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16. ABSTRACT Cast-in-Steel-Shell (CISS) piles are widely used in Caltrans bridge foundations, yet the influence of the soil plug that remains inside the shell after partial cleanout on the mobilized geotechnical resistance has not been verified with field measurements. This study quantified the effect of the remaining soil plug length on the axial load transfer behavior of CISS piles and evaluated the API design procedure required by the California Amendments to the AASHTO LRFD Bridge Design Specifications, together with the alternative Model 3 method developed by Caltrans DRISL. Four nominally identical open-ended steel shells (3 ft diameter, 40 ft embedment) were driven and cleaned out to target plug lengths of 0, 2D, 4D, and 6D, and were tested in two series of top-down compression load tests to 600 kips and 1,150 kips. Distributed fiber optic sensing (DFOS) installed in four configurations survived impact driving and provided high-resolution axial strain profiles, from which layer-wise t-z and q-z relationships were back-calculated by inverse analysis. Finite-element simulations were performed to interpret the pile-soil interaction mechanisms. Geotechnical failure was not reached in any test, and the load-displacement response remained largely linear. The back-calculated ultimate shaft capacity exceeded the Model 3 and API predictions by factors of approximately 2.8 and 5.7, respectively, indicating that both design methods are substantially conservative for shaft resistance under the tested site conditions. Finite-element inverse analyses required soil moduli 15 to 29 times the CPT-based estimates, showing that shell driving substantially strengthens and stiffens the surrounding soil relative to its pre-installation state. Because the mobilized soil resistance greatly exceeded the design estimates, the test loads, which were sized using the design methods, were carried almost entirely by shaft resistance at small displacements, and the pile tip and soil plug were only marginally mobilized. Consequently, within the applied load range, the remaining plug length from 0 to 6D had no measurable influence on pile head displacement or shaft resistance, and the end-bearing and plug contributions could not be separated from the measurements.		
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Evaluation of Soil Plug Geotechnical Resistance in the Design of CISS Piles

Final Report

Gyu-Beom Shin, Connor Geudeker, Kenichi Soga

September 11, 2026

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1 Introduction

Cast-in-Steel-Shell (CISS) piles are a deep foundation system in which a steel shell is driven to serve as a permanent form for a cast-in-place reinforced concrete pile. Their high structural capacity and seismic performance make them attractive in (1) soft or loose ground (e.g., bay mud or loose sands), (2) sites with liquefaction or scour potential that may create long unsupported pile lengths, and (3) locations where large lateral ground movements are anticipated (Caltrans, 2026).

Figure 1 summarizes a typical CISS pile construction sequence. In Stage A, the steel shell is advanced to the target depth by impact driving, and a soil plug may develop inside the shell. Stage B marks drive completion and preparation for internal excavation. In Stage C, soil is removed from within the shell while leaving a designed soil plug length; this remaining plug condition can strongly influence mobilized shaft and tip resistance. In Stage D, the reinforcing cage is installed and concrete is placed to form the final composite pile section. In Stage E, the superstructure load is transferred to the pile and resisted through outer shaft resistance, tip resistance, and any contribution associated with the remaining soil plug.

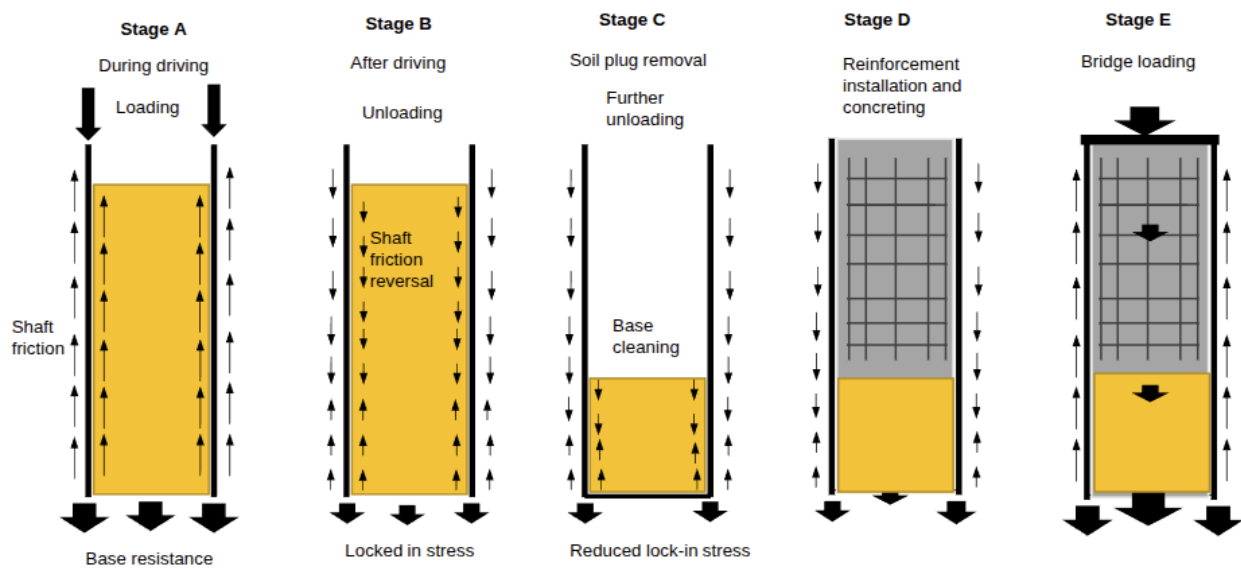


Figure 1: Typical construction sequence for Cast-in-Steel-Shell (CISS) piles (Stages A–E).

Despite their widespread use, predicting the geotechnical resistance of CISS piles remains challenging due to the complex pile–soil interaction during construction. The combined effects of construction stages, including soil state alterations during impact driving as well as the formation and subsequent partial removal of a soil plug, significantly influence the pile load transfer behavior. The California Amendments to the AASHTO LRFD Bridge Design Specifications (Caltrans, 2019) require that CISS piles be designed in accordance with the design procedure specified by the American Petroleum Institute (API, 2000). While the API design procedure provides a rational basis for estimating axial resistance for open-ended steel pipe foundations, its applicability to Caltrans practice for CISS piles where the initial driven state is significantly modified by partial cleanout to a target plug length followed by reinforcement and concrete placement has not been fully verified using field measurements. Therefore, a project-specific evaluation is needed to assess the suitability of the API design approach for CISS piles constructed using Caltrans practice.

The objective of this project is to quantify how the remaining soil plug condition affects mobilized shaft and tip resistance and to evaluate whether current design assumptions provide a reasonable representation of measured behavior. To address this objective, instrumented full-scale top-down load tests were performed

on piles at Stage C with varying soil plug lengths. Load–displacement response and axial strain profiles were measured, and the results were interpreted to develop load transfer relationships (including t – z and q – z curves). The measured behavior was then compared with predictions from the API design method and from an alternative design method (Model 3) developed by Caltrans DRISI (Shantz, 2019), providing a basis to assess design conservatism and identify where consideration of construction-stage effects may improve design reliability.

1.1 Scope of Work

The scope of work completed to fulfill the project objectives can be grouped into three major components:

- **Review of current practice and design procedures:** Review Caltrans practice for CISS piles and summarize the design procedures relevant to geotechnical resistance, focusing on the API (2000) procedure required by the California Amendments to the AASHTO LRFD Bridge Design Specifications and the alternative Model 3 approach.
- **Full-scale field testing and interpretation:** Plan and execute an instrumented field load-testing program to evaluate the influence of remaining soil plug length on CISS pile geotechnical resistance. This effort includes design and construction of test and reaction piles, definition of test conditions with varying target plug lengths, implementation of instrumentation, performance of top-down load tests, and interpretation of measured load–displacement behavior and axial strain profiles to assess load transfer and resistance components.
- **Finite-element analysis:** Conduct finite-element simulations to support interpretation of pile–soil interaction mechanisms and to provide a complementary basis for comparing measured responses with design predictions.

1.2 Organization of Report

This report is organized into six chapters. Chapter 1 introduces the project background, objectives, scope of work, and report organization. Chapter 2 summarizes the current design frameworks for CISS piles and focuses on the static analysis procedures used to estimate axial pile resistance, including the API-based design procedure and the Model 3 approach. Chapter 3 describes the full-scale load test program, including site conditions and subsurface characterization, overall test setup and procedures, and instrumentation. Particular emphasis is placed on the installation and configuration of the Distributed Fiber Optic Sensor (DFOS) system used as a primary monitoring technique in this study. Chapter 3 also presents preliminary pile resistance estimates based on CPT-derived soil properties to support test planning and interpretation. Chapter 4 presents the field test results and interpretation. The overall pile response is first summarized using load–displacement behavior, followed by detailed evaluation of axial strain profiles measured by DFOS. Inverse analyses are then performed to back-calculate load transfer relationships (i.e., t – z and q – z curves), and the interpreted resistances are compared with predictions from the API and Model 3 procedures. Chapter 5 describes the finite-element modeling conducted to provide additional insight and support interpretation of the field results. This chapter documents the modeling assumptions, boundary conditions, and analysis procedures; presents calibration of constitutive model parameters through comparisons with measured responses; and discusses key simulation outcomes and their implications for CISS pile behavior. Chapter 6 summarizes the main conclusions and provides recommendations for future work.

2 Background

2.1 Caltrans Practice and Design Procedures for Soil Plugs

This section summarizes Caltrans requirements and guidance for the construction and design of CISS piles based on the California Amendments to the AASHTO LRFD Bridge Design Specifications (Caltrans, 2019), the Caltrans Geotechnical Manual (Caltrans, 2021), the Memo to Designers (MTD) (Caltrans, 2014), the Bridge Design Memo (BDM) 10.7: Piles and Shafts (Caltrans, 2026), the Caltrans Standard Specifications (Caltrans, 2025), and the Caltrans Foundation Manual (Caltrans, 2015). Caltrans practice specifies that CISS piles may be constructed using either open-ended or closed-ended steel shells. Because this research mainly focuses on the soil plug effect, the discussion below emphasizes open-ended CISS piles, for which soil may enter the shell during driving and form a soil plug.

For construction, Caltrans requirements and recommendations related to soil plugs are provided in the Caltrans Standard Specifications and BDM 10.7. The Standard Specifications require cleanout inside the shell after driving and state that the shell must be free of materials deleterious to the bond between concrete and steel prior to concrete placement. The Standard Specifications further state that the bottom 8 ft of an open-ended CISS pile must not be cleaned out, thereby requiring a remaining soil plug (or soil column) to be maintained near the pile tip. BDM 10.7 provides practical guidance on the soil plug and construction considerations. It notes that a soil plug is left intact at the bottom of open-ended CISS piles to prevent undermining during the cleanout process, that the soil plug may also be utilized to counteract the hydrostatic forces of groundwater when the pile is internally cleaned out, and that the top of soil plug elevation or soil plug length is specified in the project Foundation Report. The superseded Memo to Designers (MTD) 3-1 further stated that a soil plug of approximately two pile diameters in length will often maintain water control and that leaving as much of the soil plug in place as possible is desirable from a geotechnical standpoint (Caltrans, 2014).

For pile design, the California Amendments require that steel pipe piles, including CISS piles larger than 18 in. in diameter, follow the static analysis method specified by API (2000). In the Caltrans Geotechnical Manual, the contribution of the internal soil plug is evaluated by comparing the soil plug tip resistance and the internal soil plug side resistance, and the lesser value is adopted as the plug-related resistance component. This procedure is conceptually motivated by the distinction between plugged and unplugged behavior: in a plugged condition, axial resistance is associated with external shaft resistance and end bearing mobilized over the full cross-sectional area, whereas in an unplugged condition, resistance may include external shaft resistance, internal shaft resistance along the soil plug and end bearing mobilized over the annular area at the pile tip. For calculating the internal soil plug side resistance, the contributing plug length is limited to four times the pile diameter. The detailed calculation procedure specified by API (2000) is provided in the following section.

2.2 API Design Procedures

This section summarizes the static analysis method presented in API (2000) for estimating the ultimate bearing capacity of CISS piles. The ultimate bearing capacity of a CISS pile is determined as

$$Q_u = Q_s + Q_t + Q_{\text{plug}}, \quad (1)$$

where Q_u denotes the ultimate bearing capacity, Q_s is the external shaft resistance along the outer surface of the pile, Q_t is the tip resistance mobilized on the annular pile tip area, and Q_{plug} is the soil plug-related resistance component. Following the API design procedure, the plug-related resistance is taken as

$$Q_{\text{plug}} = \min(Q_{\text{plug},s}, Q_{\text{plug},t}), \quad (2)$$

where $Q_{\text{plug},s}$ denotes the internal shaft resistance along the soil plug (friction between the soil plug and the inner pile wall) and $Q_{\text{plug},t}$ is the tip resistance associated with the soil plug.

Accordingly, the ultimate bearing capacity can be expressed as

$$Q_u = fA_{s,o} + qA_{t,ann} + \min(fA_{s,i}, qA_{t,plug}), \quad (3)$$

where f is the unit shaft friction, q is the unit tip resistance, $A_{s,o}$ is the outer surface area of the pile, $A_{s,i}$ is the inner surface area of the pile, $A_{t,ann}$ is the annular tip area of the pile wall, and $A_{t,plug}$ is the end area of the soil plug corresponding to the inner diameter of the pile. The detailed calculation procedure specified by API (2000) is provided in the following section.

Cohesive soils For cohesive soils, the API method computes the unit shaft friction as

$$f = \alpha c, \quad (4)$$

where c is the undrained shear strength and α is a dimensionless adhesion factor. The parameter α is defined in terms of

$$\psi = \frac{c}{p'_o}, \quad (5)$$

where p'_o is the effective overburden pressure. The API expressions for α are

$$\alpha = 0.5 \psi^{-0.5}, \quad \psi \leq 1.0, \quad (6)$$

$$\alpha = 0.5 \psi^{-0.25}, \quad \psi > 1.0, \quad (7)$$

with the constraint $\alpha \leq 1.0$. For tip resistance in cohesive soils, the unit tip resistance is computed by

$$q = 9c. \quad (8)$$

Cohesionless soils For cohesionless soils, the unit shaft friction and the unit tip resistance are expressed as

$$f = \beta p'_o, \quad (9)$$

$$q = N_q p'_{o,tip}, \quad (10)$$

where β denotes the dimensionless shaft friction factor, p'_o is the effective overburden pressure at the depth in question, and N_q is the dimensionless bearing capacity factor. Recommended β and N_q values are presented in Table 1. For long piles, it is recommended to limit f and q to f_{lim} and q_{lim} given in Table 1, because f and q may not increase linearly with overburden pressure.

2.3 Model 3

Model 3 is an SPT-based pile resistance model developed by Caltrans DRISI for driven piles in cohesionless soils (Shantz, 2019). The method was developed from the Caltrans pile load-test database to improve the prediction of axial resistance in sand, particularly for large-diameter open-ended pipe piles where conventional design methods were found to overpredict resistance. Unlike the API procedure, which estimates the shaft and tip resistances using effective overburden stress and empirical bearing capacity factors, Model 3 uses the energy-corrected SPT blow count, N_{60} , as the primary soil resistance index. The model also explicitly considers three important mechanisms: friction fatigue along the pile shaft, reduced shaft resistance in uplift loading, and the effect of coring and plugging for open-ended piles. Because Model 3 was developed for cohesionless soils, the sand formulation is applied to cohesionless layers. For mixed soil profiles, cohesive layers should be evaluated separately using an appropriate clay resistance model.

Table 1: Design parameters for cohesionless soils (API Table 6.4.3-1).

Relative density	Soil description	β	f_{lim} (ksf)	N_q	q_{lim} (ksf)
Very Loose	Sand				
Loose	Sand				
Loose	Sand–Silt				
Medium Dense	Silt				
Dense	Silt				
			Not applicable		
Medium Dense	Sand–Silt	0.29	1.4	12	60
Medium Dense	Sand				
Dense	Sand–Silt	0.37	1.7	20	100
Dense	Sand				
Very Dense	Sand–Silt	0.46	2.0	40	200
Very Dense	Sand	0.56	2.4	50	250

Shaft resistance For cohesionless soils, the unit shaft resistance is evaluated as

$$f_s(z) = aN_{60}^*(z)F_{\text{dir}}F_{\text{ff}}(z)F_{\text{OE}}, \quad (11)$$

where $f_s(z)$ is the unit shaft resistance at depth z , a is an empirical coefficient, N_{60}^* is the corrected SPT blow count used in the calculation, F_{dir} is a loading-direction factor, F_{ff} is a friction-fatigue factor, and F_{OE} is a correction factor for open-ended piles. Following the Model 3 recommendation, N_{60} is limited to a maximum value of 100:

$$N_{60}^*(z) = \min[N_{60}(z), 100]. \quad (12)$$

The loading-direction factor is defined as

$$F_{\text{dir}} = \begin{cases} 1.0, & \text{compression loading,} \\ 0.75, & \text{uplift loading.} \end{cases} \quad (13)$$

The friction-fatigue factor accounts for the reduction in shaft resistance with increasing distance above the pile tip. It is expressed as

$$F_{\text{ff}}(z) = \min \left[3^{-b} \left(\frac{h}{D} \right)^b, 1 \right], \quad (14)$$

where h is the distance above the pile tip, D is the pile diameter, and b is an empirical coefficient controlling the friction-fatigue effect. This formulation limits F_{ff} to unity near the pile tip and gradually reduces the shaft resistance at locations farther above the tip.

For open-ended piles, the reduced radial stress caused by coring action during driving is considered using

$$F_{\text{OE}} = \left(1 - \frac{D_i^2}{D_o^2} PLR^* \right)^g, \quad (15)$$

where D_i and D_o are the inside and outside pile diameters, respectively, g is an empirical coefficient, and PLR^* is a modified plug length ratio. For closed-ended or solid piles, $F_{\text{OE}} = 1.0$. The modified plug length ratio is estimated as

$$PLR^* = \begin{cases} 0.036 + 0.0364D, & D < 24 \text{ in.}, \\ \min(0.75 + 0.0067D, 1.0), & D \geq 24 \text{ in.}, \end{cases} \quad (16)$$

where D in Eq. (16) is expressed in inches.

The total shaft resistance is then obtained by integrating the unit shaft resistance over the external pile–soil contact area:

$$Q_s = \int_{A_{s,o}} f_s(z) dA_{s,o}, \quad (17)$$

where $A_{s,o}$ denotes the external shaft area along the outside of the pile. For a circular pile, this may be written as

$$Q_s = \int_0^L f_s(z) \pi D_o dz. \quad (18)$$

End-bearing resistance The end-bearing resistance is evaluated using an equivalent tip blow count, N_{tip} , which represents the average N_{60} near the pile tip. The unit end-bearing pressure is calculated as

$$q_b = d N_{\text{tip}}, \quad (19)$$

where d is an empirical coefficient that converts the equivalent tip blow count into the ultimate end-bearing pressure. For solid or closed-ended piles, the end-bearing resistance is

$$Q_b = q_b A_p, \quad (20)$$

where A_p is the pile base area.

For open-ended piles, the end-bearing resistance is calculated as the sum of the annular tip resistance and the plug resistance:

$$Q_b = q_b A_{\text{ann}} + Q_p, \quad (21)$$

where A_{ann} is the annular area of the pile wall and Q_p is the plug resistance. The plug resistance is taken as the smaller of the geotechnical end-bearing resistance over the plug area and the frictional resistance of the soil plug:

$$Q_p = \min (F_{\text{mob}} q_b A_{\text{plug}}, Q_{p-\text{fric}}), \quad (22)$$

where A_{plug} is the cross-sectional area inside the pile and F_{mob} is a mobilization factor that accounts for the reduction in end-bearing resistance caused by coring action during driving. The frictional plug resistance is estimated as

$$Q_{p-\text{fric}} = d N_{\text{max},3D} A_{\text{plug}} \max (1 - PLR^*, 0.1), \quad (23)$$

where $N_{\text{max},3D}$ is the maximum N_{60} value evaluated along the lower portion of the pile penetration, considering that plug resistance is mainly developed near the lower several pile diameters.

For layered soil profiles, N_{tip} is not taken simply as the blow count at the pile tip. Instead, Model 3 evaluates an equivalent value by averaging N_{60} over two intervals around the pile tip and taking the smaller value:

$$N_{\text{tip}}(z) = \min [\bar{N}_{60}(z - 3D \text{ to } z + 1.5D), \bar{N}_{60}(z - 1.5D \text{ to } z + 8D)]. \quad (24)$$

For open-ended piles, the diameter used in the N_{tip} averaging intervals is replaced by an effective diameter,

$$D^* = \sqrt{D_o^2 - PLR^* D_i^2}, \quad (25)$$

to account for the reduced displacement associated with coring. In contrast, the pile diameter D is retained in the friction-fatigue factor for consistency with the Caltrans Model 3 formulation.

Model coefficients The empirical coefficients in Model 3 were calibrated from the Caltrans pile load-test database. These coefficients were selected through a grid-search procedure to obtain approximately neutral bias, defined as a mean measured-to-predicted resistance ratio close to unity, while minimizing the standard deviation of the model error. For the recommended Model 3, the coefficients are

$$a = 150 \text{ psf/blow}, \quad b = -0.5, \quad d = 5.5 \text{ ksf/blow}, \quad g = 0.30, \quad (26)$$

where a and d both carry units of resistance per blow count, in psf/blow and ksf/blow respectively, whereas b and g are dimensionless. Specifically, a converts the corrected SPT blow count, N_{60} , into a unit shaft resistance, d converts the equivalent tip blow count into an ultimate end-bearing pressure, b controls the friction-fatigue effect, and g controls the open-ended pile correction associated with coring and plugging. In this study, Model 3 is used as an alternative SPT-based reference method for estimating pile resistance and comparing with the measured load-test results. However, its application to CISS piles with partially cleaned-out soil plugs should be interpreted with caution because Model 3 was developed primarily for driven piles in cohesionless soils, and its plug-related terms are based on the plugging and coring behavior that develops during pile driving rather than the subsequent construction-stage removal of the soil plug.

