Pile Driving Setup for Ohio Soils

Prepared by:
Ömer Bilgin, Ph.D., P.E.
Saeed Alzahrani, M.S.
Peter Narsavage, P.E.
Jamal Nusairat, Ph.D., P.E.
Jerry DiMaggio, P.E.

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Department of Civil and Environmental Engineering and Engineering Mechanics
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Prepared by:

Ömer Bilgin, Ph.D., P.E.
Principal Investigator

Saeed Alzahrani, M.S.

University of Dayton
300 College Park, KL 421
Dayton, OH 45469-0243

and

Peter Narsavage, P.E.
Jamal Nusairat, Ph.D., P.E.
Jerry DiMaggio, P.E.

E.L. Robinson Engineering of Ohio Inc.
950 Goodale Boulevard, Suite 180
Grandview Heights, OH 43212

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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 yards	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)		
°F	Fahrenheit temperature	5(°F-32)/9 or (°F-32)/1.8	Celsius temperature	°C
		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 or psi	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	35.71	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)		
°C	Celsius temperature	1.8°C + 32	Fahrenheit temperature	°F
		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 or psi

* SI is the symbol for the International Symbol of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised September 1993)

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LIST OF ABBREVIATIONS

A	Setup constant (Skov and Denver)
A_s	Pile shaft surface area
AASHTO	American Association of State Highway Transportation Officials
BOR	Beginning of restrike
CIPP	Cast-in-place pipe
COV	Coefficient of variation
D	Pile diameter
EOID	End of initial drive
F_{fine}	Fine fraction
F_{silt}	Silt fraction
F_{clay}	Clay fraction
FHWA	Federal Highway Administration
L	Pile length
LL	Liquid limit
N	Number of data
N60	Standard penetration test N60 number
ODOT	Ohio Department of Transportation
PL	Plastic limit
PI	Plasticity index
Q_{EOID}	Pile total resistance at EOID
$Q(t)$	Total resistance at time, t, after EOID
Q_m	Measured total resistance
Q_p	Predicted total resistance
$Q_{s,EOID}$	Pile side resistance at EOID
$Q_s(t)$	Side resistance at time, t, after EOID
$Q_{s,m}$	Measured side resistance
$Q_{s,p}$	Predicted side resistance
R	Setup ratio for total resistance
R_s	Setup ratio for side resistance
R^2	Coefficient of determination
SRP	Side resistance percentage at restrike
Std. Dev.	Standard deviation
t	Time passed since EOID
t_0	Reference time (Skov and Denver)
UBV	Ultimate bearing value (= nominal pile resistance)
V	Soil volume displaced during pile installation
w	Moisture content

CHAPTER 1. INTRODUCTION

Pile setup is an increase in the nominal axial resistance that develops over time predominantly along the pile shaft. Pore pressures increase during pile driving due to a reduction of the soil volume, reducing the effective stress and the shear strength. Other terms such as *pile freeze* or *side shear setup* are also used for pile setup. Pile setup has been observed in a variety of driven pile types and broad range of soil profiles. Both soil- and pile-related properties contribute to the amount of pile setup.

Before 2020, pile setup was not commonly considered in ODOT's standard driven pile design procedures. It is not easily quantified or predicted. If substantial pile driving losses are encountered during pile installation, ODOT often stops pile driving for some time to determine if pile setup will occur. This time delay may negatively affect the construction schedule and unforeseen costs are incurred when additional pile lengths need to be installed to meet project requirements.

The ODOT Bridge Design Manual (2021) requires that pile setup be accounted for in design if the subsurface geotechnical conditions indicate the potential for pile setup on friction piles. The setup needs to be accounted for if the estimated driving resistance indicates driving losses that would increase the length of the pile during driving by more than 10 ft based on the resistance measured at the end of initial drive (EOID) compared to the required pile capacity, known as the ultimate bearing value (UBV). The UBV corresponds to the nominal pile resistance for Load and Resistance Factor Design (LRFD) or the ultimate pile capacity in Allowable Stress Design. The ODOT Bridge Design Manual (2021) also refers to publication FHWA-NHI-16-009/010, Geotechnical Engineering Circular 12, GEC 12, (Hannigan et al. 2016) "Design and Construction of Driven Pile Foundations" for guidance on the potential for pile setup for various soil types.

Pile setup can result in a significant increase in the pile resistance over time compared to the EOID resistance. If the loss of resistance during driving is not accounted for in design and the piles are driven to the required ultimate bearing value (UBV), this can result in substantial pile quantity overruns during construction. Realization of pile setup in design can therefore result in pile quantity savings and prevent construction delays. This can also help to avoid change orders to furnish and drive additional pile lengths and construct pile splices.

This research project was undertaken to develop more reliable pile setup models to be used in design to better predict pile driving losses that may occur during installation. The project involved collecting data from existing projects around the state, performing investigations at selected field sites, and conducting comprehensive data analysis to investigate the mechanism of pile setup.

The main findings of this project are:

1. Existing pile setup models cannot properly predict setup observed for fine-grained Ohio soils,
2. Construction activities and pile driving sequence at project sites can significantly affect pile resistance obtained from pile load tests, and

3. The current setup factors recommended for side friction in ODOT's Bridge Design Manual, which were provided based on FHWA GEC 12, are significantly lower than the factors observed in Ohio soils through this study. The side friction setup factors are about 50 to 100% more than the factors currently recommended in the ODOT Bridge Design Manual.

The project provided recommendations to ODOT for: 1) load testing practices to obtain more accurate pile resistance, 2) refinement to ODOT driven pile design procedures to better account for setup, and 3) setup prediction models for pile total and side resistance gains over time based on the end of initial driving resistance.

The report consists of five chapters. This chapter, Chapter 1, contains introductory information and discusses the problem that was addressed by this research. Chapter 2 presents the goals and objectives of the research and tasks performed to accomplish the goals. Chapter 2 also includes a summary of key literature search findings. Chapter 3 explains the methodology and steps taken to conduct the research. Chapter 4 presents the results and conclusions of research findings. Chapter 5 provides recommendations to implement the research findings. Results of the literature review is provided in Appendix A. Field projects and the effect of construction activities on pile resistance are summarized in Appendix B. Appendix C provides statistical information on the data collected for the analyses. The analysis of setup ratios for setup factors recommendations is provided in Appendix D. The pile setup database is provided in Appendix E. Supplemental Documents section at the end includes documents, such as load test reports, from the field projects used for this research.

CHAPTER 2. RESEARCH BACKGROUND

This chapter presents the goals and objectives of the project. The specific project tasks performed to accomplish the goals and objectives are provided in addition to a summary of the literature review on the pile setup.

2.1 Project Goal

The goal of this research project was to develop a prediction methodology for the magnitude of setup, or driving losses, that can be anticipated during driving of closed-end cast-in-place reinforced concrete pipe (CIPP) piles driven in predominantly fine-grained soils to account for setup in design.

When piles are expected to perform as friction piles, CIPP piles are recommended as a first option by ODOT. This is due to the fact that pipe piles are likely to develop more friction due to the larger soil displacement they exhibit during driving. Therefore, pile setup is experienced mainly on CIPP piles in ODOT projects.

In this report, “predominantly fine-grained soils” is defined as more than two-thirds of pile length being in fine-grained soil layers. AASHTO and ODOT soil classifications are used to determine fine-grained soils, i.e., more than 35% passing the #200 sieve.

2.2 Project Objectives

The proposed objectives of this research project were:

- Perform a comprehensive literature review to gather information on pile setup, previous research studies and available data, existing prediction methods, and factors contributing to pile setup,
- Collect data from previous ODOT projects where pile setup was evaluated and restrike data was available,
- Conduct comprehensive analysis on the field measurements to investigate the mechanism of pile setup,
- Observe pile load tests and collect data from selected active ODOT construction projects,
- Investigate the potential effect of site construction activities on pile load test results,
- Analyze test data, identify critical soil parameters contributing to pile setup, and correlate pile setup behavior to the critical soil properties,
- Propose formulae to predict pile resistance gains over time,
- Recommend setup factors to be used in design to estimate driving losses, and
- Prepare a final report documenting the findings and recommendations for implementation of this research project.

The execution of the objectives was initially planned to be accomplished in two separate phases. During the first phase, additional ODOT projects were added to the data collection task to increase the number of projects including setup data for the analysis. Dynamic pile load test reports with initial drive and restrike resistance data were provided by third parties who performed the tests. Identifying ODOT projects with restrike tests, gathering the project load test reports, and delivering them to the research team occurred over a long period of time due to the large number of projects and reports. The bridge projects identified and used in this study were constructed between 2006 to 2020. The second phase of the project was initially planned to cover selection of sites from upcoming ODOT projects and to conducted tests at those sites. Because of these factors and associated time constraints, a decision was made by ODOT to conduct both phases concurrently.

2.3 Project Tasks

The project objectives were achieved through the following six major tasks:

- Literature review,
- Collection of existing data,
- Data analysis and interim report,
- Field tests and new data collection,
- Analysis of setup behavior, and
- Formulating pile setup prediction and final report.

Most of these tasks were initially planned to be performed sequentially with little overlap. The data collection from previous ODOT projects were planned to be completed during the first quarter of the project. There was a significant increase in the number of projects to be reviewed when gathering reports from third parties were added to the work scope early in the project. The data collection from previous projects continued almost through three quarters of the project and most of the tasks had to be conducted concurrently.

2.4 Literature Review Summary

Driven piles have been used as deep foundations for both inland and offshore structures for centuries. A unique performance aspect of driven piles is that following the initial installation, the pile resistance provided by the soils around the pile may change with time. An increase in the resistance with time is referred to as pile setup. These time-related changes are permanent and therefore would benefit projects if accounted for in design and installation of driven pile projects.

Pile setup can be measured during construction by performing dynamic pile testing during initial pile driving and restrike testing after some time. Restrike tests can be performed after several hours, days, or weeks after initial driving depending on soil and pile conditions at the site. Pile setup has been observed in a variety of pile types, pile sizes, and a broad range of subsurface profiles (Rausche et al. 1996, Budge 2009). Case studies have demonstrated that pile setup can continue to develop for a long-time following installation and can account for resistance increases of up to 12 times that of initial driving estimates (Titi and Wathugala 1999).

Other studies have confirmed that considering setup during pile design routinely increases nominal pile resistance, reduces the number of piles and lengths (and potentially the number of splices), reduces pile sections, allows for use of smaller driving equipment or reduced installation time (Komurka et al. 2005).

2.4.1 Pile Setup Mechanism

When piles are driven in saturated cohesive soils, excess pore pressures develop due to the disturbance, compression, and displacement of soils by the pile penetration. Both the shearing and radial compression of soils during driving cause pore pressures to increase. The effective stresses along the pile length decrease due to the increased excess pore pressures during pile driving. Therefore, driving losses are observed during driving and at the end of initial drive (EOID) compared to the ultimate bearing value (UBV).

After pile driving is completed, the excess pore pressures begin to dissipate and effective stresses on the pile increase. The dissipation of excess pore pressures primarily occurs through the radial flow of pore water away from the pile (Randolph et al. 1979). The dissipation of excess pore pressures results in an increase of pile resistance. The increase in resistance continues until the excess pore pressures are completely dissipated and equilibrium conditions are reached.

The decrease in excess pore pressure has been determined to be inversely proportional to the square of the distance from the pile (Pestana et al. 2002). In other studies, it has been determined that the time to dissipate excess pore pressure is proportional to the square of the horizontal pile dimension (Soderberg, 1961; Holloway and Beddard, 1995) and is inversely proportional to the horizontal coefficient of consolidation of the soil (Soderberg, 1961). Consequently, larger diameter piles require longer setup times than smaller diameter piles (Wang and Reese 1989; Long et al. 1999).

In addition to the dissipation of excess pore pressures developed due to the disturbance, compression, and displacement of soils by pile penetration, several other factors also contribute to the setup. These factors are liquefaction of loose soils due to the dynamic pile motions and vibrations, soil remolding frequently found in clays, soil fatigue practically smoothing the surface of the hard cohesive soil, and loss of cemented structure in calcareous soils (Rausche et al. 2004).

Komurka et al. (2003) suggests that there are three stages of pile setup: 1) logarithmically nonlinear rate of excess pore pressure dissipation, 2) uniform, logarithmically linear rate of excess pore water pressure dissipation, and 3) aging. As summarized by Haque et al. (2014), the first two stages are similar to the consolidation process and the duration depends on the soil type, soil properties, pile properties, pile type, and pile size. The third stage is due to a combined effect of creep, particle interference, and clay dispersion. Other studies have shown that the soil type and properties affect the setup magnitude and rate, and therefore layered soils provide additional challenges in determining pile setup.

2.4.2 Pile Setup Models and Soil Setup Factors

There have been several, mostly empirical methods proposed by researchers for estimating pile setup. These models mainly predict pile resistance with time using the resistance obtained during initial driving, i.e., EOID resistance. One of the most widely known and used empirical models for determining pile setup was proposed by Skov and Denver (1988). The model to predict the total resistance at any time, $Q(t)$, after the end of initial drive (EOID) is given as:

$$Q(t) = Q_{EOID}\left[1 + A\log\left(\frac{t}{t_0}\right)\right] \tag{1}$$

where, Q_{EOID} is resistance at the EOID, A is pile setup factor, t is time of interest (in days), and t_0 is reference time. For the pile setup constant, A, Skov and Denver (1988) recommended 0.2 for sandy soils and 0.6 for clayey soils. The reference time, t_0, is given as 0.5 and 1.0 days for sandy and clayey soils, respectively. The parameters of Skov and Denver (1988) equation, A and t_0, have been commonly adjusted in subsequent studies by others in order to find the most suitable parameters for given soil conditions. A comprehensive literature review and several other pile setup models proposed by earlier studies are provided in Appendix A.

There are also approaches to estimate losses in soil resistance during pile installation (Hannigan et al. 2016, ODOT 2021), rather than estimating the gain in pile resistance with time after installation. The approach of estimating driving loss is primarily used to incorporate setup during the design stage. Recommended soil setup factors are used to estimate potential driving losses. These soil setup factors are based on a study performed by Rausche et al. (1996) and they are provided in many design manuals and guidelines, such as the ODOT Bridge Design Manual (2021) and FHWA Geotechnical Engineering Circular No. 12: Design and Construction of Driven Pile Foundations (Hannigan et al. 2016). The soil setup factors originated from Rausche et al. (1996) are given in Table 1.

Table 1. Soil setup factors (after Hannigan et al. 2016, based on Rausche et al. 1996)

Predominant Soil Type Along Pile Shaft	Range in Soil Setup Factor	Recommended Soil Setup Factors*	Number of Sites and (Percentage of Database)
Clay	1.2-5.5	2.0	7 (15%)
Silt - Clay	1.0-2.0	1.0	10 (22%)
Silt	1.5-5.0	1.5	2 (4%)
Sand - Clay	1.0-6.0	1.5	13 (28%)
Sand - Silt	1.2-2.0	1.2	8 (18%)
Fine Sand	1.2-2.0	1.2	2 (4%)
Sand	0.8-2.0	1.0	3 (7%)
Sand - Gravel	1.2-2.0	1.0	1 (2%)

* Confirmation with Local Experience Recommended.

The soil setup factors given in Table 1 were obtained by dividing the failure load from static load test by the end of initial drive resistance from a dynamic load test (FHWA GEC-12, Hannigan et al. 2016).

CHAPTER 3. RESEARCH APPROACH

Previous studies on pile setup show that both soil- and pile-related properties contribute to pile setup. Therefore, soil- and pile-related data were collected from previous ODOT projects where pile setup was observed and from active ODOT projects where pile setup was expected. In all these projects, the restrike tests were used to quantify setup magnitude. In some cases, multiple restrike tests were performed.

The data collection included dynamic load testing reports, soil boring logs, project plans, and geotechnical engineering reports. Construction records could have also provided important information about the activities around the pile location prior to the load tests. However, the construction records were not available for the projects completed prior to the start of this research work. One of the major findings of this research project was that construction activities, such as recently installed nearby piles, can significantly affect the soil resistance during dynamic testing. The effect of construction activities will be discussed later in the report.

Three bridge construction sites were selected to collect detailed and controlled data, as well as to conduct additional testing. Cone penetration testing with piezocone (CPTu) and static pile load tests were also performed at these three sites.

The data collected from the projects were used to assemble a database for the pile setup analysis. Analyses concentrated in two main areas:

1. Developing setup factors to be used in design to account for losses during pile driving, and
2. Developing setup models to predict pile resistance over time following the end of initial driving resistance obtained from a dynamic test.

For both cases, the pile setup analyses and investigations were performed for pile total and side resistances. The applicability of existing pile setup models to Ohio soils was also investigated.

3.1 Data Collection from Previous Projects

Dynamic test reports and relevant project documents were collected in coordination with ODOT. Dynamic test reports were received from four different sources: ODOT, GRL Engineers Inc. (Solon, OH), CTL Engineering Inc. (Columbus, OH), and G2 Consulting Group (Troy, MI). Project documents, such as construction plans and boring logs, were obtained from ODOT for the bridges where the dynamic load testing was performed. The reports obtained from G2 Consulting Group were for the projects located in the State of Michigan. This report covers only the data collected from projects in Ohio and the analysis performed based on Ohio soils.

The pile load test reports, construction drawings, and exploration logs were reviewed to identify piles for this study and assemble relevant information pertaining to pile setup. The data collected covered information in four main areas:

- General project information:
  Project name, number, coordinates, county, and district
- Pile information:
  Diameter, length
- Load test data:
  Resistance at EOID, resistance at restrike(s), restrike time(s) (total resistance if only CASE method results were available; total, side, and tip resistances if CAse Pile Wave Analysis Program, CAPWAP, analysis results were available)
- Soil information:
  Moisture content, Atterberg limits (liquid limit, plastic limit, plasticity index), SPT-N60, particle size distribution (fine, silt, and clay percentages).

3.2 Field Projects, Tests, and Data Collection

Several active project sites were considered for detailed field investigations. The following three sites were selected for this purpose:

- HAM-75-1292 (HAM-75)
- LUC-75-0130 (LUC-75)
- SUM-76-0580 (SUM-76)

The locations of the field project sites are shown in Figure 1. Detailed investigations for research purposes at these sites included cone penetration testing using piezocone (CPTu), installing vibrating wire piezometers to monitor pore water pressure during driving and for several months during dissipation of excess pore pressures, dynamic pile load tests (at the end of initial drive and three restrikes), and static pile load tests. The static load tests were conducted nine days after test piles were installed at the HAM-75 and LUC-75 sites, and eight days later at the SUM-76 site.

As mentioned, construction records were not available for the previously completed projects. The three active field projects allowed construction activities, specifically pile driving activities and the installation sequence, to be observed and documented. The field observations revealed that construction activities, such as recently installed nearby piles, can significantly affect the resistance obtained from dynamic pile load tests.

Field projects and the effect of construction activities on pile resistance are described in detail in Appendix B.

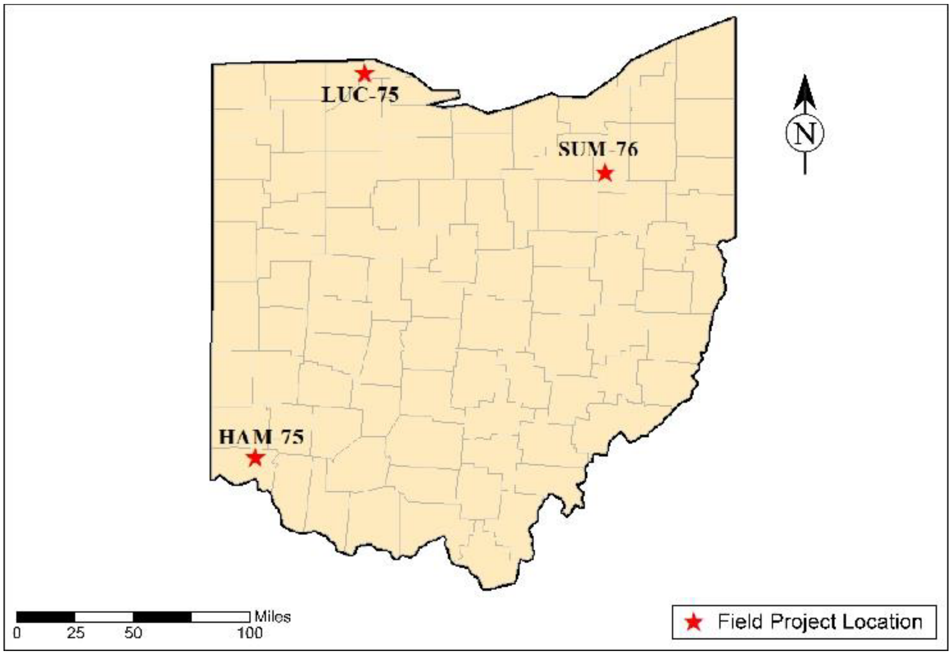

Figure 1. Location map of field project sites with static load testing in Ohio

3.3 Database

The data collected from the completed and active construction projects were used to assemble a database for the pile setup analysis. Battered piles were not included in the database. All the piles in the database were:

- Closed-end CIPP piles,
- Driven predominantly in fine-grained soils (more than two-thirds of pile length is in fine-grained soil layers, i.e., percent passing #200 sieve is more than 35%), and
- First pile driven on each substructure in a project.

ODOT generally uses steel H-piles driven to refusal on bedrock when bedrock is within an economical depth for H-piles to be used. Additionally, ODOT does not use precast concrete piles. Therefore, pile setup is experienced mainly on CIPP piles in ODOT projects.

It should be noted that the pile driving sequence was determined based on the load test reports and covered only the piles tested. Therefore, the pile identified as the first pile driven on a substructure may not reflect the actual conditions in the field.

The final database included 87 piles from 59 projects across the State of Ohio. There were a total of 109 restrike tests in the database, due to the multiple restrikes performed on some tested piles.

3.3.1 Project Locations

The projects in the database were located across Ohio as shown in Figure 2. The projects were located in 28 counties in Ohio. Hamilton County was the largest contributor to the database with 11 projects (19% of total projects) and 21 piles (24% of total piles). The distribution of projects and piles per county are shown in Table 2 and Figure 3. The marked locations in Figure 3 are representative locations for each county, not the actual project locations.

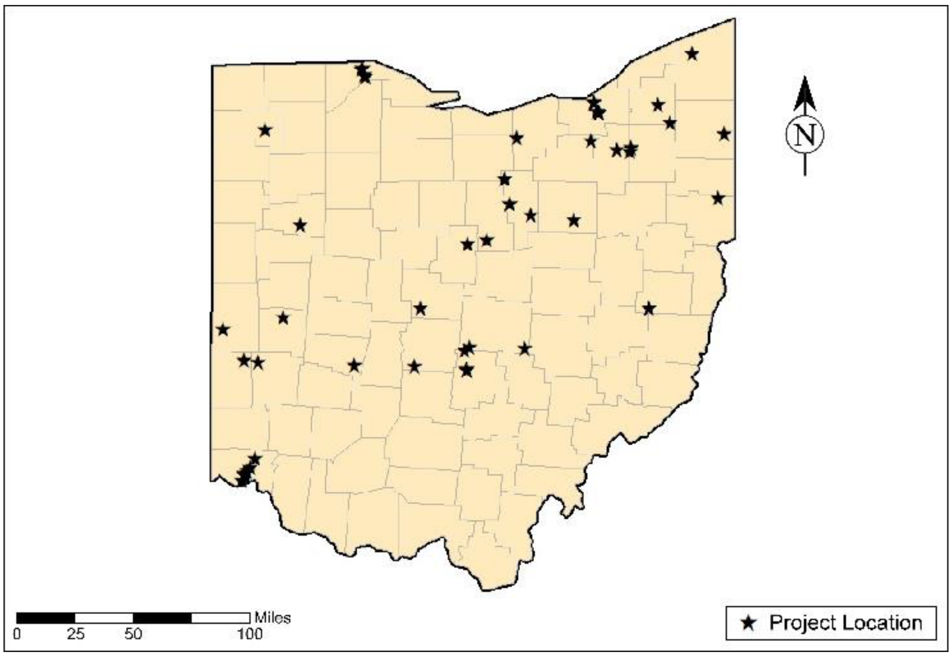

Figure 2. Location map of Ohio project sites

There were not any projects located in south-central or southeastern Ohio. This part of the state was not glaciated, and rock is relatively shallow. Consequently, foundations for transportation projects in south-central and southeastern Ohio generally consist of steel H-piles driven to refusal on rock or drilled shafts socketed into rock.

Table 2. Distribution of number of projects and piles per county*

County	Number of Projects	Number of Piles	County	Number of Projects	Number of Piles
Allen	2	2	Licking	2	2
Ashland	4	6	Lorain	1	1
Ashtabula	1	1	Lucas	7	8
Clark	1	1	Mahoning	1	2
Columbiana	1	1	Medina	1	1
Cuyahoga	6	17	Miami	1	1
Darke	1	1	Montgomery	1	1
Defiance	1	1	Morrow	1	1
Delaware	1	1	Portage	1	1
Fairfield	4	6	Preble	1	1
Franklin	1	1	Richland	1	1
Geauga	2	2	Summit	1	2
Hamilton	11	21	Trumbull	1	1
Harrison	2	2	Wayne	1	1

* CIPP piles driven in predominantly cohesive soils

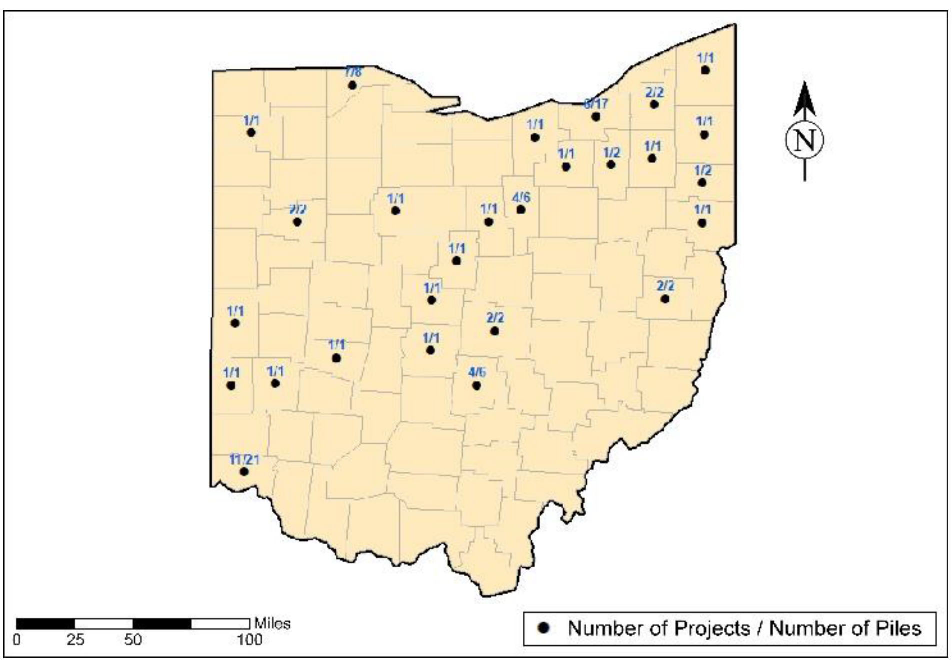

Figure 3. Number of projects and piles per county in Ohio

3.3.2 Database Statistics

The pile and soil property data are summarized in Table 3. The soil properties are weighted averages based on soil layer thicknesses along the pile length. Pile resistance-related data are also summarized in the table. Most of the parameters have 87 data points, which was the total number of piles included in the database. Due to the multiple restrikes on some of the piles, there are 109 data points. The minimum, maximum, mean, median, standard deviation, and the coefficient of variation for the parameters considered are provided in Table 3. As shown in Table 3, the pile and soil properties had wide ranges that potentially would affect pile setup behavior. Detailed statistics of the database are provided in Appendix C.

