic	Lava		Rhyolite 25 ± 5 Andesite 25 ± 5	Dacite 25 ± 3 Basalt 25 ± 5	
IGNEOUS	Volcanic	Pyroclastic	Agglomerate 19 ± 3	Volcanic Breccia 19 ± 5	Tuff 13 ± 5	

*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 7. APPROPRIATE LRFD DESIGN METHODOLOGY AND RESISTANCE FACTORS

INTRODUCTION

The allowable stress design (ASD) method has been well understood and applied in the design of structures in civil engineering. However, it is based on the accumulated experience rather than the scientific assessment. To address the major disadvantage of the classical ASD method, the LRFD method based on statistical and reliability analyses has been proposed and implemented for a few decades. The most unique benefit of the LRFD is that the various uncertainties are considered with probability and reliability. The uncertainties in loading are separated from those in resistance and the procedures based on probability theory are used to ensure a prescribed margin of safety (Minnesota Department of Transportation 2013). This has been a new standard in the design of geotechnical structures recently.

LRFD AND RESISTANCE FACTORS

In general, the factored resistance can be determined by the equation below (Paikowsky et al. 2004).

$$R_r = \phi R_n \geq \sum \eta_i \gamma_i Q_i \tag{25}$$

Where:

R_r = factored resistance

R_n = ultimate resistance

ϕ = resistance factor

η_i = factors to account for effects of ductility, redundancy, and operation importance ≥0.95

The resistance factor reduces the ultimate resistance, which is equivalent to the factor of safety in the ASD. However, the resistance factor is determined based on statistics and reliability, which are related to the structure (e.g., materials, dimension, etc.). However, uncertainties that can affect the resistance factors other than the pile type exist, which are (1) site conditions, (2) soil/rock properties, and (3) construction methods and their quality (Paikowsky et al. 2004).

The current study needs to explore the same challenges. Even though well-known theories and equations are available for piles, the currently available resistance factors would not present well the actual capacity because the site conditions are unique (i.e., seating on rock layer); rock properties are uncertain or at least not as much obtained and correlated as soil; and the construction method is unique, requiring a pilot hole with a fill.

The other unique design consideration in estimating the pile capacity in rock comes from its higher strength. It requires much higher load to cause the rock to fail. Therefore, a load test to fail the rock requires a lot of weight or reaction force, if the pile does not fail first. This is the most challenging issue for piles in rock. Collecting appropriate load test data or conducting appropriate tests will be very difficult, and thus, calibration of the resistance factors could be difficult as well.

When the pile is driven to refusal, it indicates the bedrock is of good quality, and thus, capacity of the pile will be governed by the structural capacity of the pile or the rock capacity.

Table 21 shows a summary of the resistance factors adopted by the selected state DOT agencies. AASHTO is in the columns next to some states due to the state document referring users to the AASHTO LRFD Specifications. Although several states indicated the use of PDA for a verification method of piles on rock, several did not specify whether it was for soft (weathered) rock or hard rock. In addition, none of them uses a resistance factor specifically for the pile with a pilot hole except Ohio. Ohio DOT uses AASHTO's structural resistance factors (i.e., 0.50 and 0.60) for the strength limit state for driven piles. They also use 0.95 of the resistance factor for a pile prebored into bedrock and filled with concrete because they consider it as a steel column and do not conduct field verification of the bearing capacity.

Table 21. Status of PDA application and resistance factors for piles in rock.

State	Ground Conditions	Pilot Hole	Test Type	Resistance Factor
AK	Weak Rock	Yes	PDA	AASHTO
CO	Rock	Yes	PDA	AASHTO
MT	Rock	Yes	PDA	AASHTO
PA	Soft Rock	Yes	PDA	AASHTO
FL	Rock	Yes	PDA	0.65–0.75
NC	Weathered Rock	Yes	PDA	0.6–0.75
SC	Soft Rock	Yes	PDA	0.55–0.7
AL	Rock	Yes	—	AASHTO
IL	Rock	Yes	—	AASHTO*
GA	Rock	Yes	—	AASHTO
WY	Rock	Yes	—	AASHTO
IA	Rock	Yes	—	0.7
MA	Weathered Rock	Yes	—	0.65
OH	Rock	Yes	—	0.50–0.95
WV	Rock	Yes	—	0.5

* AASHTO *LRFD Bridge Design Specification* for drilled shaft axial resistance in rock.

SUMMARY

The resistance factors currently listed in state design-related documents vary from 0.4 to 0.8, whereas those for rock with PDA range from 0.5 to 0.75. The PDA provides useful information when it is driven through weak rock layer or to hard rock layer for seating confirmation; however, it appears that PDA for rock has not been applied widely yet. Among seven states that indicated the use of PDA for piles in rock, four states (CO, FL, PA, and SC) recommend a larger hole, while two states (AK and MT) recommend a smaller hole than the size of the pile.

When considering Georgia's five neighboring states only (AL, FL, NC, SC, and TN), four are using a pilot hole and the average resistance factor is 0.67 with PDA.

The average resistance factor for a pile on rock without PDA was 0.59 excluding the 0.95 of Ohio. However, this is not a clear distinction because some states did not specifically state if the PDA is applied to both hard and weathered rock or not. In addition, application of the AASHTO resistance factor for rock could be 0.45 as a driven pile end bearing in rock, 0.50 as a drill shaft tip resistance in rock, or 0.65 as a driven pile with a PDA. Regardless of the resistance factors, if the rock is competent enough, the nominal resistance will be most likely dominated by the structural capacity of the pile if the rock mass quality is good.