3 Load Test Program

3.1 Test Site and Site Characterization

The site was located in the storage yard of Foundation Constructors, Inc. (FCI), selected for its accessibility and availability of previous exploratory borings that could be utilized for estimation of the pile load capacity. Borings indicated that the subsurface was composed primarily of medium dense sands with interstitial layers of a silty clay and a dense sand, which will be discussed in more detail below. Proximity to the Sacramento-San Joaquin river delta led to a water table observed at approximately 4 ft below grade with slight variation due to tidal influence.

Following the pile load tests, additional cone penetration tests (CPTs) were performed to obtain a more detailed subsurface characterization, as the capacities estimated from the initial boring data did not fully agree with the observed pile response. The test site and subsurface investigation locations are shown in Figure 2.

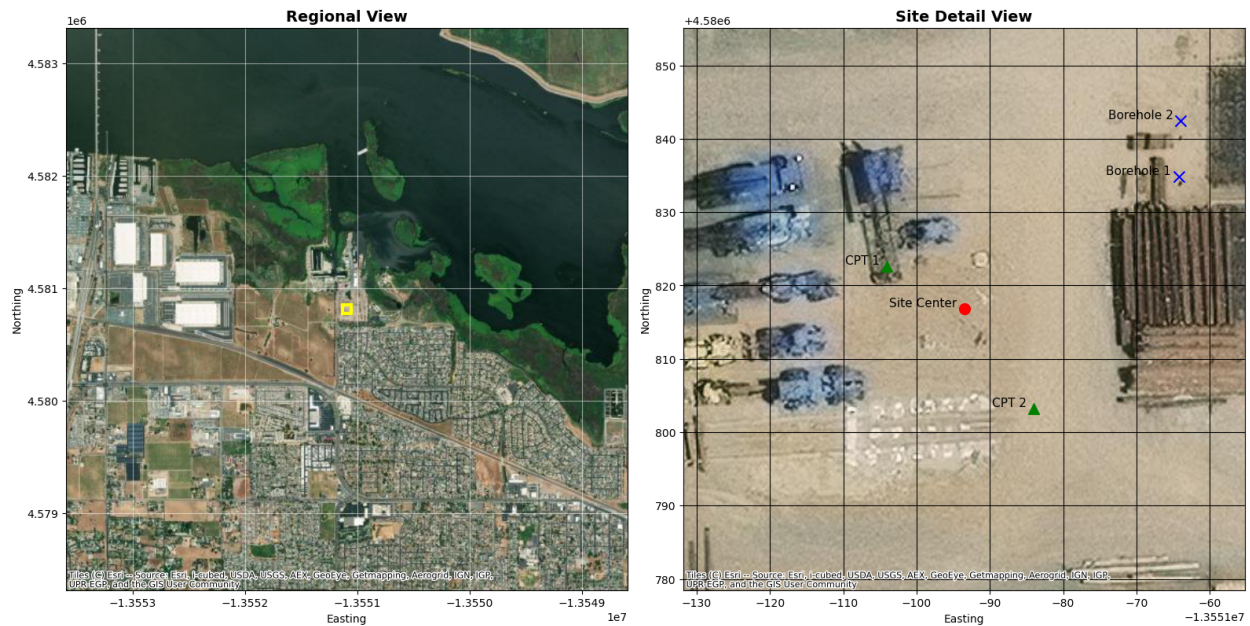


Figure 2: Site map indicating locations for subsurface investigation

To develop the finalized subsurface profile, data from two test borings were compiled and reviewed. These borings extended to a depth of 71.5 ft below ground surface, and a total of 12 soil samples were collected for laboratory testing. The formal boring logs and sample testing reports are provided in Appendix A and Appendix B, respectively. Figure 3 summarizes the boring log data, including the gradation curves, Atterberg limits, and interpreted subsurface profiles. Similar subsurface conditions were observed in the two borings to a depth of approximately 64 ft below ground surface, which is below the primary depth range of interest for this project.

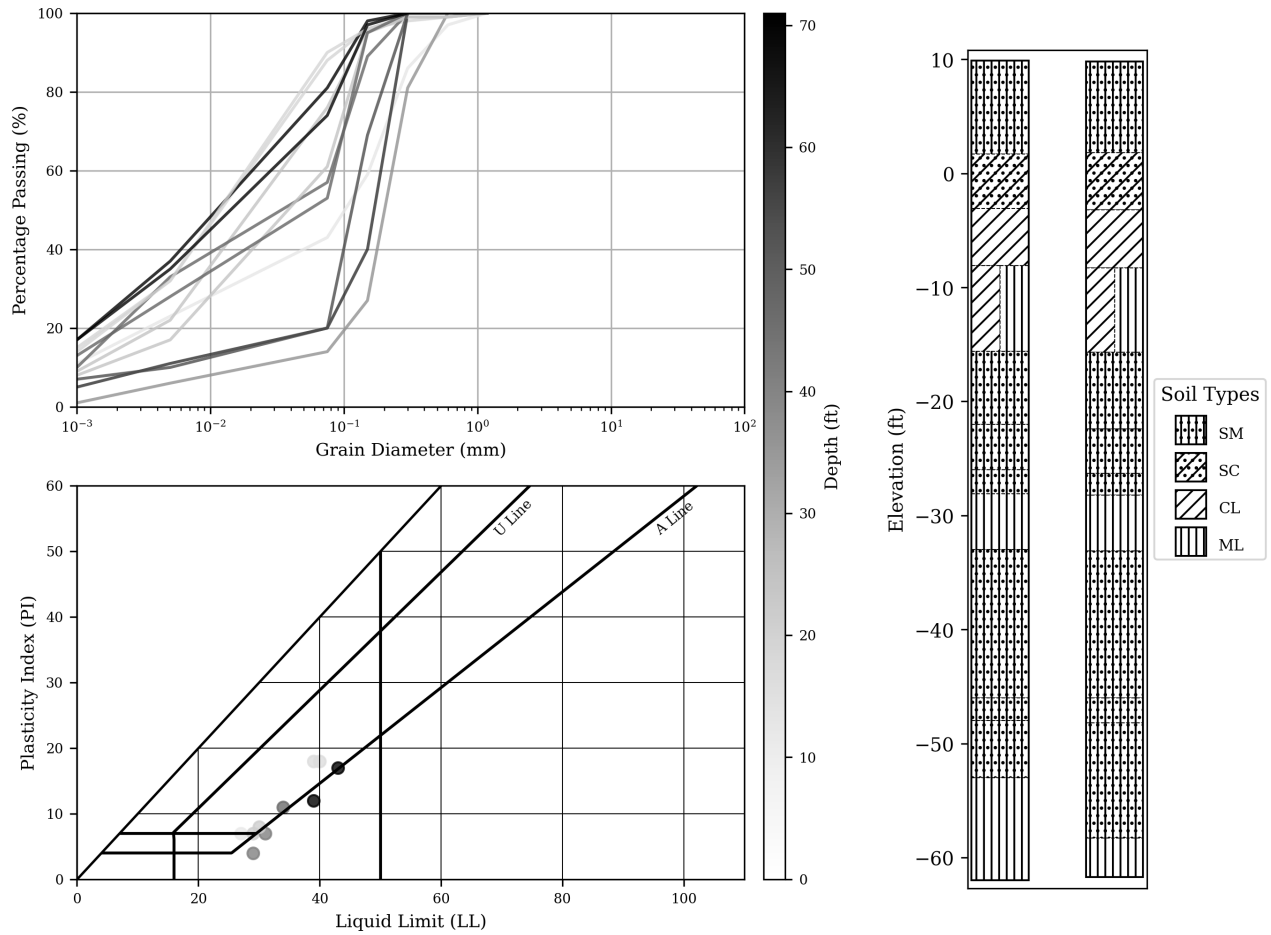


Figure 3: Boring log data summary: gradation curves (top left), Atterberg limits (bottom left) and boring log profiles (right).

Following pile testing, two cone penetration tests were pushed to a depth of 60 feet. The report provided by the contractor is provided in Appendix C. Data from the second has significant disturbance effects as it fell near the installation of a reaction pile from the second set of load tests, so only data from the first will be considered.

Using empirical relationships, a suite of soil parameters can be found. In accordance with the above design approach, comparisons of the interpreted friction angle and cohesion from the subsurface testing methods are shown in Figure 4. We see that, in general, there is agreement between the results, however the improved sampling rate of the CPT shows peak values that were not properly captured from the boring interpretation. The consequence of this will be highlighted in Section 3.5.

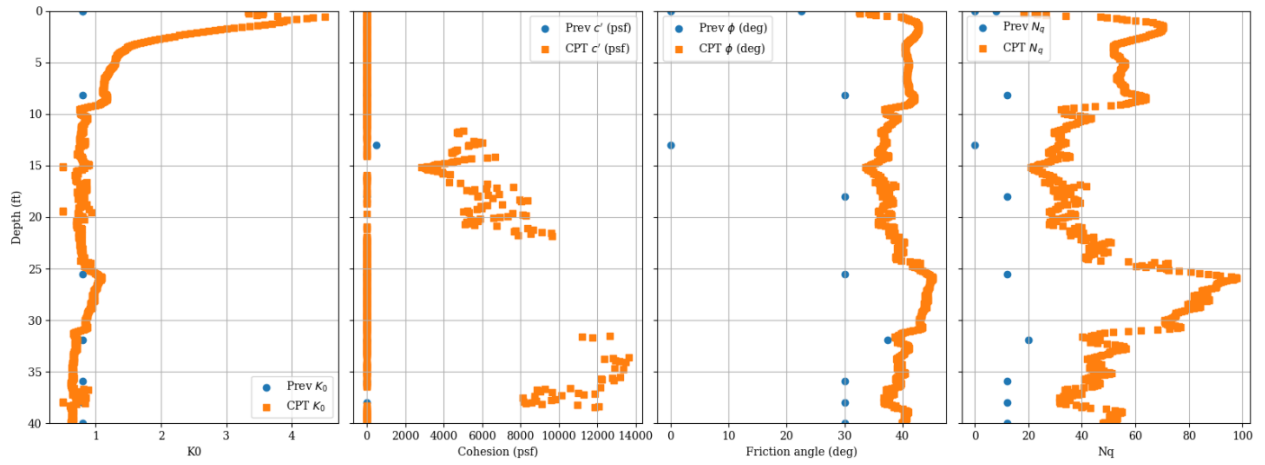


Figure 4: Comparison of boring interpreted and CPT interpreted design parameters.

3.2 Test Program and Soil Plug Test Conditions

The primary experimental variable in this study is the length of the soil plug remaining inside the shell after cleanout (Stage C in Figure 1). To isolate the effect of the remaining plug on the mobilized geotechnical resistance, four nominally identical open-ended test piles (outer diameter $D = 3$ ft, wall thickness 0.5 in, embedment depth 40 ft) were installed using the same driving procedure and were then cleaned out to different target plug lengths of 0, $2D$ (6 ft), $4D$ (12 ft), and $6D$ (18 ft), measured upward from the pile tip.

These target lengths were selected to bracket the plug-length range relevant to Caltrans design practice. The $2D$ condition corresponds to the plug length of approximately two pile diameters cited in the Caltrans MTD 3-1 as typically sufficient for water control (Section 2.1), and the $4D$ condition corresponds to the maximum plug length contributing to internal shaft resistance in the Caltrans/API design calculation. The $6D$ condition extends beyond this design limit to examine whether additional plug length provides measurable resistance, while the fully cleaned condition ($0D$) provides a reference response in which only external shaft resistance and annular tip resistance are mobilized. The $0D$ pile is hereafter also referred to as the control pile.

Two series of top-down compression pile load tests (PLTs) were performed. In the first series (PLT 1), all four plug conditions were tested to a target maximum load of 600 kips, established from the preliminary capacity estimates described in Section 3.5. Because the measured response remained essentially linear at this load level, a second series (PLT 2) was performed on the $2D$ and $4D$ piles to an increased target load of 1,150 kips to mobilize larger displacements. Table 2 summarizes the test matrix.

Table 2: Summary of test conditions and loading program.

Plug condition	Plug length (ft)	PLT 1 target load (kips)	PLT 2 target load (kips)
No plug ($0D$, control)	0	600	—
$2D$	6	600	1,100
$4D$	12	600	1,100
$6D$	18	600	—

3.3 Instrumentation

3.3.1 Distributed Fiber Optic Sensors

Due to the destructive nature of the pile driving process, two different fiber optic cables were installed in a total of three configurations to improve odds of cable survival. Regardless of cable type, the termination of the cable was protected by an angle bar filled with automotive grease. The decreased viscosity of the grease helped to ensure that the uncoupled termination end of the cable would not be aggravated during pile driving, which proved to be an effective technique with only one cable failing at termination, rendering it unusable for high resolution measurements.

A steel coated fiber optic (S-FO) was proposed due to its common use in the oil and gas sectors in high temperature and high intensity situations. The cable's cross-section is shown in Figure 5a, where of note two central fiber optics are available. Only the tight buffered fiber was utilized for these experiments as it is better suited for strain transfer. Due to the cable stiffness, a combination of welded tabs and epoxy was used to couple the cable with the pile surface. The opposite side of the cable is completely exposed to the subsurface soil. A polyethylene coated fiber optic (P-FO) cable, manufactured by Nansee Sensing Technology, was used due to its easier workability and cost effectiveness. The cable's cross section is shown in Figure 5b, where we see a single tight buffered fiber optic. Two cables were installed on each pile with slightly different installation approaches. One followed that of the S-FO approach though only epoxy was used for coupling to the pile. The alternative had the same coupling approach, however it was fully covered by an angle iron to protect it during the driving process. This ensured that one cable was certain to be readable post-driving and would match the typical installation approach for vibrating wire strain gauges.

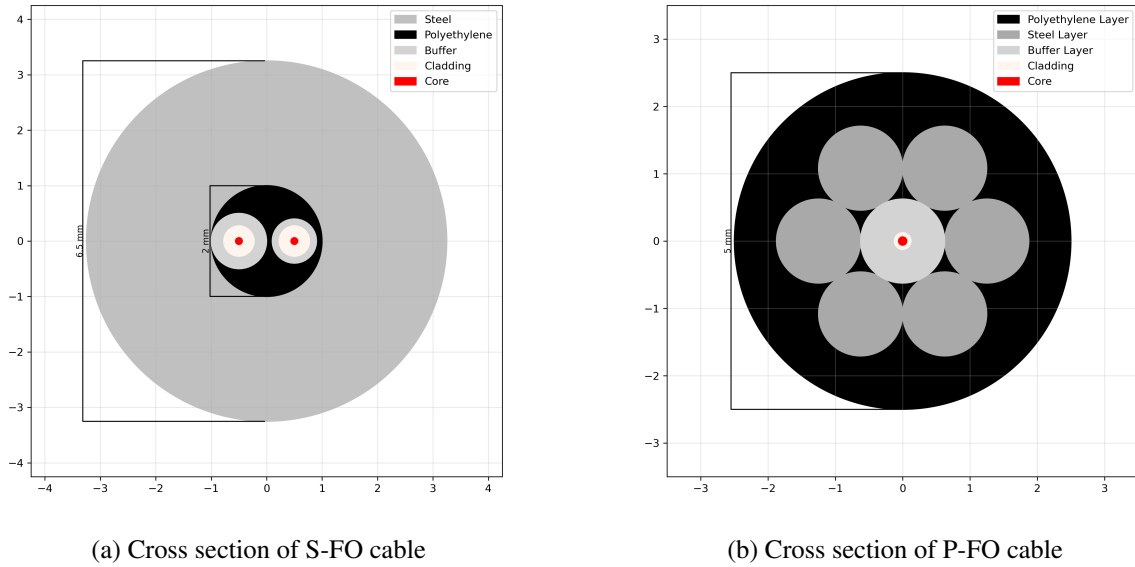


Figure 5: Cross-sectional configurations of the S-FO and P-FO fiber optic cables used for instrumentation.

The fiber optics are configured on the pile surface as seen in Figure 6. The cables are organized approximately 15-20 degrees off horizontal. This configuration was due to physical limitations that would allow for pile maneuverability for installation.

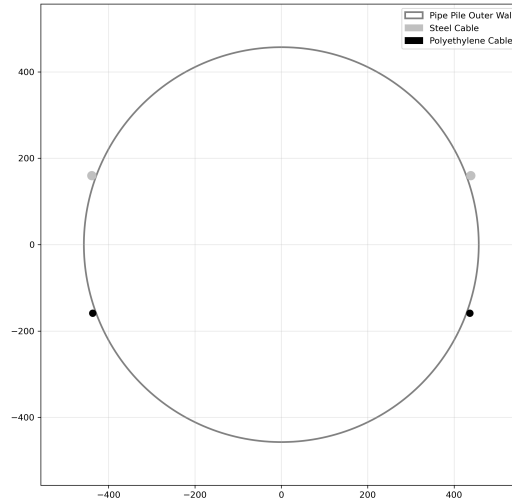


Figure 6: Pile cross section with 3x scaled fiber optic diameter

In summary, four DFOS measurement configurations were installed on each pile: the exposed and covered installations of the polyethylene-coated Nanzee cable (denoted EN and CN, respectively), and the two channels of the steel-coated cable (denoted S1 and S2). These designations are used throughout the interpretation of the load test results in Chapter 4.

3.3.2 Vibrating Wire Instruments

Vibrating wire piezometers, model EPP-40V from Encardio, were used to monitor pore pressure development around the pile tip. As seen in Figure 7, each test pile had a total of four piezometers installed in opposite pairs at 1.5 and 3 ft above the pile tip. On the control (0D) pile, only two piezometers were installed at 1.5 ft on the pile surface. Each piezometer was installed within a porous steel tube, secured by packing the instrument with geotextile cloth.

For near tip strain, model 4150 spot weldable vibrating wire strain gauges from Geokon were installed underneath angle irons approximately 6 inches above the pile tip as shown in Figure 7. The need for this data point is from the limited coverage of the DFOS cables in this region due to need for cable termination. Additionally, it provides an independent evaluation of strain, which helps to affirm the DFOS measurements.

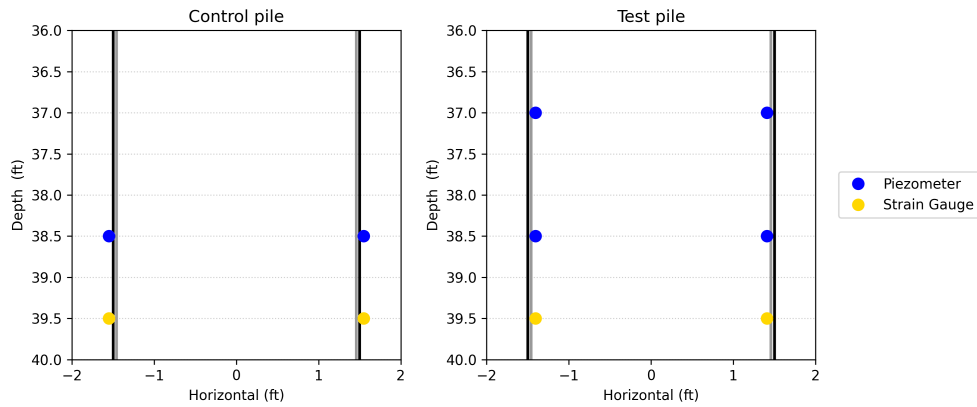


Figure 7: Vibrating wire instrument locations

3.4 Construction of Test and Reaction Piles

The configuration shown in Figure 8 was designed by FCI engineers to reach the expected failure load of 600 kips adopted for PLT 1 (Section 3.2). According to standard practice, pile spacing was set to slightly above 5D on center between test and reaction piles to eliminate observance of group effects.

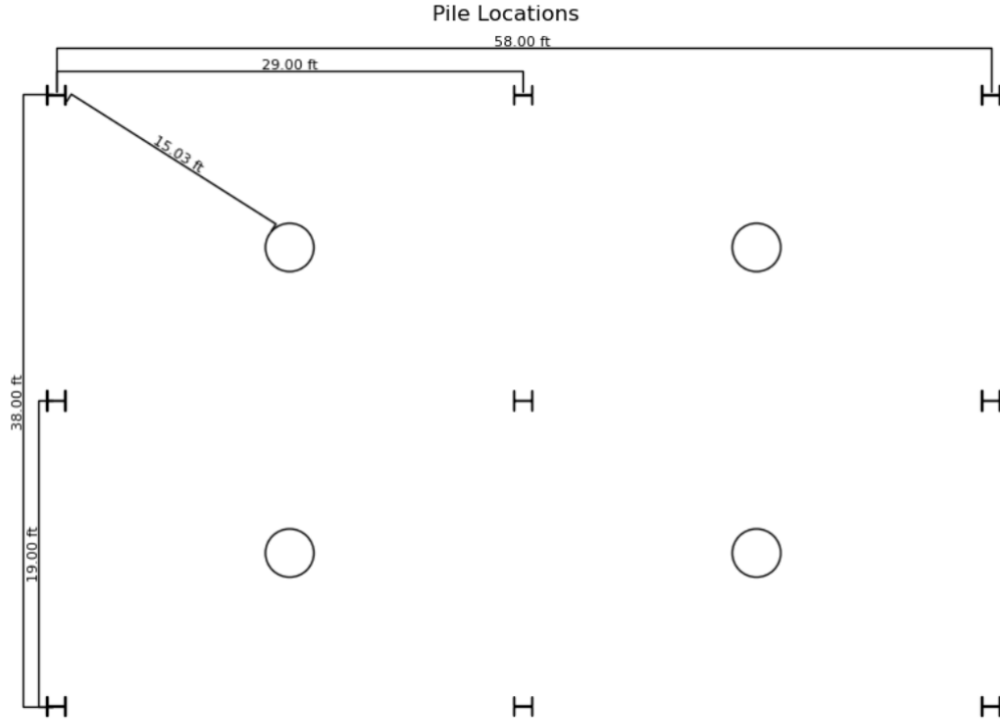


Figure 8: Initial test and reaction pile configuration.