Table 3. Summary of data collected for pile setup analysis

Parameter	No. of Data	Min.	Max.	Mean	Median	Std. Dev.	COV (%)
Pile Properties:							
Pile diameter, D (in)	87	12.00	18.00	14.44	14.00	1.95	13.53
Pile length, L (ft)	87	15.00	190.00	64.63	55.00	35.17	54.42
Soil Properties:							
Moisture content, w (%)	87	8.67	32.33	19.80	19.55	4.97	25.12
Liquid limit, LL (%)	85	16.59	49.07	29.09	29.45	5.80	19.95
Plastic limit, PL (%)	85	10.41	21.20	17.94	18.00	3.14	17.49
Plasticity index, PI (%)	85	4.24	29.28	11.19	10.97	4.04	36.07
SPT-N60 (blows/ft)	87	6.00	57.66	22.77	21.12	8.99	39.47
Fine fraction, F_{fine} (%)	87	42.37	100.00	72.45	74.50	16.33	22.54
Silt fraction, F_{silt} (%)	87	0.00	75.00	39.31	37.62	12.10	30.77
Clay fraction, F_{clay} (%)	87	3.68	67.84	32.82	32.18	14.01	42.69
Pile Resistance:							
UBV (kips)	87	76	1,100	371	291	275	74
Q_{EOID} (kips)	87	47	830	222	204	138	62
Q(t) (kips)	109	85	1,672	447	350	321	72
Restrike time, t (days)	109	0.04	57.00	7.56	3.94	11.39	150.62

3.4 Data Analysis

In this report, the change in pile resistance with time is presented as setup ratio, R, and it is given as:

$$\text{Setup Ratio, } R = \frac{Q(t)}{Q_{EOID}} \tag{2}$$

where $Q(t)$ is the pile resistance at any time, t, obtained from a restrike test and Q_{EOID} is the resistance at the end of initial drive (EOID). A setup ratio, R, greater than one indicates increase in pile resistance over time, i.e., pile setup. The setup ratio is similar to the setup factor used by FHWA GEC-12 (Hannigan et al. 2016) and the ODOT Bridge Design Manual (2021), based on Rausche et al. (1996) study. The difference between the setup ratio and setup factor is that the setup factor is based on long term pile resistance. On the other hand, the setup ratio is based on pile resistance at any time after the end of initial drive. Basically, the setup factor is the long term or ultimate setup ratio.

For many piles in the database, restrikes were performed shortly after the initial drive. As shown in Table 3, the median restrike time was less than four days, i.e., half of the restrikes were performed within four days. For fine-grained soils, i.e., silts and clays which are the soils covered in this report, AASHTO LRFD Bridge Design Specifications (2020) recommends a 7 to 14-day waiting period before a restrike can be performed following pile installation. The specifications also note that delay periods even longer than those recommended are sometimes required for fine-grained soils. In practice, restrike times are usually much shorter than the AASHTO recommended 7 to 14-day waiting period due to the tight construction schedules. Because the economic cost of longer waits outweighs the economic cost of ordering and driving more pile.

The change in pile side resistance with time was also investigated and it is presented as side setup ratio, R_s:

$$\text{Side Setup Ratio, } R_s = \frac{Q_s(t)}{Q_{s,EOID}} \tag{3}$$

where $Q_s(t)$ is the pile side resistance at any time, t, obtained from restrike test and $Q_{s,EOID}$ is the resistance at the end of initial drive (EOID). Both $Q_s(t)$ and $Q_{s,EOID}$ were obtained from the CAPWAP analysis results.

CAPWAP analysis results were used as the basis for pile total and side resistances in the database. For piles where CAPWAP analysis was not performed, the CASE method results were used for total resistance. The side setup data could not be obtained for those cases since only the total resistance can be obtained by the CASE method. While the total setup ratio was available for 87 piles, side setup ratio was available for 58 piles from the CAPWAP analysis. The total and side setup ratios observed for the piles in the database are shown in Figure 4. There is a large scatter in both setup ratios shown in Figure 4, because these data came from many projects with varying pile and soil properties rather than controlled field tests.

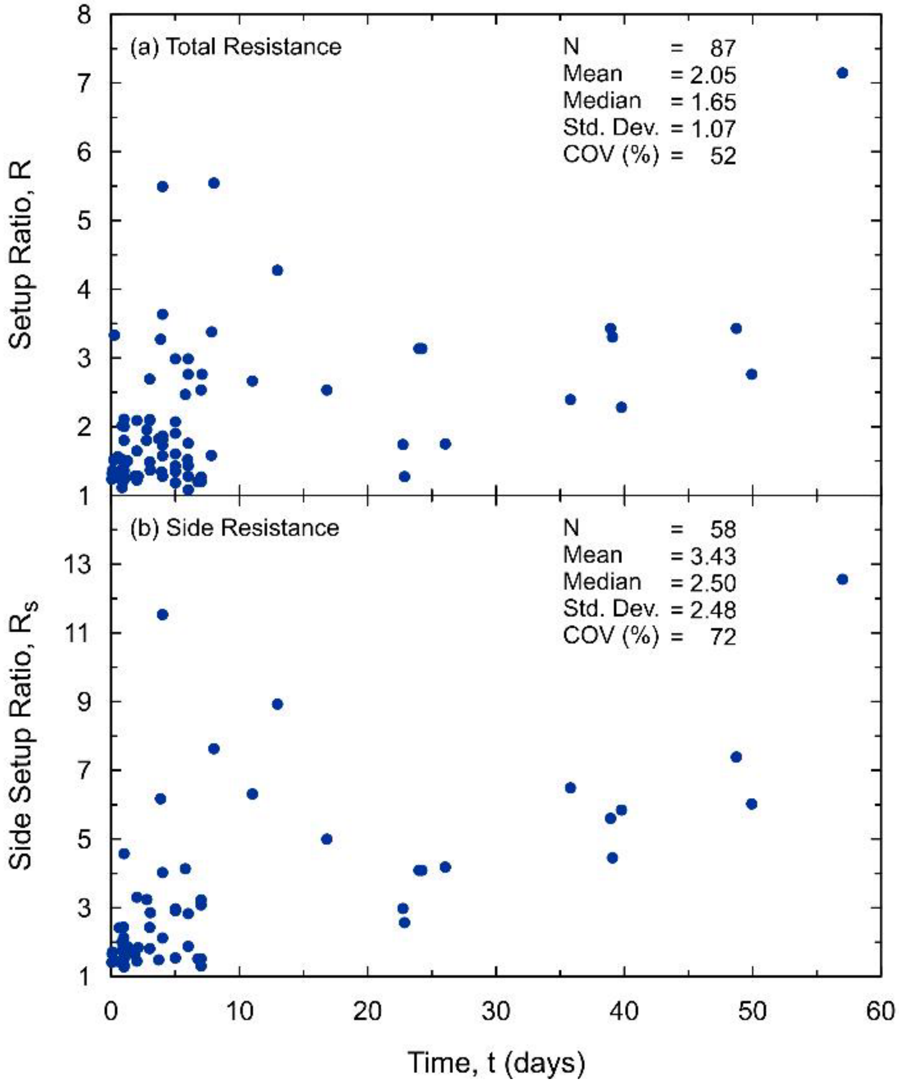

Figure 4. Setup ratios observed for piles in the database; (a) total resistance and (b) side resistance

The setup ratios, for both total and side resistances, were evaluated by comparing with soil properties in the database to investigate possible trends and correlations. Although some trends were observed, there were no good correlations observed. Some of the trends observed were: the setup ratio increases as the moisture content, liquid limit, or soil fine fraction increase; the setup ratio decreases as the soil strength increases (increasing SPT-N60 values). These trends were observed for both total and side setup ratios. Detailed setup ratio versus soil property data are provided in Appendix C for both total and side setup.

3.5 Analysis and Formulation of Pile Setup

3.5.1 Evaluation of Existing Pile Setup Models

There have been several empirical, analytical, and numerical methods proposed by researchers for estimating pile setup. These methods are almost exclusively for the setup of pile total resistance, not for side resistance setup. A recent study by Haque et al (2016a) investigated side setup and proposed models which are based on cone penetration test results and undrained shear strength of soil.

The applicability of several pile setup models to Ohio soils was investigated. Four of the methods listed in Appendix A were selected for this purpose. The Skov and Denver (1988) model was selected because it is the most commonly cited and used pile setup model. The Khan and Decapite (2011) model was included because it was developed by using soils in Ohio. The other two methods, Svinkin and Skov (2000) and Yan and Yuen (2010), were selected because of the availability of parameters used in those models in the database. The remaining models listed in Appendix A were not used for the evaluations primarily because not all the parameters needed for those models were available in the database.

The findings of the applicability evaluation of these existing pile setup models to Ohio soils are presented in Chapter 4.

3.5.2 Developing Pile Setup Prediction Models

Almost all the existing pile setup models are based on predicting pile resistance over time following installation and require the end of initial drive soil resistance obtained from dynamic testing. The ODOT Bridge Design Manual (2021) also requires restrike testing to verify setup when pile setup is accounted for during design.

Developing potential pile setup models for Ohio soils was investigated using the database. Correlations of pile setup with time, soil, and pile properties were analyzed. Several statistical software packages were used as tools to identify significant parameters and develop pile setup models. Numerous multiple regression analyses were performed using the readily available statistical software JMP to determine the significant parameters contributing to pile setup behavior. JMP was developed by SAS Institute Inc. of Cary, North Carolina.

The parameters included in the multiple regression analyses were: pile diameter, D, and length, L; soil moisture content, w, liquid limit, LL, plastic limit, PL, plasticity index, PI, standard penetration test N60, total fines percentage, silt and clay fraction percentages; total and side resistances at EOID and restrikes; and restrike times. Some of these parameters are directly obtained from the documents, and some were calculated as average values along the pile length. Some additional parameters were calculated and included as data for the regression analyses. Surface area along the pile shaft, $A_s=\pi DL$, where side resistance occurs, soil volume displaced during pile installation, $V=\pi D^2L/4$, and percentage of side resistance at the time of restrike, $SRP=Q_s(t)/Q(t)\times100$, were calculated for each pile and added to the database.

Several empirical models incorporating soil and pile properties were developed to estimate the pile total and side resistance setup for Ohio soils. The models proposed for estimating the total and side resistances are presented in Chapter 4.

3.5.3 Establishing Setup Factors

Accounting for setup in design relies on setup factors. The setup factors for different soil types were developed by Rausche et al. (1996). These setup factors are also included in FHWA and ODOT manuals and specifications to account for setup during design. The setup factors developed by Rausche et al. (1996) were based on:

- Pile total resistance, i.e., side plus tip resistance,
- Predominant soil type at pile locations, i.e., soil type along each pile that contributed the most resistance, and
- Piles with more than 50% side resistance, i.e., tip resistance contribution was less than the side resistance contribution.

Rausche et al. (2004) indicated that it would be more reasonable to have two setup factors available: one for the side resistance and one for the tip resistance. Rausche et al. (2004), however, also indicated that while it is indeed more correct to separate the setup factors for side and tip, this is normally too difficult to accurately assess and therefore little accuracy would be gained with this more sophisticated approach.

Although the setup factors given by Rausche et al. (1996) were developed based on the pile total resistance, they were used as side resistance setup factors in guidelines and design manuals, e.g., FHWA GEC-12 (Hannigan et al. 2016) and the ODOT Bridge Design Manual (2021). Those factors have been cited in these and other documents, and are also applied by practicing engineers in projects as side setup factors rather than the setup factors for total resistance as developed originally.

Based on the setup ratios obtained from the data collected, the analyses were conducted to investigate and propose setup factors for total and side resistances for piles installed in predominantly fine-grained soils.

The analysis of setup factors for total and side resistances are presented in Chapter 4.

CHAPTER 4. RESEARCH FINDINGS AND CONCLUSIONS

The results and findings of the research project are provided in this chapter. The findings are presented in the following four main areas:

- Effect of construction activities on pile resistance,
- Applicability of existing pile setup models to Ohio soils,
- Proposed new pile setup models for Ohio soils, and
- Setup factors for Ohio soils.

4.1 Effect of Construction Activities on Pile Resistance

Assessment of pile resistance due to a disturbance caused during initial driving should be performed after equilibrium conditions have been re-established to have an accurate assessment. While the main focus is usually on the disturbance caused by the pile installed, other construction activities, such as installation of nearby piles can also affect the soil conditions and pore water pressures. As a result, the time required for equilibrium conditions to return would be affected and therefore should be considered in the assessment of pile resistance. "Construction activities" consisting of the installation of other piles was investigated and covered in this report. However, other activities that can cause disturbance and affect the soil layers and pore water pressures could have similar effects.

The HAM-75 site was the first field project of this research study. It helped to observe and better understand the effect of construction activities on pile resistance obtained from dynamic load tests. The project revealed how a pile resistance can be affected not only during restrike testing but also during initial driving due to the pile installation activities at a site. The findings of investigations at the HAM-75 site is summarized in the following text and covered in detail in Appendix B.

Before the piles at the site were driven, three vibrating-wire piezometers were installed at a depth of 24.6 ft. This depth was selected because of the presence of soil layers with potentially high setup ratios. These piezometers were located 2, 5, and 10 ft from the static load test pile.

Four piles were driven and dynamically tested over a five-hour period at the HAM-75 site (refer to Figure 18 in Appendix B for the pile layout plan). All driving system details were the same for all four piles. The second pile, driven 16 ft away from the first pile, required 25% greater length to reach a resistance similar to the first pile. The fourth pile was driven within 30 ft of all the previous three piles and only 8 ft away from the third pile driven. The fourth pile had more than 50% lower resistance during the initial drive compared to the other piles. The pore pressure readings indicated significant increases during the installation of test piles and nearby reaction piles. Interestingly, the pore pressure within 2 ft of the test pile dropped to normal levels within a day or two of the pile driving, while the piezometers further from the test pile were slower to return to normal (refer to Figure 13 in Appendix B for the piezometer readings at the HAM-75 site).

Two restrike tests were performed on all four piles, at 9 and 17 days after the EOID. During the 9-day restrikes, all piles gained resistance as expected. The fourth pile exhibited the largest gains in part due to the lower resistance observed during initial drive. On the 17-day restrikes, the third and fourth piles showed continued increases in resistance, reaching similar resistance values. Although there was more than 100% resistance difference during initial drive between these two piles, i.e., third and fourth piles, the difference after 17 days was about 10%, indicating that equilibrium conditions in the soil are being approached. The similar resistances also indicate that the soil conditions were similar at the location of these two piles, which were 8 ft apart and practically had the same length. The 100% difference in resistance observed during the initial drive was due to the construction activities, not different soil conditions.

The first and second piles, however, experienced lower resistance during the 17-day restrikes compared to the previous restrikes at 9-days. The resistance of the first pile during the 17-day restrike was lower than the EOID resistance, suggesting pile relaxation. However, the lower resistances obtained from the dynamic load tests at 17-day restrikes were due to the other construction activities at the site. About 10 to 16 hours prior to the 17-day restrikes, five piles were installed around the first and second piles. These additional pile installations introduced new soil disturbance and increase in pore pressures which were in the process of being re-established after the installation of the first and second piles.

The construction activities (installation of other piles) affected the pile resistances obtained from load tests such that the setup ratios for these four piles ranged between 0.90 and 2.53, which are not true setup ratios, or setup factors, as discussed above. The LUC-75 and SUM-76 test sites were also instrumented with piezometers to further investigate the effect of construction activities. The piezometers at both sites also observed significant increases in pore pressures during pile driving. The observations confirm the fact that the construction and pile driving activities at a site can significantly affect the true assessment of pile resistance obtained from load tests.

The investigations showed that the dynamic testing and pile restrikes should be planned and performed on piles such that the influence of other pile installations are eliminated, or at least minimized. This can be achieved by performing dynamic load testing on the first pile driven at a site and by avoiding major construction activities, such as other pile installations, until the restrike tests are performed. However, this may be difficult to implement in many projects due to tight construction schedules.

Additional details of the effects of construction activities on the load test results are provided in Appendix B.

4.2 Applicability of Existing Pile Setup Models

The following four existing empirical pile setup models were investigated for their applicability to Ohio soils:

- Skov and Denver (1988) : $\dfrac{R_t}{R_{EOID}} = A\log\left(\dfrac{t}{t_0}\right)+1$

- Svinkin and Skov (2000) : $\dfrac{R_t}{R_{EOID}} = B\left[\log(t)+1\right]+1$
- Yan and Yuen (2010) : $\dfrac{R_t}{R_{EOID}} = 1 + C\log(1+t)$
- Khan and Decapite (2011) : $R_t = 0.9957 R_{EOID} t^{\alpha}$ (where, $\alpha = 0.087$)

where, R_t is pile resistance at time, t, after EOID; R_{EOID} is pile resistance at EOID; A, B, and C are soil type dependent setup constants; and t_0 = reference time (refer to Table 5 in Appendix A for details). The measured pile total resistances, Q_m, during restrike plotted against the predicted pile total resistances, Q_p, obtained using the empirical equations of these models are presented in Figure 5. The results in Figure 5 show that Skov and Denver (1988), Yan and Yuen (2010), and Khan and Decapite (2011) models significantly under predict the actual pile capacities observed during restrike tests in Ohio. The Skov and Denver (1988) and Yan and Yuen (2010) models yield similar results and both underestimate the pile setup by approximately 20%. The Khan and Decapite (2011) model results in the largest under prediction of pile setup, by 38%. The Svinkin and Skov (2000) model provides the closest predictions with about 10% over prediction, but with 43% coefficient of variation. All these methods have very large scatters in predicted time dependent pile resistances, as shown in Figure 5, and have coefficients of variation of 42% and higher.

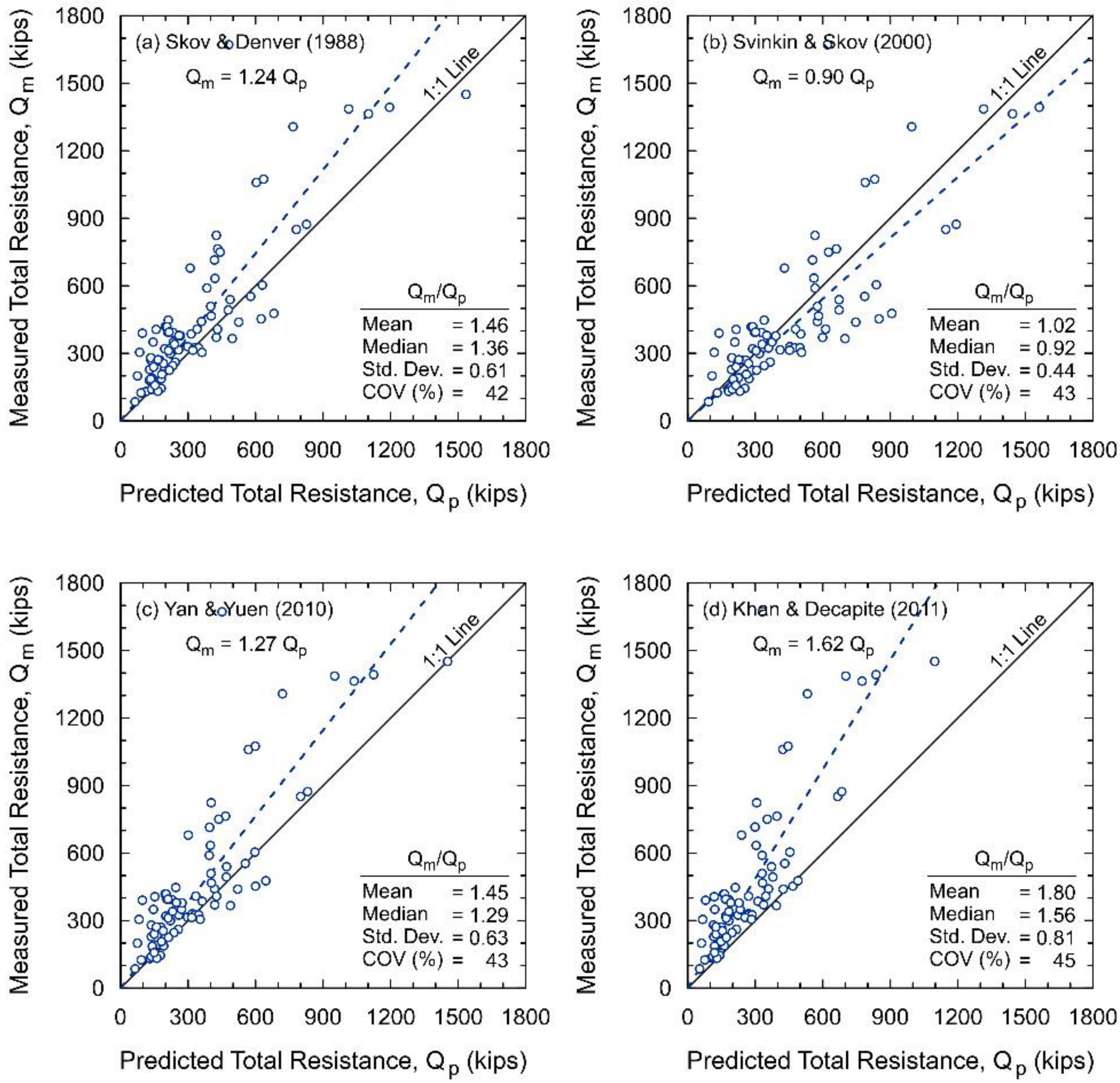

Figure 5. Measured versus predicted pile resistances using existing setup models

Some previous studies, such as Yang and Liang (2006), concluded that the Skov and Denver (1988) empirical relationship can be used to predict setup for driven piles in clay. Their study included various pile types from different geographical locations. Based on the findings of this project as shown in Figure 5(a), Skov and Denver (1988) empirical model is not suitable to predict setup for closed-end CIPP piles driven in fine-grained soils of Ohio.

4.3 Proposed New Pile Setup Models for Ohio Soils

New models to predict pile total and side resistance over time were developed by using the database information. Numerous models, amounting to several hundred, have been investigated and analyzed to optimize the setup prediction models. The best models developed are presented in this section, two for total resistance and two for side resistance. These models are used to predict pile resistance gain after the pile is installed and the initial driving resistance is obtained. They should not be used to predict driving losses. The models were developed with the pile resistance at the end of initial drive, Q_{EOID} or $Q_{s,EOID}$, being an independent variable. Because of the scatter in the data used to develop the models, reverse use of the models to predict driving loss could result in unstable solutions in some cases.

4.3.1 Setup Models for Total Resistance

The multiple regression analyses helped to identify significant parameters for time-dependent pile resistance. All models showed that the two most significant parameters were the resistance at the end of initial drive and time. In addition to these two parameters, the first model used pile shaft surface area and silt fraction as the additional significant variables, and the second model used the soil volume displaced during pile installation and percentage of side resistance as the additional significant variables. The new empirical setup models developed for total resistance are:

<u>Setup Model #1:</u>

$$\begin{aligned} Q(t) &= f\left(Q_{EOID}, t, A_s, F_{silt}\right) \\ Q(t) &= Q_{EOID} + t^{1.665} + \left(0.26\ A_s\right) + \left(t^{0.333}\ F_{silt}\right) \end{aligned} \tag{4}$$

<u>Setup Model #2:</u>

$$\begin{aligned} Q(t) &= f\left(Q_{EOID}, t, V, SRP\right) \\ Q(t) &= \left[Q_{EOID} + \left(2\ t^{1.5}\right) + 123.31\right] \log\left[\left(V^2 + SRP^{1.89}\right)^{0.25}\right] \end{aligned} \tag{5}$$

where $Q(t)$ is pile total resistance at any time, t, after EOID (resistance in kips and time in days), Q_{EOID} is pile total resistance at (EOID) (kips), A_s is area of pile shaft (ft^2), F_{silt} is silt fraction of soil (%), V is soil volume displaced during pile installation (ft^3), and SRP is percentage of side resistance (%).

The measured pile total resistance, Q_m, during restrikes is plotted against the predicted pile total resistance, Q_p, by using the two proposed models given in Eq. 4 and Eq. 5. The results are presented in Figure 6, and they show that the proposed models can predict pile total resistance with a coefficient of determination, R^2, of 0.98.