CHAPTER 8. CONCLUSIONS AND RECOMMENDATIONS

INTRODUCTION

This study was carried out to understand how piles with a pilot hole in rock are being considered in design and construction, especially with the LRFD recommendations. The research team conducted a literature review to understand how the pilot hole and bearing rock layer have been considered in the design and how they can be considered with the new design approach. In addition, a thorough review of the design approaches in the United States was completed by sending out two rounds of survey: (1) an initial survey to all 50 states, and (2) a second survey to 20 selected states. The response rate was very high and provided very useful information about how the pile is being considered and used in different states. Furthermore, field tests to evaluate the possibility of the use of PDA as a verification method were added with the extension of the project.

RESULTS

After thoroughly reviewing the literature regarding each task, the major findings and conclusions are summarized below.

Task 1: The Current Design Methodology for the Construction of Piles in Rock

- Various static and dynamic methods are available for driven piles.
- The static methods include the Canadian Geotechnical Method introduced in the AASHTO LRFD Specifications, the Federal Highway Administration RQD toe resistance method, the Illinois Department of Transportation static method, and the Tomlinson and Woodward method.

- The dynamic methods include the FHWA modified Gates formula, the Engineering News formula, the Washington State Department of Transportation pile driving formula, and wave analysis.

Task 2: The Effects of the Pilot Hole on the Pile Capacity and Behavior, along with the Associated Design and Construction Considerations

- From the collection of survey responses from 48 out of 50 states, it was found that most states have guidelines on how they use a pilot hole (or pile driving assisting methods in a broader term). However, there seems to be no consensus in the terminology, or the design and construction methods related to the use of the pilot hole.
- Some states were identified to review further for specific information through a second survey.

Task 3: The Current Specifications and Verification Methods for Pile Installation in Rock with a Pilot Hole and the Available Equipment/Methods

- The second-round survey was sent to 20 states, and 14 of those states responded. Two of the responding states were dropped due to their ambiguous statements regarding their guidelines, and 12 states were evaluated.
- Most of the responding states use one or more of the dynamic methods (e.g., WEAP, PDA, CAPWAP) for their quality control and quality assurance in the field and AASHTO resistance factors for the design. Only a few states responded that they use empirical methods such as bearing refusals.
- Concrete is often filled to the level of the rock, while other low-cost materials are used to fill the hole all the way to the ground level.

Task 4: The Field Data on Driven and Seated Piles in Georgia

- Ten projects in Georgia were selected, of which eight were reviewed for their design and verification methods, if available.
- One of the projects includes one set of static load tests and three sets of pile driving analyzer tests.
- All three piles were prepared with a pilot hole. However, due to the geological conditions confronted during the driving, one of them (Bent 1 Pile 5) was sliced and driven deeper (about 16 ft) into the ground. The other two piles seated on the rock layer and were tested as planned.
- PDA with CAPWAP has been applied to some of the projects already.

Task 5: The Verification Methods for Quality Assurance of Rock

- Rock quality has been assessed by collecting core samples in the field and conducting laboratory tests.
- In situ rock quality tests are limited, especially for the pile with a pilot hole, mainly due to its size.
- The traditional rock mass classification methods and uniaxial compressive strength can provide enough information to estimate the end bearing of the pile on rock.

Task 6: Development of Appropriate LRFD Design Methodology and Resistance Factor

- The resistance factors could not be calibrated for use in Georgia due to the number of data necessary for the statistical and reliability analyses.

CONCLUSIONS

1. Geotechnical aspects control the design in soft rock, whereas structural aspects control the design in hard rock.
2. PDA can be applied to the piles with a pilot hole on rock to check the internal stress to avoid the damage during striking. Moreover, it can verify the structural capacity of the pile if not the geotechnical capacity due to the higher bearing capacity on rock.
3. Many states use the resistance factor suggested in the AASHTO *LRFD Bridge Design Specifications*. However, some states also use the revised resistance factor for their current pile design on rock, whether the pilot hole is being used or not.
4. For most states using a pilot hole, the skin friction is ignored for the nominal resistance because the pilot holes are larger than the pile, which seems to be a reasonable and conservative assumption.
5. Some states do not run a field test but use refusal/end of driving criteria for piles driven into rock. Similar criteria could be applied to the pile with a pilot hole in rock as a supplement or replacement of PDA.

RECOMMENDATIONS

The piles reviewed in this study have unique design approaches in which the skin friction has been ignored for the nominal resistance. The skin friction was ignored since the pile seated on rock and the pile seated in a pilot hole larger than the pile were considered a reasonable and conservative assumption.

According to the findings and conclusions, this study recommends the followings.

1. Use PDA testing and the AASHTO resistance factor for the driven pile with dynamic testing, for the pile with a pilot hole in rock.
2. Do not consider the skin friction unless further research is conducted to evaluate the load transfer mechanisms and the effects of the pilot hole size and filling on the piles.
3. Perform the drivability analysis using the wave equation software (e.g., GRLWEAP). Check the hammer setting (e.g., stroke or energy) to avoid overstressing of the pile.
4. Collect the PDA test results along with the strength properties of the rock mass.
5. If the PDA test is not available, the driving refusal criterion (e.g., 5 blows per 0.5 inch) can be used. However, it is still recommended that the correlations between the refusal guidelines and rock properties are verified with PDA.
6. An alternative option could be using the resistance factor for the drilled shaft for tip resistance in rock to the pile with a pilot hole on rock.