While the standard pile installation procedure was followed, it is reiterated here for clarity and ease of reference. After configuring the vibrating wire devices and establishing the fiber optic cable connections, 50-

ft piles were installed using an impact hammer to a depth of 39 ft below grade. The piles were then allowed to set up for one week to permit soil strength recovery and gain in shaft resistance. Following this setup period, restrike driving was performed, advancing the piles an additional foot to achieve a final embedment length of 40 ft. Pile load testing was conducted approximately 1.5 weeks after restrike, allowing for installation of the reaction pile system, modification to the test piles, and expected setup of the soil resistance. Modifications to the test piles included removal of approximately 8 ft from the pile head and drilling out the soil plugs to the target plug lengths defined in Section 3.2 to achieve the intended test configurations. Reaction piles were driven to a depth of 55 ft below grade to provide sufficient tensile capacity and ensure adequate resistance during load testing.

3.5 Preliminary Design of Pile Capacity

Prior to testing, design calculations using the API method (Section 2.2) were based solely on interpretation of the readily available boring data and a suite of relevant assumptions. Of note, a constant soil density, γ , of 120 pcf and lateral earth pressure coefficient, K_0 , was assumed. Friction angle, corresponding to interface angle, and bearing capacity factor, N_q , were all based on the provided soil description from the field engineer. Value discrepancies for these and other critical parameters have been visualized in Figure 4.

A pile length of 40 feet had been chosen to limit costs and avoid the dense sand layers located at depths greater than 45 feet. Following this, the expected total bearing capacity can be calculated as a function of plug length, as shown in Figure 9. Note that since the tip resistance of the soil plug typically exceeds the internal shaft friction, the internal shaft friction is expected to govern the plug-related resistance, as would be desired to evaluate its exact performance by way of field testing.

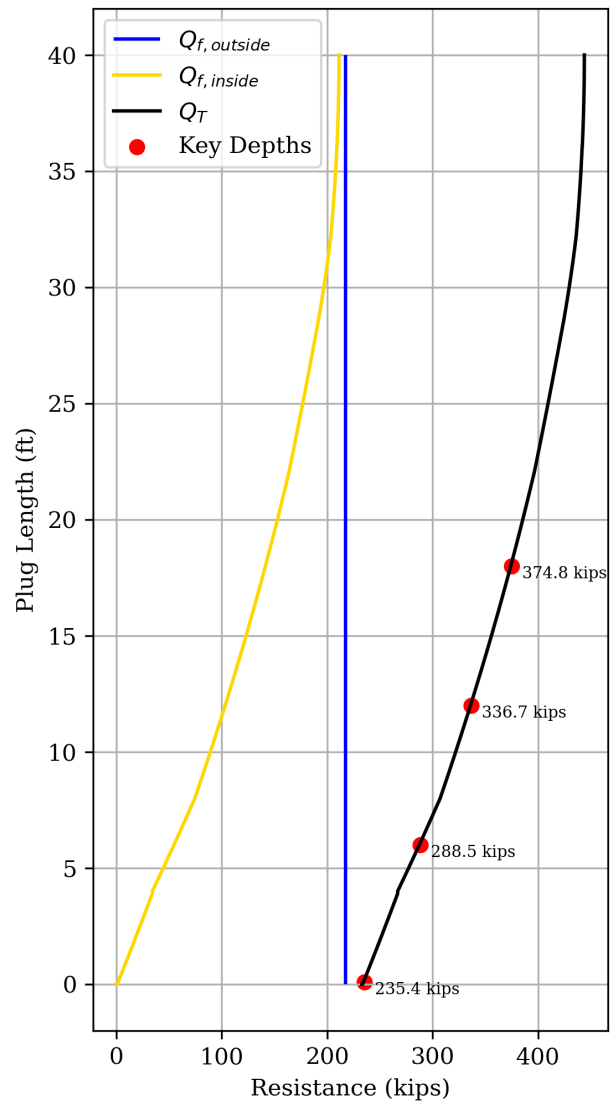


Figure 9: Bearing Capacity Calculation from Boring Data

Utilizing CPT inferred values as displayed in Figure 4, the bearing capacity is re-evaluated below according to the API method. Incorporating the peak values not captured by the boring logs led to an increase of approximately 175 kips across the key soil plug lengths.

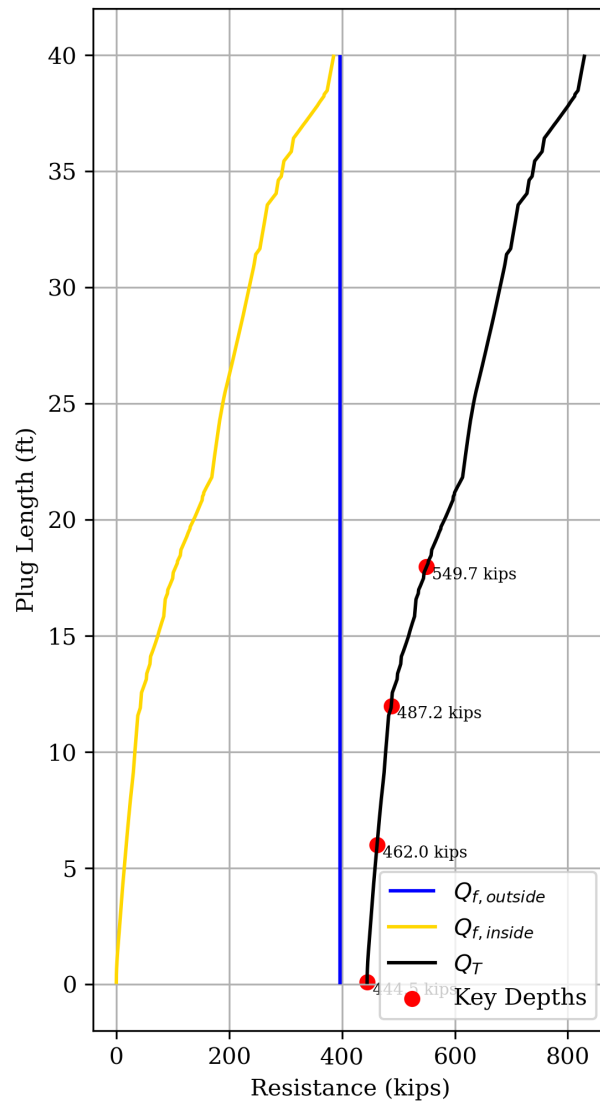


Figure 10: Bearing Capacity Calculation from CPT Data

A dynamic installation analysis was also run using the GRLWeap software by FCI engineers, provided in Appendix D. The analysis calculates resistance at 5 ft increments, indicating a total resistance of 540 to 560 kips assuming the pile retains the plug along the entire length. Assuming a linear variation, the incremental capacity can be interpolated to 1 ft increments to calculate the capacity for the target plug lengths. The plot of the resistance as a function of plug length from this analysis is provided in Figure 11.

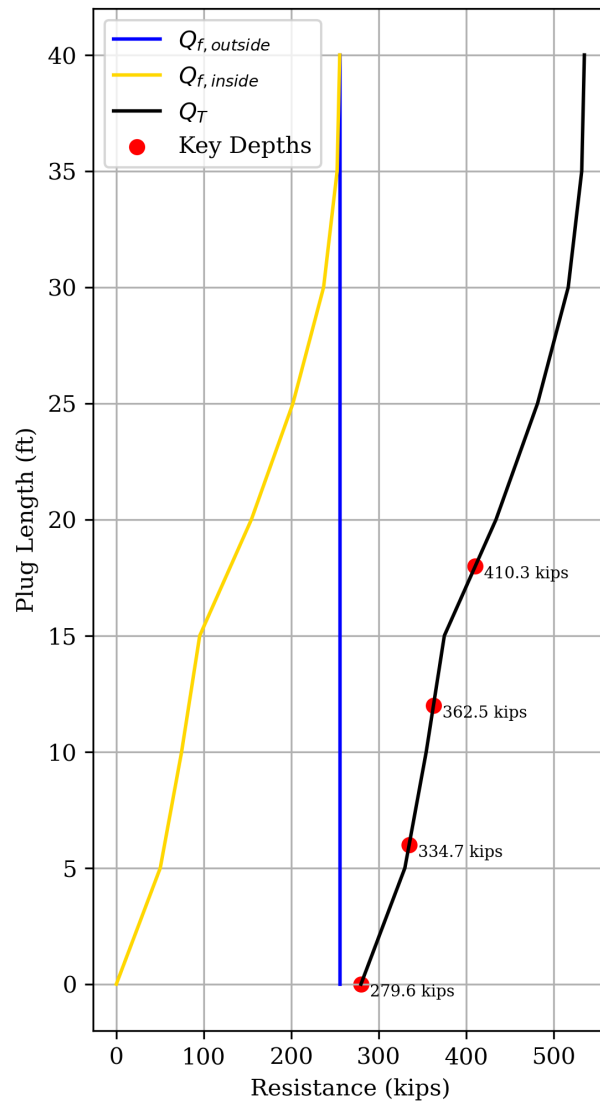


Figure 11: Bearing Capacity Calculation from GRL Analysis

Finally, the resistance was calculated according the Model 3 procedure developed by Caltrans. Following the procedure detailed in Section 2.3, the profile is provided in Figure 12. Based on the philosophy of Model 3 it is assumed that the primary contribution of any remaining soil plug will happen over the last 9 ft (3D). Accordingly, any remaining plug above that height will not yield greater resistance and it assumed that plugs below 3D can be linearly interpolated between the 3D expectation and 0 contribution.

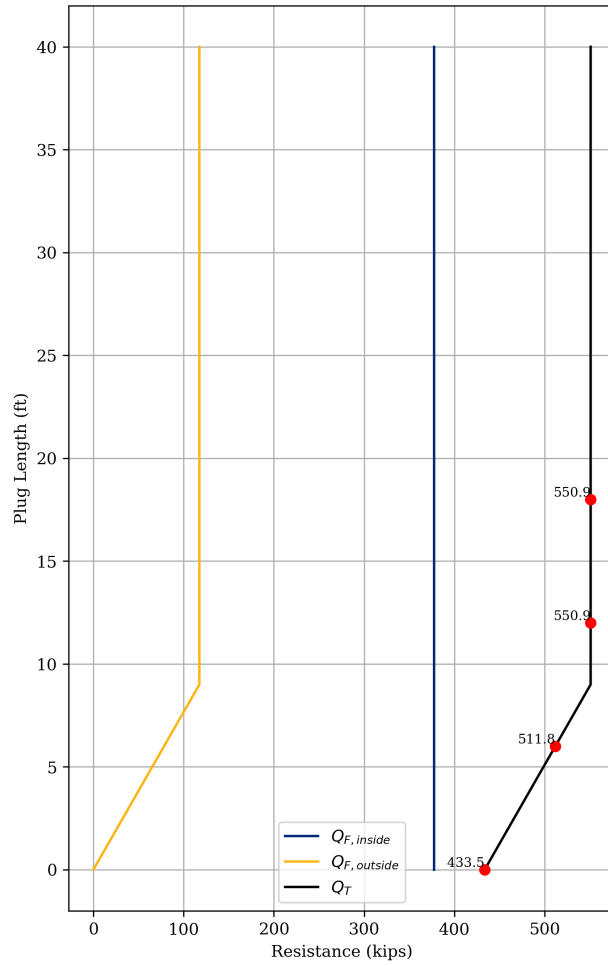


Figure 12: Bearing Capacity Calculation from Model 3 Approach

Finally, a summary table for the expected resistance calculations at key plug lengths is provided in Table 3.

Table 3: Summary of resistances; all values in kips.

Method	Data Source	$Q_{f,o}$	Q_t	0D		2D		4D		6D	
				$Q_{f,i}$	Q_T	$Q_{f,i}$	Q_T	$Q_{f,i}$	Q_T	$Q_{f,i}$	Q_T
API	Boring	217.0	18.4	0	235.4	53.1	288.5	101.3	336.7	139.4	374.8
API	CPT	394.5	50.0	0	444.5	17.5	462.0	42.7	487.2	105.2	549.7
GRLWeap	Boring	255.5	24.1	0	279.6	55.1	334.7	82.9	362.5	130.7	410.3
Model 3	Boring	377.3	56.2	0	433.5	78.3	511.8	117.4	550.9	117.4	550.9

3.6 Load Test Procedure

An image of the pile load test setup is provided in Figure 13. As can be seen, load is provided by pressurizing the hydraulic jack and independently measuring the output via the load cell, both of which had

been previously calibrated. The applied load was transferred directly to the pile head and into the load beam assembly, which was connected to the reaction piles. This configuration placed the reaction piles in tension, thereby providing the necessary resistance to mobilize axial compression in the test pile.



Figure 13: Example of pile load test configuration during testing.

To monitor load test system response, an array of independent sensors was installed to capture displacement behavior under increasing applied loads. Pile head displacement was measured using both automated and manual linear variable differential transformers (LVDTs). The LVDTs were fixed to independent angle-iron beams positioned to the left and right of the pile head, which served as reference to ensure that only pile displacement was captured. Displacement of the reaction piles was monitored using an optical tracking system with fixed reference instrumentation. Targets mounted on the reaction piles were incrementally observed to verify stability of the reaction system and to ensure that excessive uplift or tensile failure did not occur during testing. The loading protocol consisted of 5% incremental increases in applied load until the target maximum load for each test series was reached (600 kips for PLT 1 and 1,150 kips for PLT 2; Table 2). Upon reaching the peak load or geotechnical failure, unloading was performed in 20% decrements until the system returned to the initial condition. Each load increment was held for five minutes to allow for displacement and mobilization of resistance along the soil–pile interface.

4 Field Test Results

4.1 Field Observations

Most pile load tests progressed through the prescribed loading schedule without error. Load increments were applied and held as planned, and undamaged instrumentation functioned as intended throughout the majority of the testing program. Two exceptions to the standard loading progression occurred and are

summarized in Table 4. It is not expected that these errors had significant influence on the outcome of the pile response.

Table 4: Errors observed during pile load tests

PLT #	Plug Length	Description
1	2D	The test was reset after reaching the first loading stage due to faulty reading of the manual LVDTs.
1	2D	After the 8th loading step, miscommunication led to a larger load increase than expected. Upon realization the load was reduced to the expected and held for five minutes, returning to the expected program.

For all tests, geotechnical failure criteria were not reached within the applied loading range, as defined by standard displacement-based interpretation methods. Although measurable pile head displacement occurred and some displacement was retained upon unloading, the response remained controlled and did not indicate soil–pile interface failure. Consequently, testing was terminated upon reaching the target maximum load.

4.2 Load–Displacement Behavior

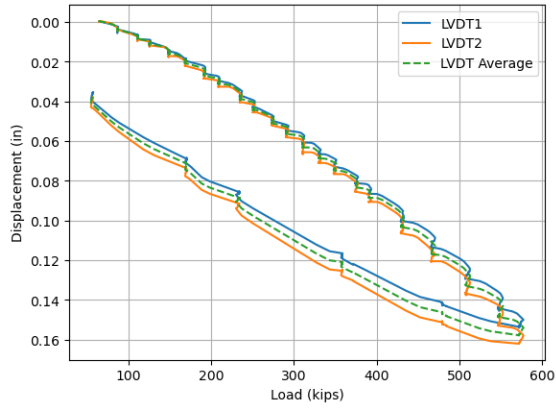
A principal deliverable of the static pile load testing is the load-displacement curves measured at the pile head, providing a direct representation of the pile-soil interaction. According to engineering practice, geotechnical failure was not observed for these tests leading to a mostly linear elastic response as shown in Figure 14. Furthermore, no meaningful correlation can be made between the observed pile displacement and the remaining plug length with peak results being summarized in Table 5. Of potential interest is the difference in displacement response for PLT 1 on the 2D and 4D plugs, which may be the result of equipment malfunction (via calibration or reference setup), eccentric loading, or a nonuniform resistance response. The results of PLT 2 largely dissuade the latter two possibilities, leading to expectation that the former is the main cause.

Even after increasing the load test program to reach 1,150 kips, only slight geotechnical failure appears to have begun, as seen in the slight vertical drop offs of the last load step(s) of Figures 14e and 14f. Looking at peak displacements, it can also be seen that the values are largely comparable between these two tests and the trend has reversed in comparison to the 600 kips tests where the 4D plug saw larger head displacements. All this to say that it remains difficult to determine the influence of the variable plug length without the observation of the geotechnical failure stage.

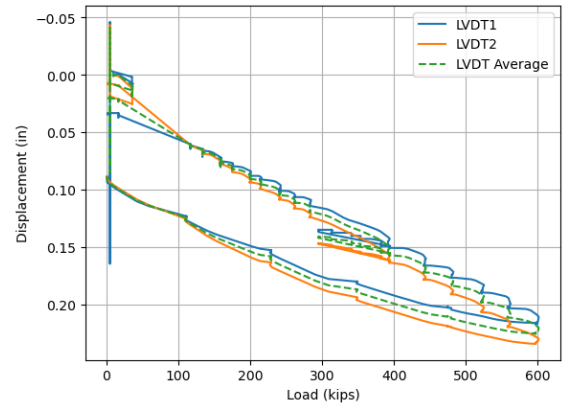
However one may consider the alternative interpretation, that the lack of correlation implies that the most significant contribution of resistance from the soil plug is below the 2D segment. While determining the validity of this assumption is best left to Section 4.4, assuming it is correct would require any contribution of resistance from additional soil plug to be less than the variability one may expect due to variable soil conditions for each pile. Given the confined area of the site and consistency of exploratory investigation results however, this should be an unlikely scenario.

Test series	PLT 1	PLT 1	PLT 1	PLT 1	PLT 2	PLT 2
Plug condition	0D	6D	4D	2D	4D	2D
Plug length (ft)	0	18	12	6	12	6
Peak load (kips)	600	600	600	600	1,150	1,150
Peak avg. displacement (in)	0.157	0.223	0.193	0.158	0.371	0.431

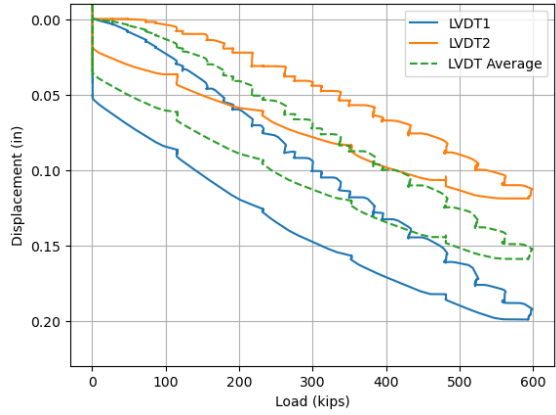
Table 5: Summary of peak average displacements at the pile head



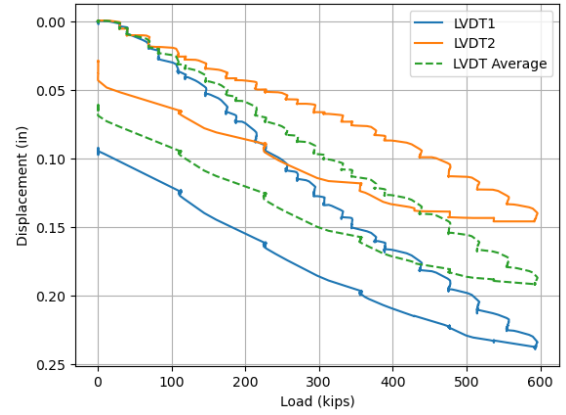
(a) PLT 1: No plug (0D)



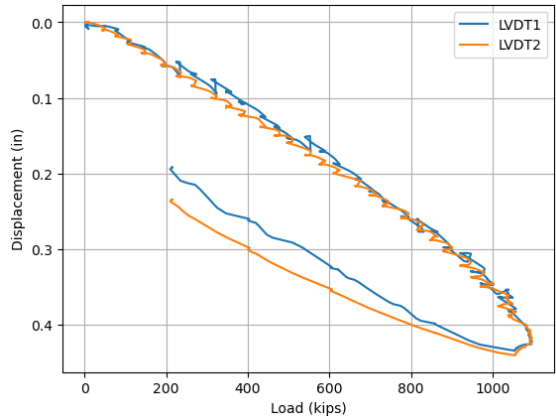
(b) PLT 1: 6D plug



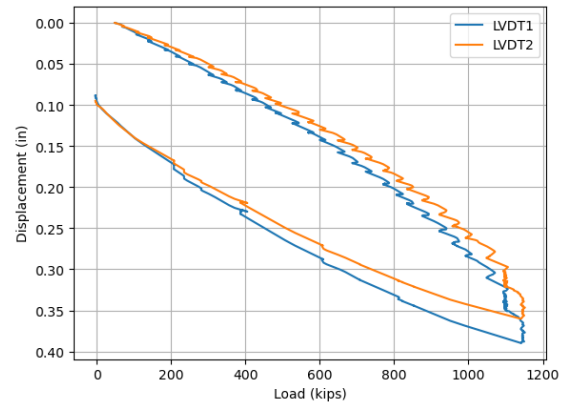
(c) PLT 1: 2D plug



(d) PLT 1: 4D plug



(e) PLT 2: 2D plug



(f) PLT 2: 4D plug

Figure 14: Load-displacement curves from LVDTs at pile cap.