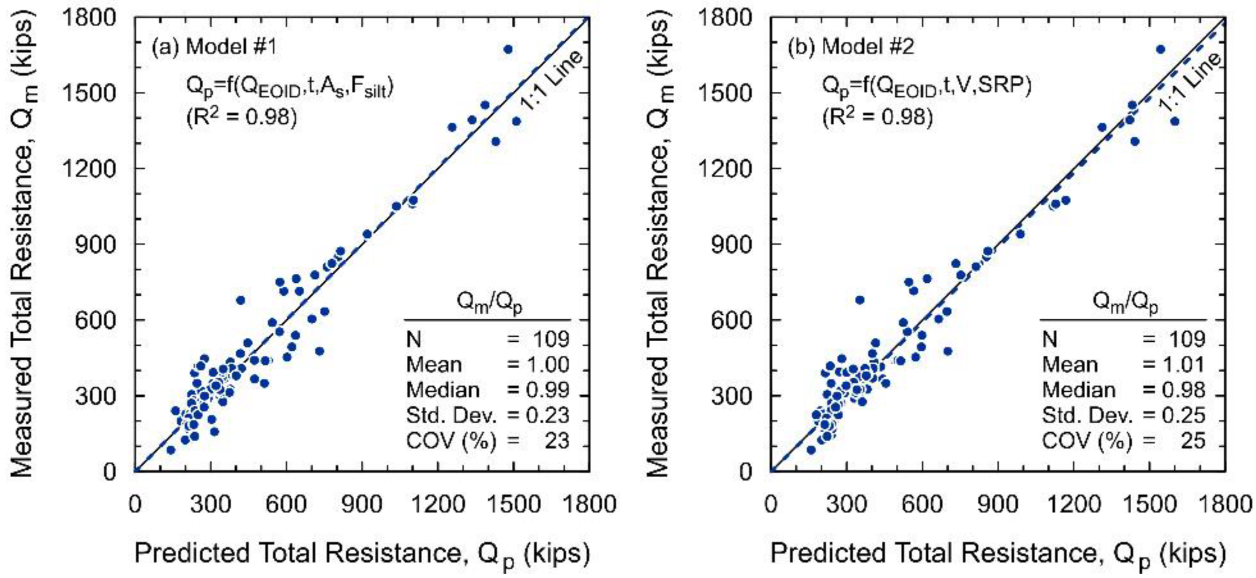

Figure 6. Measured versus predicted pile total resistances using proposed setup models

The data in Figure 6 show that both models predict the time-dependent pile total resistance very well, with very high coefficient of determination $R^2 = 0.98$ and with relatively low coefficient of variation of (COV) about 25%.

The results showed that both models may over-predict the total resistance when piles have less than 200 kips resistance. Therefore, these models should only be used for piles with more than 200 kips of predicted resistance.

4.3.2 Setup Models for Side Resistance

The side resistance at the end of initial drive and time were also the most significant parameters identified for the side resistance setup models, similarly to the models for total resistance. In addition to these two parameters, soil volume displaced during pile installation, and soil moisture content were the other two influential parameters in both models. The difference between the two models was the fifth parameter; while the first model used clay fraction, the second model used plasticity index. These new empirical setup models developed for side resistance are:

Setup Model #1:

$$Q_s(t) = f\left(Q_{s,EOID}, t, V, w, F_{clay}\right)$$
$$Q_s(t) = 0.98\left[Q_{s,EOID} + \left(V - F_{clay}\right) + \left(t + w\right)\right]^{0.5}\left[10.85\,w + \left(Q_{s,EOID} - F_{clay}\right) + t^2\right]^{0.5} \tag{6}$$

Setup Model #2:

$$Q_s(t) = f\left(Q_{s,EOID}, t, V, w, PI\right)$$

$$Q_s(t) = 0.98\left(Q_{s,EOID} + \frac{t^2}{PI - 3.311} + V + 62.33\right)\log\left[\left(w+t\right)\frac{t}{V} + 6.404\right] \tag{7}$$

where $Q_s(t)$ is side resistance of pile at any time, t, after EOID (resistance in kips and time in days), $Q_{s,EOID}$ is side resistance at the end of initial drive (EOID) (kips), V is soil volume displaced during pile installation (ft^3), w is soil moisture content, F_{clay} is clay fraction of soil (%), and PI is soil plasticity index (%).

The measured pile side resistance, $Q_{s,m}$, during restrikes is plotted against the predicted total side resistance, $Q_{s,p}$, by using the two proposed models given in Eq. 6 and Eq. 7. The results are presented in Figure 7, and they show that the proposed models can predict pile total resistance with a coefficient of determination, R^2, of 0.97.

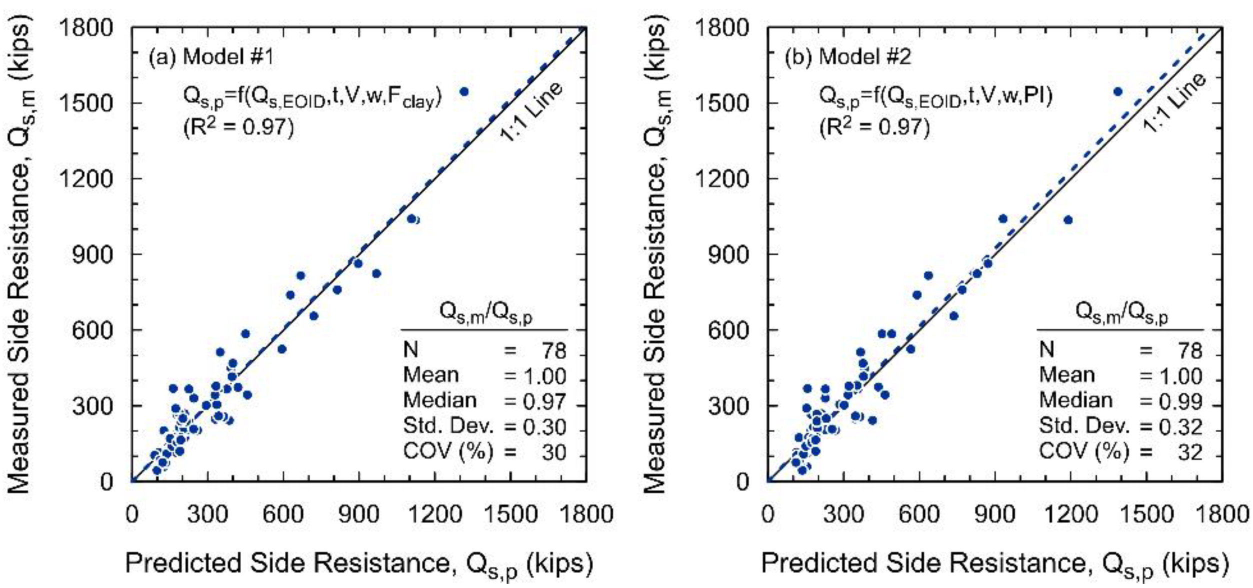

Figure 7. Measured versus predicted pile side resistances using proposed setup models

Both models can predict the time-dependent side resistance of piles very well, with very high coefficient of determination $R^2 = 0.97$ and with relatively low coefficient of variation (COV) of about 30%, as shown in Figure 7. Similarly to the setup models proposed for total resistance, these setup models for side resistance should only be used for piles with 200 kips or more total resistance.

4.4 Setup Factors for Ohio Soils

The setup ratios, for both total and side resistances, for the 87 piles in the database were shown in Figure 4. The range of restrike times after the end of initial drive ranged from 0.04 days (almost one hour) to 57 days, with an average of 7.56 days and median of 3.94 days. Based on the restrikes performed within this wide time range, the average and median of setup ratios for pile total resistance were 2.05 and 1.65, respectively. For the side resistance of piles, the average and median setup ratios were 3.43 and 2.50, respectively. In other words, half of the piles in the database experienced more than 65% increase in total resistance and 150% increase in side resistance.

These setup ratios do not represent setup factors to predict driving losses, because they are not long-term or ultimate setup ratios. Half of the restrikes in the database were performed within four days following the installation of piles. Only 26% of the restrikes were performed after seven days, which is the minimum waiting period recommended by the specifications (AASHTO 2020) for silty and clayey soils, the soil types investigated in this project.

The setup ratios have been analyzed based on various restrike times to provide recommendations for setup factors. The results are summarized in Table 4. The effect of restrike times on the setup ratios are also presented in Figure 8. It is important to note that the setup ratios in Table 4 and Figure 8 are based on the overall pile resistance, not individual soil layers along the pile length.

Table 4 and Figure 8 show that both total and side resistance setup ratios are increasing as the short duration pile restrike times are eliminated from the cluster analysis. For the piles with seven-day or higher restrike time, which is the minimum waiting period recommended (AASHTO 2020), the average setup ratio for total resistance was 2.81 and for side resistance was 5.16. These numbers are 2.95 and 5.48 for the piles with 14-day or higher restrike times.

The setup ratios observed in this project are compared to the setup factors provided by FHWA-NHI-16-009 based on the work performed by Rausche et al. (1996). The study by Rausche et al. (1996) was based on a wide range of pile types, pile sizes, pile materials, and most importantly a broad range of geologic deposits. Only the setup factors for fine-grained soils were used for comparisons since these materials were investigated in this project. The study by Rausche et al. (1996) included 19 sites nationwide (Table 1) and this project included 59 projects in the State of Ohio. There were 86 piles used for comparison purposes, one outlier data point was not considered in comparisons. The results are presented in Figure 9. For the setup ratios obtained using pile total resistances, there is an excellent match with the results obtained in this study compared to the observations made by Rausche et al. (1996) and recommended by FHWA-NHI-16-009. When the setup ratios for the side resistance is compared to the total resistance setup values, the range is almost doubled with a significant increase in the average value as shown in Figure 9. The figure does not include the side resistance setup ratio for Rausche et al. (1996), because that study only used the total resistances to investigate setup ratios, or setup factors.

Table 4. Summary of setup ratios based on various restrike time criteria

Criteria[a]	Total/Side Resistance	No. of Piles	Min.	Max.	Mean	Median
$t > 0$ (All data)						
	Total, R	87	1.08	7.15	2.05	1.65
	Side, R_s	58	1.28	12.56	3.43	2.50
$t \geq 1$ day						
	Total, R	71	1.08	7.15	2.17	1.80
	Side, R_s	48	1.28	12.56	3.76	2.97
$t \geq 7$ days						
	Total, R	23	1.20	7.15	2.81	2.67
	Side, R_s	20	1.31	12.56	5.16	4.73
$t \geq 14$ days						
	Total, R	13	1.27	7.15	2.95	2.76
	Side, R_s	13	2.57	12.56	5.48	5.00
$1 \leq t < 7$ days						
	Total, R	64	1.08	5.49	1.77	1.51
	Side, R_s	38	1.28	11.53	2.52	1.85
$7 \leq t < 14$ days						
	Total, R	10	1.20	5.55	2.64	2.60
	Side, R_s	7	1.31	8.93	4.57	3.24

[a] t is pile restrike time after EOID

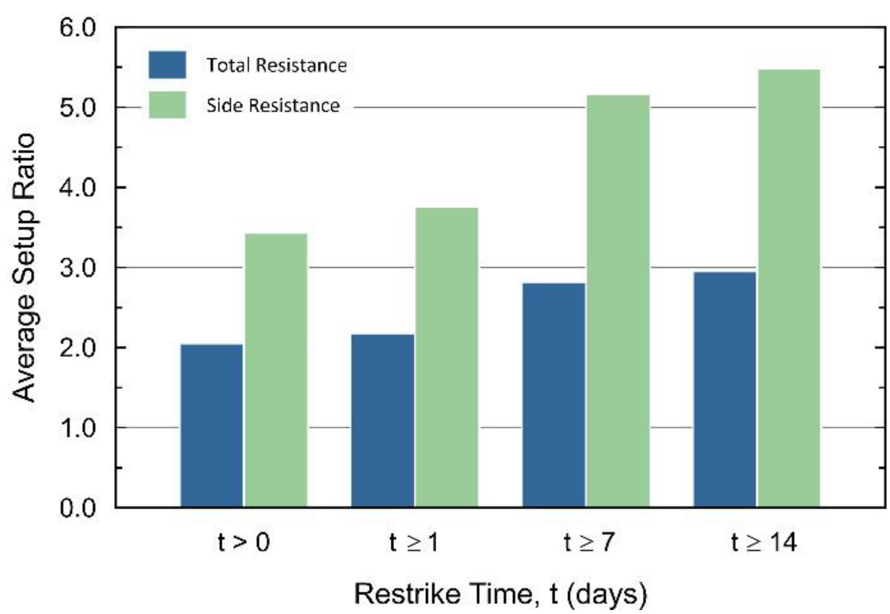

Figure 8. Average setup ratios for restrike time clusters

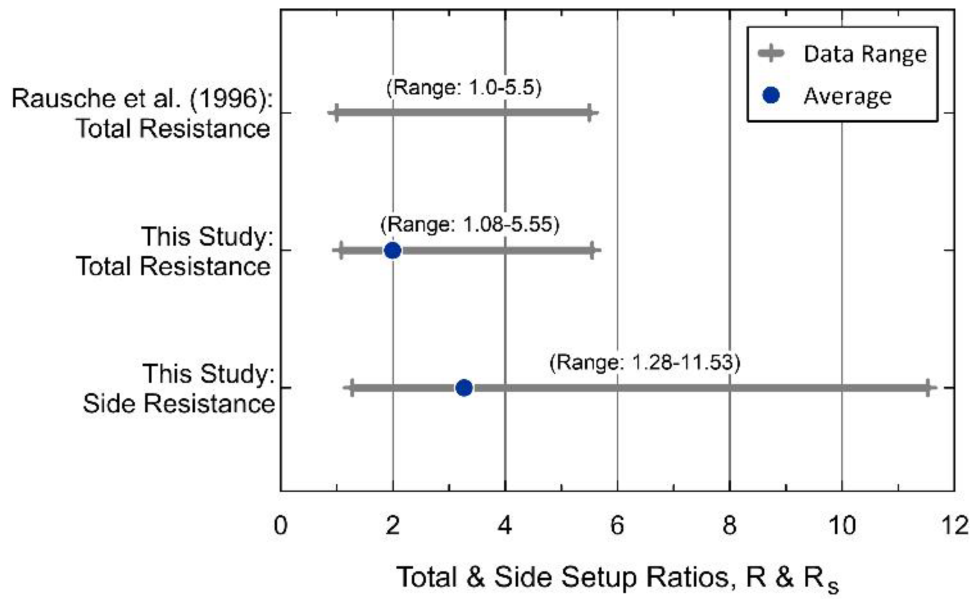

Figure 9. Comparison of setup factors

Additional analyses of setup ratios were performed on 1) piles with multiple restrikes and 2) soil layers along the pile shaft of some individual piles. The results of these analyses are provided in Appendix D.

Based on the data presented here and in Appendix D, pile driving setup factor of 2.00 for the total resistance and 3.00 for the side friction are recommended for CIPP piles driven in fine-grained Ohio soils, with better than 95% confidence levels.

4.5 Conclusions

This study was performed on CIPP piles driven in predominantly fine-grained soils (more than two-thirds of pile length is in fine-grained soil layers, i.e., percent passing #200 sieve is more than 35%). Subsurface investigations and site characterization are important in properly identifying soil layers and obtaining soil properties at a project site to accurately implement the findings of this project. In addition, an improved and adequate site characterization generally reduces risks associated with design, construction, and operation of transportation infrastructure by reducing likelihood of encountering differing site condition claims, increasing reliability of estimated soil and rock properties, and decreasing uncertainty of subsurface conditions during construction.

The main conclusions drawn from this project are:

- The construction activities and installation of other piles at a project site can significantly affect the true resistance assessment of the piles tested,

- The most commonly cited and used Skov and Denver (1988) pile setup model, and several other existing models analyzed, are not suitable to predict setup for closed-end CIPP piles driven in fine-grained soils in Ohio. Most of them significantly underestimate pile setup (i.e., overconservative estimate of resistance) and all have large variability in predicting the pile resistance gains,

- If a pile tested during driving is not the first pile driven at the site, and the dynamic test results show lower resistance than anticipated, the pile should not be driven longer than the estimated design length just because the load test shows low resistance,

- Setup-related time-dependent total and side resistances of driven piles can be reasonably predicted well after the end of initial drive with the new models proposed. The proposed models depend on both pile- and soil-related parameters along with the pile resistance measured during installation and time passed since the pile installation,

- For setup of pile **total resistance**, the current setup factors recommended for fine-grained soils by FHWA GEC-12 and the ODOT Bridge Design Manual, which were based on a wide range of pile types, pile sizes, pile materials, and most importantly a broad range of geologic deposits, are in very good agreement with the data obtained from projects in Ohio,

- For setup of pile **side resistance**, the current setup factors recommended for fine-grained soils by FHWA GEC-12 and the ODOT Bridge Design Manual, are significantly lower than what piles driven in Ohio soils experience. The setup factors given by Rausche et al. (1996) were developed based on total resistance of piles and use of them as setup factors for side friction results in underestimated driving losses,

- For CIPP piles in fine-grained Ohio soils, a pile driving setup factor of 2.00 for the total resistance and 3.00 for the side friction are recommended with more than 95% confidence levels, and

- The new proposed setup factors, for both total and side resistances, will help ODOT to better account for pile setup during design and predict setup observed in the field.

CHAPTER 5. RECOMMENDATIONS FOR IMPLEMENTATION

***Key recommendations for implementation*:**

- Dynamic testing should be performed on the first pile driven at the site. If there were recent construction activities that have an effect on the subsurface pore water pressure at the site prior to driving the first pile, such as fill placement or preloading, the possible effect of those activities on the subsurface conditions and on the dynamic test results should be evaluated.

- Dynamic load testing on the second pile driven at the site should be performed at least seven days later, preferably 14 days, or on the pile furthest, preferably at least 100 ft, away from the first pile. If these time or distance recommendations cannot be followed at the site, any unexpected dynamic load test results from second or consecutive piles driven at the site should not be used to make decisions on extending pile lengths.

- Pile restrikes should be performed at least 7 days, preferably 14 days later, on piles driven in predominantly fine-grained soils.

- The existing empirical pile setup equations, such as Skov and Denver (1988) model, should not be used for the CIPP piles driven in fine-grained soils of Ohio. Based on the data collected, this model and several others assessed usually underestimate the pile setup and may result in additional unnecessary pile length to meet the resistance.

- The current setup factors for side friction recommended by the ODOT Bridge Design Manual, 2020 Edition (2021) should be updated. For the CIPP piles in fine-grained Ohio soils, pile driving setup factors of 2.00 for the total resistance and 3.00 for the side friction are recommended with more than 95% confidence levels. There was insufficient data and evidence to recommend different setup factors for different fine-grained soil types. These setup factors are recommended for all soils classified as fine-grained based on ODOT and AASHTO soil classifications.

***Expected benefits from implementation*:**

Realization of pile setup in design with use of recommended setup factors and improved pile setup prediction models will result in pile quantity savings and prevent construction delays. Their use can also help avoid change orders to furnish and drive additional pile lengths and construct pile splices.

***Potential risks to implementation and strategies to overcome*:**

The recommended setup factors and pile setup prediction models are based on the data collected from projects across the State. However, there are possible conditions that may not have been captured within the data analyzed. In addition, the models are based on existing data and empirical approaches. The setup factors selected are based on better than 95% confidence levels. It is likely that there will be projects where the observations in the field will be different than what is predicted during design. However, due to the 95% confidence levels used for setup factor recommendations, it is anticipated these situations will be very limited.

The use of restrike tests, which are already addressed in the ODOT Bridge Design Manual, would overcome any potential risks, if any unexpected behavior is observed in the field.

***Suggested timeframe for implementation*:**

ODOT already implemented one of the major findings of this project in the Construction and Materials Specifications (C&MS) by requiring dynamic testing to be performed on the first pile driven at the site.

The use of the recommended setup factors and pile setup prediction models can begin as soon as ODOT is ready to implement them.

***Additional recommendations*:**

It is recommended that ODOT communicate with FHWA and suggest updates to the setup factors in their manuals. The factors in their manuals used for side setup were developed based on pile total resistance, not for the side resistance as currently used in FHWA manuals.

It is recommended that ODOT consider investigations to revise setup factors listed in the Bridge Design Manual for coarse-grained soils, since the setup factors in the manual were not originally proposed for side friction.

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APPENDICES

- APPENDIX A: Detailed Literature Review
- APPENDIX B: Field Projects and Effect of Construction Activities
- APPENDIX C: Detailed Database Statistics and Analysis
- APPENDIX D: Analysis of Setup Ratios for Setup Factors
- APPENDIX E: Pile Setup Database

APPENDIX A: Detailed Literature Review

The axial geotechnical resistance of driven piles may change following installation. An increase in the axial geotechnical resistance with time is referred to as setup, and a decrease is commonly called relaxation. Other terms such as pile freeze or side shear setup are also used to refer to pile setup. It is important to note that these time-related changes are permanent and therefore must be accounted for in the design and installation phases of all driven pile projects.

Pile setup was first documented in the literature in the year 1900 by Wendel. When appropriately accounted for during the design phase of a project, the integration of pile setup can lead to more cost-effective pile design as it can reduce the pile length, pile section, and size of driving equipment.

On modern day projects pile setup can be measured during construction by performing dynamic monitoring using a Pile Driving Analyzer (PDA) during both initial driving and restrike driving. Restrike dynamic tests are typically performed after several hours, days, and up to few weeks after initial driving. For projects with a large number of driven piles, the savings in pile costs significantly exceed the cost of testing needed to characterize setup. However, the extensive testing may not be economical for smaller projects (Tarawneh and Imam 2014) if setup is the sole reason for dynamic monitoring (dynamic monitoring is usually specified by owners to verify the axial geotechnical resistance and driven pile integrity).

Pile setup has been observed in a variety of pile types, pile sizes and a broad range of soil profiles (Budge 2009). A study on pile setup performed by Haque et al. (2014) showed 1.77 times increase in the geotechnical resistance of prestressed concrete piles driven in cohesive soils compared to the end of initial driving. Other case studies have demonstrated that pile setup can continue to develop for a long time following installation and can account for resistance increases of up to 12 times that of initial driving estimates (Titi and Wathugala 1999).

Additional studies have confirmed that considering setup during pile design routinely increases nominal pile resistance, reduces the number of piles and lengths (and potentially the number of splices), reduces pile sections, and allows for using smaller driving equipment or reduced installation time (Komurka et al. 2005).

Ng and Sritharan (2016) found that when pile setup is used in load and resistance factor designs (LRFD), foundation cost can be reduced by 1) reduced pile lengths, fewer piles, and/or smaller pile caps and 2) smaller installation systems (possibly with less fuel consumption), and reduced construction labor.

A.1 Pile Setup Mechanism

During the pile driving process, the soils surrounding the pile are significantly displaced, disturbed, and remolded. Also, excess pore pressure is generated in saturated soils. With the passage of time, the excess pore pressure dissipates; the soils gain strength, and the pile's permanent axial geotechnical resistance changes (increases or decreases).

Several researchers have recommended that setup should be included in the static analysis prediction methods used to determine pile geotechnical resistance during design. Bullock et al. (2005), for example, recommended use of a conservative design approach to include side resistance due to setup. In their approach, the predicted setup capacity was assumed to have the same degree of uncertainty as that of the measured capacity, and a single factor of safety was used to account for all uncertainties of loads and resistances. It is important to note that the majority of static analysis equations used in practice already include some percentage of setup depending on the time period of the static test following initial installation. This means that the time-related changes in geotechnical resistance (setup or relaxation) are already included in the static analysis models.

In other studies, it has been observed that the initial resistance of piles driven in dense sands and stiff clays can be significantly higher than long term capacity due to relaxation (Sawant 2013). Relaxation is caused by the development of negative pore pressures resulting in a temporary increase in effective stresses and shear strength of the soil during driving due to soil dilation.

As previously stated, setup is primarily associated with the increased side shear, or shaft resistance, over time occurring primarily due to the dissipation of excess pore pressure (Komurka et al. 2005). Ng and Sritharan (2016) reported a 55% increase within seven days in the resistance of steel H-piles driven in cohesive soils. Soils around the pile lose their strength during pile driving due to the increased pore pressures resulting in reduced effective stresses and remolding of soil structure. Strength gain of the disturbed soils surrounding the pile and increase in lateral soil stresses also contribute to the pile setup (Ng and Sritharan 2016). The contribution of tip resistance to setup has been found to be minimal compared to the side resistance. Haque et al. (2014) found that 90% of the total increase in the resistance of prestressed concrete piles driven in cohesive soils was due to side resistance. These studies also showed that closed-ended pipe piles and other types of displacement piles exhibited larger setup increases compared to small displacement piles, such as H-piles.

Komurka et al. (2003) suggested there are three stages of pile setup: 1) logarithmically nonlinear rate of excess pore pressure dissipation, 2) uniform, logarithmically linear rate of excess pore water pressure dissipation, and 3) aging. As summarized by Haque et al. (2014), the first two stages are similar to the consolidation process and the duration depends on the soil type, soil properties, pile properties, pile type, and pile size. The third stage is due to a combined effect of creep, particle interference, and clay dispersion. Other studies have shown that the soil type and properties affect the setup magnitude and rate, and therefore layered soils provide additional challenges in determining pile setup.

Lee et al. (2010) studied the results of 43 dynamic load tests using the Pile Driving Analyzer (PDA) conducted over a period of five months on H-piles and closed-ended pipe piles driven into layered soils. The existing soils primarily consisted of very loose to medium dense sand, sandy loam and soft to hard silty clay loam and silty clay. The groundwater table was relatively shallow at a 4 ft depth from the ground surface. The test results showed that the magnitude and rate of setup were quite different from those observed in other studies. The study concluded that pile setup with time in a layered soil profile was very minimal up to about 40 days after initial driving, and then started to increase.

A.2 Methods to Predict Pile Setup

Dynamic restrike tests and static load tests are used to account for pile setup. However, both are time-consuming, are conducted during the construction project phase and may not be feasible for some projects. It is desirable to account for setup during the design stage of a project.

There have been several empirical, analytical, and numerical methods proposed by researchers for estimating pile setup (e.g., Skov and Denver 1988; Svinkin 1996). The semi-logarithmic formula, proposed by Skov and Denver (1988), has been broadly used to predict the setup increase with time. Several of these methods and their respective limitations are summarized in Table 5.

Ng et al. (2011) performed a study to predict setup using the effect of pore pressure dissipation based on extensive field evaluations and pile setup measurements. The study concluded that setup can be satisfactorily estimated by using pile resistance at end of initial driving (EOID), soil properties, and pile geometry.

Chen et al. (2014) noted that the majority of the available empirical setup equations are based on a limited database and parameters; therefore, site-specific (or local) calibration is essential. Ng et al. (2013b) proposed a procedure for incorporating setup into load and resistance factor design (LRFD) of driven piles using a closed-form first-order second-moment (FOSM) framework.

Lim and Lehane (2015) investigated the effect of time on the shaft friction of displacement piles in sandy soils by conducting several pile load tests. The results revealed that side resistance decreased significantly due to pile installation disturbance. The study proposed a new model that accounts for aging characteristics.

Yang and Liang (2006) performed a study to incorporate the setup into a reliability-based LRFD approach for driven piles. A database of pile setup resistance compiled by Yang and Liang (2006) showed that the empirical equation proposed by Skov and Denver (1988) could be used to predict pile setup. It should be noted that Skov and Denver (1988) equation, provided in Table 5, is the most commonly used empirical equation to predict pile setup.