Even though larger resistance factors are being used by some states, it is not recommended that Georgia adopt the larger factors from other states at this point. Instead, additional studies are recommended to further investigate the use of PDA and CAPWAP for hard rock to ensure the appropriate use of the technology. In addition, collecting the data for PDA, penetration per blow, and rock properties are strongly recommended so they can be correlated to each other eventually.

FUTURE RESEARCH RECOMMENDATIONS

This study was able to provide an overview of how piles with pilot holes are considered in state agencies in the U.S. and how those are designed and constructed. Although most states use the resistance factors recommended by AASHTO (2020), some states have improved their resistance factors for their states for piles on rock.

Therefore, this study suggests the following future work:

- Set up a rock mass properties database from the construction sites.
- Collect PDA test results for piles on rock in Georgia for future research recommendations.
- Create the integrated load test database especially for piles on rock that drive through weak rock or seat on hard rock.
- Correlate the PDA results with other properties, such as design methods, static load test results, or rock properties.
- Investigate the effect of pilot hole size and socket depth to the pile with a pilot hole.
- Compare the cost and benefits for a larger resistance factor with no test versus a smaller resistance factor with PDA tests.

APPENDICES

APPENDIX A. LIST OF STATE DOCUMENTS REVIEWED

Table 22. Documents reviewed, listed by state.

State	Title	Year
AL	Construction Manual	2000
	Structural Design Manual	2019
	Standard Specifications for Highway Construction	2018
AK	Bridge and Structures Manual	2017
	Standard Specifications for Highway Construction	2017
AZ	Standard Specifications for Road and Bridge Construction	2008
AR	Design–Build Guidelines and Procedures	2015
	Standards Specification for Highway Construction (Division 800 Structures)	2014
CA	Foundation Manual	2015
	Bridge Design Practice (Chapter 16 – Deep Foundations)	2015
CO	LRFD Bridge Design Manual	2019
	Standard Specifications for Road and Bridge Construction	2019
CT	Geotechnical Engineering Manual	2005
	Bridge Inspection Manual	2001
DE	Construction Manual	N/A
	Standard Specifications for Road and Bridge Construction	2016
	Bridge Design Manual	2019
FL	Standard Specifications for Road and Bridge Construction	2020
GA	Bridge and Structures Design Manual	2019
HI	2005 Standard Specifications	2005
ID	Bridge Design Manual	N/A
	Structures Book	N/A
	Standard Specifications for Highway Construction	2018
IL	Geotechnical Manual	2020
	Construction Manual	2020
	Bridge Manual	2012
IN	Design Manual	2012
	Standard Specifications	2020
IA	LRFD Bridge Design Manual	2019
	Standard Specifications for Highway and Bridge Construction	2017

State	Title	Year
KS	Bridge Construction Manual	2013
	Construction Manual	2014
KY	Standard Specifications for Road and Bridge Construction	2008
	Structural Design (Chapter 3)	N/A
LA	Standard Specifications for Roads and Bridges	2016
ME	Bridge Design Guide	2003
MD	Construction Manual	2002
	Standard Specs	2018
	Substructure Repairs (SR – SUB)	N/A
MA	LRFD Bridge Manual (Part I and Part II)	2013
	Standard Specifications for Highways and Bridges	2020
MI	Design Manual Bridge Design	2019
	Standard Specifications for Construction	2012
MN	Standard Specifications for Construction	N/A
	LRFD Bridge Design Manual	2014
	Bridge Construction Manual	N/A
	Geotechnical Manual	N/A
MS	Geotechnical Guidance Manual	2007
	Standard Specifications for Road and Bridge Construction	2017
	Bridge Design Manual	2010
MO	Bridge Design Manual	2020
	Standard Specifications for Highway Construction	2019
MT	Standard Specifications for Road and Bridge Construction	2014
	Geotechnical Manual	2008
	Structures Manual	2004
	Bridge Inspection and Rating Manual	2018
NE	Construction Manual	2019
	Geotechnical Policies and Procedures Manual	2012
NV	Structures Manual	2008
	Standard Specifications for Road and Bridge Construction	N/A
NH	Bridge Manual	N/A
	Standard Specifications for Road and Bridge Construction	N/A
NJ	Design Manual for Bridges & Structures (Sixth Edition)	2016
	Standard Specifications for Road and Bridge Construction	2007
NM	Standard Specifications for Highway and Bridge Construction	2019
NY	Geotechnical Design Manual	2018
	Standard Specifications	2020
NC	Standard Specifications for Roads and Structures	2012
	LRFD Driven Pile Foundation Design Policy	2014