4.3 Porewater Pressure Response

An example response of the porewater pressure response during pile load testing is provided in Figure 15. In accordance with the expected subsurface conditions a range of only approximately 1.1 kPa was

observed during the entirety the pile load test of the 6D pile at the location nearest the pile tip.

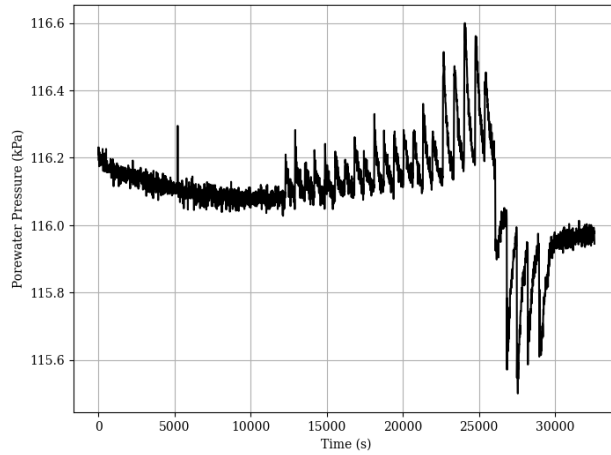
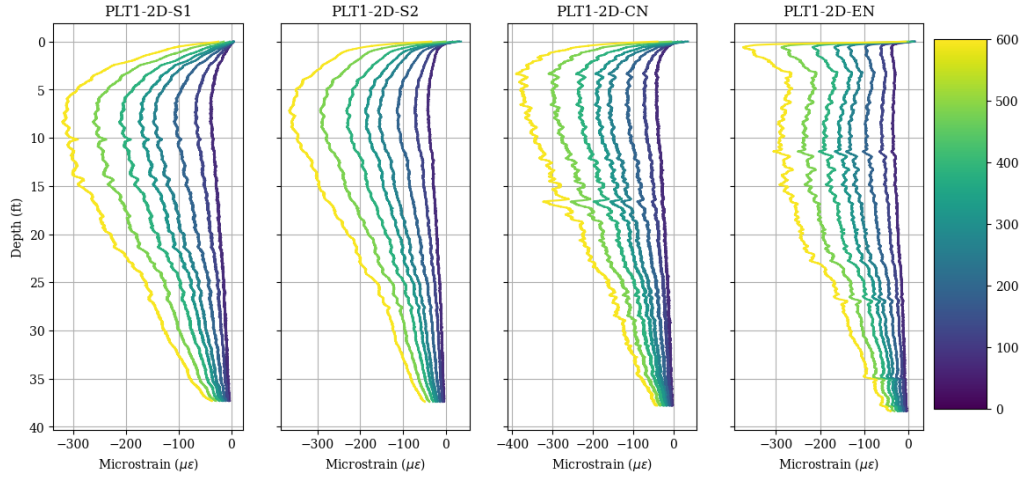


Figure 15: Porewater pressure response for PLT1-6D

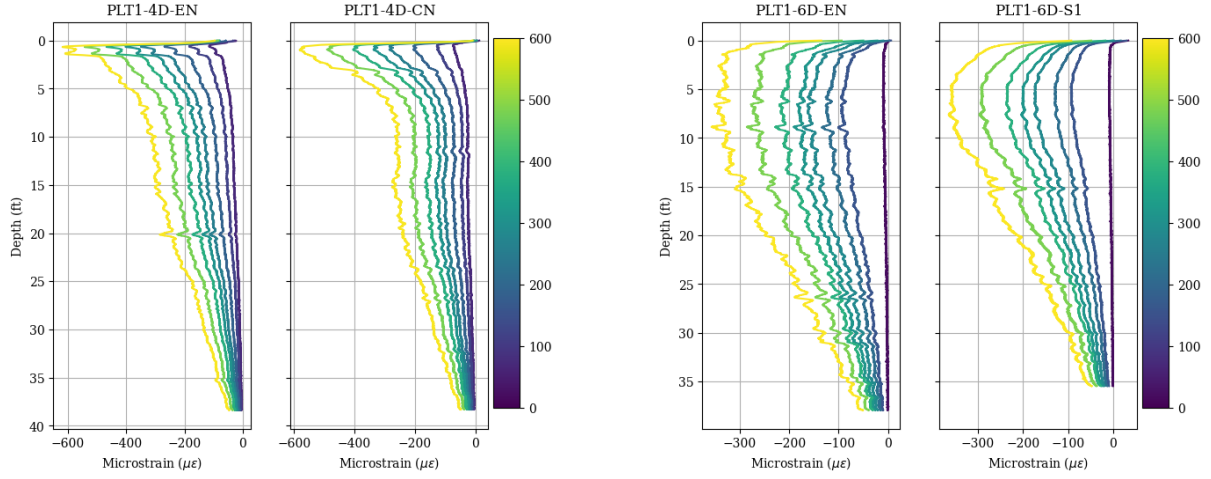
This degree of change is expected to be ultimately inconsequential on the response of the system, so all numerical analysis will assume drained conditions.

4.4 Strain Profiles

The installed fiber optic cables allow us to look directly at the strain profile measured during the course of testing. Those obtained during the first series of pile load tests (PLT 1) are presented in Figure 16. The results are shown for each of the tested pile configurations, with the profiles progressing from lower to higher applied load. These measurements provide insight into the development and distribution of axial strain along the pile as the applied load increases.

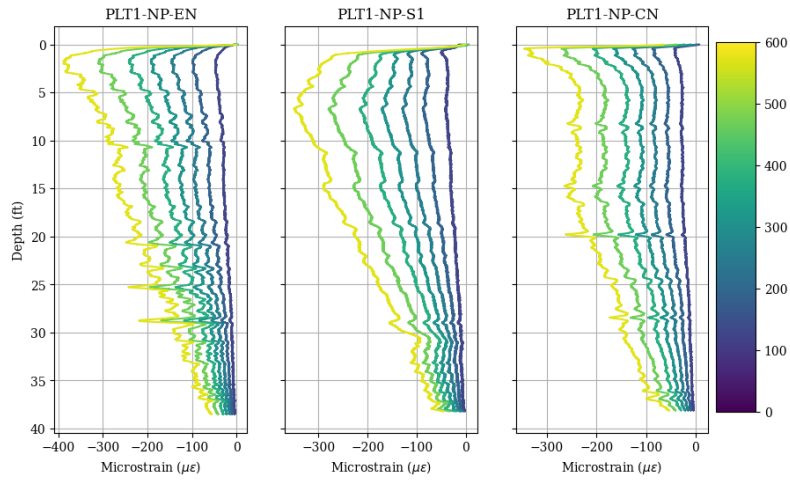


(a)



(b)

(c)



(d)

Figure 16: Strain profiles from PLT 1.

The strain profiles from the second series of pile load tests (PLT 2) are presented in Figure 17. As mentioned previously, these results are limited to the 2D and 4D plug configurations due to limitations associated with increasing the maximum load.

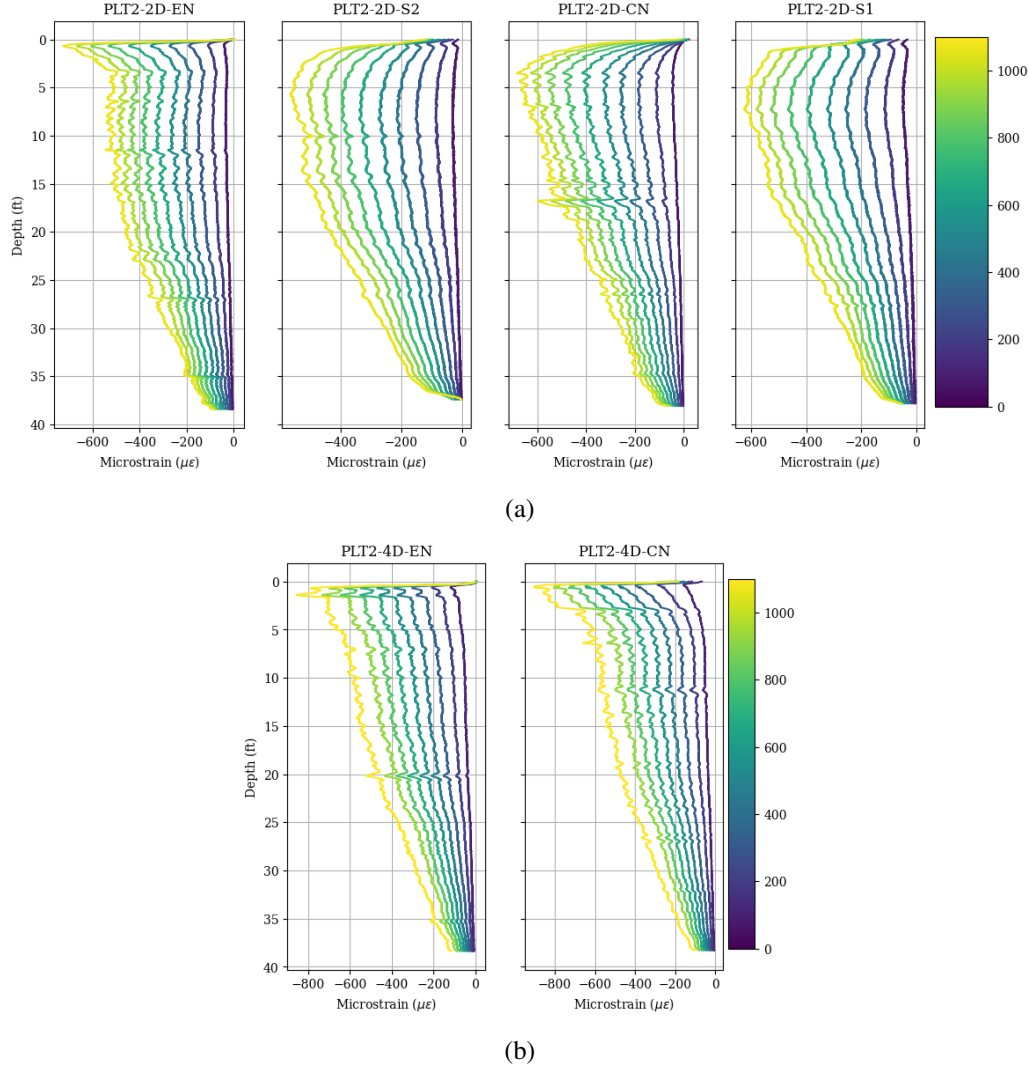


Figure 17: Strain profiles from PLT 2.

Several features of the measured strain profiles can be examined to evaluate the pile response. In particular, the magnitude and distribution of strain along the pile provide an indication of how the applied load is transferred through the pile-soil interface. However, there are variable factors that can influence the observed strain levels, as is indicated by the variable maximal strain values observed for the different load tests.

First let's consider with some care the near surface strain levels, assuming minimal surface influence of the soil we should expect near analytical strain values to be observed. For the 600 kip and 1100 kip maximal load we should expect strain values of $371 \mu\epsilon$ and $680 \mu\epsilon$, respectively. In general, the installed steel cables failed to reach this value near surface. This discrepancy is expected to do with the comparative stiffness mismatch of between the cable and the coupling epoxy, as these cables would typically be installed using welding techniques and are mostly utilized for temperature sensing. At depth, the lateral pressure of the

soil improves the ability of the cable to match the pile response, typically with a peak strain value observed at around 6 ft of depth. The shape of the steel response was mirrored by the CN cable in PLT1-2D and PLT2-2D response, likely as a result of the near surface coupling being disturbed during the construction process. With the exception of this pile, the response of the EN and CN cable installations were relatively similar and better matched the analytical results near surface. However, for both PLT1-4D and PLT2-4D, these cables measured a larger than expected strain in the near surface. This deviation is likely influenced by many factors, including thermal effects, eccentric loading or others which have not been realized, though based on the data gathered no definitive conclusion can be drawn as to the source.

At depth, the decreasing observed strain is a result of the load being transferred to the surrounding soil. Local disturbances observed, i.e. at 20 ft for PLT1-4D-EN and PLT2-4D-EN or at 17 ft in PLT1-2D-CN and PLT2-2D-EN, are associated with imperfections on the pile surface. Specifically these were spiral welds along the pile length and a butt weld that had connected to piles of smaller length. The inconsistency of the imperfection locations is also heightened by the fact that some weld locations had been sanded down in anticipation of the fiber coupling. The response of fiber around the welds indicates a decrease in stiffness associated with the commonly observed increase in strain. Near the pile tip of the S1 and S2 cables, there is some notable decrease of the observed strain that is associated with poor coupling under the covered tip of the cable.

4.5 Load Transfer Behavior of CISS Piles

To interpret the load-transfer response and to derive representative $t-z$ and $q-z$ relationships from the field tests, a one-dimensional beam-spring model was developed. Figure 18 illustrates the adopted idealization. The pile is modeled using axial beam elements, while nonlinear springs are introduced to represent mobilized shaft resistance along the pile and end bearing at the pile tip. For CISS piles, the resistance mechanism can vary depending on the extent to which the soil plug retained inside the shell participates in load transfer. Following the conceptual distinction described in API (2000), two cases were considered: an *unplugged* case and a *plugged* case. In the unplugged case, an additional set of springs is placed along the inner wall of the shell to represent internal shaft resistance mobilized by the retained soil plug. The tip spring is then assumed to act only over the annular area associated with the steel shell. In contrast, for the plugged case, the internal shaft springs are deactivated, while the tip spring is applied over the full cross-sectional area defined by the pile outer diameter. These two idealizations are used to explore possible load-transfer responses of CISS piles.

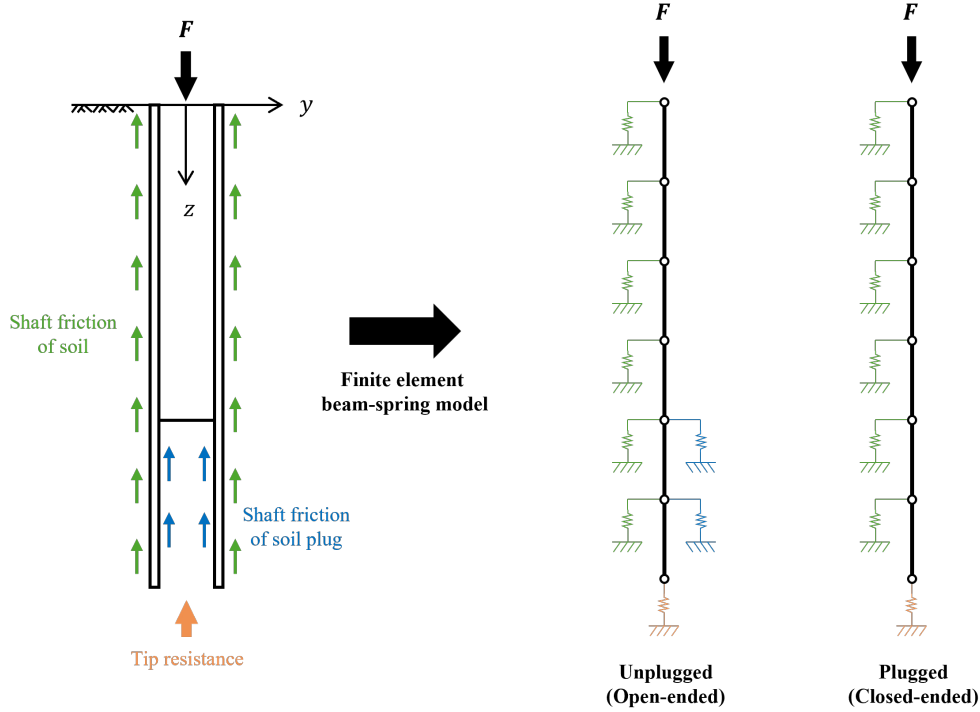


Figure 18: Beam-spring model for CISS pile.

The nonlinear spring response was described using the hyperbolic load transfer relationships proposed by Pyke (1979) for shaft friction (t - z) and tip resistance (q - z). This formulation captures the initial stiffness, progressive stiffness reduction, and overall nonlinear mobilization (including hardening/softening trends) using a small set of parameters. The hyperbolic model is written as

$$R(\delta) = \frac{k \delta}{\left[1 + \left(\frac{k \delta}{R_{\max}} \right)^{hd} \right]^{1/d}}, \quad (27)$$

where $R(\delta)$ denotes the mobilized resistance (i.e., t for shaft friction or q for end bearing); δ is the relative displacement between the pile and the surrounding soil (or between the shell and the soil plug for the internal springs); k is the initial stiffness; $R_{\max}$ is the ultimate resistance (i.e., $t_{\max}$ or $q_{\max}$); and h and d are shape parameters controlling the curvature and stiffness degradation. The axial equilibrium of the pile is governed by

$$EA \frac{d^2 u(z)}{dz^2} - \pi D t(u(z)) = 0, \quad (28)$$

where $u(z)$ is the pile axial displacement, EA is the axial stiffness of the pile, D is the pile diameter, and $\pi D t(u)$ represents the mobilized shaft resistance per unit pile length. At the pile head, the applied axial load is imposed through a Neumann boundary condition, and the tip spring (q - z) is assigned at the pile base. No Dirichlet boundary conditions are enforced at the head or the tip; the displacement response is governed by compatibility with the nonlinear springs along the shaft and at the base. The pile was discretized into 100 finite elements, and the resulting nonlinear system was solved using a Newton-Raphson iterative scheme. Spring forces were evaluated at Gauss integration points within each element, and the external load was applied incrementally to match the loading stages used in the field tests. Convergence was monitored using the residual force norm. Using the beam-spring model as the forward solver, an inverse

analysis was conducted to estimate load-transfer parameters for the surrounding soil and the retained soil plug. Specifically, the inverse analysis seeks the parameter vector $\mathbf{p}$ that minimizes the objective function $J(\mathbf{p})$, defined in terms of the mismatch between model predictions and field measurements, including pile head displacement and axial strain profiles measured by DFOS at selected loading stages. The objective function was defined using normalized squared errors in axial strain and pile head displacement:

$$J(\mathbf{p}) = \sum_{n=1}^{N_{\text{steps}}} \frac{\left(\epsilon_{\text{(BS)}}^{(n)}(z; \mathbf{p}) - \epsilon_{\text{(exp)}}^{(n)}(z) \right)^2}{\epsilon_{\text{(exp)}}^{(n)}(z)^2} + \frac{\left(u_{\text{head, (BS)}}^{(n)}(\mathbf{p}) - u_{\text{head, (exp)}}^{(n)} \right)^2}{\left(u_{\text{head, (exp)}}^{(n)} \right)^2}, \quad (29)$$

where n is the loading-stage index and $\mathbf{p}$ is the parameter vector. Normalization by the experimental quantities provides balanced weighting between the strain and head-displacement terms. The target parameters for the inverse analysis are the hyperbolic parameters defining the t - z and q - z curves, including the initial stiffness parameters for the shaft springs (k_t) and the tip spring (k_q), and the ultimate resistances (t_{max} , q_{max}). Following Pelecanos et al. (2017), the shape parameters were held constant at $h = 1$ and $d = 3$ to reduce the degrees of freedom and improve optimization robustness. For computational efficiency, the soil profile was divided into eight layers, each approximately 5 ft thick (1.5 m). Within each layer, the shape parameters were fixed ($h = 1$, $d = 3$), and only the initial stiffness and ultimate resistance were optimized. The parameter vector $\mathbf{p}$ can be written as

$$\mathbf{p} = [k_{t,1}, t_{\text{max},1}, \dots, k_{t,8}, t_{\text{max},8}, k_q, q_{\text{max}}, k_{t,sp}, t_{\text{max},sp}], \quad (30)$$

where $k_{t,i}$ and $t_{\text{max},i}$ are the initial stiffness and ultimate unit shaft resistance of the surrounding soil in layer i , respectively; k_q and q_{max} define the tip spring; and $k_{t,sp}$ and $t_{\text{max},sp}$ define the internal shaft springs associated with the soil plug (activated only for the unplugged case). A hybrid Genetic Algorithm (GA) implemented in MATLAB was used for the optimization. The GA first performed a global search using population-based crossover and mutation, and a subsequent gradient-based local search was used to refine the best solution. This hybrid strategy improves robustness in exploring the parameter space while providing efficient local convergence (Abushama et al., 2024). Because the GA is a stochastic optimization method, each inverse analysis was repeated 10 times under identical conditions to confirm the consistency of the converged solutions. It should be noted that an additional inverse analysis using a finite-element model is described in Section 5.4. The beam-spring optimization presented here is intended to estimate representative t - z and q - z curves, rather than to uniquely identify detailed soil constitutive properties.