The pile load test data (both static and dynamic) collected by Yang and Liang (2006) showed that pile setup is significant, continues to occur for a long time after initial driving, but diminishes after 100 days following EOID. A normal distribution was shown to adequately represent the probabilistic characteristics of the predicted pile setup. Using the first-order reliability method (FORM), they developed separate resistance factors to account for different degrees of uncertainties associated with measured short-term resistance and predicted setup resistance at various reliability levels.

Predicting pile setup using artificial neural network and machine learning has also been investigated by researchers during recent years. Several studied showed that such methods can be used to model pile setup with (Jeon and Rahman 2007; Alkroosh and Nikraz, 2014; Tarawneh, 2018). However, these methods generally produce complex models which are quite difficult to adopt for use in practice.

A.3 Factors Affecting Pile Setup

The increase of pile geotechnical resistance with time depends on many factors including the soil type and the properties, type, and size of the driven pile.

A.3.1 Effect of Soil Type and Properties

Pile setup has been reported in a variety of soil types from cohesive to cohesionless soils. The soil types include organic and inorganic saturated clays, loose to medium dense silt, sandy silts, silty sands, and fine sand (Long et al. 1999, Astedt et al. 1992, Hannigan et al. 1997). Soft clays typically exhibit larger magnitudes of setup than stiff clays (Long et al. 1999, Sawant 2013). However, long-term pile setup has been shown to not be significant in very silty low plasticity cohesive soils, sands, and gravel as compared to cohesive soils (Holloway 1995, Walton and Berg 1998, Yang 1970). ODOT, on the contrary, has encountered substantial setup ratios in very silty low plasticity cohesive soils.

Piles driven into clay commonly experience larger geotechnical resistance gains than piles driven into sand and silt deposits. Piles may also experience relaxation (reductions of geotechnical resistance) when driven into dense and saturated sand and silt deposits (Long et al. 1999).

The soil surrounding the pile undergoes radial deformations during the pile driving process, which results in the development of significant excess pore water pressure (PWP) within the influence zone (Randolph et al. 1970, Long et al. 1999). In cohesive soils, due to their low permeability, the developed excess PWP dissipates slowly. As a result, a small percentage of setup occurs during the first logarithmically nonlinear dissipation Phase 1, while the majority of setup occurs during the logarithmically linear dissipation (Phase 2). In cohesive soils, a small amount of setup can be related to aging (Phase 3). Figure 10 illustrates the three phases pile setup.

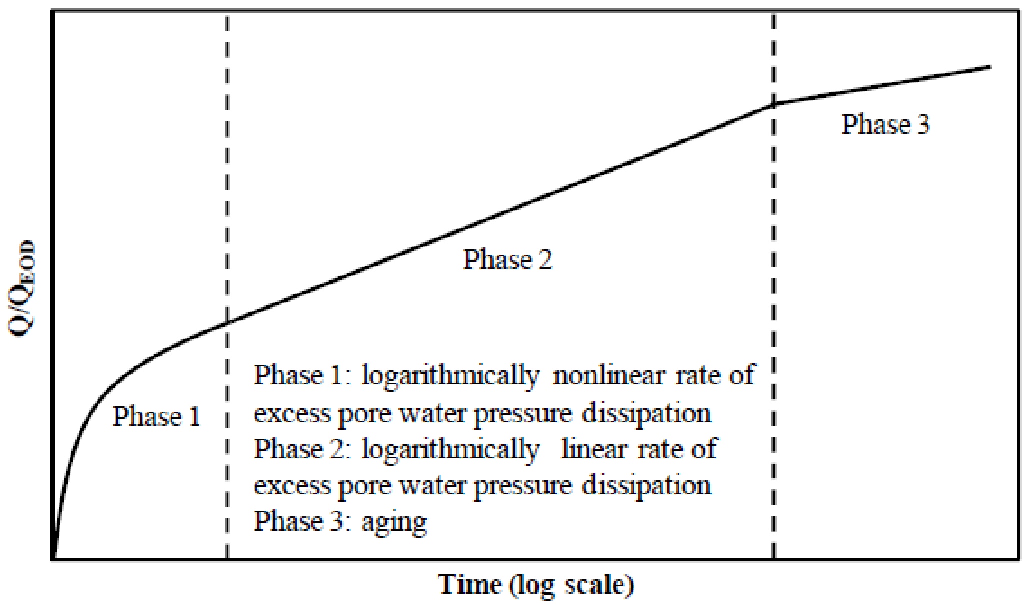
Randolph et al. (1979) stated that for piles driven in cohesive soils, the soil's shear strength change decreases with the logarithmic distance from the pile until it equals the initial soil strength at about 10 pile radii. In silts and fine sands, the developed excess PWP around the pile dissipates at a relatively faster rate than in cohesive soils (i.e., almost while driving occurs). As a result, some setup may occur during the logarithmically linear dissipation (Phase 2), while the majority of setup occurs during the aging (Phase 3) in these soils (Yang 1970, Axelsson 2002). Either, or both, of these phases, may begin immediately after driving (Titi and Wathugala 1999, Abu-Farsakh et al. 2015).

Table 5. Summary of pile setup estimation methods for fine-grained soils

Reference	Setup Model	Soil Type	Comments
Pei and Wang (1986)	$\frac{R_t}{R_{EOID}} = 0.236\left[\log(t)+1\right]\left(\frac{R_{\max}}{R_{EOID}}-1\right)+1$	Shanghai clay	- Purely empirical - No soil and pile properties - Difficult to determine R_{max}
Zhu (1988)	$\frac{R_{14}}{R_{EOID}} = 0.375S_t + 1$	Shanghai clay	- Predicts only resistance at 14 days
Skov and Denver (1988)	$\frac{R_t}{R_{EOID}} = A\log\left(\frac{t}{t_0}\right)+1$	Clay, chalk, or sand	- Wide range of A constant - Need A parameter for local soils
Lukas and Bushell (1989)	$\Delta R = \sum_{i=1}^{n}\left[S_{ai}(long) - S_{ai}(short)\right]\times A_s$	Silty clay	- Difficult to obtain correct pile adhesion
Svinkin and Skov (2000)	$\frac{R_t}{R_{EOID}} = B\left[\log(t)+1\right]+1$	General cohesive soil	- Challenges with B parameter - No clear relationship between B parameter and soil or pile properties - Parameter B similar to A of Skov and Denver (1988) model
Bullock et al. (2005)	$\frac{R_t}{R_{EOID}} = A\log\left(\frac{t}{t_0}\right)+1$	Sand and silty clay	- $A = 0.1$ in absence of tests - $t_0 = 1.0$ day
Karlsrud et al. (2005)	$\frac{R_t}{R_{100}} = A\log\left(\frac{t}{t_{100}}\right)+1$ $A = 0.1 + 0.4\left(1-\frac{PI}{50}\right)OCR^{-0.8}$	Clay	- Assumes complete consolidation in 100 days - Use of R_{100} is not practical
Yan and Yuen (2010)	$\frac{R_t}{R_{EOID}} = 1 + C\log(1+t)$	Sand and clay	- Parameter C related to soil setup rate - $C = 0.524$ (clays); $C = 0.418$ (sands)
Khan and Decapite (2011)	$R_t = 0.9957R_{EOID}t^{\alpha}$ $\alpha = 0.087$	Cohesive soil	- No soil or pile properties - No correlations between soil setup and SPT-N
Haque et al. (2016a)	$f_s = f_{s0}\left[A\log\left(\frac{t}{t_0}\right)+1\right]$ $A = 0.57e^{-0.05q_t}$	Cohesive soil	- Developed for side setup - Requires CPT
Haque et al. (2016b)	$\frac{f_s}{f_{s0}} = 1 + \left[\frac{0.79\left(\frac{PI}{100}\right)+0.49}{\left(\frac{s_u}{1\text{ tsf}}\right)^{2.03}+2.27}\right]\log\left(\frac{t}{t_0}\right)$	Cohesive soil	- Developed for side setup - Good correlations between setup and soil properties

R_t = pile resistance at time, t, after EOID; R_{EOID} = pile resistance at EOID; R_{max} = maximum pile resistance after soil consolidation completed; R_{14} = pile resistance at 14 days after EOID; S_t = soil sensitivity; A, B, and C are soil setup factors; t_0 = reference time; S_{ai} = pile adhesion; A_s = pile shaft area; R_{100} = pile resistance at 100 days after EOID; PI = plasticity index; OCR = overconsolidation ratio; f_s = unit side resistance at time, t, after EOID; f_{s0} = unit side resistance at EOID; q_t = corrected cone tip resistance; and s_u = undrained shear strength.

The rate of pile setup in granular soils depends on many factors including soil density, soil grain characteristics (particle size, shape, and gradation), soil shear modulus, moisture content, pile soil dilatancy, and in-situ stress level (Chow et al. 1998, Haque et al. 2014, York et al. 1994). Koutsoftas (2002) reported a 125 to 150% increase of pile resistance in dense sand. Generally, setup is greater for dense and well-graded sands, than for loose and uniform sands (York et al. 1994; Koutsoftas 2002).



(Q = Pile capacity at any time after driving, Q_{EOD} (also referered to as Q_{EOID}) = Pile capacity at the end of intital drive)

Figure 10. Three phases of pile setup (after Komurka et al. 2003)

Axelsson (1998) showed that pile driving in sand can generate strong arching effects, even at significant depths, and then the arching deteriorates with time due to stress relaxation and results in a horizontal stress increase. The increase in horizontal stress due to stress relaxation can continue for several months and is approximately linear with the logarithm of time. In addition, the soil aging phenomenon causes the reorientation of particles, leading to interlocking. In other words, both soil particles interlocking with pile surface roughness and stress relaxation provide an explanation of the large setup effects in non-cohesive soils.

Chow et al. (1998) suggested the best explanation for the large setup effects on driven piles in cohesionless soils is that sand creep rather than climatic- or tide-related changes in pore pressure weakens the arching mechanisms surrounding the pile shaft, increasing horizontal stresses while also producing larger dilation effects.

A.3.2 Effect of Pile Type

Setup has been reported to occur in almost all pile types including prestressed concrete (PSC) piles, tapered and fluted steel piles, H-piles, open-end and closed-end pipe piles, and in treated and untreated wood piles. Studies have shown the setup rate decreases as pile size increases (Camp III 1999).

When concrete and timber piles are driven into clays, excess PWP can dissipate along the pile interface, causing excess PWP in the soil adjacent to the pile surface to dissipate faster than soil a further distance from the pile surface (Komurka et al. 2003). When subjected to a load that causes pile soil deformation, movement occurs at some distance away from the pile surface rather than directly at the pile-soil interface. Because PSC piles have higher soil/pile interface friction, they usually exhibit larger setup than steel piles (Preim et al. 1989). Chow et al. (1997) stated that a portion of the setup for steel piles installed in sands is attributed to the corrosion-induced bonding of the sand particles with the steel.

A.4 Pile Setup Research by State DOTs

<u>Florida:</u>

Bullock et al. (2005) conducted a detailed pile load tests to investigate the side shear setup for soil layers. The tests have been performed on different soil types including clay, silt, and mixed soils. An increase of 80% in the resistance of prestressed concrete pile driven in alluvial sand showed that pile setup is also a consideration for piles driven in cohesionless soils. The study results concluded that the setup constant "A" ranges between 0.12-0.32 which is less than those constants recommended by Skov and Denver (1988). Using a minimum setup constant $A = 0.1$ in the absence of pile load tests, and higher values in the presence of static or dynamic load tests, were recommended.

<u>Indiana:</u>

Basu et al. (2009) conducted research for the Indiana DOT to develop a method to account for pile setup and relaxation. The study concluded: 1) resistance increase due to the pile setup ranges from roughly 1.2 to 1.4 times the EOID, 2) changes in soil behavior due to pile driving is very complex and cannot be modeled in a simplistic way, 3) the setup factor depends on the effective overburden pressure and overconsolidation ratio (OCR), and 4) setup duration depends on the permeability and undrained shear strength of the soil and pile diameter.

<u>Iowa:</u>

An Iowa DOT research project examined LRFD design procedures for piles supporting bridges. Two analytical pile setup quantification methods based on soil properties and a new calibration procedure to incorporate pile setup into LRFD were developed (Ng et al. 2011). A survey of State DOTs showed that steel H-piles is the most common pile type used for bridges, therefore the project investigated setup of five H-piles driven in cohesive soils in Iowa

(AbdelSalam et al. 2010, Ng et al. 2013a). The results showed the amount of setup at a given time depends on soil properties, including the coefficient of consolidation, undrained shear strength, and the SPT-N value. The results also suggested that including CPT pore-water pressure dissipation measurements as part of the site investigation can help determine the coefficient of consolidation and allow the estimation of the change in pore water pressure that influences pile setup.

Ng et al. (2013b) developed a model to predict pile setup for steel H-piles driven in cohesive soils by incorporating the end of initial driving pile capacity (EOD) and the standard penetration test (SPT-N) value. This model is limited to predict pile setup up to 30 days because the data used to model setup was limited to 30 days.

<u>Louisiana:</u>

A Louisiana DOT research study on estimating setup of driven piles into Louisiana clayey soils was documented in Wang et al. (2009). In general, the piles on one of the projects showed an increase of more than 90 percent of the nominal shaft resistance within two weeks after installation. An empirical relationship between the measured pile resistance at 24-hour restrike and the calculated pile resistance based on the Cone Penetration Test (CPT) was determined during the study. The study indicated that a predictive model developed using an entire restrike database with varying restrike times may result in an under-prediction of resistances at longer time restrikes or static load testing. This behavior is based on the usual practice of having a larger number of restrikes at shorter times following EOID when setup is not fully developed.

Haque et al. (2016a) proposed a model to predict soil setup for layered soils using piezocone cone penetration test data (CPTu). The logarithmic setup factor "*A*" was evaluated for soil layers using the unit side resistance. The results showed that the setup factor *A* ranges between 0.31 and 0.15 for clayey and sandy layers, and that the setup factor "*A*" decreases when the cone tip resistance increases. A simple linear regression model was developed to estimate side resistance setup for individual soil layers. Another study by Haque et al. (2016b) using prestressed concrete piles driven in different soil conditions proposed a model to predict pile setup by incorporating different soil properties, such as plasticity index (PI), undrained shear strength (s_u), soil sensitivity (S_t), overconsolidation ratio (OCR), and coefficient of consolidation (C_v).

Herrington (2018) investigated the applicability of incorporating pile setup into driven pile design procedure. The logarithmic setup factor "*A*" was back-calculated using historical database of driven piles in Louisiana. Using the first-order reliability method (FORM), resistance factors were developed to account for pile setup. Moreover, a case study was performed to check the accuracy of this approach. The results indicated that the number of required piles in the design stage could be lowered by 20% when pile capacity at 24 hours of initial driving is used.

<u>Minnesota:</u>

A Minnesota DOT research project investigated various methods of predicting the magnitude and/or rate of setup (Budge 2009). The study summarized case studies published by others and discussed by Komurka et al. (2003). The case studies presented a wide range of results on pile setup and concluded that pile setup can occur in both cohesive and cohesionless soils. The study by Budge (2009) recommended the use of CPT for predicting setup, dissipation tests at sites where setup is anticipated and restrikes will be performed and conducting a series of restrikes especially at sites where dissipation tests indicate significant time for pore pressure dissipation.

Ohio:

A student research study was sponsored by the Ohio Department of Transportation for the prediction of pile setup for Ohio soils (Khan and Decapite 2011). The report summarized thirty references related to pile setup, listed eight pile setup equations existing in the literature, and analyzed setup behavior using data obtained from 23 piles. Conclusions from the study include: 1) some degree of setup occurred in 91% of the cases and some degree of relaxation occurred in 9% of the cases, 2) the data analyzed did not yield any correlations between pile setup and the soil properties used, 3) no correlations were observed between setup and SPT-N values, and 4) pile setup showed an increasing trend with increasing pile lengths.

Wisconsin:

Wisconsin DOT sponsored a research project to investigate pile setup and compiled a summary of predictive setup methods. (Komurka et al. 2003). A number of empirical equations used to predict setup and a number of subsurface exploration field tests offering potential value in predicting setup were discussed. The study indicated that setup occurs in a variety of soil types, organic and inorganic saturated clay, and loose to medium dense silt, sandy silt, silty sand, and fine sand, and is related to both soil and pile properties. The study noted that as the permeability of soil increases the setup rate increases and the setup rate decreases as pile size increases due to the longer time needed for lateral consolidation.

A.5 Summary

The overview of research studies conducted by several State DOTs and other literature demonstrate that considering setup during pile design can routinely: increase nominal geotechnical resistance, reduce the number of piles and length (and potentially the number of splices), reduce the required pile sections, and allow for smaller driving equipment or reduce installation time.

Dynamic restrike tests and static load tests are used to account for pile setup, however, both are time-consuming and may not be feasible for some projects. There are several empirical, analytical, and numerical methods proposed by various researchers for estimating pile setup, and they all have their limitations. The majority of the empirical equations used for pile setup are based on a limited database and parameters; therefore, site-specific (or local) calibration is essential. Empirical methods to account for pile setup are not currently addressed as part of the AASHTO LRFD specifications because of these uncertainties and limitations. The availability of a methodology that can be used in design for setup magnitude prediction can have significant benefits by avoiding construction delays and unforeseen costs.

APPENDIX B: Field Projects and Effect of Construction Activities

B.1 Ohio Geology

Much of Ohio was covered by glaciers in the past. These glaciers covered about two-thirds of Ohio and played a major role in forming the soil conditions in the state. Pre-Illinoian glaciation occurred at greater than 300,000 years ago, then the Illinoian glaciation occurred between 300,000 to 130,000 years ago, followed by the Wisconsinan glaciation about 24,000 to 14,000 years ago. The glaciers covered the north, central, and western parts of the state, while the Illinoian glacier extended further in the southwestern part of the state, around Hamilton, Clermont, and Brown counties (refer to Figure 11). Traces of the Pre-Illinoian glaciation remain only in parts of Hamilton county.

Before the onset of continental glaciation, the Ohio landscape was dominated by rolling hills and deeply incised, mature rivers and streams. Erosion and deposition by Ice-Age continental glaciers advancing into northern and western Ohio produced a low-relief land surface compared to the unglaciated, high-relief land surface of southeastern Ohio.

Natural soil deposits in Ohio generally consist of one of five different types. Alluvial deposits are mainly gravel, sand, and silt associated with the floodplains of modern streams and rivers or outwash plains produced by meltwater originating from melting glaciers. Lacustrine deposits consist of sands, silts, and clays deposited within ancient and modern lakes. These deposits tend to contain alternating laminations that are clay rich and silt rich. Varves are thin, alternating layers of sand or silt rich sediment overlain by clay rich sediment that represent and annual deposit in a lake or other body of water. These deposits are commonly associated with the ancestors of Lake Erie, such as Lake Maumee, but also occur along the margins of some of the larger, interior, glacially-related lakes. Glacial drift is a general term that applies to all clay, silt, sand, gravel, pebble, cobbles, and boulders transported by glaciers or deposited by glacial ice or from meltwater derived from glacial ice. Residual soil consists of material that is a result of the weathering or decomposition of rock that has not moved from its original location. Colluvium is a loose mixture of soils and bedrock fragments that forms on hillsides and moves downslope to form thicker deposits at the bases of hills.

The thickness of glacial drift (thickness of the soil occurring above rock) in western and northern Ohio is highly variable, a consequence of numerous geologic factors acting in combination. In some areas, drift has been deposited on a relatively flat bedrock surface and changes in drift thickness are primarily the result of variations in the amount of glacial material deposited. In other areas, drift has infilled a deeply incised buried bedrock surface, and changes in drift thickness are primarily the result of variations in bedrock-surface elevation.

Distinct, narrow linear patterns of thick drift in western and central Ohio are the result of deep incisions in the underlying limestone and dolomite bedrock by a large, northwest flowing drainage system, the Teays Valley system, that existed before and during early glaciations. The main Teays Valley entered the state in Scioto County, where the remnants of the Teays Valley are still evident at the surface. Near Ross County, the valley disappears under glacial sediments which cover western Ohio. The valley continues north and west toward Indiana. While drift thickness might be less than 100 ft for most of the state, the Teays Valley contains much thicker areas of drift. Most of the valleys are in excess of several hundred feet, with some valleys approaching 700 ft of drift thickness.

In northeastern Ohio, narrow thick drift areas south of Lake Erie were caused by preglacial bedrock valleys. These valleys were partially filled with thick deposits of till and glaciolacustrine (glacial lake) sediment and then re-excavated by later northward flowing rivers such as the Cuyahoga River and the East Branch of Rocky River.

In northwestern Ohio, repeated scouring of the relatively soft bedrock surface by glacial ice flowing southwestward from the Lake Erie Basin destroyed most pre-existing drainage systems. In this part of Ohio, the bedrock surface is smooth and the upper surface of the drift has been planed off by wave action and deposition by a post-glacial, high level ancestral Lake Erie. In the extreme northwest corner of Ohio, the drift thickens considerably because of numerous moraines that formed along the northwestern edge of the Erie Lobe.

In western Ohio, draping linear features of thick drift, called ridge moraines, formed along the temporarily stationary ice-front as glacial sediment was released from the ice. These ribbons of thick drift define the lateral dimensions of glacial ice lobes, particularly those of the last Wisconsinan ice sheet. Many ridge moraines in western and northeastern Ohio have a draped appearance because of south flowing ice, impeded by bedrock highlands, moved more easily along major lowlands.

Southeastern Ohio is unglaciated and devoid of ice deposited sediment (glacial till). Many southeast Ohio valleys, however, carried huge volumes of glacial meltwater away from the ice front toward the Ohio River. In the process, many of these valleys were at times made deeper by the erosive force of fast flowing meltwater streams, and at other times were partially filled with sediment. Some valleys in unglaciated Ohio contain thick deposits of clay and silt that accumulated on the bottoms of lakes that formed when glacial ice blocked the flow of rivers or when rapidly accumulating meltwater sediments blocked the mouths of smaller tributaries.

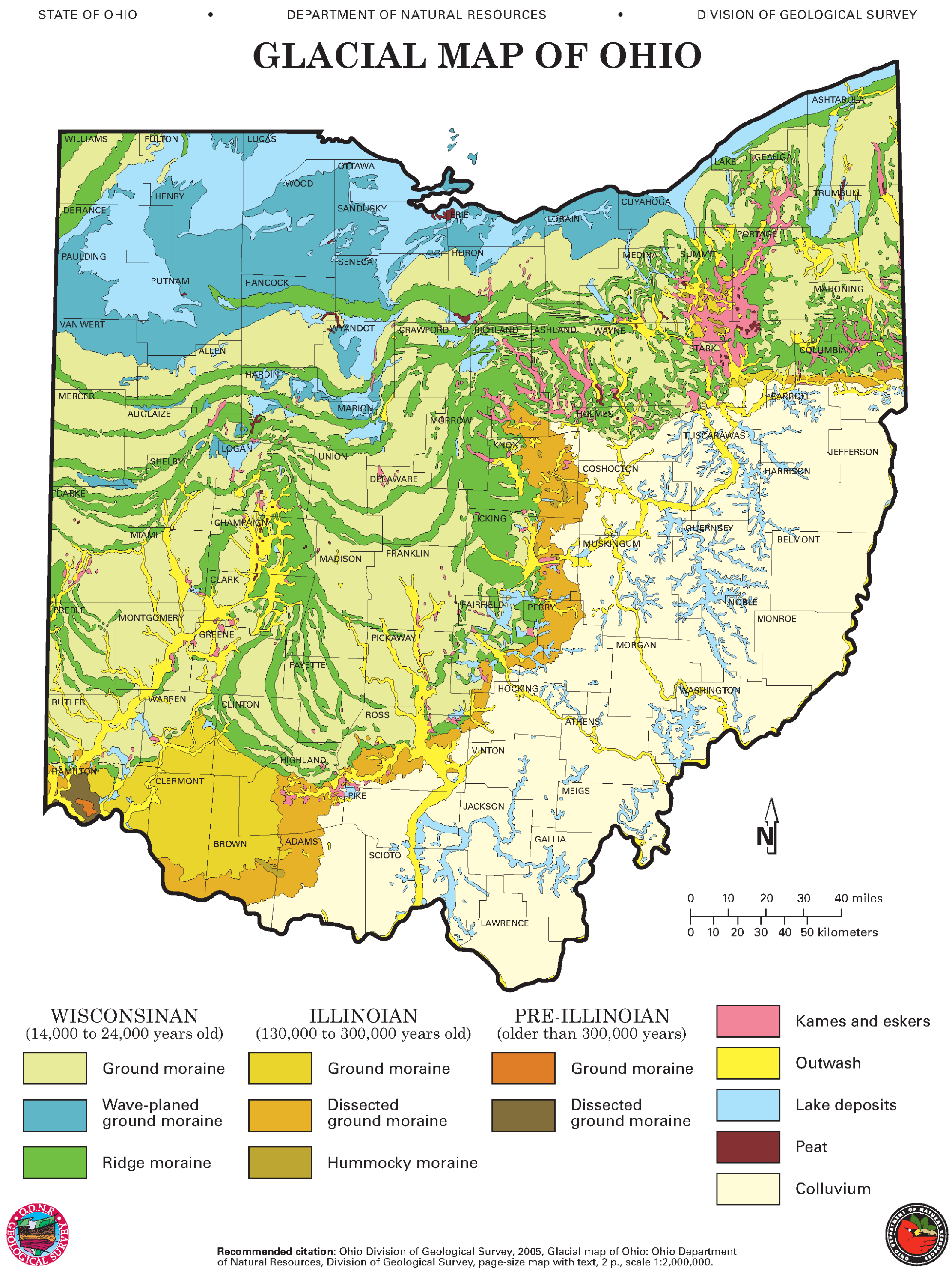


Figure 11. Ohio Glacial Geology (Ohio Dept. of Natural Resources)

B.2 Ohio Foundation Types

Ohio transportation projects are largely founded on three types of foundations: spread footings, drilled shafts, and driven piles. Spread footings are often used in areas where rock is near the surface or where there is competent soil and the structural loads are relatively light. Drilled shafts are almost always socketed into rock, and consequently are often used in areas of relatively shallow rock (less than 25 feet), which mostly correlate to the unglaciated portions of the state. Driven piles are used in most other cases. Driven piles are frequently used in the glaciated portions of Ohio where more fine-grained soils are found. Ohio frequently uses two types of driven piles: H-piles or closed-end CIPP piles. Ohio does not currently have construction specifications for precast concrete piles and the authors are aware of only one transportation project supported on precast concrete piles in Ohio. Per the Ohio DOT Bridge Design Manual, closed-end CIPP piles shall not exceed a 24 inch nominal diameter. Although not used very often, the DOT defines large-diameter open ended pipe piles as having a minimum nominal diameter of 36 inches.