State	Title	Year
ND	Design Manual	N/A
	Standard Specifications for Road and Bridge Construction	2014
OH	Bridge Design Manual	2019
	Construction and Material Specifications	2019
OK	Standard Specifications for Highway Construction	2019
OR	Bridge Design Manual	2019
	Geotechnical Design Manual (Chapter 8 – Foundation Design)	2019
PA	Geotechnical Engineering Manual	2018
	Design Manual, Part 4 (DM-4)	2019
RI	LRFD Bridge Design Manual	2007
	Standard Specifications for Road and Bridge Construction	2004
SC	Construction Manual	2004
	Geotechnical Design Manual (Version 2.0)	2019
SD	Structures Construction Manual	2018
	Standard Specifications for Roads and Bridges	2015
TN	Standard Specifications for Road and Bridge Construction	2015
	Geotechnical Manual	2016
TX	Geotechnical Manual	N/A
	Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges	N/A
UT	Standard Specifications for Road and Bridge Construction	2017
VT	Structures Design Manual	2010
	Standard Specifications for Construction	2011
VA	Road and Bridge Specifications	2016
WA	Bridge Design Manual (LRFD)	2019
	Standard Specifications for Road, Bridge, and Municipal Construction	2020
	Geotechnical Design Manual	2010
WV	Bridge Design Manual	2004
	Construction Manual	2002
WI	Bridge Manual (Chapter 11 – Foundation Support)	2019
WY	Construction Manual	2017
	Standard Specifications for Road and Bridge Construction	2010

APPENDIX B. SURVEY QUESTIONS

First Survey Questions

The first 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 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.

Second Survey Questions

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?

APPENDIX C. SUMMARY OF SECOND SURVEY

Alabama:

Q1: Field verification is used to verify the pile capacity in answer to the first question.

Q2: Static analysis programs are used to determine pile capacity in the design process. Additionally, resistance factors developed by AASHTO are used.

Q3: 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.

Alaska:

Q1: Verification depends on if the foundation is considered a shaft or a pile. If the foundation is on competent, strong rock, that it will likely be considered a shaft and verification would not 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 applied. If expected driving stresses are low, then there is no need for dynamic testing.

Q2: 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 if available. Additionally standard resistance factors published by AASHTO LRFD are utilized for design.

Q3: 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 replied 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.

Colorado:

Q1: The Pile Driving Analyzer (PDA) or CAPWAP are used to verify the capacity. Any QA/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.

Q2: Treat the pile like a column in the predrilled location. We use AllPile or similar program for more detailed analysis for zero moment locations. All resistance factors are sourced from AASHTO LRFD Bridge Design Manual.

Q3: 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.

Additionally, they provided useful information in Survey 1 as included below.

PDA and WEAP are used to determine pile strength after the piles have been driven.

- CDOT requires a boring to reach bedrock, SPT test and geotechnical report.
- The WEAP is used for drivability and hammer size.
- Designers use program driven for driven pile or presumptive equation with blow counts for pile depth and pay base.
- During pile installation PDA is used for actual driving refusal, capacity verification during construction.
- PDA with CAPWAP provides end bearing as well as side shear for future database and backchecking purpose.

Florida:

Q1: Dynamic testing methods are used to verify capacity such are pile driving analyzer or embedded data collectors.

Q2: 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.

Q3: There is not a preferred method of backfilling a hole and it depends on the project; the hole is backfilled the entire performed depth.

Iowa:

Q1: Wave Equation Analysis is performed on all driven piles to check for driving stresses and determine capacity in the field. The contractor is to hit the pile with an approved hammer to “seat” the pile on rock to confirm the rock is solid.

Q2: The pile capacity is determined using LRFD methods considering structural resistance and geotechnical resistance. A resistance factor of 0.7 is used based on local research.

Q3: Predrilled holes are backfilled with concrete. The pile is a minimum of 3 ft into sound rock, and above the concrete can be sand.

Kentucky:

Q1: 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.

Q2: 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. Regarding the 2nd question, 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.

Q3: Predrilled holes are backfilled with sand, gravel, and concrete depending on what is needed. If axial strength is needed, only sand and gravel are used, and if lateral strength is needed, concrete is used. The hole is backfilled the entire length.

North Carolina:

Q1: If the rock is crystalline rock with ($N \geq 60$ blows in 0.10 ft), 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.

Q2: 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.

Q3: For non-integral end bents/abutments, the hole is filled with concrete, grout, or flowable fill. For integral end bents/abutments the hole is generally filled to the natural ground ±3 ft.

Ohio:

Q1: 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 (4,000 psi) 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 are no verifications of firm contact with rock other than counting pile driving hammer blows.

Q2: 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.

Q3: The pre-bored hole is always larger than the pile and is backfilled with class "QC Misc" (4,000 psi) 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.

Oklahoma:

Q1: A pile driver is placed, and Gates Equation is checked or practical refusal is used to verify pile capacity.

Q2: Rock in Oklahoma is not deep and pile designs are governed by structural design. In addition, AASHTO LRFD resistance factors are used along with local ODOT factors.

Q3: Sand is typically used as backfill material and is filled to the full depth.

Pennsylvania:

Q1: Wave equation analysis or dynamic testing are used to approve capacity in the field per Design Manual Part 4 (DM-4) 10.7.3.8.4 and 1.7.5.1 and/or dynamic pile monitoring are used. Please reference Section 1005.5b of Publication 408 (Pennsylvania Specifications) for dynamic monitoring criteria. Piles are driven to absolute refusal (20 blows per inch prior to placement of concrete. For piles bearing on rock. Piles are driven to absolute refusal/end of driving full driving criteria and/or full restrike, if restrike is required per the project special provisions, and prior to placement of concrete.