4.5.1 Representative inverse analysis results

A representative result is presented for PLT 2 with a 2D soil plug, interpreted from the EN cable (Section 3.3.1) using the plugged idealization. This condition is shown as representative because it reached the highest applied load in the program and therefore mobilized the largest displacements. Figure 19 compares the measured and back-calculated responses for the two quantities that define the objective function in Eq. (29): the pile head load-settlement behavior and the axial strain profiles at representative load levels. The optimized beam-spring model reproduces both targets closely. The agreement for the axial strain profiles, which provide the spatially distributed measure of load transfer, is quantified by a coefficient of determination of $R^2 = 0.969$ and a root-mean-square error of $26.9 \mu\epsilon$, computed between the measured and back-calculated strains pooled over all load steps and measurement depths. This level of agreement confirms that the hybrid genetic-algorithm optimization converged to a parameter set that consistently captures the measured load transfer behavior.

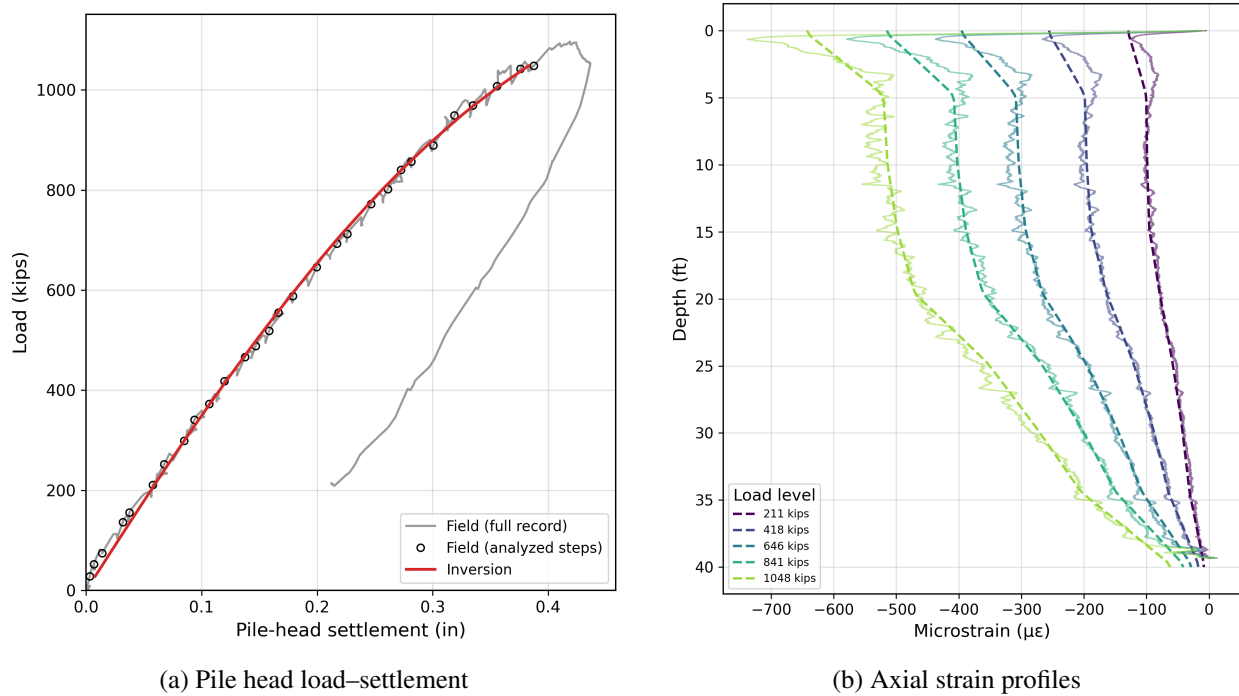
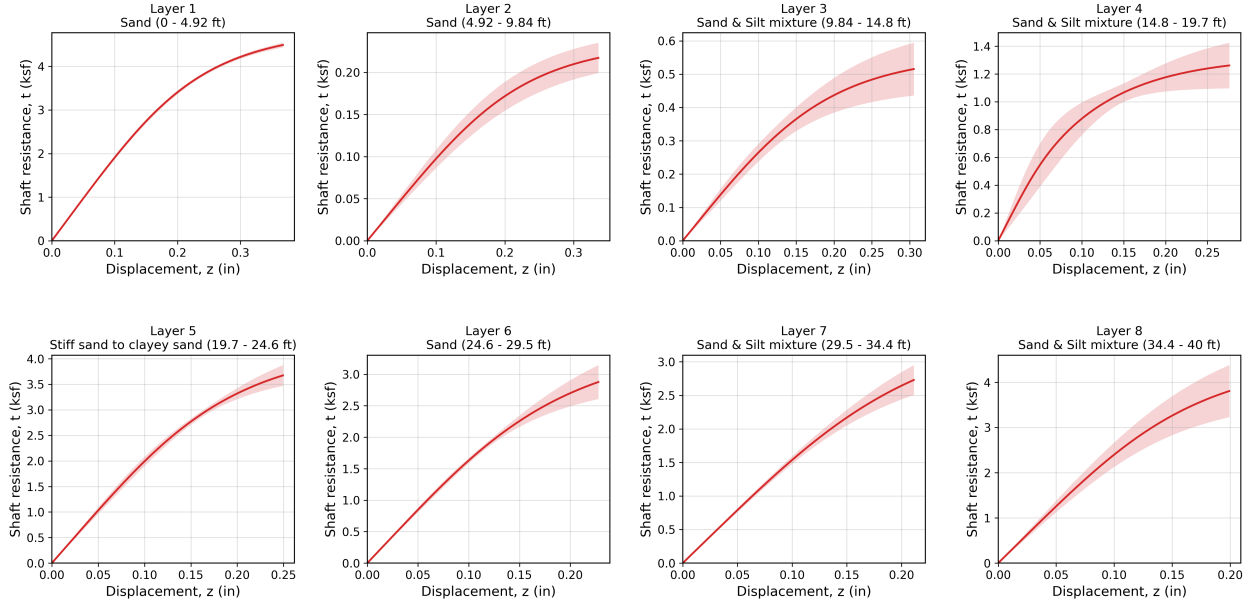
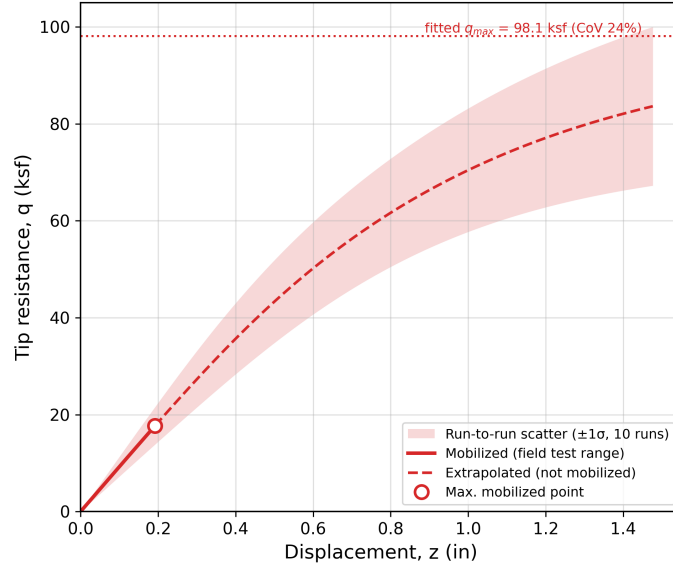


Figure 19: Measured versus back-calculated response for the representative condition (PLT 2, 2D plug, EN cable, plugged idealization). Solid lines denote field DFOS measurements and dashed lines denote the inverse-analysis result.

The corresponding back-calculated load transfer curves are shown in Figure 20, reported as the mean of the ten repeated optimizations together with a ± 1 standard-deviation band. The layer-wise t - z curves (Figure 20a) show little difference among the ten runs and form narrow bands in all layers, indicating that both the initial stiffness and the ultimate resistance of the shaft springs are well constrained by the measured strain profiles. The tip q - z response (Figure 20b) shows larger variability among the runs. In this panel, the solid line denotes the portion of the curve mobilized during the test, while the dashed line is the extrapolation toward the optimized ultimate resistance, $q_{\max} = 98.1$ ksf. Because only about 18% of $q_{\max}$ was mobilized, the extrapolated ultimate resistance varies considerably between runs and is therefore less reliable, and $q_{\max}$ is not interpreted here as a measured capacity given the limited tip mobilization. The back-calculated resistances are compared with the API and Model 3 design predictions separately in Section 4.5.5.



(a) Layer-wise t - z curves (mean ± 1 standard deviation of ten optimizations)



(b) Tip q - z curve (mobilized portion solid, extrapolation dashed; ± 1 standard-deviation band across ten optimizations)

Figure 20: Representative back-calculated load transfer curves (PLT 2, 2D plug, EN cable, plugged idealization).

This contrast between a well-constrained t - z response and a poorly-constrained q - z response was consistent across the testing program and can be explained by the degree to which each spring was mobilized. Figure 21 pools every shaft layer and every tip spring from all test conditions and plots the coefficient of variation of the mobilized resistance across the ten repeated optimizations ($\text{CoV} = \sigma/\mu \times 100\%$, where μ and σ are the mean and standard deviation of the resistance mobilized at the maximum test displacement) against the mobilization ratio (mobilized resistance divided by fitted ultimate). The reliability of the back-calculated load transfer clearly improves with mobilization. The shaft springs were mobilized to 40–95%

of their ultimate and are tightly constrained (mean CoV $\approx 14\%$), whereas the tip springs were mobilized to only 2–26% and show much larger scatter (mean CoV $\approx 56\%$), because the tip carries only a small fraction of the load and the strain measurements are nearly insensitive to it. Consistent with this trend, the tip scatter decreases from the lower-load PLT 1 to the higher-load PLT 2. Accordingly, the back-calculated $t-z$ curves are considered reliable, whereas $q_{\max}$ is poorly constrained and should be regarded as an extrapolated rather than a measured quantity, reflecting that the applied loads, which were sized using the design methods, were insufficient to mobilize the pile tip.

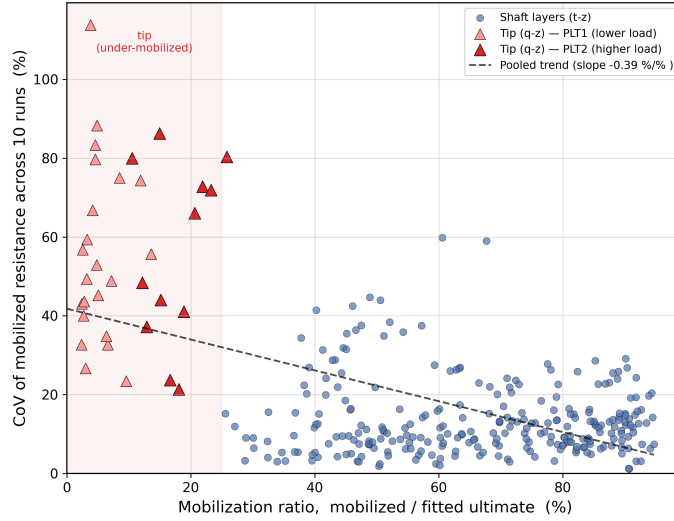
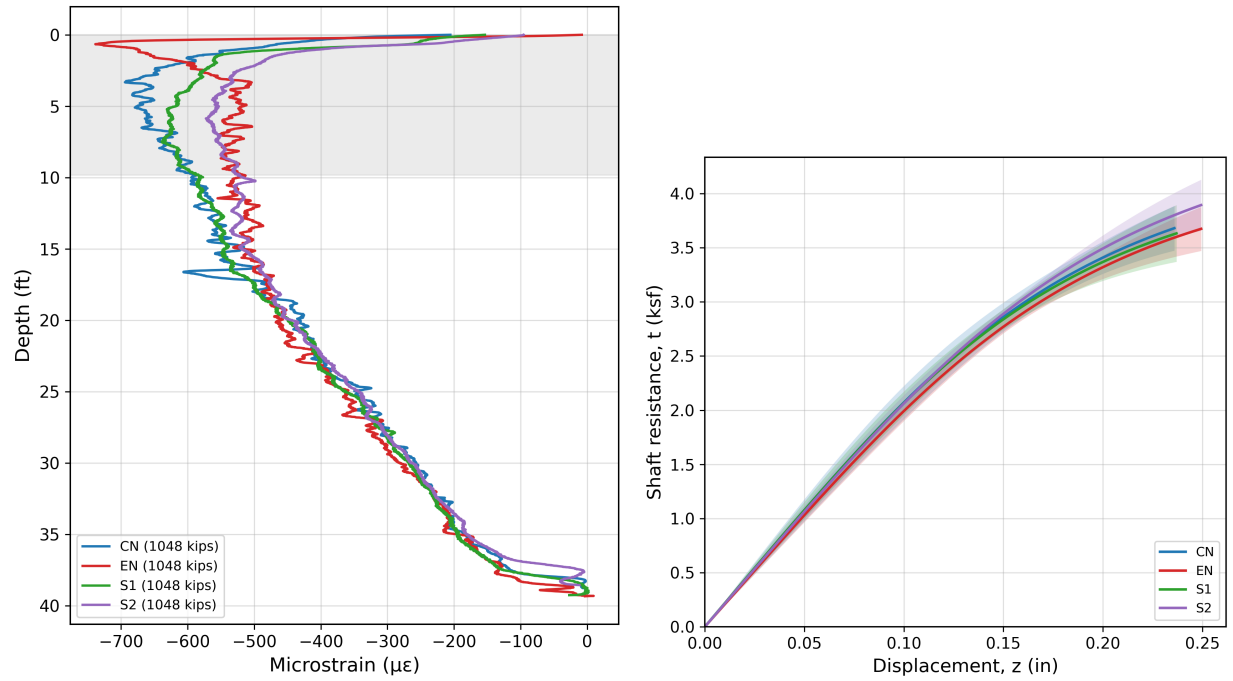


Figure 21: Reliability of the back-calculated springs across all test conditions: coefficient of variation of the *mobilized* resistance (over ten repeated optimizations) versus mobilization ratio.

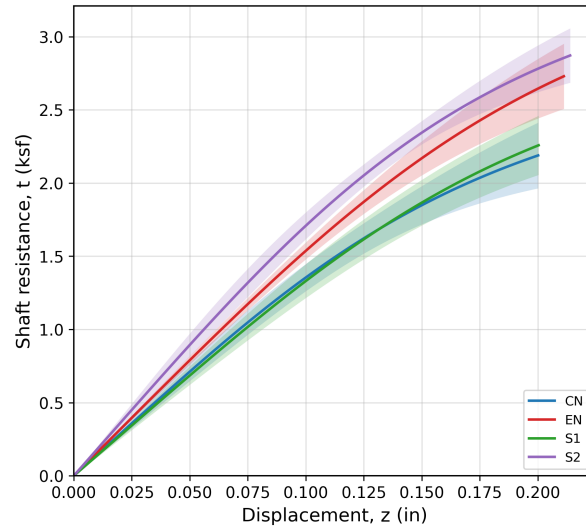
4.5.2 Effect of DFOS Installation Method

The influence of the DFOS installation method was examined by comparing the four cable configurations (EN, CN, S1, and S2) defined in Section 3.3.1, which were installed on the same pile. Figure 22 presents, for a representative condition (PLT 2, 2D plug), the measured strain profiles near the maximum load together with the resulting back-calculated $t-z$ curves for two deeper layers (Layer 5 and Layer 7). Because the load was applied in a top-down configuration, the axial force and the corresponding measured strains are largest near the pile head and decrease with depth as shaft resistance is progressively shed. In the near-surface region (roughly the top 10 ft), the measured strains, and therefore the back-calculated shaft resistance, differ somewhat among the cable configurations, reflecting differences in the cable-to-pile attachment and strain transfer where the strains are largest. Because these near-surface differences reduce confidence in the interpreted response at shallow depths, only the deeper-layer results are presented here. Toward the pile tip, the measured strain profiles converge (Figure 22a) and the back-calculated $t-z$ curves become essentially consistent across all four cable configurations (Figures 22b and 22c), indicating that the DFOS interpretation at depth is robust to the installation method.



(a) Measured strain profiles near maximum load

(b) Back-calculated t - z , Layer 5



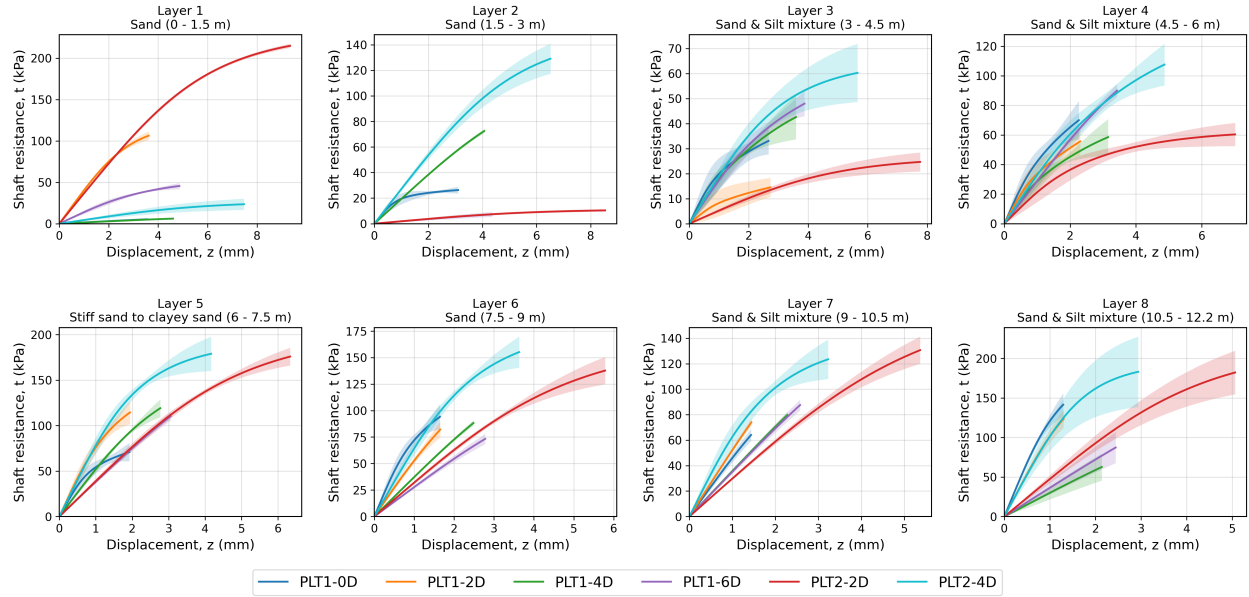
(c) Back-calculated t - z , Layer 7

Figure 22: Effect of DFOS installation method (PLT 2, 2D plug) for the four cable configurations.

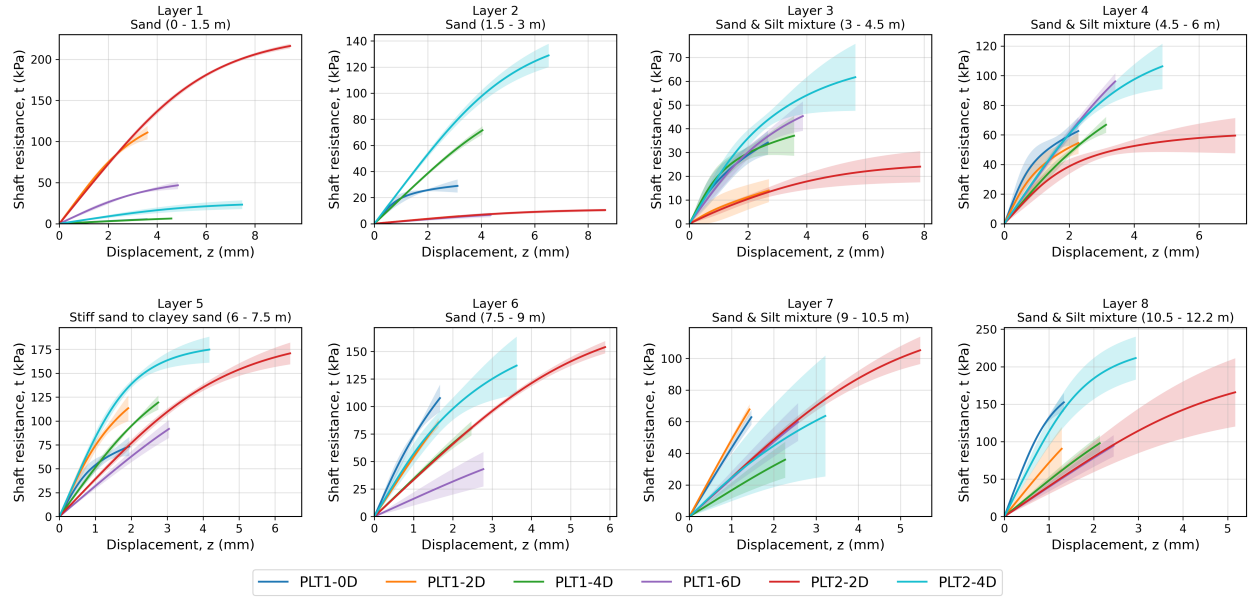
4.5.3 Effect of soil plug length

The effect of soil plug length was evaluated by comparing the back-calculated t - z curves across all test conditions interpreted from the EN cable, which was available for the full set of plug lengths. To account for the two idealizations of load transfer, the comparison was performed separately for the plugged and unplugged cases (Figures 23a and 23b). In both cases, the layer-wise t - z curves do not exhibit a systematic dependence on plug length. This is shown more directly in Figure 24, where the fitted ultimate unit shaft resistance $t_{\max}$ for the PLT 1 series (0D, 2D, 4D, and 6D plugs) is plotted against plug length for each layer;

the values scatter without a consistent monotonic trend. The absence of a clear plug-length effect is consistent with the load–displacement observations in Section 4.2: because geotechnical failure was not reached and the applied loads remained relatively small, the mobilized resistance distribution was not sufficiently developed to reveal a distinct influence of the retained soil plug. Within the tested load range, no clear effect of soil plug length on the back-calculated shaft resistance could be identified.