When piles are driven to bedrock, H-piles are recommended as a first consideration due to their strength. When piles are expected to perform as friction piles, pipe piles are recommended as a first option. This is due to the fact that pipe piles are likely to develop more friction due to the larger soil displacement they exhibit during driving.

B.3 Pile Load Testing

One of the benefits of driven piles is the ability to predict capacity during driving. Since piles must be driven by a hammer to a proposed depth, many correlations have been developed between the amount of energy used to drive a pile and its capacity. Traditionally, pile capacity was estimated by counting both number of hammer blows as well as the stroke of the hammer. While this method is still employed to determine driving criteria, other methods of estimating pile capacity are currently used. Two of these methods include static and dynamic load testing. Static load testing consists of building a reaction frame around a test pile and slowly applying a load while measuring the displacement of the test pile. The pile is eventually tested to “failure” or a certain percentage (e.g., 200%) of the design load.

Dynamic load testing is a much more rapid test that is typically performed as the pile is driven. Before pile driving, a few sensors are installed to the top of the pile. The pile is then set and installed via normal pile driving practices. During installation, the hammer blows striking the pile send a wave of energy down the pile, which is translated into pile displacement and reflected back to the top of the pile. The installed sensors measure this reflected wave energy and engineers are able to correlate these measurements to an axial geotechnical resistance. There are both benefits and costs to these two testing methods. Both testing methods allow engineers to employ higher resistance factors in pile design, which allows for a more economical design. However, this is not the whole story. Static load tests, due to their duration and load frame design and construction, are much more costly than dynamic load tests. For this reason, ODOT only requires them if the project in question has more than 10,000 linear feet of piling at the same capacity on a project. For projects below the 10,000 linear foot threshold, ODOT requires dynamic tests on a minimum of 2% of driven piles. The results of these tests can be used to confirm or adjust the driving criteria for the rest of the piles.

Dynamic pile testing is preferred due to its lower cost. One other problem that exists with pile driving is the issue of relaxation and setup, which is the primary function of this research. As piles are driven, pore water pressures increase which can cause capacity loss during driving. This is commonly referred to as driving loss. Conversely, piles driven in heavily over-consolidated clays, dense saturated silts and fine sands, or to refusal on clayey shale can experience capacity reduction after driving. This is commonly referred to as relaxation. Pile relaxation is typically neglected in Ohio, as its effects are muted since a pile can regain the lost capacity with a very small displacement. Driving losses are a different matter. Currently, ODOT deals with these losses by prescribing restrikes. Restrikes mean reperforming a dynamic load test after the increased pore water pressure has dissipated. This usually means waiting from one day to a week, but could be even much longer (greater than four weeks) if setup is required to meet the required pile capacity. However, since the economic cost of longer waits outweighs the economic cost of ordering and driving more pile, increasing pile length is typically the preferred option by ODOT.

In order to understand the issue of pile setup, the research team conducted static and dynamic load testing with pore water pressure measurements on several projects around the state. Selected projects were located in the southwest corner of Ohio (Cincinnati), northeast Ohio (Akron), and northwest Ohio (Toledo). The procedures at these sites included performing cone penetrometer testing (CPT) and using the CPT rig to push in piezometers to a depth where soils with high setup potential were present. These piezometers were located approximately 2, 5, and 10 ft (depending on the project) away from the proposed test pile. Pore water pressure was then measured during initial pile driving as well during the static load test and during any subsequent restrikes.

Movement of the pile top during the test was measured using 4 dial indicators. The base of each dial indicator was attached to a steel reference beam and the plungers for each dial indicator rested on steel plates welded perpendicular to the test pile. The supports for the reference beam were at least 8 ft away from the test pile and as far as practical from the reaction piles. A second, independent measurement of the pile top movement was made using a steel wireline, scale, and mirror. The scale and mirror were attached to the test pile and the wireline was stretched between two supports away from the test pile. The mirror was used to eliminate parallax error by lining up the wire and its image in the mirror when taking a reading. Movements of the four reaction piles closest to the test pile were also measured using dial indicators.

Strain measurements along the length of the pile were obtained using 8 vibrating wire sister bar strain gages (Model 4911) manufactured by Geokon. The strain gages are attached by the manufacturer to a three-foot long piece of #4 rebar (the “sister bar”). The strain gage was attached to a length of #5 rebar by wire tying the sister bar to the #5 rebar.

The readings from the vibrating wire sister bar strain gages were used to back-calculate the force in the pile at each of the strain gage locations. This allows the determination of the load distribution along the pile with depth, which allows estimation of the side resistance along the length of the pile and the amount of tip resistance at the base of the pile. The incremental rigidity method described by Komurka and Moghaddam (2020) was used to convert the strain gage readings to force in the pile. For strain gages embedded in concrete, when the change of force during a load increment is divided by the change in strain during the same load increment (the incremental rigidity), the points eventually follow a linear relationship. The linear parameters (slope and intercept) of this best fit line can then be used to calculate the force from the strain gage reading using the formulas shown below:

Best Fit Line for Incremental Rigidity: $\frac{\Delta Q}{\Delta \varepsilon} = A\,\varepsilon + B$ (8)

Force in Pile: $F = 0.5A\,\varepsilon^2 + B\,\varepsilon$ (9)

B.3.1 HAM-75 Project Site

The static load test was located at the foundation for the north pier of proposed Bridge No. HAM-75-1292 in Cincinnati, Ohio. The bridge carries Shepherd Lane over I-75. The piles were 14-inch diameter CIPP piles with a wall thickness of 0.32 inches. The pipes were driven with a plate covering the pile tip and then filled with concrete. The required pile capacity (known as the ultimate bearing value, UBV) was 226 kips. This corresponds to the nominal pile resistance for Load and Resistance Factor Design or the ultimate pile capacity in Allowable Stress Design.

A steel reaction frame was constructed to resist the jacking load on the test pile and transfer the load to 8 reaction piles. The reaction frame was designed for a maximum jacking load of 452 kips, which is twice the UBV. The test pile was pile number 77, and the reaction piles were numbers 68, 70, 71, 73, 81, 83, 84, and 86. All piles were vertical. The piles were driven with an IHC S-40 hydraulic hammer. Dynamic load tests were performed by GZA on Test Pile No. 77 and Pile No. 65, 67, and 88 during the initial drive and restrikes performed 10 and 17 days after the initial drive. The results of the load tests are shown in Table 6. The results of the load tests on the test pile (#77) are shown in Figure 12.

Boring number B-003-0-10 was located at the north pier footing. This boring encountered predominantly cohesive soil to the depth of boring, which was 60 ft. The cohesive soil was stiff to very stiff above El. 549.5 and then very stiff to hard below that. A 5-foot thick medium dense non-plastic silt layer was encountered at a depth of 10.5 ft.

The ODOT Office of Geotechnical Engineering also performed a CPT exploration at the site on August 7, 2019. The CPT (with pore pressure measurements) also encountered predominantly cohesive soil to the depth of exploration, 44.5 ft. The CPT rig was used to push three vibrating wire piezometers into the ground to a depth of 24.6 ft. These piezometers were located 2, 5, and 10 ft from the static load test pile. The pore pressure readings indicated significant increases during pile driving for the test pile and nearby reaction piles. Interestingly, the pore pressure within 2 ft of the test pile dropped to normal levels within a day or two of the pile driving, while the piezometers further from the test pile were very slow to return to normal. The piezometer readings at HAM-75 site are shown in Figure 13.

Table 6. HAM-75 project load test results

		Pile #77 (Test Pile)	**Pile #65**	**Pile #67**	**Pile #88**
Date	Driven depth:	45.0 ft	41.75 ft	41.50 ft	36.33 ft
8/13/19	EOID	side: 130 kips tip: 190 kips total: 320 kips	side: 100 kips tip: 220 kips total: 320 kips	side: 50 kips tip: 100 kips total: 150 kips	side: 140 kips tip: 150 kips total: 290 kips
8/22/19	Static Load Test	side: 255 kips tip: 199 kips total: 454 kips			
8/23/19	BOR 1	side: 250 kips tip: 240 kips total: 490 kips	side: 300 kips tip: 90 kips total: 390 kips	side: 170 kips tip: 170 kips total: 340 kips	side: 170 kips tip: 180 kips total: 350 kips
8/30/19	BOR 2	side: 250 kips tip: 150 kips total: 400 kips	side: 270 kips tip: 150 kips total: 420 kips	side: 250 kips tip: 130 kips total: 380 kips	side: 120 kips tip: 140 kips total: 260 kips

Note: The ground surface was at El. 562.75, which is approximately 6 inch below the bottom of footing.

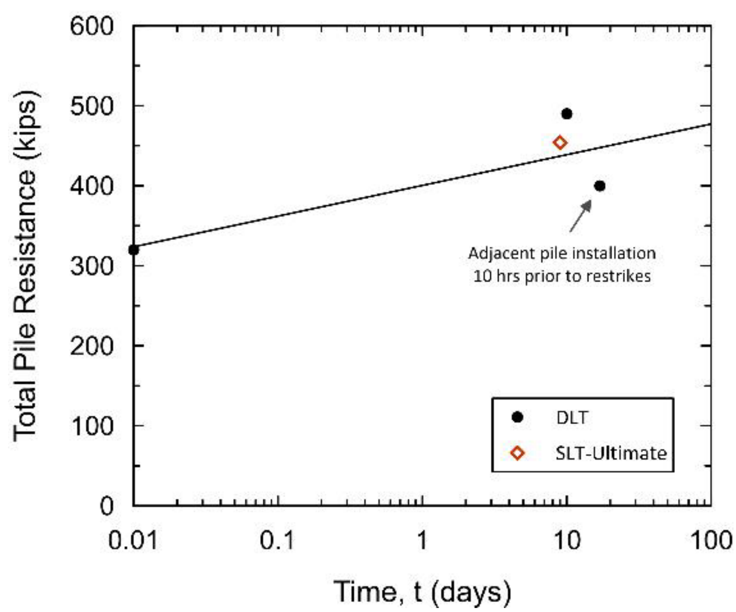


Figure 12. HAM-75 project static and dynamic load test results on test pile (Pile #77)

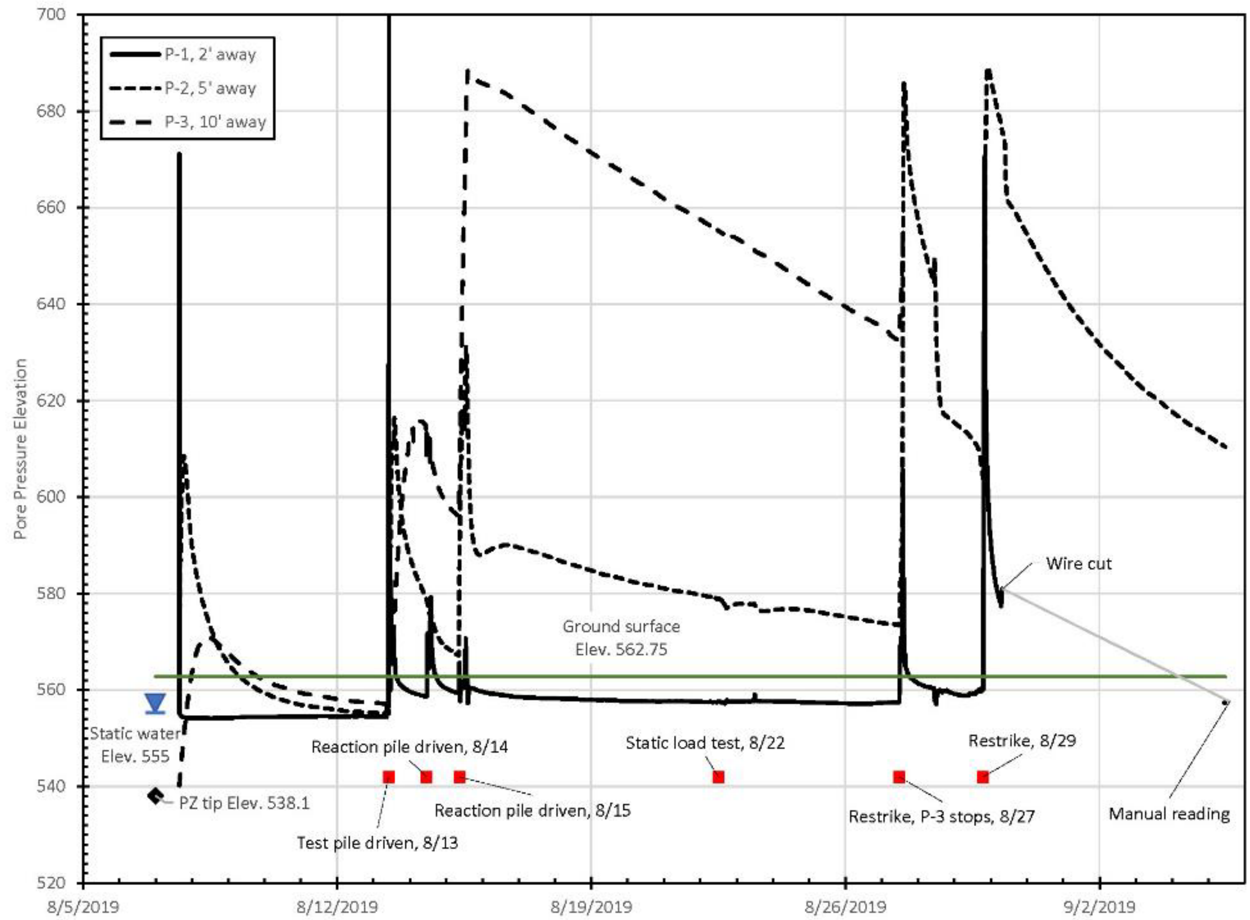


Figure 13. Piezometer readings at HAM-75 project site

The test pile supported the maximum test load of 454 kips with approximately 0.26 inch of movement at the top of the pile. In this case, the load-displacement curve did not cross the criteria line used to determine pile capacity, so the full resistance of the test pile was not mobilized under the maximum test load. Therefore, the nominal test pile resistance is greater than the maximum test load of 454 kips. The results indicate there was pile setup at the test pile, but there was also a reduction in capacity during the 17 day restrike. It is believed that this reduction is due to the driving of other piles in the foundation after the completion of the static load test.

Additional information on HAM-75 project can be found in Supplemental Documents section at the end of the report.

B.3.2 LUC-75 Project Site

The test pile was a closed-end pipe pile, 12.75 inch O.D. The compression test pile was installed on August 21, 2019 to a depth of 88 ft below existing grade (reported as El. 600 at the time of testing) and was designated as pile #43. Pile #43 was dynamically tested during initial driving and with multiple restrikes.

The static load test results are compared to the dynamic load test results in Table 7. The restrike dynamic load test that was four days after the static load test indicated higher total resistance than the static load test, although the side resistance from the dynamic load test was lower than the side resistance from the static load test. The static load test results are based on the Davisson criteria, which indicated a total resistance of 430 kips at 0.8 inches of displacement. The estimated displacements from the CAPWAP analyses of the dynamic load test results indicate an estimated displacement of 0.9 inches at the top of pile at the reported total resistance. Therefore, the displacements are comparable.

Table 7. LUC-75 project load test results (Pile #43)

Date	Test	Total (kips)	Side (kips)	Tip (kips)	Unit Side (ksf) Upper 73 ft	Unit Side (ksf) Lower 15 ft
8/21/2019	EOID	347	87	260	0.22	0.74
8/30/2019	Static	430 (D)	243	187	0.11	4.30
		562 (Ult)	308	254	0.13	5.50
9/3/2019	BOR 1	553	208	345	0.58	1.40
9/13/2019	BOR 2	604	259	345	0.69	1.90

Note: Test pile driven to 88 ft embedded length.

EOID and BOR tests are based on CAPWAP results. Static is based on the results of the static load test, with side and tip resistance based on an interpretation of the embedded strain gage readings. (D) indicates resistance based on Davisson criteria and (Ult) indicates resistance based on maximum test load. The results of the load tests on test pile (#43) are shown in Figure 14.

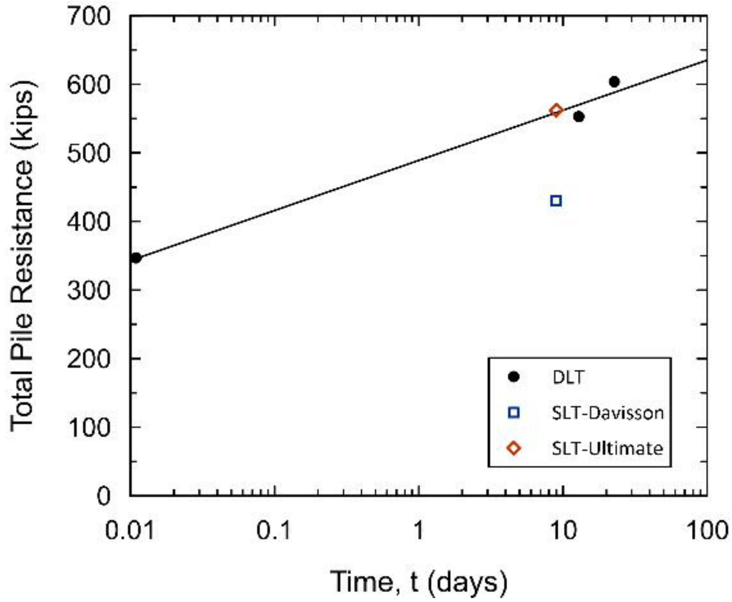


Figure 14. LUC-75 project static and dynamic load test results on test pile (Pile #43)

ODOT completed four cone penetration test (CPT) soundings at or near the test pile location, although the tip load cell was damaged on the fourth sounding, so no log was produced for it. At El. 527, the CPT encountered significantly higher cone tip resistance and sleeve friction, indicating a much stiffer soil layer. Three vibrating wire piezometers were installed at an approximate depth of 25 ft. The piezometers were attached to a data logger and readings were recorded every thirty minutes before pile driving, every 5 minutes during pile driving and static load testing (from 8/20/2019 5:00 PM to 8/30/2019 5:30 PM), and then every thirty minutes after static load testing. A graph of the piezometer readings is shown in Figure 15. The graph shows how the pore water pressures increase in response to the pile driving and restrikes and then dissipate with time. The graph shows a baseline groundwater elevation near 590 ft before pile driving. There is a significant increase in pore pressures during pile driving, with a gradual dissipation of pore pressures. However, the readings appear to be converging to a new baseline groundwater elevation between 605 and 610 ft. Rainfall data from a nearby weather station is shown in the chart in an attempt to correlate the increased groundwater level to precipitation, but that does not appear to be the case. There is no explanation at present as to why the baseline groundwater level increased.

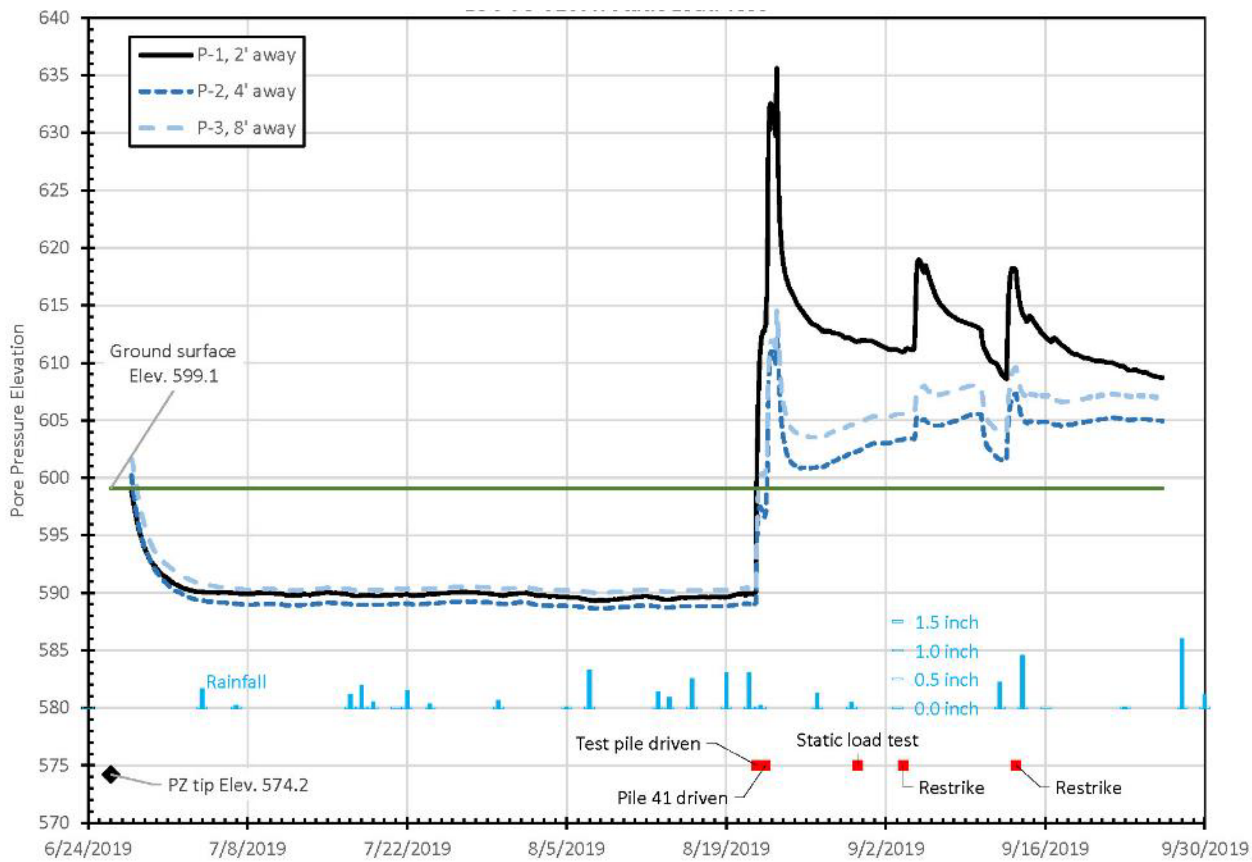


Figure 15. Piezometer readings at LUC-75 project site

Additional information on LUC-75 project can be found in Supplemental Documents section at the end of the report.

B.3.3 SUM-76 Project Site

The test pile was a 12-inch diameter closed-end pipe pile. Because this test program was for research purposes only (the pile was not used to support a permanent structure), no ultimate bearing value was assigned. The compression test pile was dynamically tested during initial installation on October 30, 2019, to the specified depth of 30 ft below existing grade (reported as EL 966.5 ft at the time of testing). The pile was advanced 6 inches during a restrike performed October 31, 2019. The static load test was performed on November 7, 2019, and the pile was subsequently dynamically tested during restrikes on November 8 and 15.

The static load test results are compared to the dynamic load test results in Table 8. Generally, the dynamic load tests during restrikes indicated higher total resistances than the static load test, although the tip resistances from the dynamic load tests were lower than the tip resistance from the static load test. However, the static load test results are based on the Davisson criteria, which indicated a total resistance of 121 kips at 0.3 inches of displacement. The estimated displacements from the CAPWAP analyses of the dynamic load test results indicate an estimated displacement of 1 to 2 inches at the top of pile at the reported total resistance. When this is compared to the maximum test load during the static load test, the total resistance results are in better agreement with the dynamic load test results.

Table 8. SUM-76 project load test results

Date	Test	Total (kips)	Side (kips)	Tip (kips)	Unit Side (ksf) Upper 17 ft	Unit Side (ksf) Lower 13 ft
10/30/2019	EOID	100	52	48	0.46	0.68
10/31/2019	BOR 1	136	80	56	0.59	1.00
11/7/2019	Static	121 (D)	55	66	0.05	1.30
		140 (Ult)	51	90	0.09	1.10
11/8/2019	BOR 2	156	101	55	0.69	1.30
11/15/2019	BOR 3	167	104	63	0.75	1.30

Note: Test pile driven to 30 ft embedded length.

EOID and BOR tests are based on CAPWAP results. Static test is based on the results of the static load test, with side and tip resistance based on an interpretation of the embedded strain gage readings. (D) indicates resistance based on Davisson criteria and (Ult) indicates resistance based on maximum test load. The results of the load tests on the test pile are shown in Figure 16.

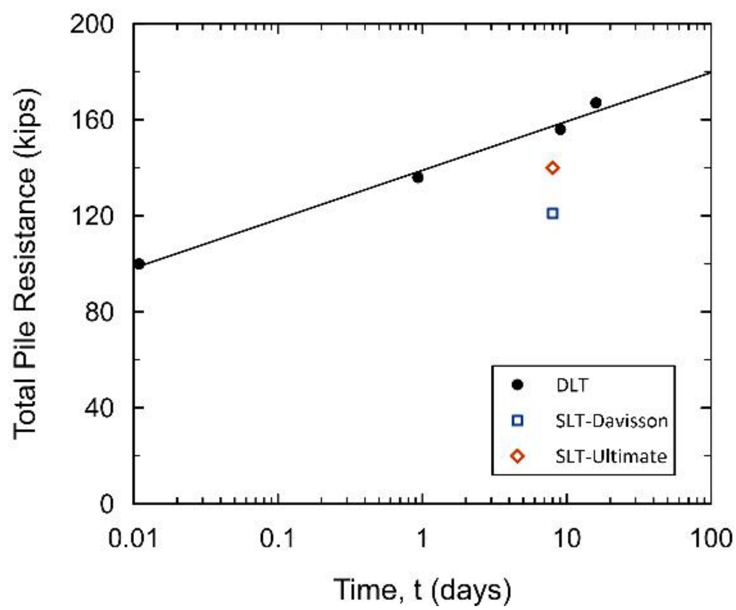


Figure 16. SUM-76 project static and dynamic load test results on test pile

ODOT completed four cone penetration test (CPT) soundings at or near the test pile location. Three vibrating wire piezometers were also installed at depths from 14 to 15 ft. The piezometers were attached to a data logger and readings were recorded every five minutes. A graph of the piezometer readings is shown in Figure 17. The graph shows how the pore water pressures increase significantly in response to the pile driving and restrikes and then dissipate with time.