In order to verify the seating of pile on rock, the top of rock elevation is estimated on the plans/structure borings, the pile is driven to absolute refusal/end of driving criteria, wave equation analysis is performed with graphs available to the structure control engineer for use in determining if the pile is seated on rock. Dynamic pile monitoring may be specified if the rock is soft/weathered or if uplift of the pile is anticipated due to the nature of the rock (claystone), etc., and would also be used to determine if the pile has achieved required capacity without overstressing.

Q2: 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. Contribution of side friction in soil for piles driven to rock is ignored.

Q3: Typically, granular material is used to backfill the piles, but concrete and grout have been used before as well.

South Carolina:

Q1: If the rock is hard and strong enough, the pile is designed using 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.

Q2: If rock is hard enough, we do a shaft design. Otherwise, we use resistance factors for piles driven to weak rock. AASHTO factors are used and the South Carolina Geotech Manual, with a typical factor of 0.6 for piles driven to rock.

Q3: 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 in the shaft design.

Wyoming:

Q1: 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.

Q2: 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 Chapter 10 of the AASHTO LRFD code.

Q3: Annular space around the pile is typically backfilled with sand or pea gravel, and the entire length of the pile is backfilled up to the cutoff elevation.

APPENDIX D. SUMMARY OF FIELD TEST REPORTS AT BANKS COUNTY

Report of PDA Testing – Banks County

TP-1 / Bent 3 / Column 1 / Pile 5

Table 23. TP-1 / Bent 3 / Column 1 / Pile 5.

TP-1 / Bent 3 (HP 14×73 (50 ksi))	
Required ultimate driving resistance	482 kips
Minimum tip elevation	633 ft
Tip elevation end of initial drive	633.09
Pile set at end of initial drive	¼ inch in 2 blows
Stroke height at end of initial drive	11.6+ ft
Fuel setting	Initial: Minimum; Final: Minimum
PDA at end of initial drive	822 kips (blow 7)
CAPWAP at end of initial drive	- CAPWAP 565 kips on blow 4 with 5.3-ft stroke height - CAPWAP 654 kips on blow 5 with 8.7-ft stroke height - CAPWAP 816 kips on blow 6 with 9.3-ft stroke height

RMX: Maximum Case Method Capacity (JC):

- During the PDA testing, the RMX method was utilized and the Case damping constant (JC) value of 0.9 was utilized for evaluating signals.
- The match quality for the signal analyzed in CAPWAP for TP-3 was 2.06.

Driving Stresses:

Compressive (mainly CSX and CSB) and tension stresses (TSX) should not exceed 45 ksi during driving. Test pile TP-1 tension, and compressive stresses remained under the maximum recommended values for grade 50 ksi steel piles, according to the PDA testing. Based on the PDA results, compressive stress in the 14×73 steel H-piles can be managed below 45 ksi if the hammer is set on the minimum fuel setting and sand stroke heights do not exceed 9 ft (with pile set approximating ⅛ of an inch per blow).

Beta Integrity:

- Beta was 100% at the end of initial driving for TP-1, therefore PDA data indicate test pile TP-1 was driven without damaging the pile.
- The toe beta was observed throughout the driving of TP-1. This toe beta observed should not be considered alarming as an indication of damage occurring to the toe of the pile because the toe beta never became a full beta at the end of the pile as it was being advanced into the hard layer and was observed throughout driving.

Capacity:

- The subsequent CAPWAP analysis and PDA testing confirmed that the pile driving system can advance the grade 50 14×73 steel H-piles to a depth where the required driving resistance is being achieved without damaging the piles.
- Test pile TP-1 advancement stopped approximately 1 inch above the minimum tip elevation due to CSB approaching 45 ksi. The pilot hole likely stopped a few inches shy of reaching the minimum tip elevation.

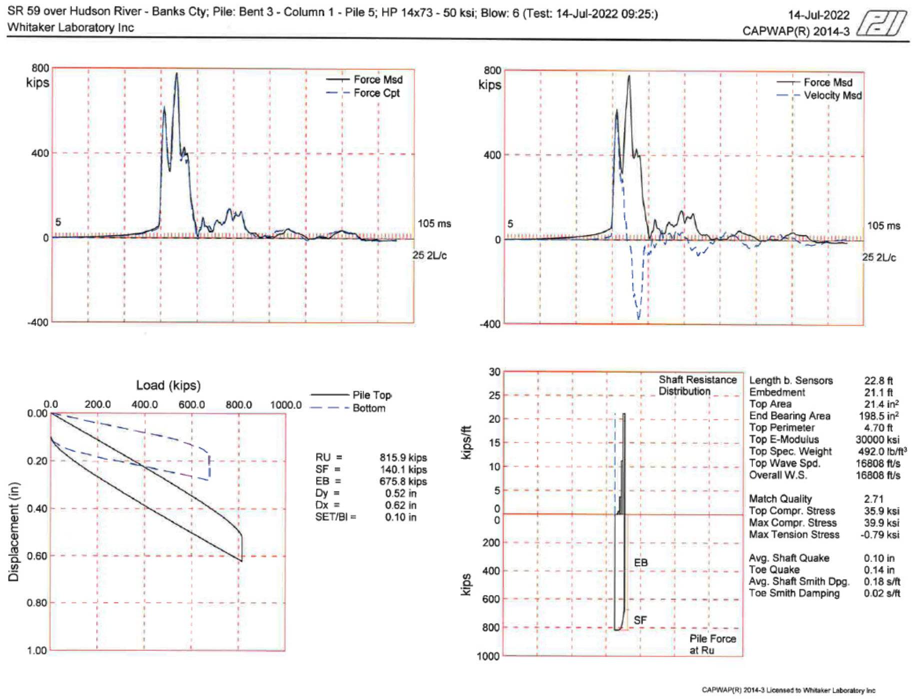

Figure 21. Graphs. PDA & CAPWAP Results (TP-1/Bent 3).