(a) Plugged idealization



(b) Unplugged idealization

Figure 23: Back-calculated t – z curves for all test conditions (EN cable), grouped by layer.

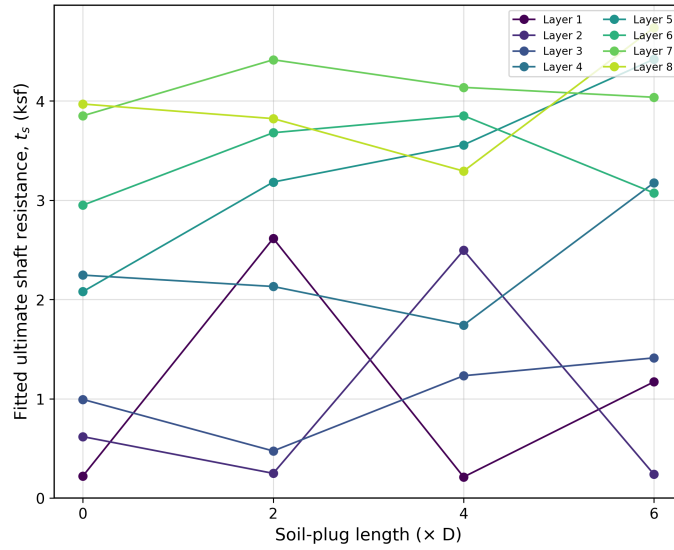


Figure 24: Fitted ultimate unit shaft resistance $t_{\max}$ versus soil plug length for each soil layer (PLT 1 series, EN cable, plugged idealization).

4.5.4 Effect of analysis method (plugged vs unplugged)

The two idealizations introduced in Section 4.5, the plugged and unplugged cases, were compared to assess whether the field test can distinguish between closed-ended and open-ended load transfer behavior. Figure 25 compares the two interpretations for the representative condition (PLT 2, 2D plug, EN cable). The back-calculated t - z curve for a deep layer (Figure 25a) and the tip q - z curve (Figure 25b) are nearly indistinguishable between the two cases over the mobilized range, confirming that both idealizations reproduce the same measured response. The key difference lies in how the mobilized head load is partitioned among the resistance components (Figure 25c). Both interpretations carry essentially the same total head load ($\approx 1,050$ kips), but the plugged case attributes the non-shaft contribution (≈ 125 kips, about 12%) to base end bearing acting over the full cross-section, whereas the unplugged case attributes a comparable contribution (≈ 150 kips, about 14%) to internal friction along the retained soil plug, with a negligible annular tip contribution (≈ 10 kips, about 1%). The pile base or tip is essentially unmobilized in both interpretations (the annular tip carries about 1% of the load in the unplugged case), which is consistent with the under-mobilized q - z response discussed in Section 4.5.1. Because the plugged versus unplugged distinction is fundamentally a difference in how the base and plug resist load, and the base was not mobilized, the two idealizations are observationally indistinguishable from the strain data alone; both reproduce the measured response by re-partitioning the same ~ 12 – 14% of non-shaft resistance. A definitive determination of plugged versus unplugged behavior would require either higher loads to mobilize the base or direct instrumentation of the plug and tip.

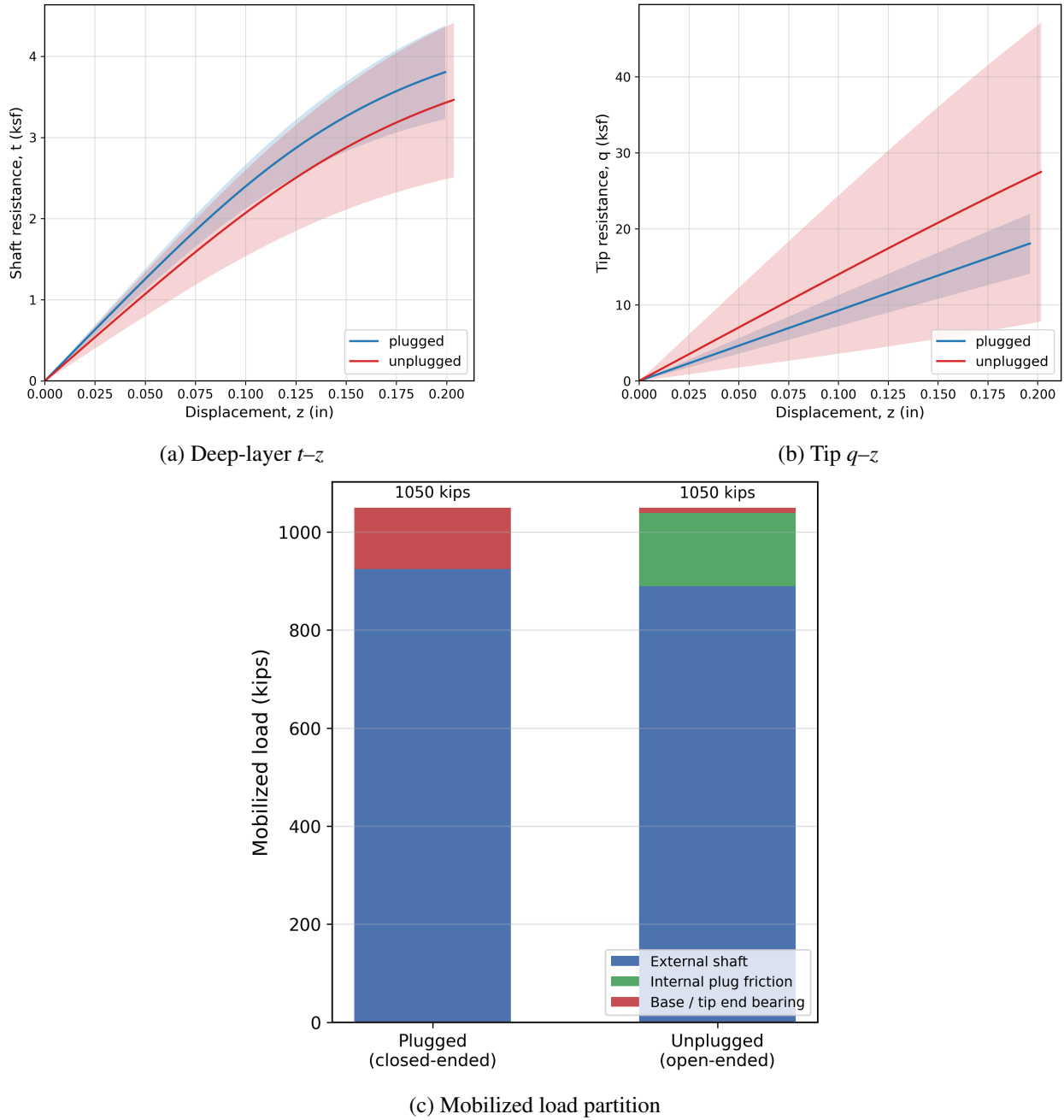
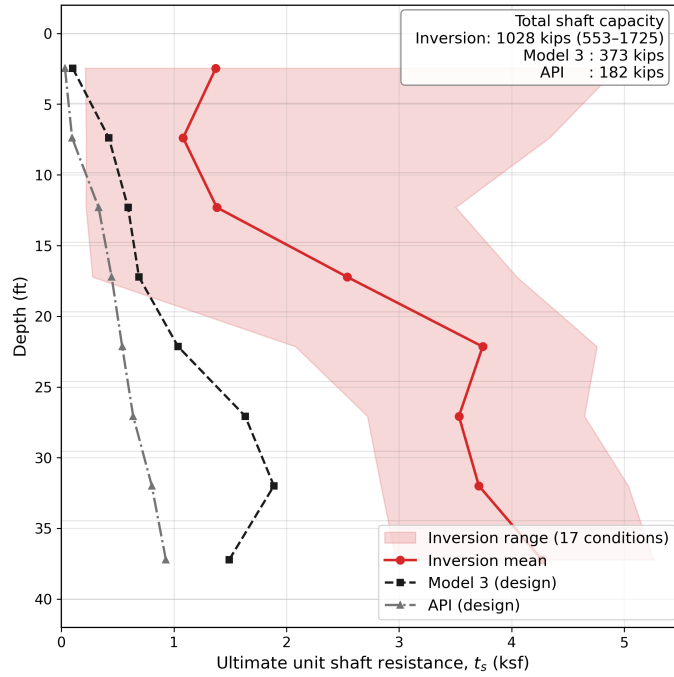


Figure 25: Plugged versus unplugged interpretation (PLT 2, 2D plug, EN cable).

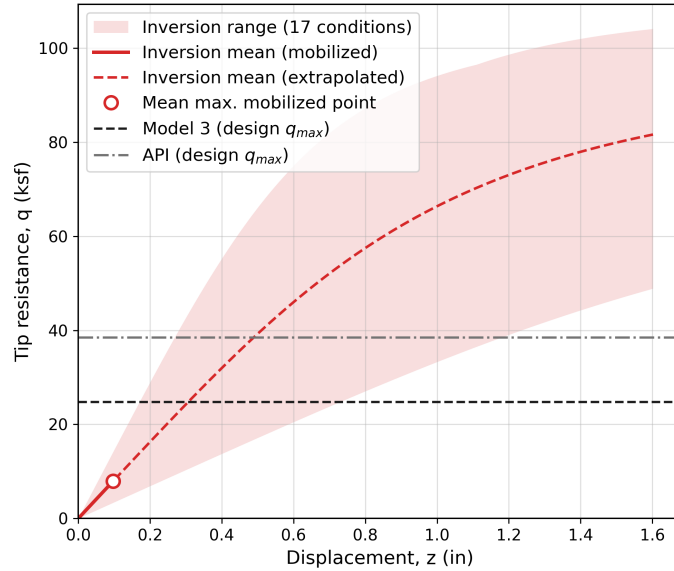
4.5.5 Comparison with API and Model 3 Predictions

The back-calculated shaft resistance was compared with the API and Model 3 design predictions. Figure 26(a) shows the fitted ultimate unit shaft resistance $t_{\max}$ as a function of depth, with the inversion results from all test conditions summarized by their mean and minimum–maximum range and the API and Model 3 design unit shaft resistances plotted for comparison. At every depth, the back-calculated shaft resistance lies well above both design predictions. Integrated over the pile, the back-calculated ultimate shaft capacity (mean of 1,028 kips, ranging 553–1,725 kips across conditions) exceeds the Model 3 (373 kips) and API (182 kips) predictions by factors of approximately 2.8 and 5.7, respectively. The shaft comparison above is

the meaningful one, because the shaft resistance was well mobilized and reliably back-calculated. For completeness, the back-calculated tip response is also compared with the design end bearing in Figure 26(b), in which the q - z curves from all plugged conditions are aggregated. The mean fitted $q_{\max}$ (93.9 ksf, ranging 72.7–109.1 ksf across conditions) exceeds both the API (38.5 ksf) and Model 3 (24.8 ksf) end-bearing values. However, as noted in Sections 4.5.1 and 4.5.4, the tip was barely mobilized (about 0.1 in on average; the solid segment in Figure 26(b)), so the fitted $q_{\max}$ is an extrapolation with large scatter across conditions (shaded band) and should not be taken as a validated end-bearing capacity. This tip comparison is therefore indicative only, and the field tests do not provide a reliable basis for assessing the design end-bearing resistance. The finding that the measured shaft resistance substantially exceeds the design estimates indicates that the in-situ soil is considerably stronger and stiffer than assumed in the design calculations, which is consistent with the installation-induced densification and increased confinement discussed in Section 5.4. This result also provides a unifying explanation for the limited plug-related observations in the preceding sections: because the soil resistance greatly exceeded the design estimates, the maximum test loads, which were sized using those design methods, were carried almost entirely by shaft resistance at small displacements, leaving the pile tip and soil plug insufficiently mobilized to clearly distinguish soil plug length effects or plugged versus unplugged behavior.



(a) Ultimate unit shaft resistance versus depth (inset: integrated total shaft capacity)



(b) Aggregated tip q - z versus design end bearing (solid = mobilized, dashed = extrapolation)

Figure 26: Back-calculated resistance compared with the API and Model 3 design predictions (all test conditions, plugged idealization).

5 Finite-Element Analysis

Although the full-scale load tests provided direct measurements of the load–displacement response and the load transfer behavior inferred from the axial strain distribution, the measured responses remained

largely within the elastic range under the CPT-based design load levels. As a result, the mechanisms associated with ultimate resistance mobilization could not be clearly examined from the field data alone. Therefore, finite-element (FE) analyses were performed to simulate the field test configuration. The analyses were used to further interpret the observed response and to evaluate the sensitivity of the pile behavior to key model parameters through controlled parametric studies.

5.1 Finite-Element Model

Finite-element analyses were conducted using Abaqus 2024 with a two-dimensional axisymmetric modeling approach. The model represents a CISS pile embedded in a soil domain, in which a soil plug was retained inside the shell near the pile tip. Consistent with the field tests, only the soil plug was modeled inside the pile, representing the Stage C condition prior to reinforcement installation and concrete placement (Figure 1).

The pile geometry was set identical to the field test dimensions, with pile radius $r_p = 1.5$ ft (diameter $D = 3$ ft), shell thickness $t = 0.5$ in, and pile length $l_p = 40$ ft. The surrounding soil domain was selected sufficiently large to minimize boundary effects, with radius $r_s = 30$ ft ($10D$) and depth $l_s = 80$ ft ($2l_p$). The soil plug length, l_{sp} , was defined as the remaining soil length inside the pile and was varied as $l_{sp} = 0, 2D, 4D, 6D$ to match the field test conditions (Section 3.2). Figure 27 illustrates the model geometry and the definition of l_{sp} .

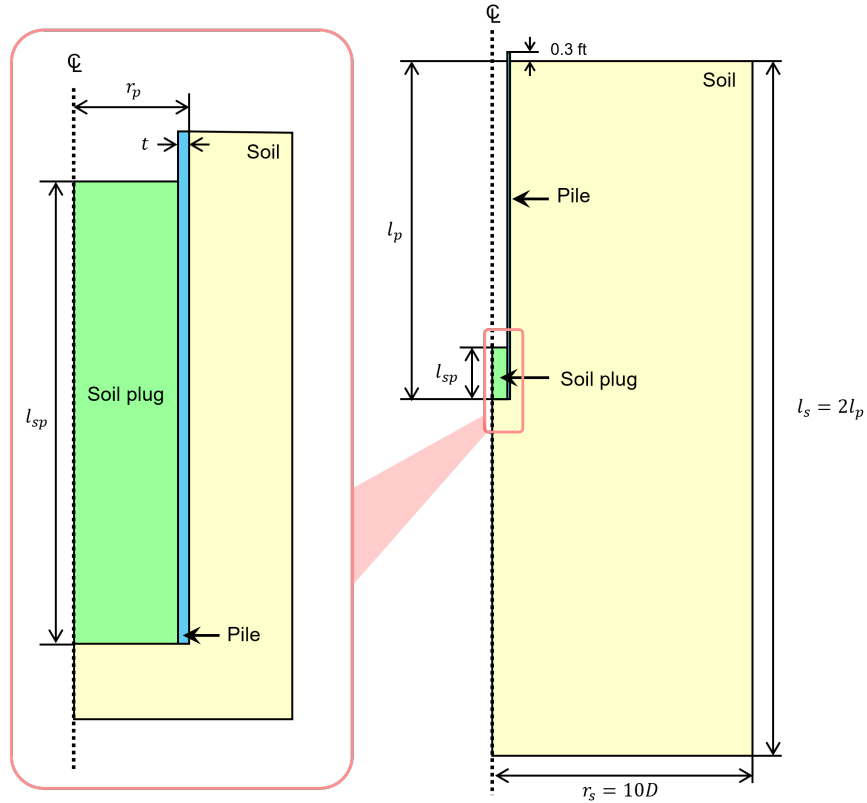


Figure 27: Finite-element model geometry and definition of soil plug length l_{sp} .

For discretization, the pile was modeled using 4-node bilinear axisymmetric quadrilateral elements with reduced integration (CAX4R), while the surrounding soil and the soil plug were modeled using 3-node linear axisymmetric triangular elements (CAX3). As shown in Figure 28, a mesh convergence study for

various mesh sizes demonstrated that the load–displacement response converged when a refined mesh with an element size of 1.97 in was used in the vicinity of the pile, together with a coarser mesh (39.37 in) in the far field. This mesh scheme was adopted in the analyses. In terms of constitutive modeling, the pile

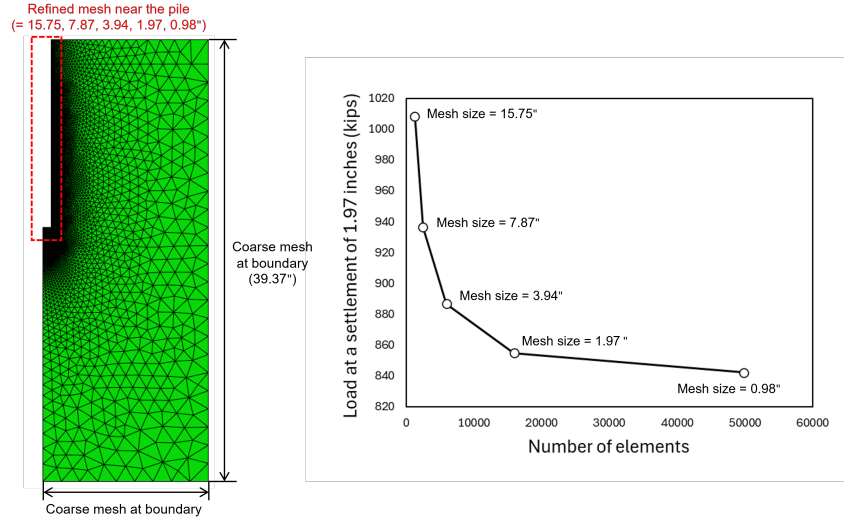


Figure 28: Mesh convergence of the load–displacement response for different refined mesh sizes near the pile.

was modeled as a linear elastic material, considering that the steel shell has much higher stiffness than the surrounding soil. Typical elastic properties of steel were assigned to the pile. The soil plug was also modeled as a linear elastic material. For CISS piles, the soil plug is created during driving and subsequent excavation (cleanout), and its condition is difficult to characterize due to construction-induced densification and disturbance. Using an elastoplastic model for the plug would introduce additional uncertain parameters and could lead to parameter-sensitive results. To reduce this uncertainty, the soil plug was idealized as elastic. This idealization appears reasonable given the elastic response observed in the field tests and the high confinement imposed by the steel shell. For this initial model setup, the plug properties were set equal to those of the surrounding soil described below, and refined later through calibration. For the surrounding soil, the Mohr–Coulomb (MC) elastoplastic model was adopted, maintaining a relatively simple parameter set for subsequent parametric studies and inverse analyses. The soil parameters for the MC model were derived from CPT-based interpretation. Figure 29 shows the soil properties estimated based on the CPT results. In the FE model, average values of unit weight γ , friction angle ϕ' , and earth pressure coefficient K_0 were adopted for soil parameters, while Young's modulus E was modeled as varying linearly with depth. Based on the CPT interpretation, Young's modulus increased approximately linearly with depth; therefore,

$$E(z) = E_0 + kz, \quad (31)$$

where $E_0 = 1450$ psi, $k = 44.21$ psi/ft, and z is the depth measured from the ground surface (ft). Table 6 summarizes the material properties of the pile, soil plug, and surrounding soil used in the analyses.

For the FE analysis, it should be noted that a drained (effective-stress) analysis was adopted assuming no excess pore pressure ($\Delta u = 0$) with a constant groundwater table at 4.43 ft depth. Water was not modeled explicitly; instead, the buoyant effect of groundwater was accounted for by using a submerged unit weight below the groundwater table, i.e., $\gamma' = \gamma - \gamma_w$.

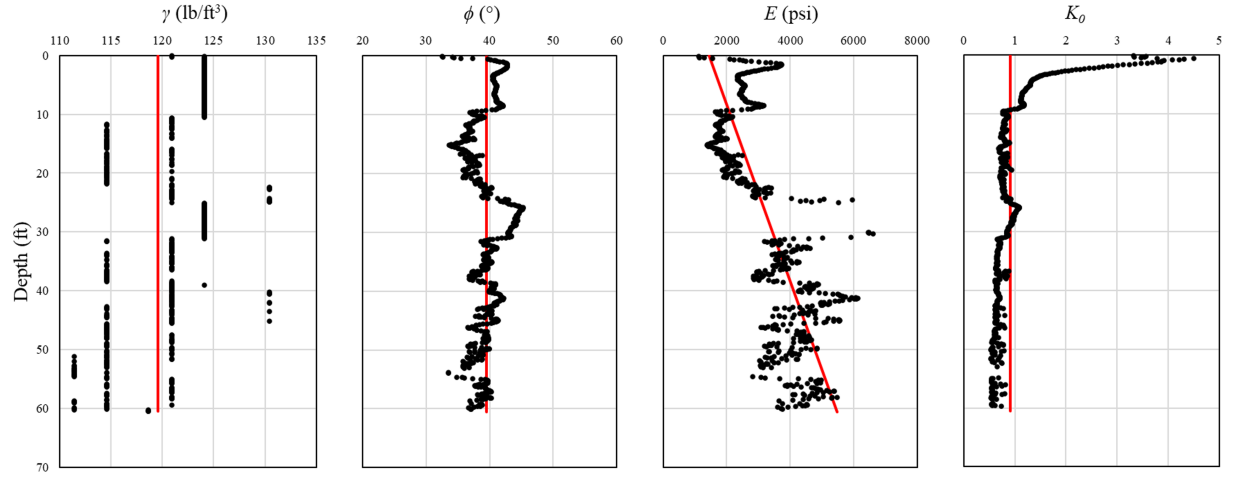


Figure 29: CPT-interpreted soil properties used to define the finite-element material parameters. The red lines indicate the average values adopted in the FE model and the linear variation of E with depth.