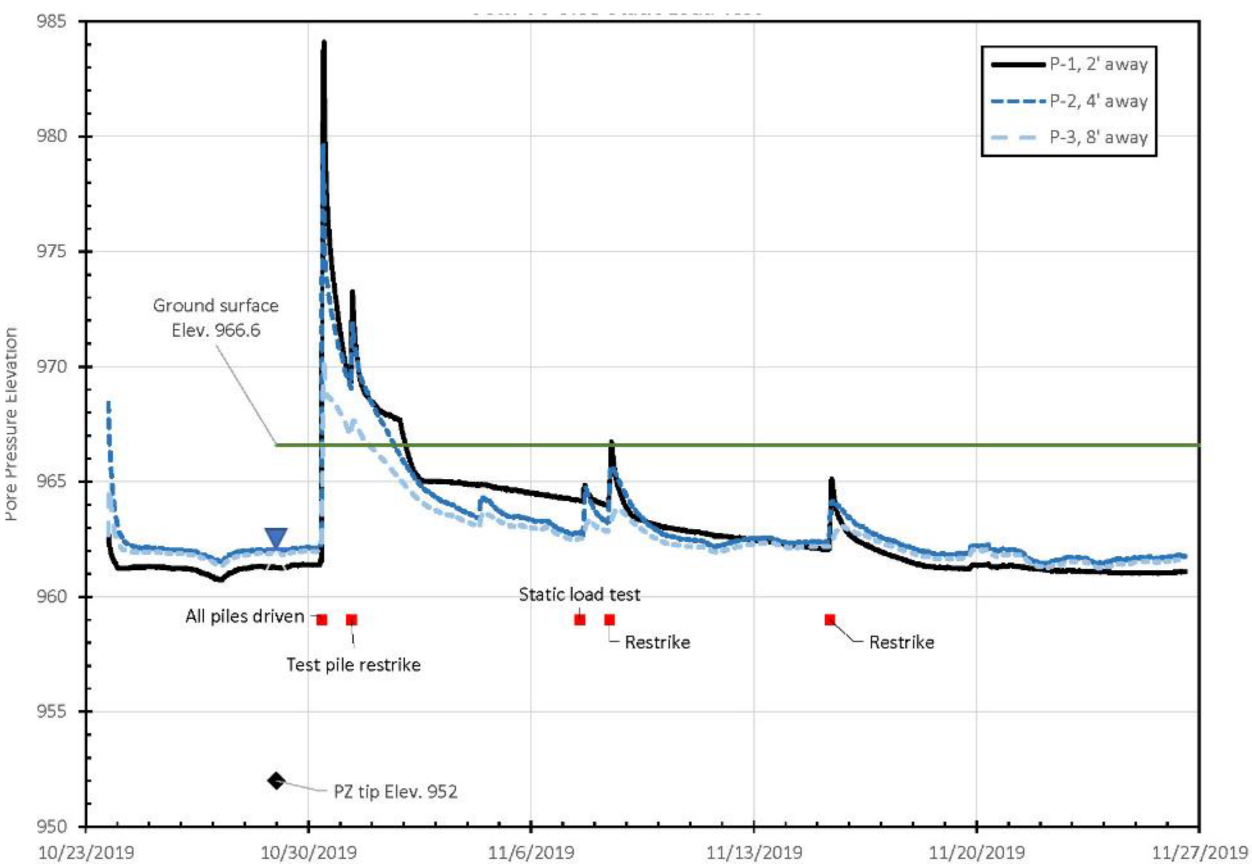


Figure 17. Piezometer readings at SUM-76 project site

Additional information on SUM-76 project can be found in the Supplemental Documents section at the end of the report.

B.4 Effect of Construction Activities on Pile Resistance

During the field projects, it was noted that the construction activities at the sites had an effect on the measured pore pressures and the measured pile resistance. At the HAM-75 project there were four piles installed at the site within a five-hour period. The layout of the piles is presented in Figure 18. The piles marked and numbered as 1st, 2nd, 3rd, and 4th show the driving sequence for the four piles. The distance between the 1st and 2nd piles was about 16 ft. The distance between the

2nd and 3rd piles was about 17 ft. The distance between the 3rd and 4th piles was the shortest distance between the piles driven, about 8 ft.

Dynamic load tests were performed on all four piles during initial driving and at two restrikes, performed at 9 and 17 days after the installation. Pile information and test results are given in

Table **9**. Pile resistance at the end of initial drive and at restrikes are plotted in Figure 19. To be able to better compare the test results, pile resistance per unit length is used in the figure. As shown in the figure, there was a difference of 113% between the third pile (#65) and the fourth pile (#67) at the time of initial driving. This difference was only 10% during the second restrikes performed after 17 days. This clearly shows how the resistance of the fourth pile (#67) was affected at the time of initial driving by the three piles driven prior to this pile, especially the 8-ft away third pile. Figure 19 also shows the effect of driving nearby piles on the resistances of piles #77 and #88. Piles #75, #76, #80, #82, and #85 were driven 10 to 16 hours before the 17-day restrikes on piles #77 and #88 The test results show that there was a relaxation in pile resistances at these two piles relative to the first restrikes performed eight days earlier. The 17-day resistance of pile #88 was even lower than the EOID resistance. The installation of the adjacent pile introduced new soil disturbance and an increase in pore pressures which were in the process of being re-established after the installation of piles #88 and #77.

Pile resistance changes with time are presented in Figure 20 as setup ratios. Due to the pile driving activities affecting the pile resistances obtained from dynamic load tests, the setup ratios obtained covered a wide range, from 0.90 to 2.53. These setup ratios would not be representative "setup factors" to be used in design.

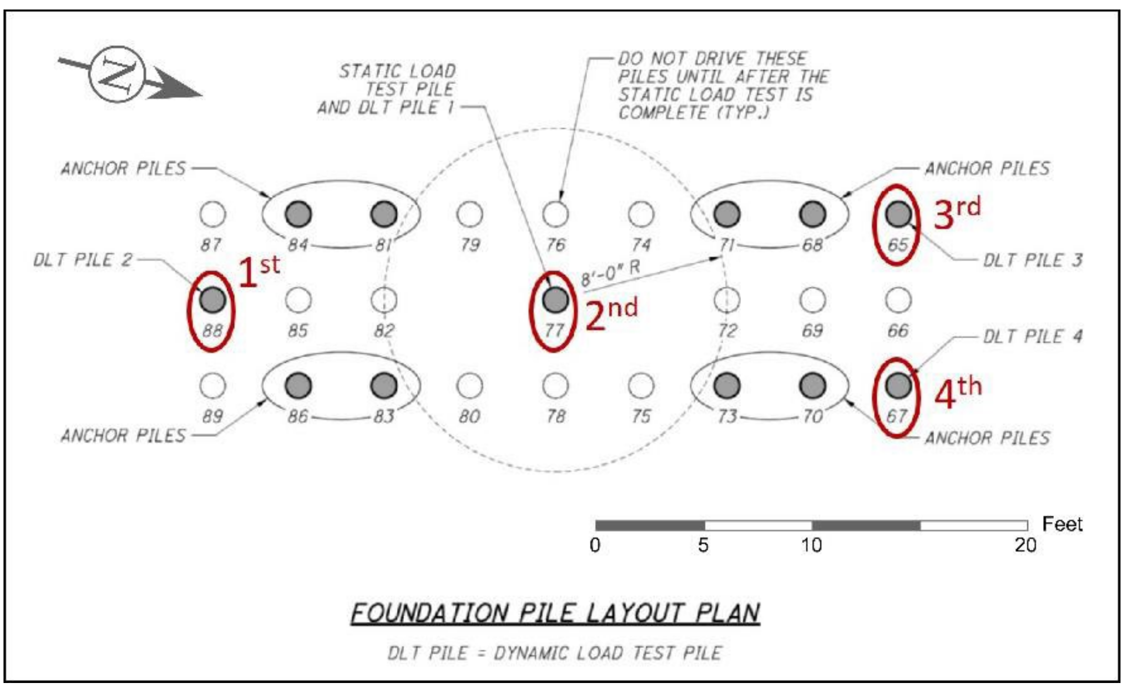


Figure 18. HAM-75 field project center pier foundation and static load test pile layout plan

Table 9. HAM-75 project dynamic load test data

Driving Sequence	Pile ID	Length (ft)	Driving Date - Time	UBV (kip)	Pile Resistance CASE / CAPWAP: EOID Q_{EOID} (kip)	First Restrike Time (day)	First Restrike Q(t) (kip)	Second Restrike Time (day)	Second Restrike Q(t) (kip)
First	88	36.3	8/13/2019 - 8:09 AM	226	310 / 290	9	380 / 350	17	280 / 260
Second	77	45.0	8/13/2019 - 9:41 AM	226	300 / 320	9	470 / 490	17	430 / 400
Third	65	41.8	8/13/2019 - 10:46 AM	226	320 / 320	9	400 / 390	17	370 / 420
Fourth	67	41.5	8/13/2019 - 12:42 PM	226	120 / 150	9	350 / 340	17	410 / 380

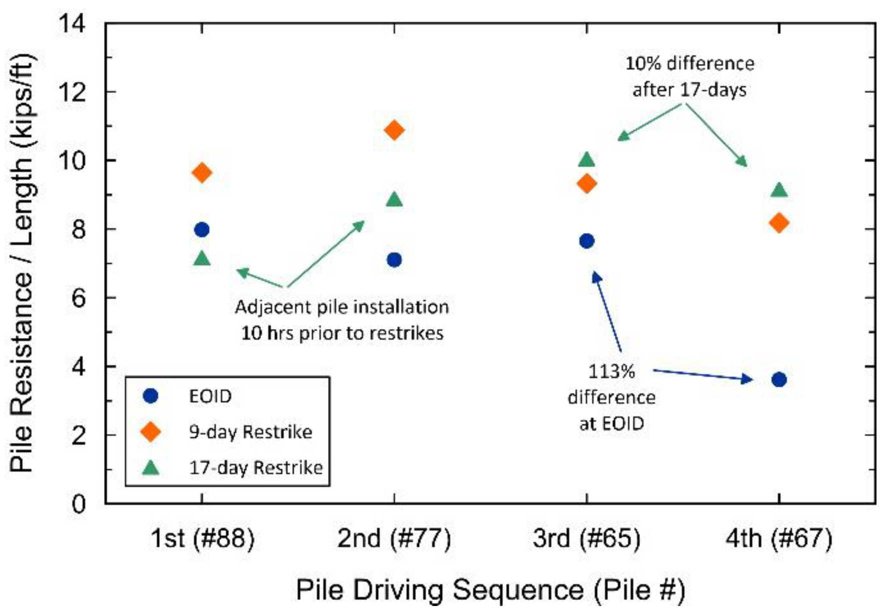


Figure 19. HAM-75 project pile driving sequence versus resistance using CAPWAP results

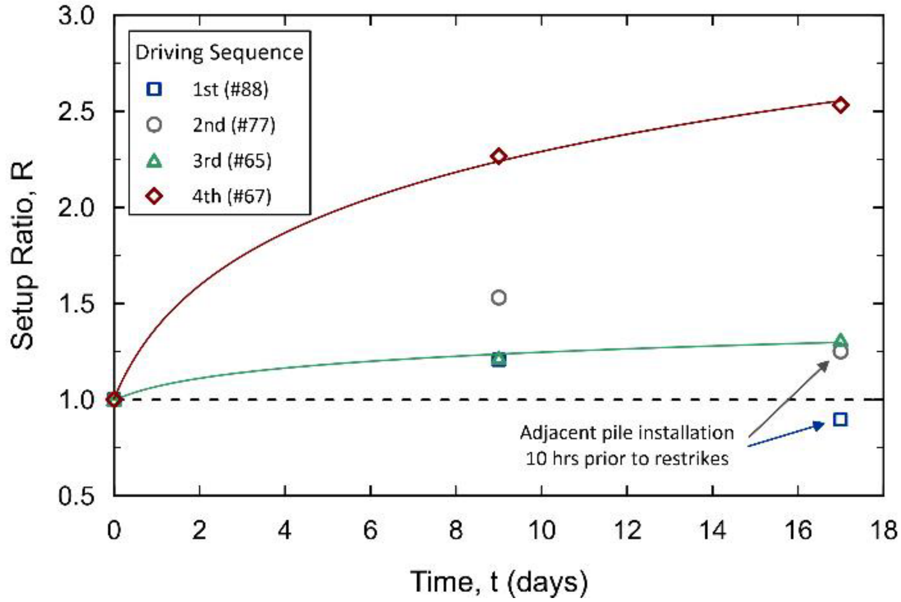


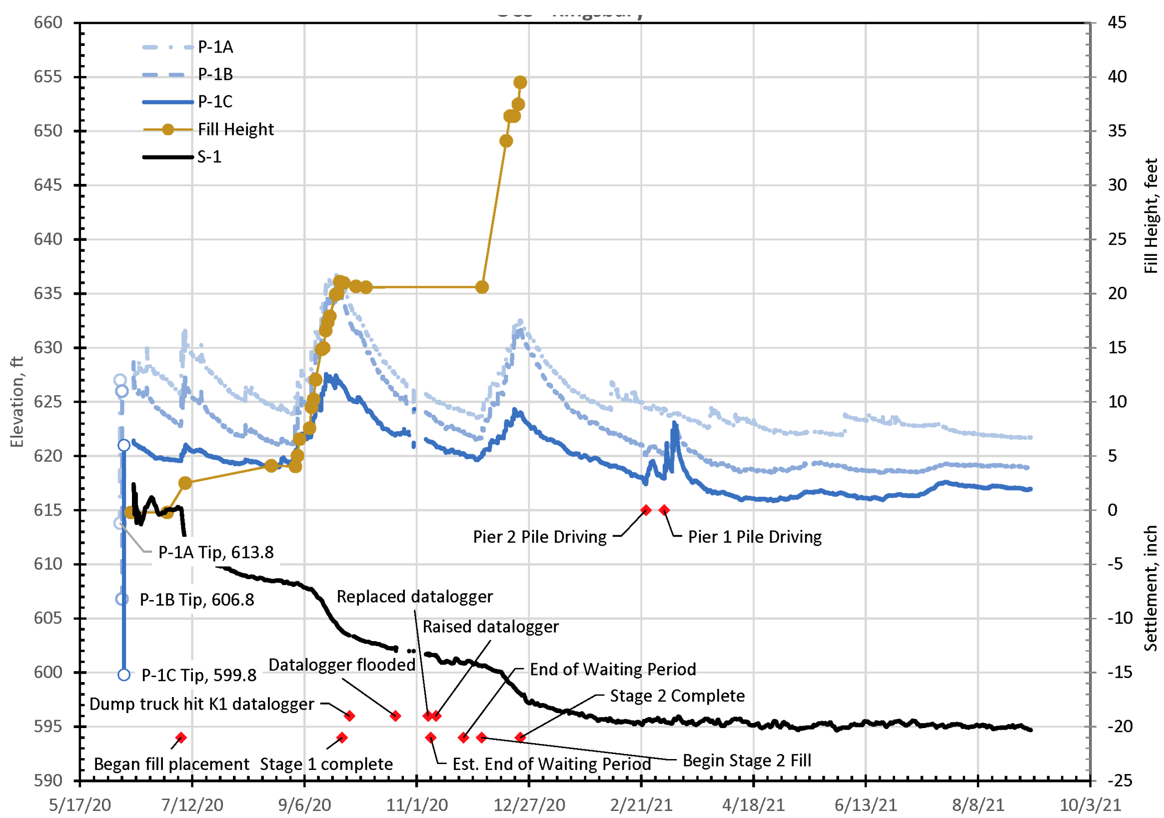
Figure 20. HAM-75 project setup ratios

It was also observed that the pore water pressures in the soil were affected by pile driving some distance away. The adjacent piezometers were installed with the CPT rig at distances of approximately 2 ft, 4-5 ft, and 8-10 ft away from the static load test pile. These piezometers were installed between 15-25 ft deep, depending on the project. As expected, the largest increase in pore water pressure was during initial pile driving, with greater distance from the pile showing a fall in pressure. As an example, the 2-ft piezometer would see a 20 ft increase in pore water pressure, with the 4-ft piezometer seeing a 15 ft increase and the 8-ft piezometer seeing an increase of 8 ft in pressure head. In general, any subsequent restrikes showed a maximum change in pore water pressure of a few feet.

Pile driving can affect pore pressures in the soil for a significant distance. On another project with which the research team was involved (OC3 Kingsbury Project), it was observed that a group of piles driven 93 or 134 ft away from a piezometer location increased the pore pressures by approximately 2 to 8 ft of pressure head. These were piles driven at two bridge pier locations, while the piezometers were measuring pore pressures beneath the approach embankments. The measurements at the rear and forward abutments are shown in Figure 21.

The results obtained from the HAM-75 field project site and the other sites emphasize the importance of coordination of construction activities prior to the load tests. This would remove or limit the effect of construction activities and their influence on test results and allow load tests to be more beneficial to the projects.

(a)



(b)

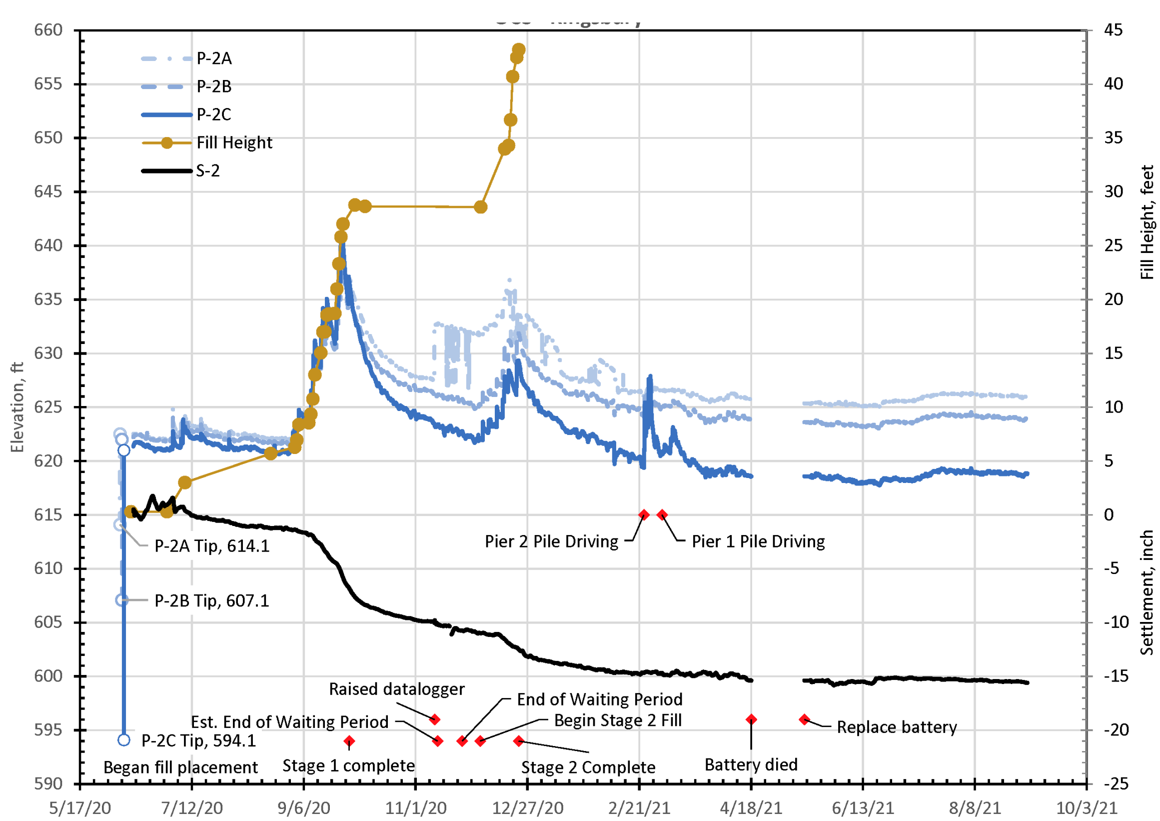


Figure 21. Pore pressures at OC3 Kingsbury site (a) rear abutment and (b) forward abutment

APPENDIX C: Detailed Database Statistics and Analysis

C.1 Database Statistics

C.1.1 Pile Properties

The piles used in this research were CIPP piles. The pile data collected are summarized in Table 10. Pile diameters ranged from 12 to 18 inches. The distribution of the pile diameters is given in Figure 22(a). The driven pile lengths had a wide range, from 15 to 190 ft, with an average and median lengths of 65 ft and 55 ft, respectively. The majority of the piles (61%) were between 40 to 80 ft long. The distribution of pile lengths is shown in Figure 22(b).

Table 10. Summary of pile properties

Parameter	No. of Data	Min.	Max.	Mean	Median	Std. Dev.	COV (%)
Pile diameter, D (in)	87	12.00	18.00	14.44	14.00	1.95	13.53
Pile length, L (ft)	87	15.00	190.00	64.63	55.00	35.17	54.42

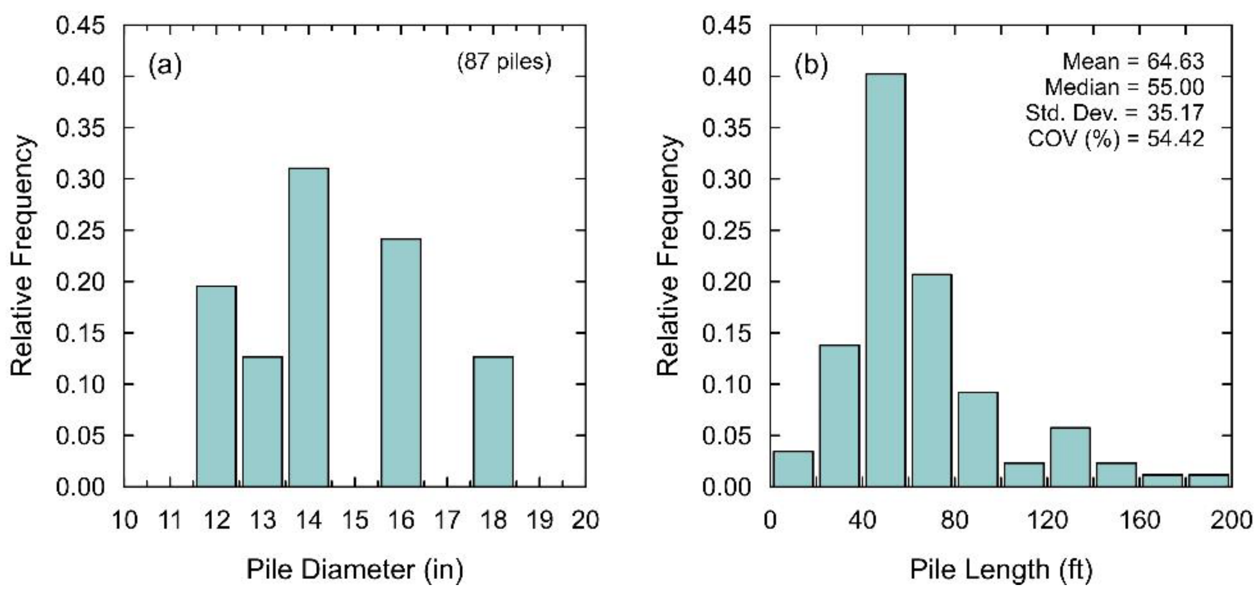


Figure 22. Distribution of pile sizes; (a) pile diameters and (b) pile lengths

C.1.2 Soil Properties

The piles investigated were installed in predominantly fine-grained soils. More than two-thirds of the lengths of the piles along their shafts were in fine grained soil layers. The properties of soils are summarized in Table 11. The properties are weighted averages based on the soil layer thicknesses along the pile length. The distributions of soil properties in the database are provided in Figure 23 and Figure 24.