Recommendations:

Based upon PDA testing and subsequent CAPW AP analysis performed on TP-1 at Bent 3, Whitaker offers the following driving criteria for production piles within Bents 2 and 3 on this project:

- 24-inch pilot hole shall be installed to the minimum tip elevation of 631 for Bent 2 and 633 for Bent 3.
- Piles shall consist of HP 14×73 piles (grade 50).
- Piles shall be driven utilizing the Pileco D19-42 hammer.
- Hammer should be set on the Minimum fuel setting.
- Piles shall be marked after every blow.

- Piles shall be terminated at or below a **minimum tip elevation of 631 for Bent 2 and 633 for Bent 3** once a 7+-ft stroke height is achieved with a pile set of ⅛ inch (or less) per blow.

<u>**Note**</u>**:** Piles shall not be overdriven on this site to avoid damaging the bottom of the piles. Pile advancement shall stop immediately (even if above minimum tip elevation) if a 9+-ft stroke height is observed with a pile set of ⅛ inch (or less) per blow.

TP-2 / Bent 4 / Pile 1

Table 24. TP-2 / Bent 4 / Pile 1.

TP-2 / Bent 4 / Pile 1 HP 14×73 (50 ksi)	
Required ultimate driving resistance	491 kips
Minimum tip elevation	648 ft
Tip elevation end of initial drive	649.75*
Pile set at end of initial drive	⅜ inch in 5 blows
Stroke height at end of initial drive	9 ft
Fuel setting	Initial: Minimum; Final: Minimum
PDA at end of initial drive	858 kips (blow 145)
CAPWAP at end of the initial drive	• Blow 95 – CAPWAP 464 kips with ½-inch travel in 5 blows with 6.5-ft stroke heights (max compressive stress 25.6 ksi) • Blow 131 – CAPWAP 541 kips with ⅜-inch travel in 5 blows with 7+-ft stroke heights (max compressive stress 34.8 ksi). • Blow 145 – CAPWAP 696 kips with ⅜-inch travel in 5 blows with 9-ft stroke heights (max compressive stress 45.8 ksi).

* Pile advancement stopped 1.75 ft above minimum tip elevation due to high CSB (compressive stress at bottom of pile).

RMX: Maximum Case Method Capacity (JC):

During the PDA testing, the RMX method was utilized. Case damping constant (JC) value of 0.9 was utilized during the testing.

Driving Stresses:

- Compressive (mainly maximum compressive stress, CSX and maximum compressive stress at bottom, CSB) and tension stresses (maximum tension stress at bottom, TSX) should not exceed 45 ksi during driving. For test pile TP-2, both tension and compressive stresses remained under the maximum recommended values for grade 50 ksi steel piles. Pile advancement stopped immediately upon CSB reaching 43.4 ksi (Blow 146).
- Based upon PDA data, compressive stress in 14×17 steel H-piles can be managed below 45 ksi if the hammer is set on the minimum fuel setting and stroke heights do not exceed 9 ft (with pile set no less than ⅜ of an inch in 5 blows).

Base Integrity:

- Beta was 100% at the end of initial driving for TP-2, therefore PDA data indicate test pile TP-2 was driven without damaging the pile. However, Beta % approximating 78% to 80% was showing up between 80–100. These Beta's were being caused by bending stresses. After the hammer was re-set and re-aligned, Beta returned to 100%.
- During early low-energy early signals, toe beta was observed. The toe beta never became a full beta at the end of the pile, as it was being advanced into the hard layer and was observed throughout driving. Due to this, the toe beta

observed in TP-2 should not be considered alarming as an indication of damage to the toe of the pile.

Capacity:

- The PDA testing and subsequent CAPWAP analysis have confirmed that the pile driving system can advance the grade 50 14×73 steel H-piles to a depth where the required driving resistance is being achieved without damaging the piles.
- Test pile TP-2 advancement stopped approximately 1.75 ft above the minimum tip elevation due to CSB approaching to 45 ksi.