Table 6: Material properties used in the finite-element analyses.

Material	γ (γ') (lb/ft ³)	E (psi)	ν	ϕ' (°)	ψ (°)	c' (psi)	K_0
Pile	492	29000000	0.29	—	—	—	—
Soil plug	(57.43)	$E_0 + kz$	0.30	—	—	—	0.92
Soil	119.86 (57.43)	$E_0 + kz$	0.30	39.47	3	0	0.92

Note: The soil plug was entirely below the groundwater table in all analyses; therefore, γ' was applied for the soil plug.

5.2 Simulation Procedure

The FE simulations were carried out using a staged procedure that reproduces the construction sequence used in the field tests. Figure 30 summarizes the simulation procedure adopted in this study. First, a geostatic stress state was established in the soil domain by imposing displacement boundary conditions along the model boundaries (zero radial displacement along the side boundary and zero vertical displacement along the bottom boundary) and applying the gravity load. After equilibrium was reached, the resulting stress state was verified against the target geostatic condition with $\sigma'_{v0} = \gamma'z$ and $\sigma'_{h0} = K_0\sigma'_{v0}$, where σ'_{v0} and σ'_{h0} are the initial vertical and horizontal effective stresses at depth z , respectively.

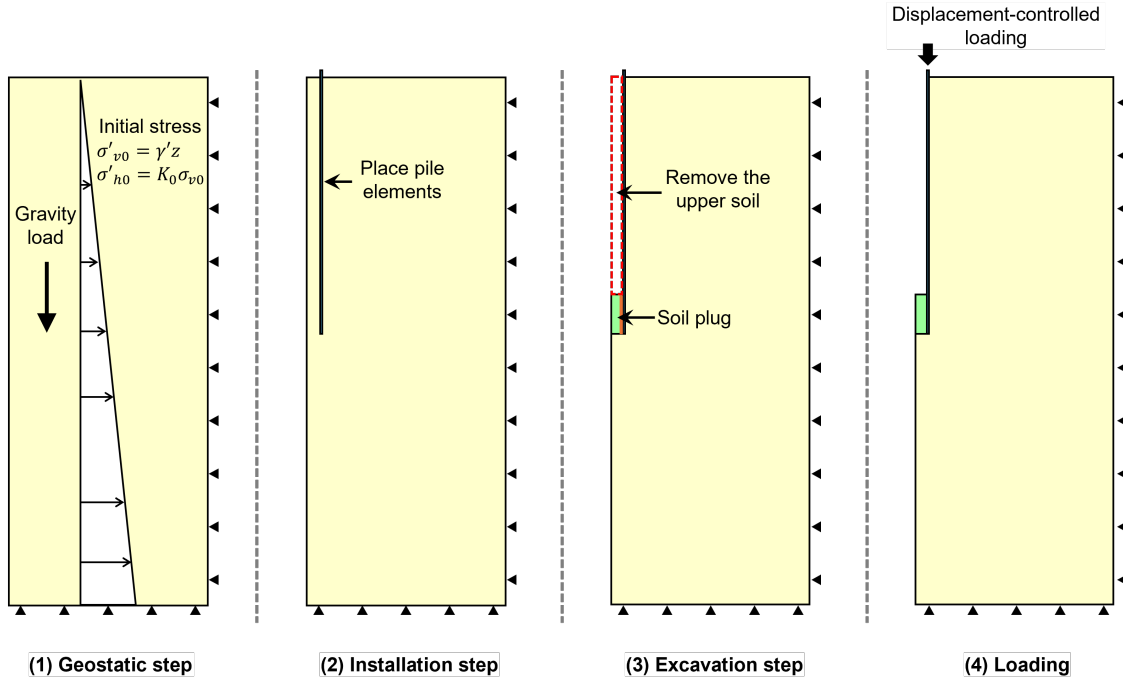


Figure 30: Simulation procedure.

Next, the pile installation was modeled in a simplified manner by deactivating the soil elements located within the pile footprint and activating the pile elements to represent the post-driving CISS configuration. Although a more detailed simulation of pile driving could be performed using advanced techniques such as adaptive meshing, this study adopted a practical idealization in which the pile is introduced into a geostatic soil mass, similar to a cast-in-place pile installation. As a result, construction-induced disturbance and soil plug formation during driving were not modeled explicitly. Instead, these effects were evaluated through parameter calibration by comparing the calibrated parameters with the CPT-interpreted estimates.

During and after the installation step, the pile–soil interface was modeled using a Coulomb friction contact formulation. In this model, the interface shear stress is limited by Coulomb friction,

$$\tau_{\max} = \mu p, \quad (32)$$

where $\tau_{\max}$ is the maximum shear stress, p is the contact pressure, and μ is the friction coefficient. Figure 31 illustrates the shear stress–slip (τ – γ) relationship for the Coulomb friction model. In this study, the allowable elastic slip γ_i was set to 0.005 times the local element length at the interface (i.e., $\gamma_i = 0.0098$ in for the adopted mesh; Abaqus default), and μ was set to 0.8 for the CPT-based simulations and subsequently calibrated.

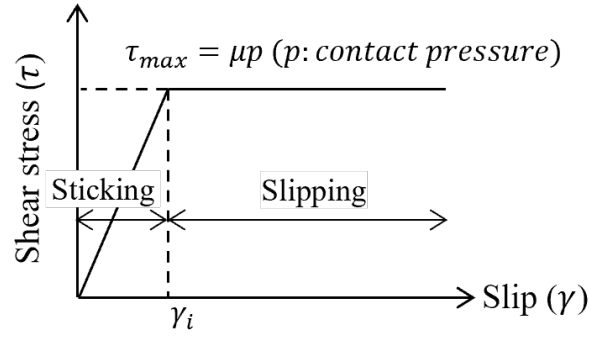


Figure 31: Coulomb friction model adopted for the pile–soil interface.

After equilibrium was reached in the installation step, the soil above the target soil plug length was removed in the excavation (cleanout) step while the prescribed soil plug was retained. Finally, axial loading was applied under displacement control by prescribing a downward displacement at the pile head, and the corresponding reaction force and internal response were monitored to obtain the simulated load–displacement behavior and load transfer response.

5.3 Comparison with Field Test Results

FE simulations were performed using the CPT-interpreted soil parameters described in Section 5.1. Because PLT 1 reached only a relatively small maximum load and the response remained near-linear, the comparison focuses on PLT 2, where a higher target load was applied to mobilize larger displacements. Simulations were conducted for the two soil plug conditions considered in the field program, $l_{sp} = 2D$ and $4D$. For a direct comparison at an identical deformation state, the measured pile head displacement from PLT 2 was prescribed at the pile head during the loading step (Step 4 in Figure 30). For each plug condition, the reaction force at the pile head was extracted to obtain the simulated load–displacement response. Axial strain profiles along the pile were also extracted at the same imposed displacement levels and compared with the field measurements.

Figure 32 compares the CPT-based FE results with the field test data. In the strain plots, results at the same displacement levels are shown in the same color, with solid lines for FE simulations and dotted lines for field measurements. The legend indicates the field load levels used to define the displacement levels prescribed in the FE simulations. The load–displacement curves show that the CPT-based simulations predict substantially lower loads than measured in the field at the same pile head displacement, indicating an underestimation of the mobilized resistance. Consistent with this trend, the simulated axial strain profiles are also smaller than those measured in the field, because the FE model predicted a smaller load at the same displacement levels.

These discrepancies suggest that the CPT-derived parameters, together with the simplified installation idealization adopted in this study, do not fully represent the stiffness and strength state of the soil surrounding the driven steel shell under the tested conditions. Therefore, parametric studies were performed to assess the sensitivity of the predicted response to key model parameters. The parametric studies considered parameters expected to strongly affect the mobilized resistance: the soil plug elastic modulus (E_{sp}), the pile–soil interface parameters (friction coefficient μ and allowable elastic slip γ_i), and the surrounding soil properties (elastic modulus E_s , friction angle ϕ' , and dilation angle ψ). Unlike the CPT-based comparison, where the measured displacement was prescribed, additional displacement was applied in the parametric studies until the simulated load reached a level comparable to the field measurements. The discussion below focuses on the load–displacement response, because comparing strain profiles under substantially different

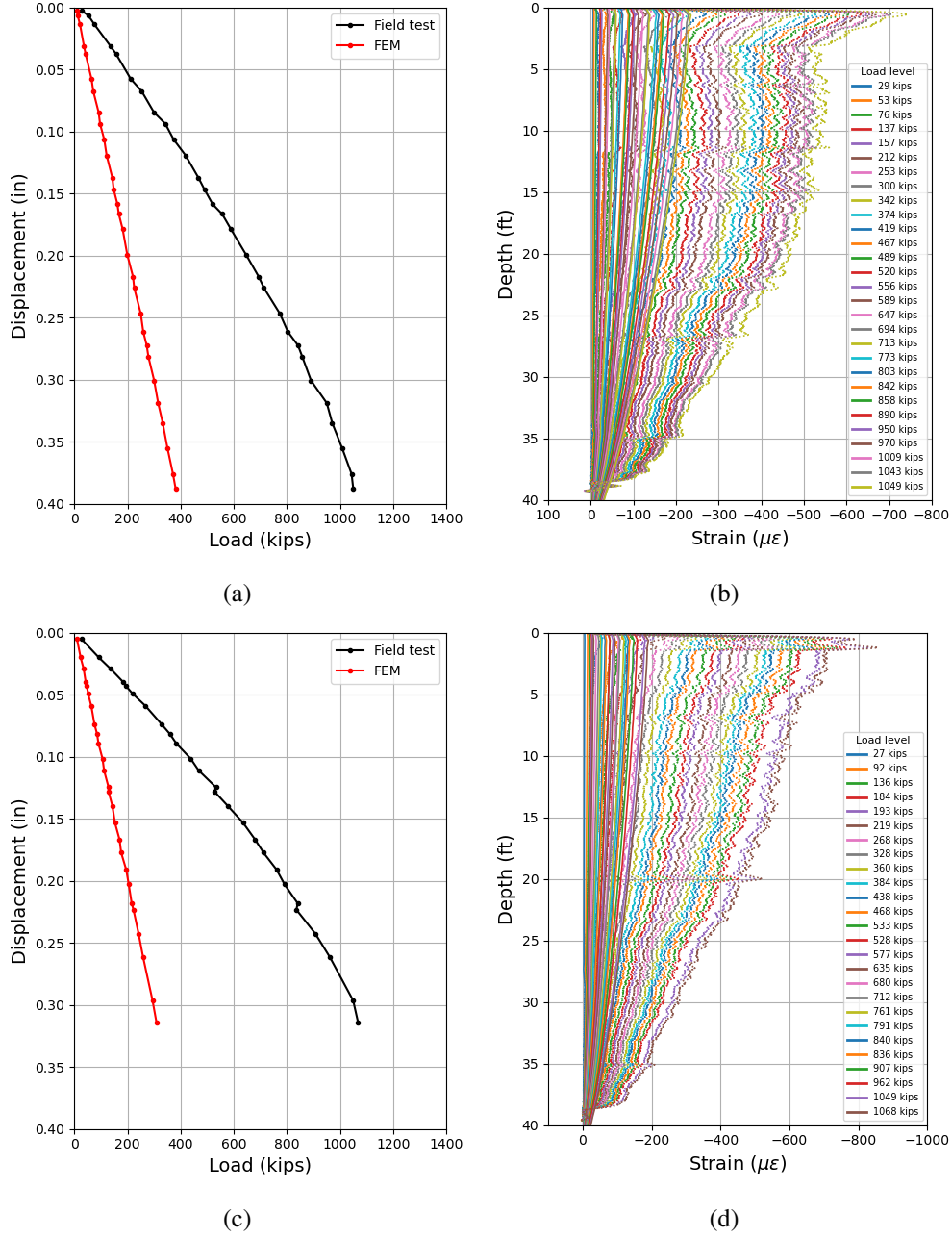


Figure 32: Comparison between FE simulations (CPT-interpreted parameters) and field test results: (a) load-displacement ($l_{sp} = 2D$), (b) axial strain profile ($l_{sp} = 2D$), (c) load-displacement ($l_{sp} = 4D$), and (d) axial strain profile ($l_{sp} = 4D$).

load levels provides limited insight. Figure 33 summarizes the results of the parametric studies. Within the investigated ranges, the soil plug stiffness showed a limited influence on the load–displacement response: adopting $E_{sp} = E_s$ and adopting E_{sp} comparable to the steel shell ($E = 29,000$ ksi) produced similar curves (Figure 33a). The interface parameters also led to relatively minor changes in the load–displacement response in most cases (Figure 33b), except for the low-friction case ($\mu = 0.4$), which resulted in a noticeably softer response. This behavior is attributed to the reduced interface shear strength ($\tau_{\max}$), which promoted extensive slip along the pile–soil interface and reduced mobilized resistance.

In contrast, the response was highly sensitive to the stiffness and shear strength of the surrounding soil. Increasing E_s relative to the CPT-interpreted values progressively stiffened the response and brought the simulations closer to the field measurements (Figure 33c). The shear strength parameters showed a similarly strong influence: increasing ϕ' stiffened the response, and increasing ψ further increased stiffness due to enhanced confinement associated with dilative behavior (Figure 33d). Overall, these results suggest that the primary source of mismatch in the CPT-based simulations is associated with the stiffness and strength state of the surrounding soil.

It should be noted, however, that the above sensitivity trends are based on the load–displacement response and the specific parameter ranges considered. The soil plug and interface parameters can still influence the distribution of load transfer (i.e., strain profiles). Therefore, to obtain a consistent set of parameters that reproduces both the measured load–displacement response and the axial strain profiles, an inverse analysis was conducted in the following section, where the surrounding-soil, soil-plug, and interface parameters were calibrated simultaneously.

5.4 Inverse Analysis for Parameter Calibration

To obtain a parameter set that consistently reproduces both the measured load–displacement response and the axial strain profiles, an inverse analysis was performed using the previously developed Abaqus finite element model as the forward analysis engine. The overall framework of the inverse analysis is illustrated in Figure 34. The procedure consists of defining a set of optimization variables, running the Abaqus simulation with a trial parameter set, evaluating the discrepancy between the numerical and measured responses through an objective function, and iteratively updating the parameters until convergence is achieved.

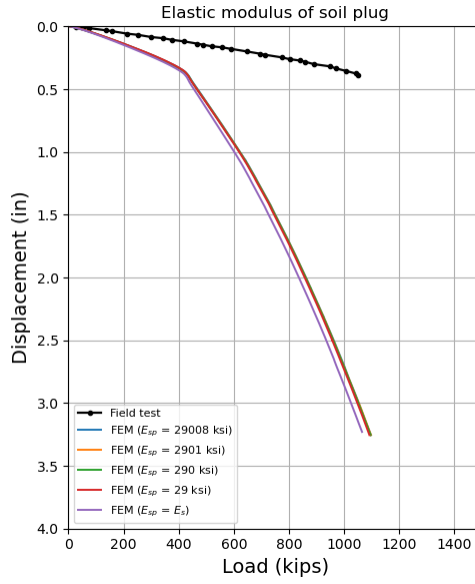
The vector of optimization variables is defined as

$$\mathbf{P} = [\phi', \psi, E_{s0}, k_s, \nu_s, E_{sp}, \nu_{sp}, \mu, K_0], \quad (33)$$

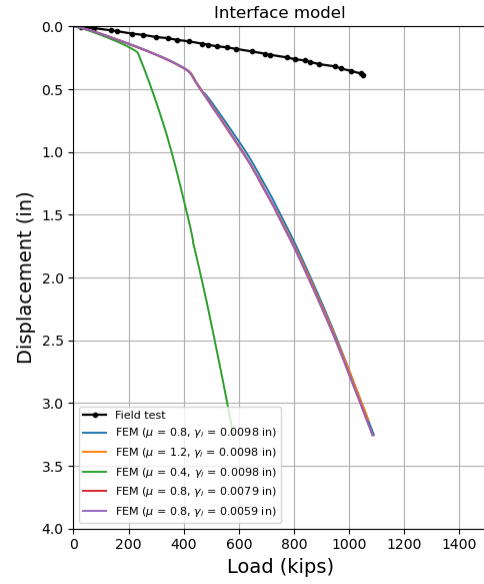
where ϕ' and ψ denote the friction angle and dilation angle of the surrounding soil, respectively; E_{s0} is the elastic modulus of the surrounding soil at the ground surface ($z = 0$), and k_s is the depth-dependent gradient of the elastic modulus such that $E_s(z) = E_{s0} + k_s z$; ν_s is the Poisson's ratio of the surrounding soil; E_{sp} and ν_{sp} are the elastic modulus and Poisson's ratio of the soil plug; μ is the pile–soil interface friction coefficient; and K_0 is the lateral earth pressure coefficient used to define the initial geostatic stress state. These parameters were selected because they govern the stiffness and shear strength state of the surrounding soil, the mechanical behavior of the retained soil plug, and the interface shear transfer mechanism, all of which influence the mobilized resistance and load transfer behavior observed in the field tests.

For a given trial parameter set $\mathbf{P}$, FE analysis was performed using the developed Abaqus finite-element model. Consistent with the comparison presented in Section 5.3, the measured pile head displacements (u_i) from the field test were prescribed in the loading step of FE model. From each simulation, the reaction force at the pile head corresponding to each imposed displacement level ($F_{\text{FEM}}(u_i; \mathbf{P})$) and the axial strain distribution along the pile at the same displacement levels were evaluated ($\epsilon_{\text{FEM}}(z_j, u_i; \mathbf{P})$). Here, z_j denotes the DFOS measurement depths along the pile length.

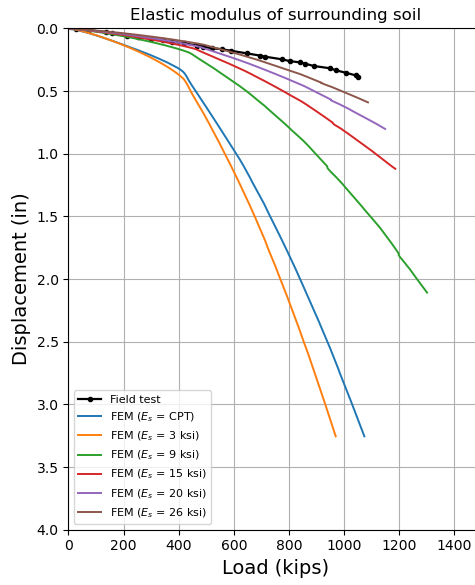
Using the evaluated simulation results, an objective function was defined to quantify the discrepancy between the simulated and measured responses. Because the displacement levels were prescribed to match the



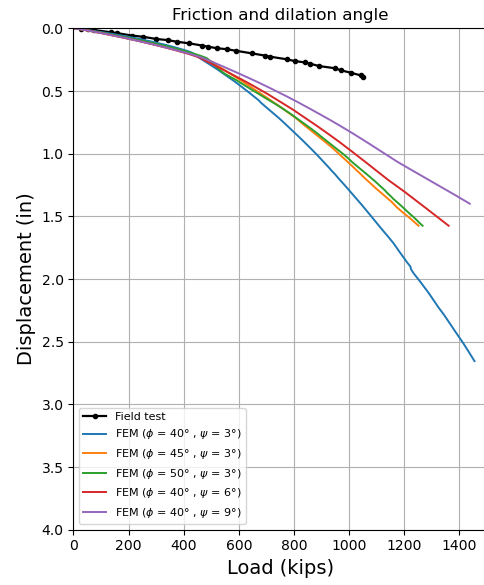
(a)



(b)



(c)



(d)

Figure 33: Load–displacement curves from the parametric studies: (a) soil plug elastic modulus, (b) interface parameters, (c) surrounding soil elastic modulus, and (d) surrounding soil friction and dilation angles.