Table 11. Summary of soil properties

Parameter	**No. of Data**	**Min.**	**Max.**	**Mean**	**Median**	**Std. Dev.**	**COV (%)**
Moisture content, w (%)	87	8.67	32.33	19.80	19.55	4.97	25.12
Liquid limit, LL (%)	85	16.59	49.07	29.09	29.45	5.80	19.95
Plastic limit, PL (%)	85	10.41	21.20	17.94	18.00	3.14	17.49
Plasticity index, PI (%)	85	4.24	29.28	11.19	10.97	4.04	36.07
SPT-N60 (blows/ft)	87	6.00	57.66	22.77	21.12	8.99	39.47
Fine fraction, F_{fine} (%)	87	42.37	100.00	72.45	74.50	16.33	22.54
Silt fraction, F_{silt} (%)	87	0.00	75.00	39.31	37.62	12.10	30.77
Clay fraction, F_{clay} (%)	87	3.68	67.84	32.82	32.18	14.01	42.69

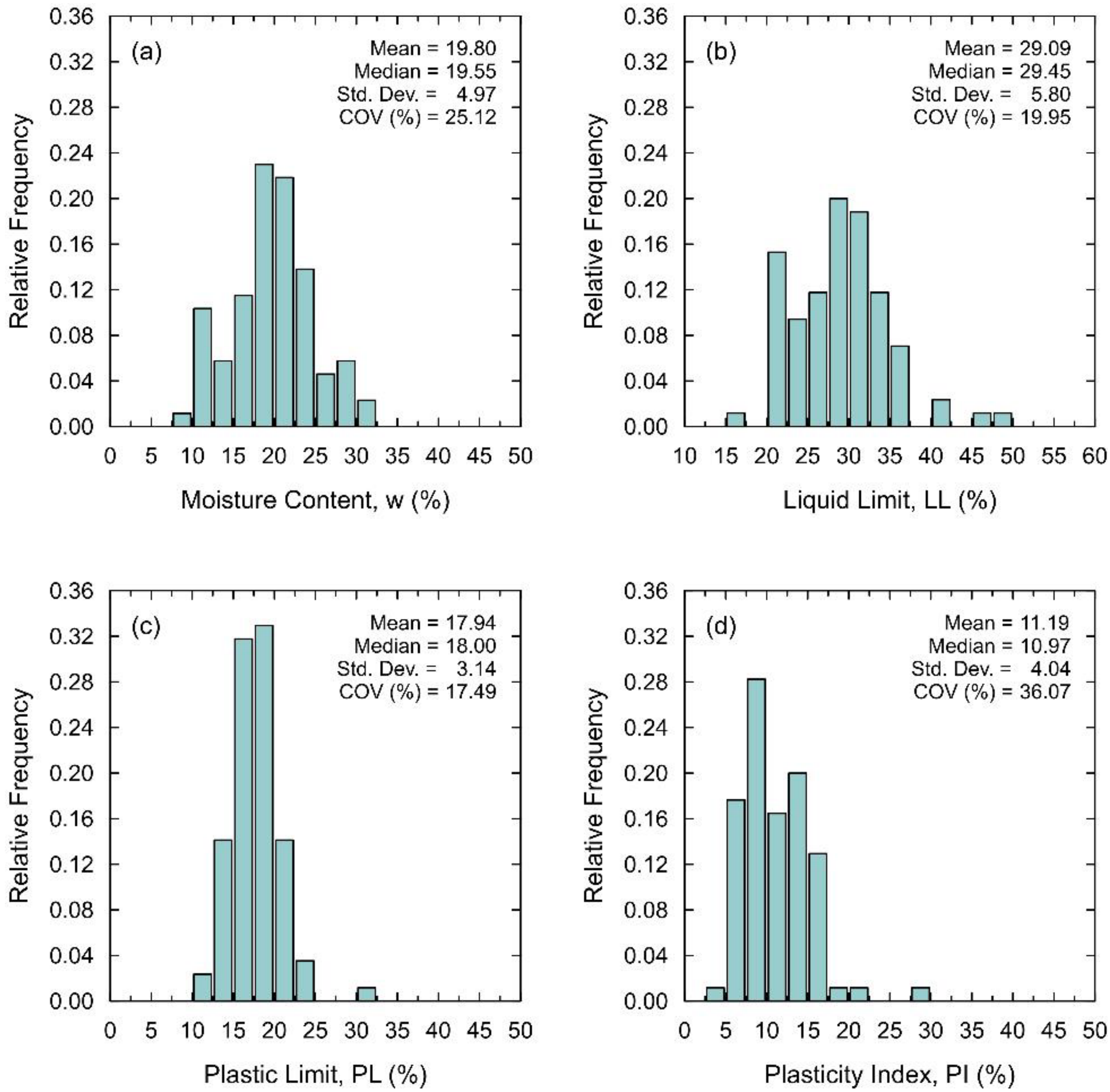


Figure 23. Distribution of soil properties data; (a) moisture content, (b) liquid limit, (c) plastic limit, and (d) plasticity index

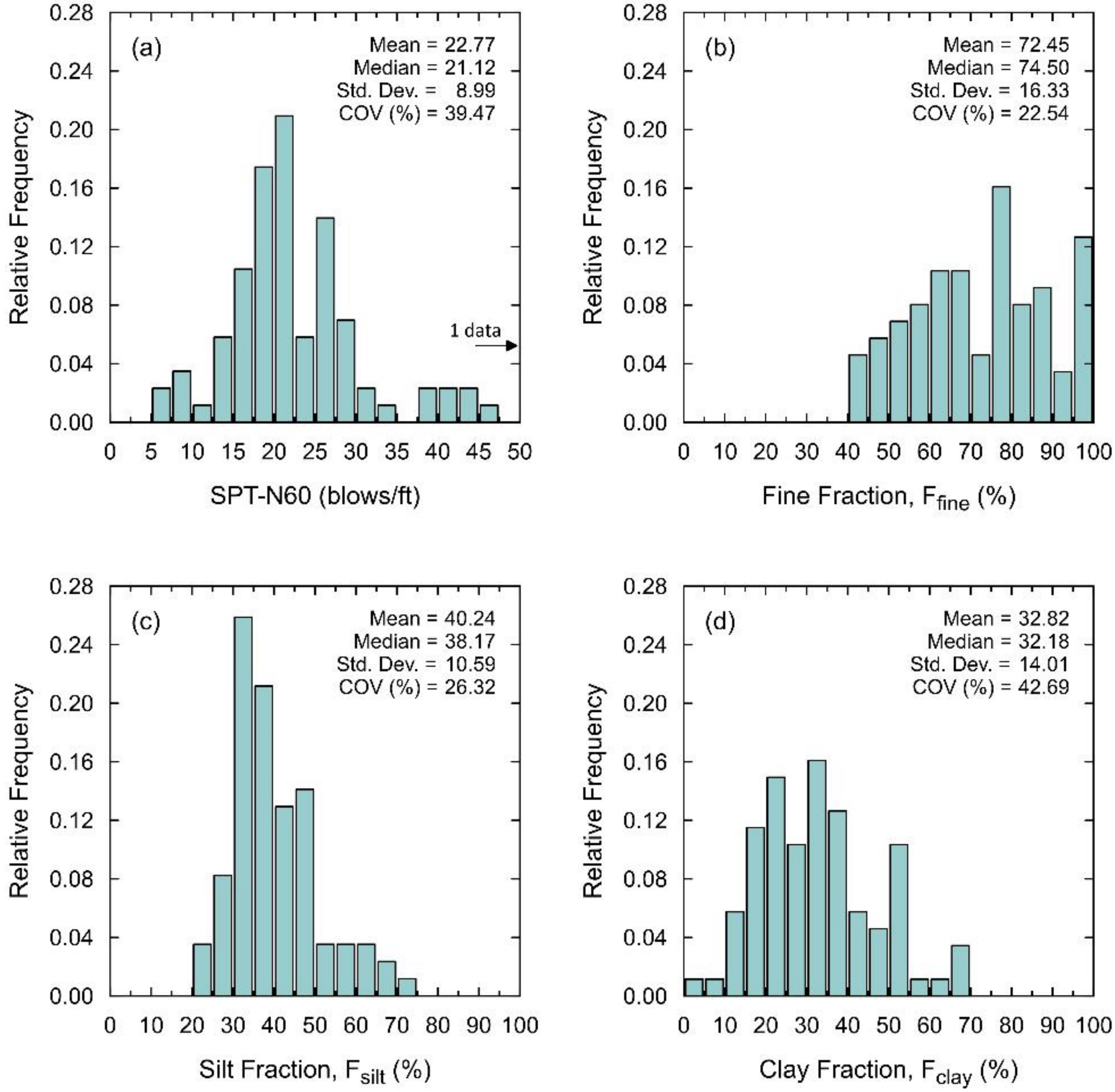


Figure 24. Distribution of soil properties data; (a) SPT-N60, (b) fine fraction, (c) silt fraction, and (d) clay fraction

C.1.3 Pile Resistance

Pile resistance-related statistics are summarized in Table 12 and the distributions of resistance data are provided in Figure 25.

Table 12. Summary of pile resistance data

Parameter	**No. of Data**	**Min.**	**Max.**	**Mean**	**Median**	**Std. Dev.**	**COV (%)**
UBV (kips)	87	76	1,100	371	291	275	74
Q_{EOID} (kips)	87	47	830	222	204	138	62
Q(t) (kips)	109	85	1,672	447	350	321	72
Restrike time, t (days)	109	0.04	57.00	7.56	3.94	11.39	150.62

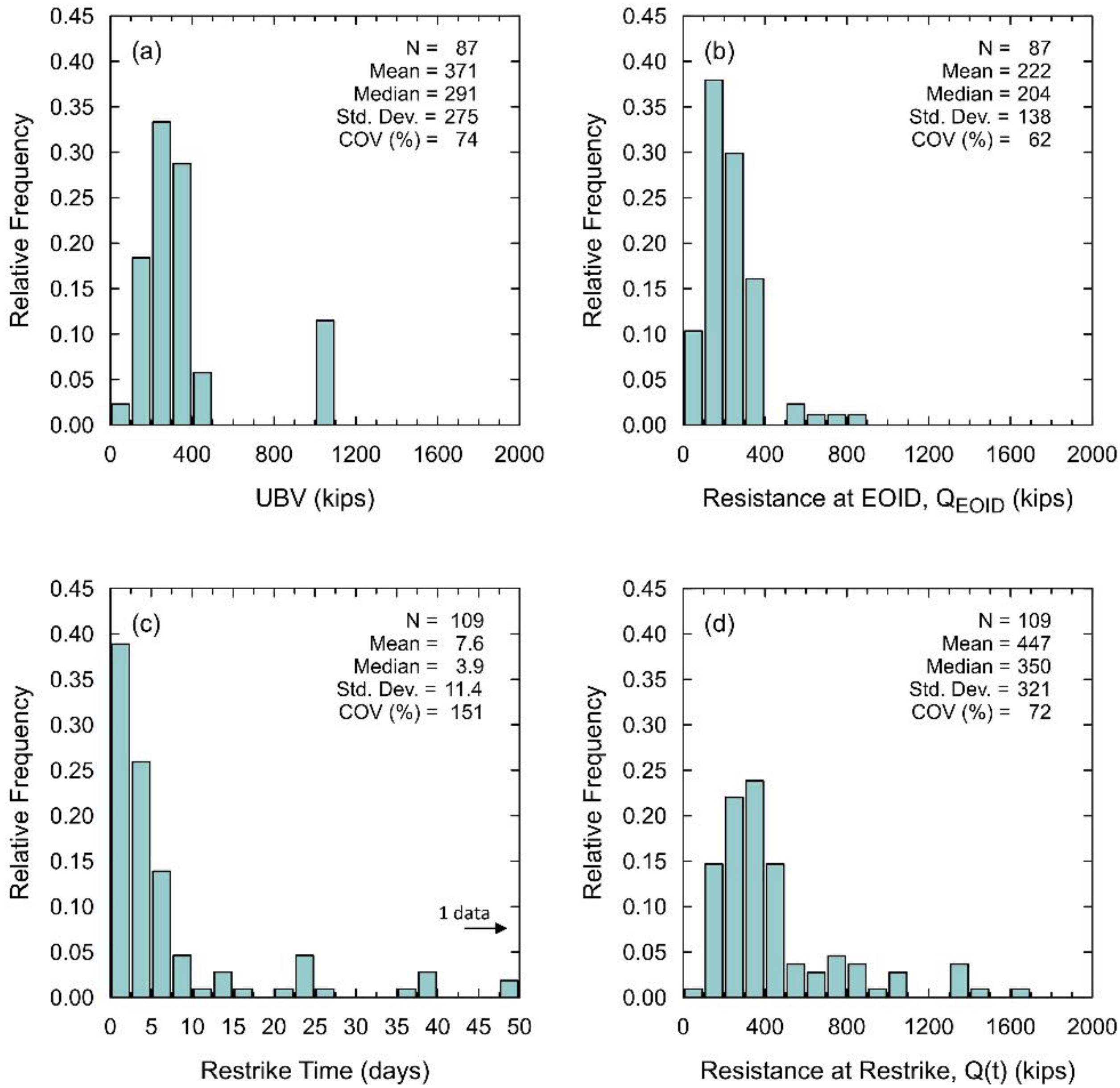


Figure 25. Distribution of pile resistance data; (a) UBV, (b) resistance at initial drive, (c) restrike time after EOID, and (d) resistance at restrike

C.2 Analysis of Data

The setup ratio versus soil properties data for the 87 piles in the database are presented in this section. Figure 26 and Figure 27 present data for setup ratio, *R*, based on pile total resistance. These figures have 87 data points coming from 87 piles in the database. For the piles with multiple restrikes, resistance from the last restrike was used to calculate setup and presented in these figures.

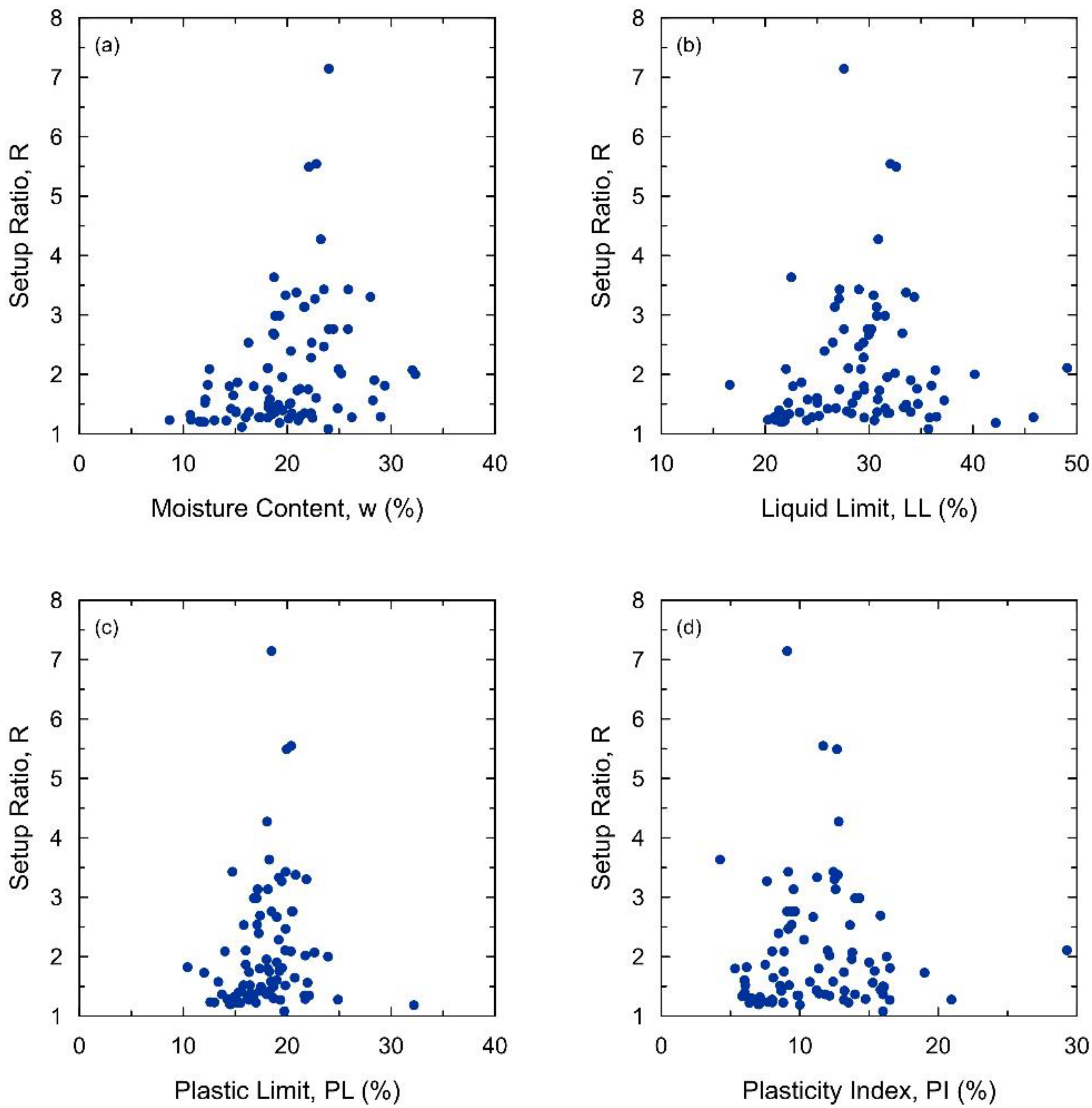


Figure 26. Setup ratio for total resistance versus; (a) moisture content, (b) liquid limit, (c) plastic limit, and (d) plasticity index

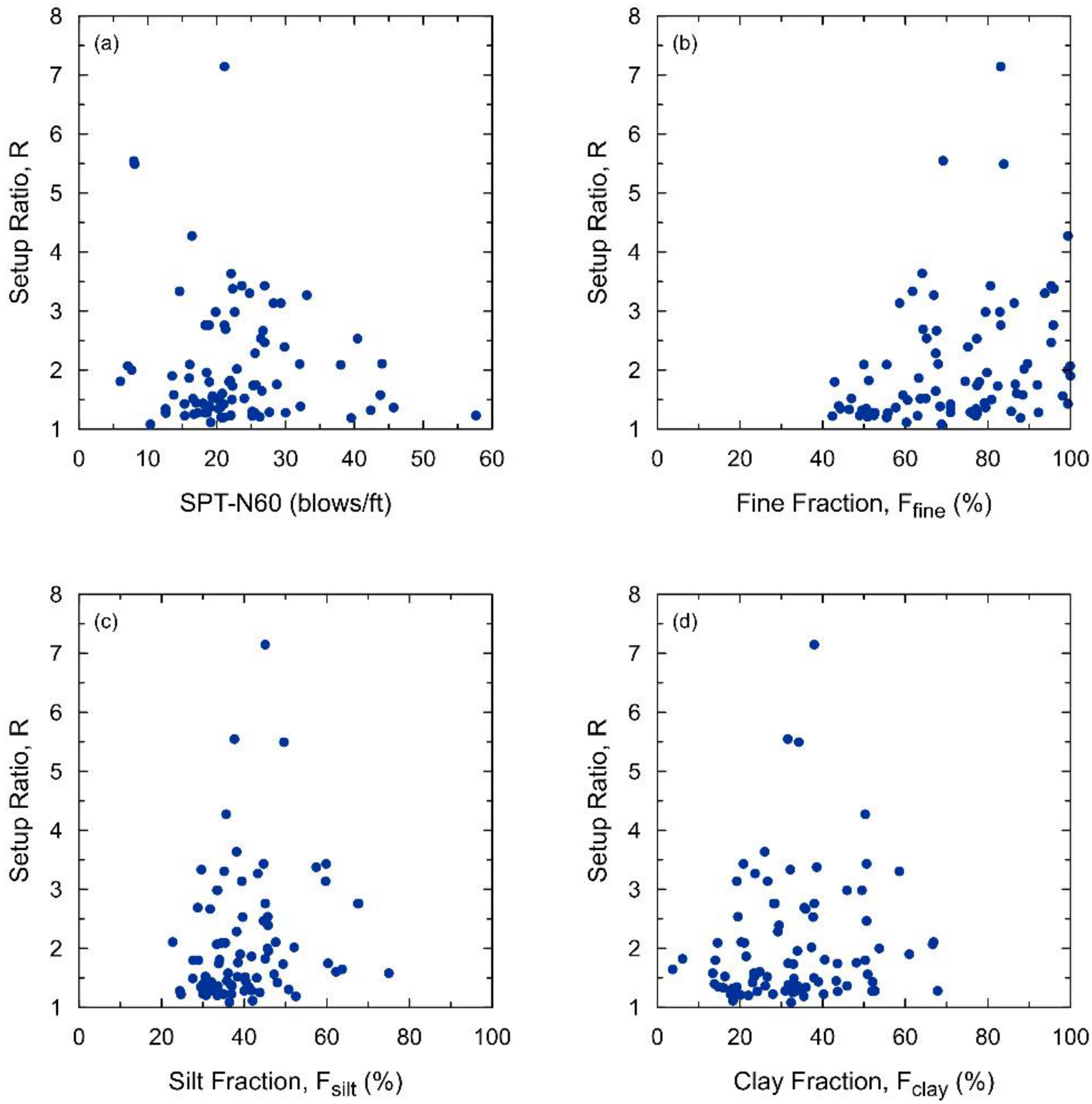


Figure 27. Setup ratio for total resistance versus; (a) SPT-N60, (b) fine fraction, (c) silt fraction, and (d) clay fraction

Figure 28 and Figure 29 presents data for side setup ratio, R_s, based on the side resistance of piles. These figures have 58 data points coming from piles with CAPWAP analysis results in the database. For the piles with multiple restrikes, resistance from the last restrike was used to calculate setup and presented in these figures.

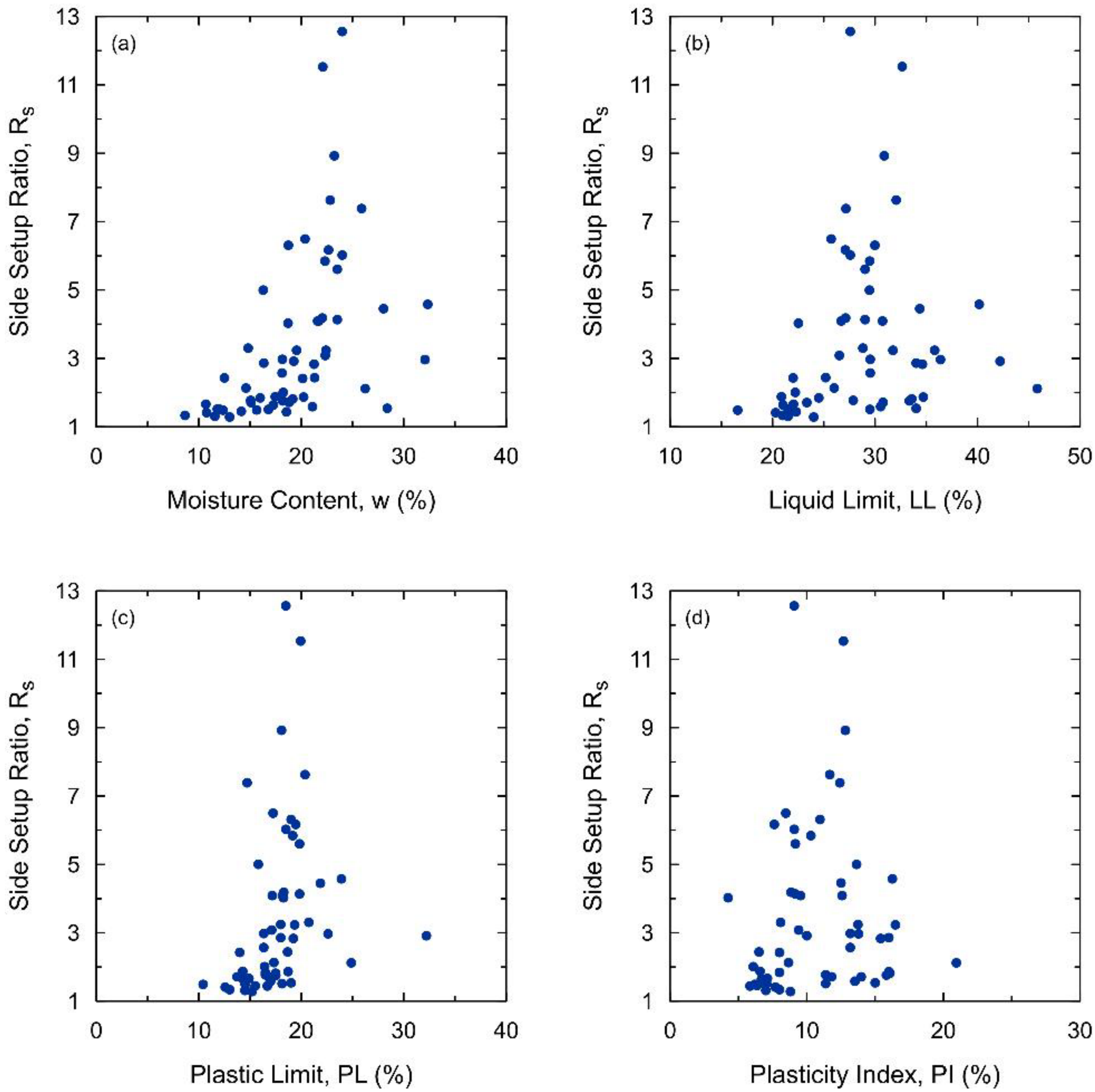


Figure 28. Setup ratio for side resistance versus; (a) moisture content, (b) liquid limit, (c) plastic limit, and (d) plasticity index

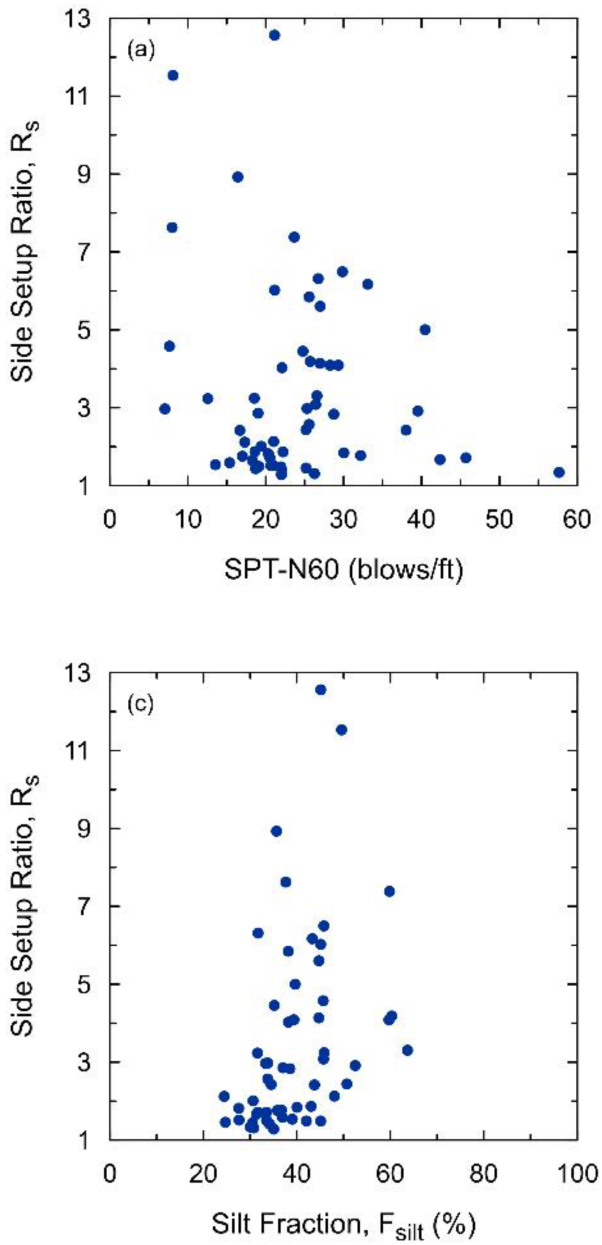


Figure 29. Setup ratio for side resistance versus; (a) SPT-N60, (b) fine fraction, (c) silt fraction, and (d) clay fraction

A detailed analysis of data presented in Figure 26 through Figure 29, did not show any strong or good correlations between the individual soil properties and setup ratios, for neither total nor side setup ratios. Although there were no correlations, there were some trends observed between the setup ratios and individual soil properties. The observed trends in Figure 26 through Figure 29 are summarized in the following:

- Moisture content, *w*: Both total and side setup ratios increase as soil moisture content increases. The increasing trend is stronger for the side setup,
- Liquid limit, *LL*: Although there is no obvious trend between setup ratio and liquid limit for the total resistance, setup ratio for side resistance increases as soil liquid limit increases,
- Plastic limit, *PL*: Both total and side setup ratios increase as soil plastic limit increases. The increasing trend is much stronger for the side setup ratio,
- Plasticity index, *PI*: No trends observed between the setup ratios, both total and side, and the soil plasticity index,
- Standard penetration test blow counts, N60: Both total and side setup ratios decrease as soil SPT-N60 increases. The decreasing trend is stronger for the side setup ratio,
- Fine fraction, F_{fine}: Both total and side setup ratios increase as soil fine fraction increases. The increasing trend is much stronger for the side setup ratio,
- Silt fraction, F_{silt}: Both total and side setup ratios increase as soil silt fraction increases. The increasing trend is significantly much stronger for the side setup ratio,
- Clay fraction, F_{clay}: No clear trends observed between the setup ratios, both total and side, and the soil clay fraction.

APPENDIX D: Analysis of Setup Ratios for Setup Factors

D.1 Setup Factors Confidence Levels

The setup ratios have been analyzed based on various restrike times to provide recommendations for setup factors. The minimum, maximum, average, and 95% confidence levels for different restrike time ranges are shown in Figure 30. Figure 30(a) is based on total resistance and Figure 30(b) is based on side resistance data. The horizontal lines in the plots show the range and the average values. The box is the range for 95% confidence limits.