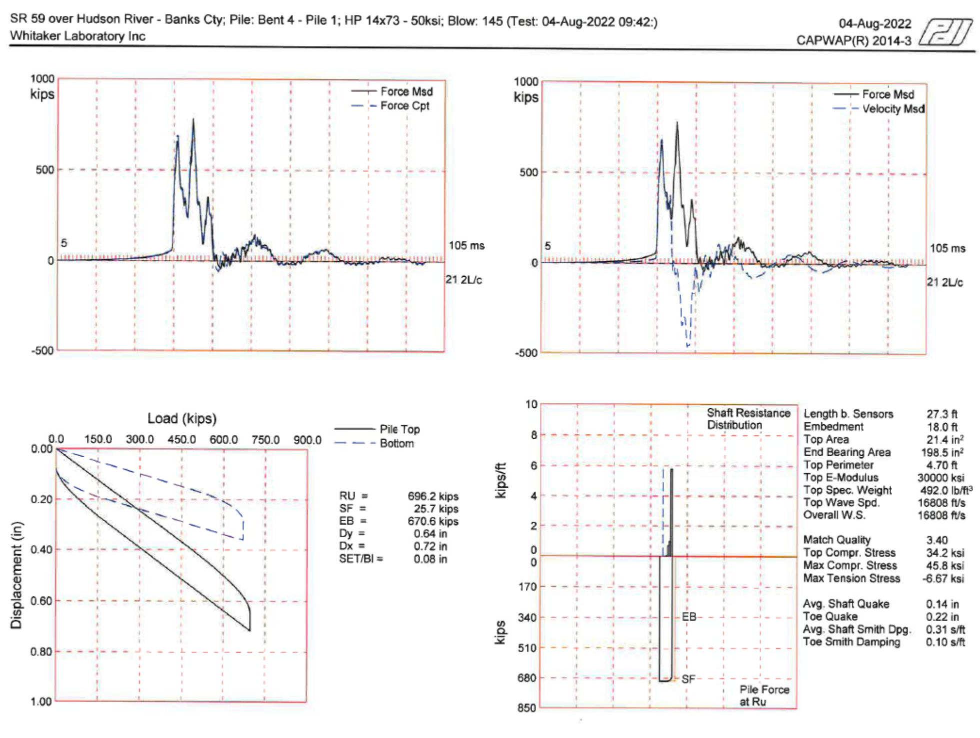

Figure 22. Graphs. PDA & CAPWAP Results (TP-2/Bent 4).

Recommendations:

Based upon PDA testing and subsequent CAPW AP analysis performed on TP-2 at Bent 4, Whitaker offers the following driving criteria for production piles within Bent 4 on this project:

- 24-inch pilot hole shall be installed to the minimum tip elevation of 661.
- 30-inch diameter casing should be installed to keep the pilot hole open (as did the test pile).
- Piles shall consist of HP 14×73 piles (grade 50).
- Piles shall be driven utilizing the Pileco D19-42 hammer.
- Hammer should be set on the Minimum fuel setting.
- Piles shall be marked every 5 blows.
- Piles shall be terminated at or below a minimum tip elevation of 648 once a 7+ ft.
- Stroke height is achieved with a pile set of ⅜ inch (or less) in 5 blows.

Note: Piles shall not be overdriven on this site to avoid damaging the bottom of the piles. Pile advancement shall stop immediately (even if above minimum tip elevation) if a 9+-ft stroke height is observed with a pile set of ⅜ inch (or less) in 5 blows.

TP-3 / Bent 1 / Pile 5

Notes:

- Pilot hole (26-inch diameter) was advanced to elevation 655.
- 30-inch diameter casing (11 ft long) was installed to keep the pilot hole open.

Table 25. TP-3 / Bent 1 / Pile 5.

TP-3 / Bent 1 / Pile 5 (HP 14×73 (50 ksi))	
Required ultimate driving resistance	491 kips
Minimum tip elevation	644 ft
Tip elevation end of initial drive	623.88
Pile set at end of initial drive	½ inch in 5 blows
Stroke height at end of initial drive	8.6+ ft
Fuel setting	Initial: Minimum; Final: Minimum + 1
PDA at end of initial drive	600 kips
CAPWAP at end of initial drive	598 kips

RMX: Maximum Case Method Capacity (JC):

During the PDA testing, the RMX method was utilized. Case damping constant (JC) value of 0.9 was utilized during the testing. The match quality for the signal analyzed in CAPWAP for TP-3 was 2.06.

Driving Stresses:

- For grade 50 steel, compressive and tension stresses should not exceed 45 ksi during driving.

- The PDA testing revealed that for test TP-3 tension and compressive stresses remained well under the maximum allowable values for grade 50 ksi steel piles utilizing the PILECO D19-42 hammer on the minimum +1 fuel setting.

Capacity:

The PDA testing and subsequent CAPWAP analysis have confirmed that the pile driving system can advance the grade 50 14×73 steel H-piles to a depth where the required driving resistance is being achieved without damaging the piles.

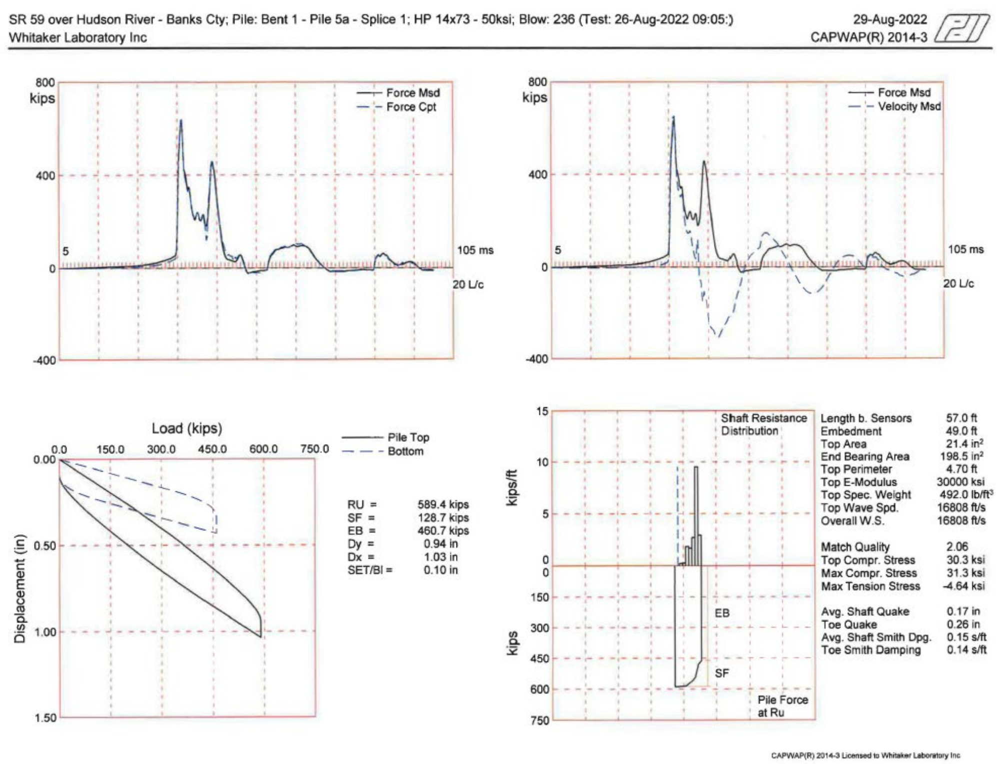

Figure 23. Graphs. PDA & CAPWAP Results (TP-3/Bent 1).

Recommendations:

Based upon PDA testing and subsequent CAPW AP analysis performed on TP-3 at Bent 1, Whitaker offers the following driving criteria for production piles within Bent 1 on this project:

- 26-inch pilot hole shall be installed to elevation 655.
- 30-inch diameter casing should be installed to keep the pilot hole open (as did the test pile).
- Piles shall consist of HP 14×73 piles (grade 50).
- Piles shall be driven utilizing the Pileco D19-42 hammer
- Hammer should be set on the Minimum +1 fuel setting.
- Piles shall be marked every 5 blows.
- Piles shall be terminated at or below a minimum tip elevation of 644 once an 8.6+ foot stroke height is achieved (for all blows) with a pile set of ½ inch (or less) in 5 blows.

Note:

- Piles shall not be overdriven on this site to avoid damaging the bottom of the piles. Pile advancement shall stop immediately (even if above minimum tip elevation) if a 9+-ft stroke height is observed with a pile set of ⅜ inch (or less) in 5 blows.

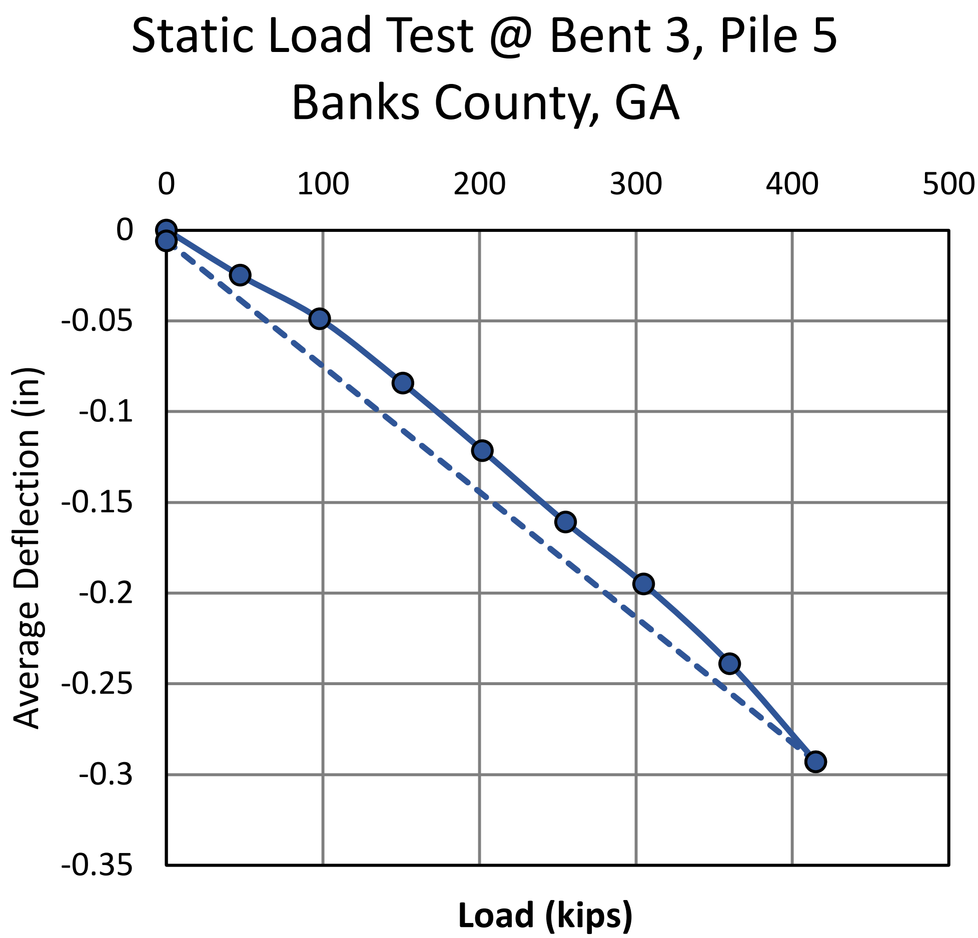

Figure 24. Graph. Xxx.

Test load: 520 kips

Maximum total deflection: 0.239 inch

Maximum applied load: 415 kips

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