As the restrike times increase, the average and the confidence level limits increase as shown in Figure 30. For the piles with restrike times more than 7 days and 14 days, lower bound of the 95% confidence level for setup ratio based on total resistance is more than 2.00. For the same time ranges, 95% confidence level for side resistance setup ratio is about 4.00.

Figure 30 shows that the recommended new setup factors of 2.00 for total resistance and 3.00 for side friction have better than 95% confidence levels.

D.2 Piles with Multiple Restrikes

There were 18 piles with multiple restrikes in the database. The trends of the resistance gain with time for the total and side resistance of the piles are shown in Figure 31(a) and Figure 31(b), respectively. In Figure 31, each same colored dots and trend line represent a different pile with multiple restrikes. The recommended new setup factors are also provided in the figures. The figure shows that the majority of piles already have setup ratios more than the recommended setup factors. For the piles that did not reach the recommended setup factors at the time of their final restrikes, their trends suggest that many of them will also reach the recommended setup factors with time. It appears that there will be a few piles that will not reach the recommended values, which is expected since these setup factors were developed based on empirical approaches and with confidence intervals of better than 95%.

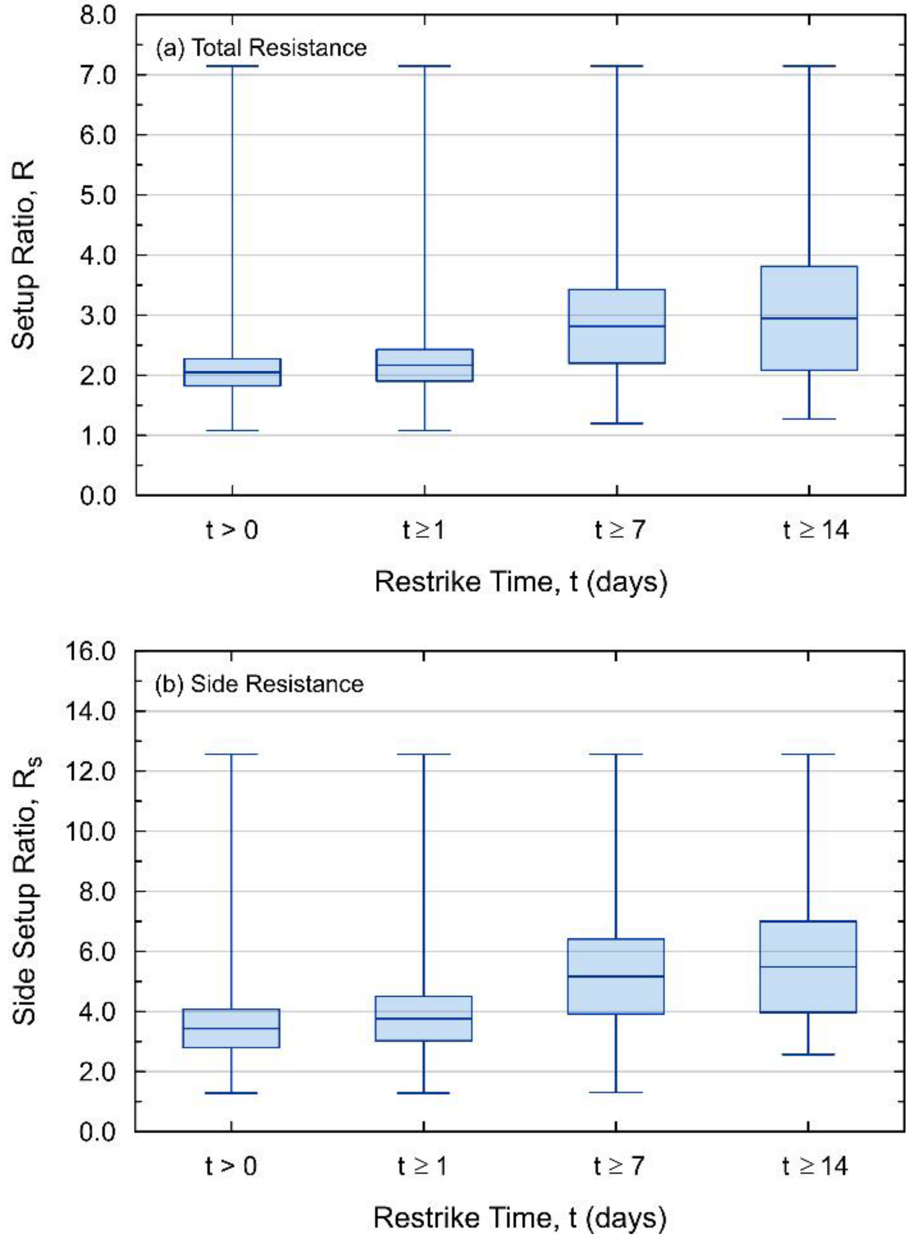


Figure 30. Setup ratios with minimum, maximum, average, and 95% confidence intervals; (a) total resistance and (b) side resistance

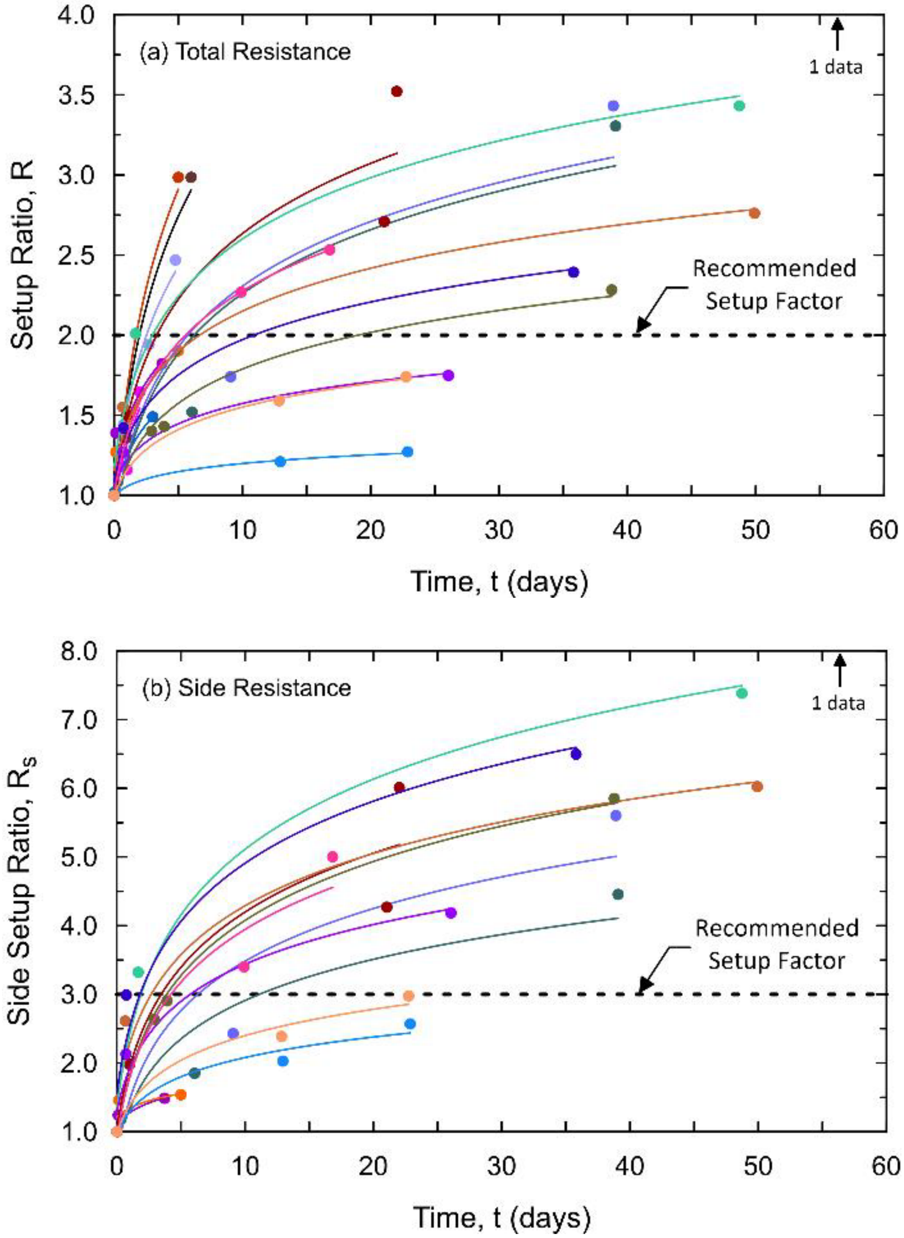


Figure 31. Resistance gain over time for piles with multiple restrikes; (a) total resistance and (b) side resistance

D.3 Investigation of Setup Behavior of Individual Soil Layers

Several piles from different counties were selected for the assessment setup behavior of individual soil layers along the pile shaft. A total of eight piles from six projects were used for this analysis. The piles used in this analysis belong to the following projects:

- HAM-75-0385
- HAM-75-0772
- HAM-52-2044
- HAM-75-1292
- LUC-75-0167R
- CUY-480-1842C

Using the boring logs and dynamic load test results, the soil types from the boring logs were matched to the unit side resistances along the pile shaft from the CAPWAP analysis results. For each layer, the setup ratios were obtained by using the unit side resistance at the time of initial drive and at final restrike. Restrike times of less than seven days are not included in the analysis. For this data set, the minimum restrike time was 7.00 days and the maximum was 40 days, with an average of 24 days. There were a total of 60 setup ratio data used for the assessment of setup ratios for the side resistance of different soil types. The setup ratios ranged from 1.56 to 15.33, with an average of 5.21.

The statistics of the side setup ratios calculated are provided in

Table **13**. The different soil types showed some differences, but all had average side setup ratios greater than 3.00, some much higher. The setup ratios for all data points are plotted by soil types in Figure 32(a). Because of the limited number of data for most soil types, it was not possible to assess the confidence levels for individual soil types. The 95% confidence limits for the combined soil types are shown in Figure 32(b). As provided in the figure, the confidence lower limit for side setup ratio is about 4.30 for the data analyzed using individual layers and soil types. This level is much higher than the new side setup factor of 3.00 recommended in this report, indicating confidence levels higher than 95% for the new setup factor recommended in this report for the side resistance.

At the same confidence level of 95%, the setup ratio for side friction from individual layer analysis, R_s = 4.30 in Figure 32(b), is higher than the one obtained from the side resistance along the total pile length, R_s = 4.00 in Figure 30(b). The piles in this study were installed in predominantly fine-grained soils. Therefore, a higher setup ratio obtained from individual fine-grained soil layers is understandable.

Table 13. Summary of side setup ratios based on individual soil layers[a]

Soil Type	No. of data	Min.	Max.	Mean	Median
A-4a	5	2.75	6.76	3.78	3.28
A-4b	8	2.46	11.13	5.91	5.07
A-6a	36	1.56	11.25	4.94	3.77
A-6b	5	2.19	15.33	7.69	7.64
A-7-6	6	2.51	13.10	4.99	3.41
All soils	60	1.56	15.33	5.21	3.88

[a] - Based on five piles from Hamilton, two piles from Lucas, and one pile from Cuyahoga counties
- Pile restrike time at least seven days after EOID

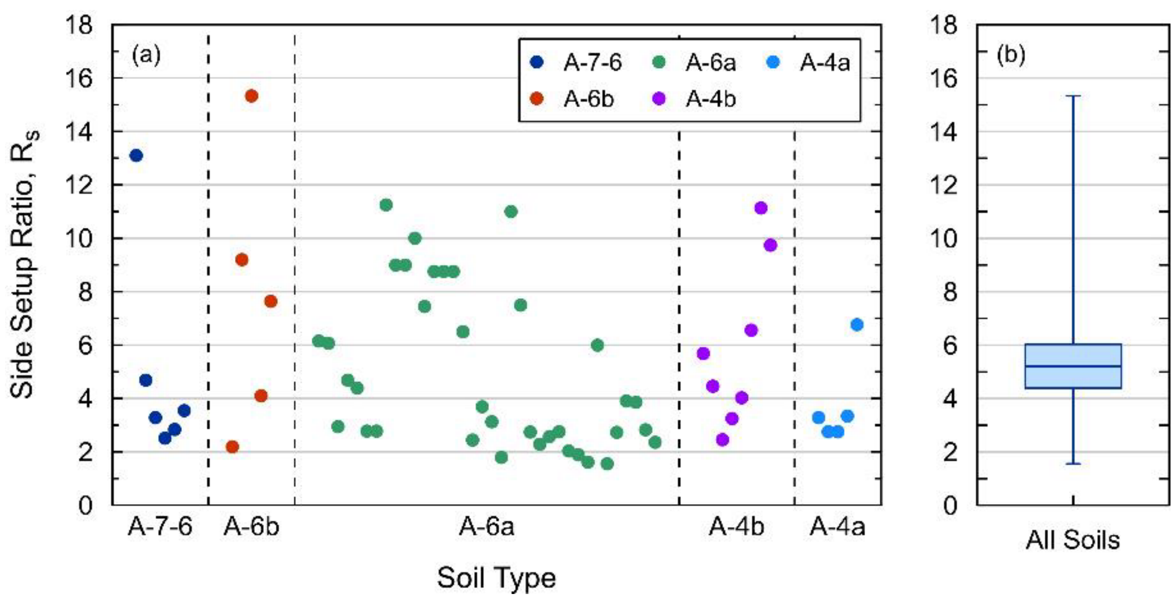


Figure 32. Soil layer based side setup ratios; (a) different soil types and (b) all soils side setup ratio limits, average, and 95% confidence interval range

APPENDIX E: Pile Setup Database

	PROJECT INFO			PILE INFO					LOAD TEST DATA									
									Resistance at EOID (kips)					Resistance at BOR (kips)				
Pile				Pile		Size	Length	UBV	CASE	CAPWAP			Restrike		CASE	CAPWAP		
No	Project	County	Year	No	Pile ID	(in)	(ft)	(kips)	Qo	Qo	Qo,s	Qo,t	No.	t (day)	Q(t)	Q(t)	Q(t),s	Q(t),t
1	LUC-75-0524	LUC	2014	1	1-Rear Abu	12.8	45.0	296	130	140	49	91	1st	0.87	152	191	84	107
													2nd	5.00	418			
2	ASD-42-0359L	ASD	2009	2	35-Fwd Abu	12	15.0	96	186	204	63	141		2.10	286	261	116	145
3				3	28-Pier 2	12	18.7	136	131	147	54	93		0.99	270	209	115	94
4	ASD-89-0294	ASD	2009	4	2-Pier 2	16	30.0	170	255	265	47	218		0.86	286	295	70	225
5	ASD-250-0377	ASD	2012	5	23-Pier 2	16	57.0	376	328	312	212	100		0.60	428	416	305	112
6				6	31-Fwd Abu	14	51.0	260	223	230	119	111		0.85	349	350	239	111
7	MAH-80-0.97	MAH	2006	7	19-Fwd Abu	14	94.0	276	210					2.76	378	378	232	146
8				8	22-Rear Abu	14	75.0	276	238	254	59	195		4.00	311	325	125	200
9	SUM-8-0911	SUM	2008	9	24-Pier	12	58.3	200	210	258	114	144		1.83	335	330	187	143
10				10	3-Rear Abu	12	56.1	186	270	290	120	170		6.00	372	370	225	145
11	FAI-33-0637	FAI	2014	11	72-Fwd Abu	12	64.3	208	152	151	85	66	1st	0.04	154			
													2nd	3.00	225	225	155	70
12				12	31-Fwd Abu	16	21.0	183	118	110	29	81		6.75	128	132	44	88
13	DAR-36-0545	DAR	2006	13	22-Fwd Abu	14	42.0	278	228	235	138	97		0.06	288	290	195	95
14	HAM-75-0385	HAM	2010	14	73-Pier 1	14	65.0	262	56	55	35	20		8.00	310	305	267	38
15				15	146-Fwd Abu	14	41.0	268	236	240	38	202		7.00	310	305	123	182
16				16	4-Rear Abu	14	80.0	268	62	71	32	40		4.00	387	390	369	21
17	HAM-050-1875	HAM	2011	17	34-Pier 1	16	45.6	328	303					5.00	408			
18	HAM-52-2044	HAM	2012	18	13-Rear Abu	14	65.0	370	143					7.08	395			
19				19	191-Pier 2	14	52.0	390	186	228	143	85		24.23	742	715	585	130
20				20	36-Rear Abu	14	37.0	261	100					7.82	158			
21				21	86-Fwd Abu	14	22.0	190	185	180	36	144		3.05	240	246	103	143
22				22	55-Ramp E-Pier 3	14	76.0	390	201					7.85	679			
23	HAM-75-0772	HAM	2013	23	174-SW Wingwall	12	33.0	176	231	102	32	70		11.00	326	272	202	70
24	COL-14-1204	COL	2007	24	7-Rear Abu	14	84.0	156	115					0.85	232			
25	GEA-422-1952	GEA	2007	25	18-Rear Abu	14	73.0	160	72					0.25	237	240	197	43
26	GEA-168-0758	GEA	2007	26	26-Pier 2	16	83.2	340	226	220	72	148		0.63	269	276	174	102
27	DEF-66-0737	DEF	2010	27	41-Rear Abu	14	41.2	359	212					1.00	435	447	64	383
28	MRW-42-21.27	MRW	2016	28	15-Rear Abu	16	33.0	426	336	328	149	179		0.08	419	433	248	185
29	FAI-33-0637	FAI	2015	29	29-S. Wall	16	28.0	176	122	120	45	75		7.00	154	145	59	86
30				30	31-Pier	16	76.0	405	104					3.00	250	280	239	41
31	POR-MR277-0.04	POR	2016	31	10-Fwd Abu	14	60.0	198	52	55	35	21		4.00	189	200	141	59
32	LOR-511-13.15	LOR	2011	32	17-Pier 1	16	33.6	164	118	113	23	90	1st	1.00	131			
													2nd	2.00	164	186	76	110
33	DEL-CR124-0438	DEL	2011	33	32-Fwd Abu	14	81.1	242	137	150	135	15		1.00	275	270	204	66
34	HAM-50-1881	HAM	2015	34	92-Pier 4	16	41.0	350	109					2.00	228			
35	HAM-050-1875	HAM	2012	35	77-Pier 3	16	46.1	316	338					6.00	366			
36				36	34-Pier 1	16	45.6	328	303					5.00	408			
37	MIA-75-1569	MIA	2016	37	42-Pier	16	34.0	372	320	314	98	216		1.00	387	386	131	255
38	ALL-75-0611	ALL	2012	38	56-Rear Abu	12	47.0	323	237	272	122	150		0.86	380	376	216	160
39	CLA-42-0627	CLA	2012	39	34-Fwd Pier	12	40.8	230	165	164	111	53	1st	0.12	242	228	138	90
													2nd	3.72	311	299	165	134
40	HAM-75-0605	HAM	2012	40	90-Pier 2	14	75.0	260	155	157	127	30		0.50	245			
41				41	9-Rear Abu	14	44.0	260	137	135	38	97		1.00	265	270	174	96
42	HAM-75-0647	HAM	2012	42	35-Rear Abu	14	70.0	268	145	138	54	84		2.78	270	270	175	96
43	ASD-250-0377	ASD	2012	43	29-Fwd Abu	14	50.0	260	253					0.95	353			
44	HAM-52-2044	HAM	2013	44	13-Rear Abu	14	65.0	370	143					6.00	395			
45				45	34-P2	14	44.0	255	152	155	107	48		7.00	405	393	330	63
46				46	191-Pier 2 South	14	52.0	390	186	228	143	85		24.00	742	715	585	130
47	HAM-75-0306	HAM	2013	47	5-Rear Abu	14	64.0	291	231	232	95	137		6.00	405	408	269	138
48	TRU-187 (HAR#25)	TRU	2013	48	P-2-Rear Abu	14	47.8	305	246	257	46	211		1.00	325	315	73	242

	PROJECT INFO			PILE INFO					LOAD TEST DATA									
									Resistance at EOID (kips)				Resistance at BOR (kips)					
Pile				Pile		Size	Length	UBV	CASE	CAPWAP			Restrike		CASE	CAPWAP		
No	Project	County	Year	No	Pile ID	(in)	(ft)	(kips)	Qo	Qo	Qo,s	Qo,t	No.	t (day)	Q(t)	Q(t)	Q(t),s	Q(t),t
49	PRE-503-2120	PRE	2012	49	17-P2	16	49.8	430	296					4.00	467			
50	LIC-70-2888	LIC	2014	50	P-36-Fwd	12	71.3	162	144	134	78	56	1st	0.13	176	170	114	56
													2nd	5.00	248	255	120	135
51	ODOT 485(14)	LUC	2015	51	1-Rear Abu	12.8	45.0	296	130	140	49	91	1st	1.00	152	191	84	107
													2nd	6.00	418			
52	ODOT 3007-15	RIC	2015	52	13-P1	16	54.0	252	177	150	71	79		5.00	324	311	211	100
53	LUC-75-04.52	LUC	2015	53	1-Rear Abu	12.8	45.0	296	130	140	49	91		1.00	152	191	84	107
54	ALL-75-0.21	ALL	2014	54	68-Fwd Abu	12	55.0	280	239	239	139	100		0.11	332	326	238	88
55	FAI-33-5.60	FAI	2015	55	31-Fwd Abu	16	21.0	183	118	110	29	81		7.00	128	132	44	88
56	LIC-310-0096	LIC	2016	56	62-P2	12.8	62.0	244	144	153	110	43		3.00	298	320	267	53
57	CUY-77-14.33	CUY	2018	57	233-P2	16	48.8	436	129					5.00	207			
58	MOT-70-0334	MOT	2018	58	74-Fwd Abu	12	53.0	311	207					0.12	266			
59	CUY-77-1409	CUY	2018	59	168-Fwd Abu	16	49.0	380	368	370	82	288		5.00	448	439	239	200
60	CUY-77-1409	CUY	2018	60	4-Rear Abu	16	59.0	383	292	294	164	130		4.00	509			
61	CUY-77-1409	CUY	2018	61	106-CP	16	45.0	350	114					3.00	230	240	169	71
62	LUC-75-0130	LUC	2019	62	164-Fwd Abu	12.8	71.0	326	242					3.94	319	325	133	220
63	HAS-22-0264	HAS	2019	63	87-P3	12	75.0	230	112	107	47	60		3.86	345	350	290	60
64	FRA-71-0308	FRA		64	118-P3	12	20.0	180	145	148	68	80		5.96	225			
65	LUC-75-0130L	LUC	2020	65	168-Fwd Abu	12.8	71.5	326	209	219	68	151		6.00	314			
66	LUC-75-0139R	LUC	2019	66	29-Rear Abu	12.8	87.0	257	230	224	99	125		1.00	325	324	174	150
67	HAS-22-0264	HAS	2019	67	118-Fwd Abu	12	60.0	210	140					0.22	212			
68	FAI-204-0346	FAI	2020	68	22-Fwd Abu	12.8	22.0	180	144	153	75	78		2.00	177	187	109	78
69	ATB 45-1913	ATB	2020	69	14-Fwd Abu	16	70.0	402	355	360	285	75		1.00	447	441	366	75
70	MED-303-19.90	MED	2020	70	8-Rear Abu	12	55.0	230	119	120	75	45		1.29	182	180	140	40
71	WAY-250-1926	WAY	2020	71	21-P2	18	45.5	278	67					4.00	159	125	56	69
72	CUY-21-1004L	CUY	2018	72	37-R Abu	12	85.0	328	98					5.00	140			
73				73	41-R Abu (SLT)	12.8	115.0	328	87	95	41	54		12.96	433	406	366	40
74				74	49-Ret. Wall	12	37.0	76	47					4.00	89	85	44	41
75	CUY-480-1842C	CUY	2018	75	P1-D_Unit1	18	140.0	1,100	333	304	124	180	1st	0.71	443	438	248	190
													2nd	2.72	591	590	380	210
													3rd	5.78	744	750	513	237
76				76	P2-1_Unit1	18	159.0	1,100	340	309	154	155	1st	9.07	524	539	374	165
													2nd	38.91		1,060	863	197
77				77	P3-1_Unit1	18	173.0	1,100	356	325	185	140	1st	6.07	498	493	343	150
													2nd	39.07		1,074	824	250
78				78	P4-D_Unit2	18	190.0	1,100	238	234	123	111	1st	1.00	385	350	243	107
													2nd	21.05	698	634	525	108
													3rd	23.03	781	824	739	85
													4th	73.22		1,672	1,545	127
79				79	P5-1_Unit2	18	140.0	1,100	425	502	172	330	1st	0.67	843	778	449	329
													2nd	49.92		1,386	1,036	350
80				80	P6-1_Unit2	18	150.0	1,100	333	381	141	240	1st	1.67	759	764	468	296
													2nd	48.73		1,307	1,041	267
81				81	P7-1_Unit2	18	123.7	1,100	845	830	195	635	1st	0.70	1,017	1,051	416	635
													2nd	26.04		1,451	816	635
82				82	P8-1_Unit3	18	122.0	1,100	570	570	101	469	1st	0.73	818	811	302	509
													2nd	35.80		1,364	656	708
83				83	P9-A_Unit3	18	120.7	1,100	715	722	148	573		0.96	935	940	361	579
84				84	P10-1_Unit3	18	110.5	1,100	591	610	130	480	1st	2.92	849	851	343	508
													2nd	3.91	829	873	378	495
													3rd	39.78		1,393	760	633
85	HAM-75-12.92	HAM	2019	85	67-Center Pier	14	41.5	226	120	150	50	100	1st	9.91	350	340	170	170
													2nd	16.81	410	380	250	130
86	LUC-75-0167R	LUC	2019	86	40-Fwd Abu	12.8	89.0	330	363	375	100	275	1st	12.94	427	453	203	250
													2nd	22.87	469	477	257	220
87				87	43-Fwd Abu (SLT)	12.8	88.0	330	331	347	87	260	1st	12.85	534	553	208	345
													2nd	22.75	598	604	259	345

SUPPLEMENTAL DOCUMENTS

The following documents are provided in the following pages:

- HAM-75 Project Dynamic Load Test Report
- HAM-75 Project Static Load Test Report
- HAM-75 Project Piezometer Chart

- LUC-75 Project Dynamic Load Test Report
- LUC-75 Project Static Load Test Results Memo
- LUC-75 Project Static Load Test Report

- SUM-76 Project Dynamic Load Test Report
- SUM-76 Project Static Load Test Results Memo
- SUM-76 Project Static Load Test Report