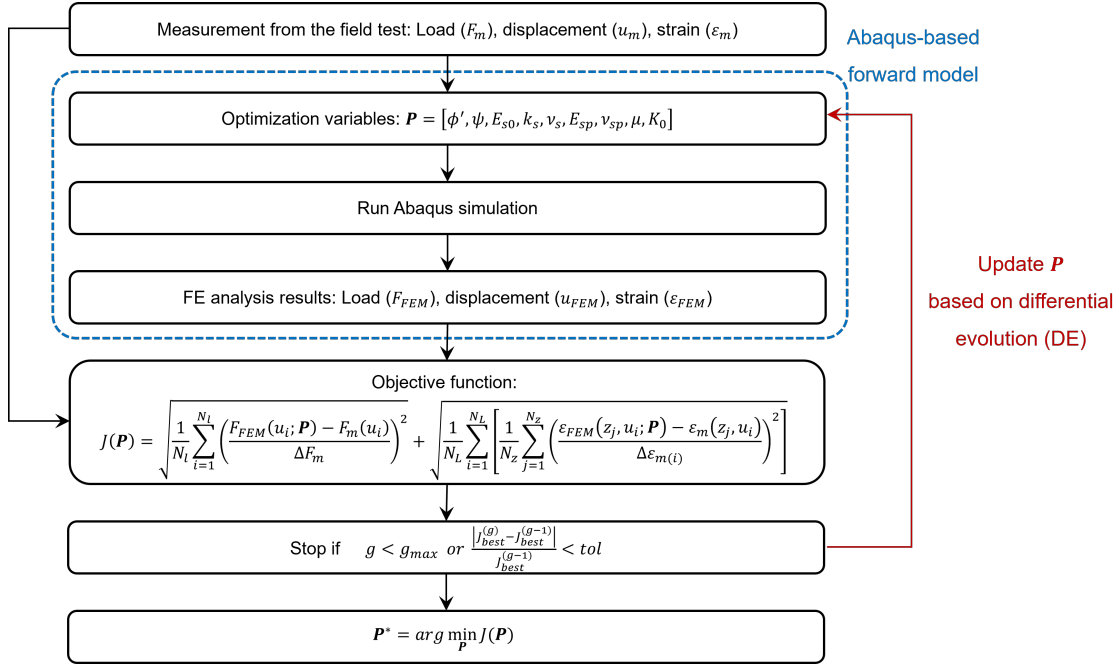


Figure 34: Framework of inverse analysis for parameter calibration.

field test, the load mismatch was evaluated as $J_F(\mathbf{P})$. In addition, to ensure that the load transfer mechanism was properly reproduced, the difference between the simulated and measured axial strain profiles ($J_\epsilon(\mathbf{P})$) was also incorporated. To provide balanced consideration of the load and strain components, normalized errors were adopted. The objective function is expressed as

$$J(\mathbf{P}) = \underbrace{\sqrt{\frac{1}{N_L} \sum_{i=1}^{N_L} \left(\frac{F_{FEM}(u_i; \mathbf{P}) - F_m(u_i)}{\Delta F_m} \right)^2}}_{J_F(\mathbf{P})} + \underbrace{\sqrt{\frac{1}{N_L} \sum_{i=1}^{N_L} \left[\frac{1}{N_z} \sum_{j=1}^{N_z} \left(\frac{\epsilon_{FEM}(z_j, u_i; \mathbf{P}) - \epsilon_m(z_j, u_i)}{\Delta \epsilon_{m(i)}} \right)^2 \right]}}_{J_\epsilon(\mathbf{P})}, \quad (34)$$

where N_L is the number of displacement steps considered, N_z is the number of strain measurement locations, $F_m(u_i)$ and $\epsilon_m(z_j, u_i)$ are the measured load and strain from the field test, respectively, and ΔF_m and $\Delta \epsilon_{m(i)}$ are normalization factors taken as the maximum measured load and the maximum measured strain at step i , respectively. For the inverse analysis, the measured strain profiles were taken from the EN cable.

The inverse analysis framework was implemented in a Python environment that integrates parameter updating, Abaqus execution, and post-processing of results. To minimize the objective function defined in Eq. (34), the Differential Evolution (DE) algorithm was adopted. DE is a population-based global optimization method that is well suited for nonlinear, multi-parameter problems where gradient information is not readily available. At each generation, candidate parameter sets were generated through mutation and crossover operations, evaluated by running the forward Abaqus simulation, and selected based on their objective function values. The optimization process was terminated when either the maximum number of generations (30) was reached or the relative change in the best objective value fell below a prescribed tolerance (1×10^{-3}). The optimized parameter set $\mathbf{P}^*$ was taken as the parameter set that yields the minimum objective value:

$$\mathbf{P}^* = \arg \min_{\mathbf{P}} J(\mathbf{P}). \quad (35)$$

Figure 35 compares the load–displacement response and axial strain profiles from the FE analyses using the optimized parameter set against the corresponding field measurements. The calibrated model reproduces the measured load–displacement response well, whereas the simulations based on the CPT-interpreted parameters substantially underpredict the load at the same displacement levels (Figure 32). In addition, the simulated strain profiles obtained with the optimized parameters show overall good agreement with the field measurements. Minor discrepancies in the strain profiles can be attributed to the sensor configura-

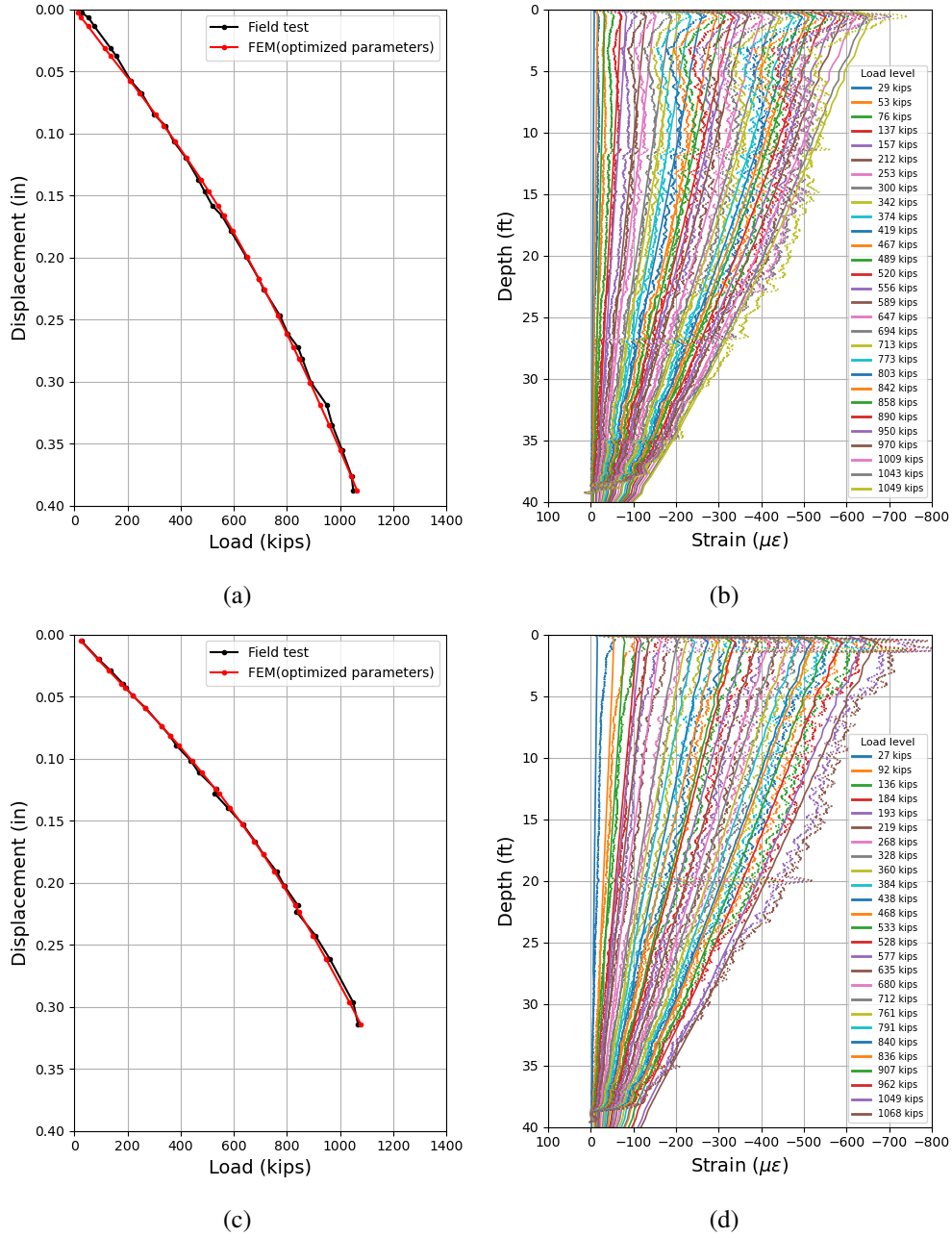


Figure 35: Simulation results with optimized parameters and field test results: (a) load–displacement ($l_{sp} = 2D$), (b) axial strain profile ($l_{sp} = 2D$), (c) load–displacement ($l_{sp} = 4D$), and (d) axial strain profile ($l_{sp} = 4D$).

tion adopted in the field test and the modeling idealizations in the FE analyses. For $l_{sp} = 2D$, a noticeable mismatch was observed near the pile head. This discrepancy is primarily associated with the use of the EN cable for the inverse analysis. As described in Section 4.5.2, the EN cable exhibits a rapid reduction in measured strain near the pile head, whereas the other DFOS cables indicate relatively larger strains in this region. Therefore, the calibrated strain profile near the pile head is expected to more closely match the results obtained from the other cables. For both $l_{sp} = 2D$ and $4D$, the FE results show relatively large compressive strains near the pile tip. In contrast, the DFOS measurements exhibit a sharp reduction in strain near the pile tip, which is primarily attributed to the installation detail at the cable termination. To avoid cable damage during pile installation, the DFOS cable end near the pile tip was intentionally left unbonded (i.e., not fully attached to the pile), and thus the measured strains in this region are considered less representative of the true pile strain. Accordingly, the actual pile strain near the pile tip is expected to be closer to the FE-predicted values than implied by the DFOS measurements. Lastly, the surrounding soil in the FE model was represented using a simplified, uniform soil profile, which tends to produce smooth and approximately linear variations in the computed strain profiles. Incorporating a layered soil profile consistent with the site stratigraphy is expected to further improve the agreement with the measured strain distributions. Table 7 summarizes the optimized parameter sets for $l_{sp} = 2D$ and $4D$, together with the CPT-derived parameters. Note that ψ , ν_s , ν_{sp} , and μ are not directly obtained from CPT interpretation; therefore, typical values were initially adopted in the CPT-based analyses and subsequently calibrated through the inverse analysis. In addition, unlike the initial properties listed in Table 6, E_{sp} in Table 7 is reported as the equivalent elastic modulus at the pile tip evaluated from the adopted linear profile. Compared with the CPT-derived parameters, the inverse analysis yielded substantially larger elastic moduli for both the surrounding soil (E_{s0}) and the soil plug (E_{sp}). This trend suggests that the installation of the CISS pile, particularly driving of the steel shell, may have densified and stiffened the soil around and within the shell relative to the CPT-based estimate. The optimized parameters also indicate a lower friction angle and a higher dilation angle than the CPT-based values. However, these shear strength parameters should be interpreted with caution, as ϕ' and ψ may trade off within the adopted constitutive model and calibration objective. Accordingly, the optimized ϕ' and ψ values are best viewed as an effective parameter combination that reproduces the measured response within the current modeling framework, rather than as a direct indication of changes in the in-situ friction angle due to installation.

Table 7: Optimized parameter sets obtained by the inverse analysis and CPT-derived material properties.

Material	ϕ' (°)	ψ (°)	E_{s0} (psi)	k_s (psi/ft)	ν_s	E_{sp} (psi)	ν_{sp}	μ	K_0
CPT-derived parameters	39.47	3.00	1,450	66.3	0.30	4,102	0.30	0.80	0.92
Optimization 2D	29.55	13.66	21,321	212.2	0.34	26,977	0.45	1.07	1.00
Optimization 4D	29.28	7.84	41,626	181.3	0.38	35,244	0.37	1.02	1.18

5.5 Discussion

To interpret the large increase in the optimized elastic modulus of the surrounding soil, the installation effect of the CISS pile was evaluated using cavity expansion theory. Cavity expansion theory provides analytical solutions to estimate stress and displacement changes in soil induced by the expansion of cylindrical (or spherical) cavities, and it has been used to idealize installation effects associated with driven piles and in-situ tests (e.g., pressuremeters). In this study, the closed-form solution commonly used for pressuremeter expansion was adopted as an idealized representation of the steel shell installation. As shown in Figure 36, the shell installation was simplified as a cavity expanding from the inner radius of the shell (r_{in}) to the outer radius (r_{out}). At $r = r_{in}$, the initial radial effective stress was taken as the geostatic lateral stress ($p_0 = \sigma'_{h0}$).

The required cavity wall pressure (p) to expand the cavity to r_{out} was then computed. The detailed formulations are provided in Yu (2013) and Liu (2011). Figure 37 presents the calculated relationship between

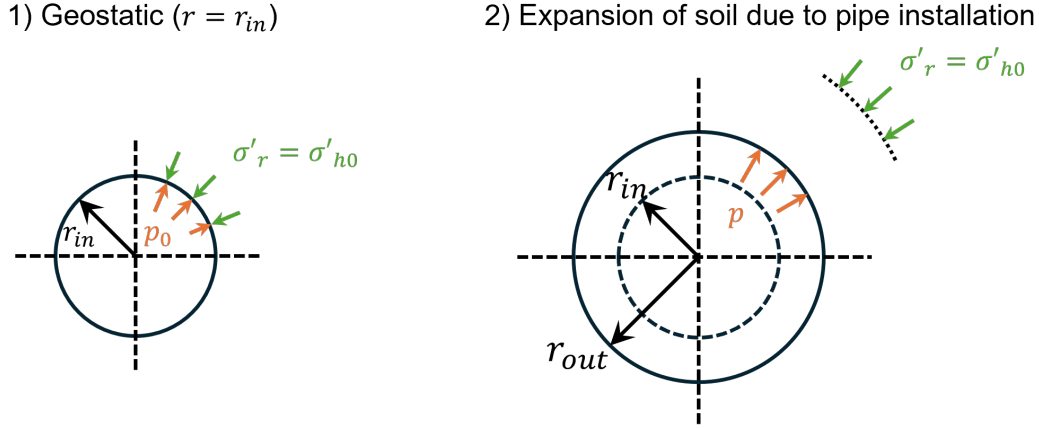


Figure 36: Cavity expansion for steel shell installation in a CISS pile.

the cavity wall pressure (p) and the cavity radius (r). As a representative example, the p – r curve at a depth of 33 ft is shown. The results indicate that expanding the cavity to the shell outer radius requires a cavity wall pressure of approximately 3.4 times the initial geostatic lateral pressure, increasing from 13.34 psi to 45.25 psi. This suggests that the installation of the steel shell can substantially increase the lateral effective stress (i.e., confinement) around the pile. For a quantitative assessment of the stiffness change associated

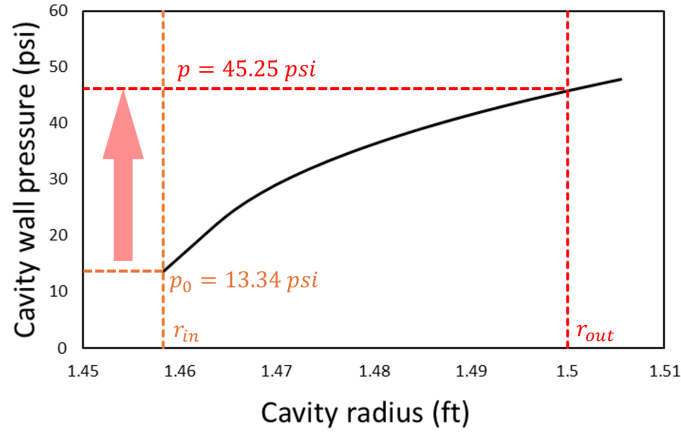


Figure 37: Pressure–radius relationship at a depth of 33 ft evaluated by cavity expansion theory.

with the increased confinement, the elastic modulus was re-estimated using the Janbu stress-dependent stiffness relationship (Janbu, 1963). In this relationship, the initial tangent modulus scales with confining stress as

$$E_i = K p_a \left(\frac{\sigma'_c}{p_a} \right)^n, \quad (36)$$

where E_i is the initial tangent modulus, K is the modulus number, p_a is the reference stress (atmospheric pressure), σ'_c is the confining pressure, and n is the coefficient (typically 0.3–0.7). Therefore, the modulus

ratio before ($E_{i(0)}$) and after ($E_{i(f)}$) installation can be expressed as

$$\frac{E_{i(f)}}{E_{i(0)}} = \left(\frac{p(r_{out})}{p_0} \right)^n. \quad (37)$$

Here, $p(r_{out})$ is the lateral effective stress (i.e., confinement) at the cavity wall after shell installation, calculated using cavity expansion theory, and p_0 is the initial geostatic lateral stress state. Figure 38 shows the increase in $p(r_{out})$ along depth and the associated modulus change estimated from this increased confinement. The modulus at $p = p_0$ corresponds to the CPT-based elastic modulus (Table 6), whereas the modulus at $p = p(r_{out})$ represents the re-estimated modulus using Janbu's relationship. For the re-estimation, $n = 0.7$ was adopted. Although cavity expansion theory predicts a clear increase in confinement and a corresponding increase in modulus (approximately 2–4 times the initial profile), the estimated increase remains much smaller than that inferred from the inverse analysis (approximately 14.7–28.7 times when comparing the optimized E_{s0} ; Table 7). This discrepancy suggests that the optimized modulus from the inverse analysis reflects not only the quasi-static volumetric expansion effect captured by cavity expansion theory, but also additional installation mechanisms (e.g., driving-induced dynamic effects) that can further densify the soil and enhance stiffness. To further interpret why the inverse analysis resulted in a substantially higher elastic

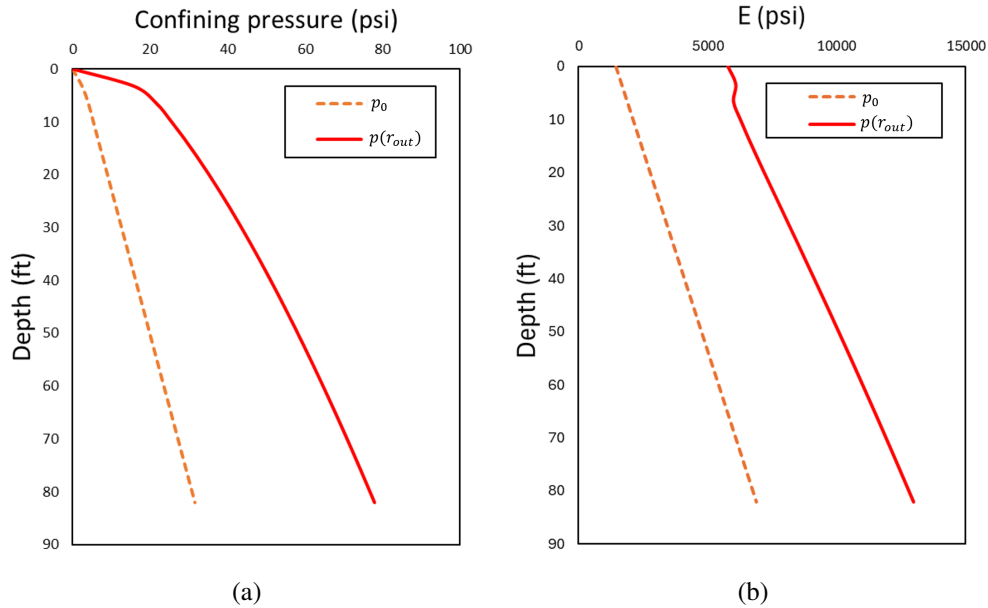


Figure 38: Installation-induced increase in confinement and the associated modulus change estimated using Janbu's relationship: (a) $p(r_{out})$ along depth from cavity expansion theory and (b) re-estimated E profile.

modulus, the strain fields from the simulations using CPT-estimated properties and the optimized parameters were examined. Figure 39 compares the logarithmic shear strain component ($LE12$) and plastic shear strain component ($PE12$) in the soil adjacent to the pile at depths of 21–27 ft. As a representative comparison, the strain contours for $l_{sp} = 2D$ are presented. As shown in Figure 39(a) and (b), the CPT-based simulation exhibits highly localized shear deformation adjacent to the pile–soil interface. $LE12$ reaches approximately 1.01 at the interface and decays rapidly with radial distance, dropping below 0.07 at $r/D \approx 0.1$. Moreover, $PE12$ is nearly equal to $LE12$ near the interface ($PE12 \approx 0.95$), indicating that the deformation is dominated by plastic shearing in the elements adjacent to the pile. Such extreme strain values are likely associated with the concentration of relative displacement into one or two elements adjacent to the pile–soil interface, forming a highly localized shear band. This indicates that the imposed pile-head displacement is

largely accommodated by intense local shearing within a narrow zone. As shown in Figure 39(c) and (d), the optimized simulation also exhibits a concentrated shear band near the pile–soil interface; however, the peak shear strain levels are markedly reduced. LE_{12} reaches approximately 0.061 at the interface and decreases to about 0.008 at $r/D \approx 0.1$. Similarly, PE_{12} is approximately 0.059 at the interface and decreases to about 0.0062 at $r/D \approx 0.1$. Relative to the CPT-based case, this corresponds to an approximately 94% reduction in LE_{12} at the interface and about 89% reduction in LE_{12} at $r/D \approx 0.1$. These results suggest that the optimized parameter set limits the intensity of shear localization and reduces excessive plastic shearing adjacent to the pile. Because the Mohr–Coulomb model with a constant elastic modulus cannot explicitly represent strain-dependent stiffness and progressive localization, the inverse analysis may increase the bulk elastic modulus as an effective parameter to reproduce the localized response around the pile. Accordingly, the large optimized modulus reflects not only installation-induced confinement and densification, but also model-form limitations associated with representing localized shear and stiffness evolution near the pile–soil interface.

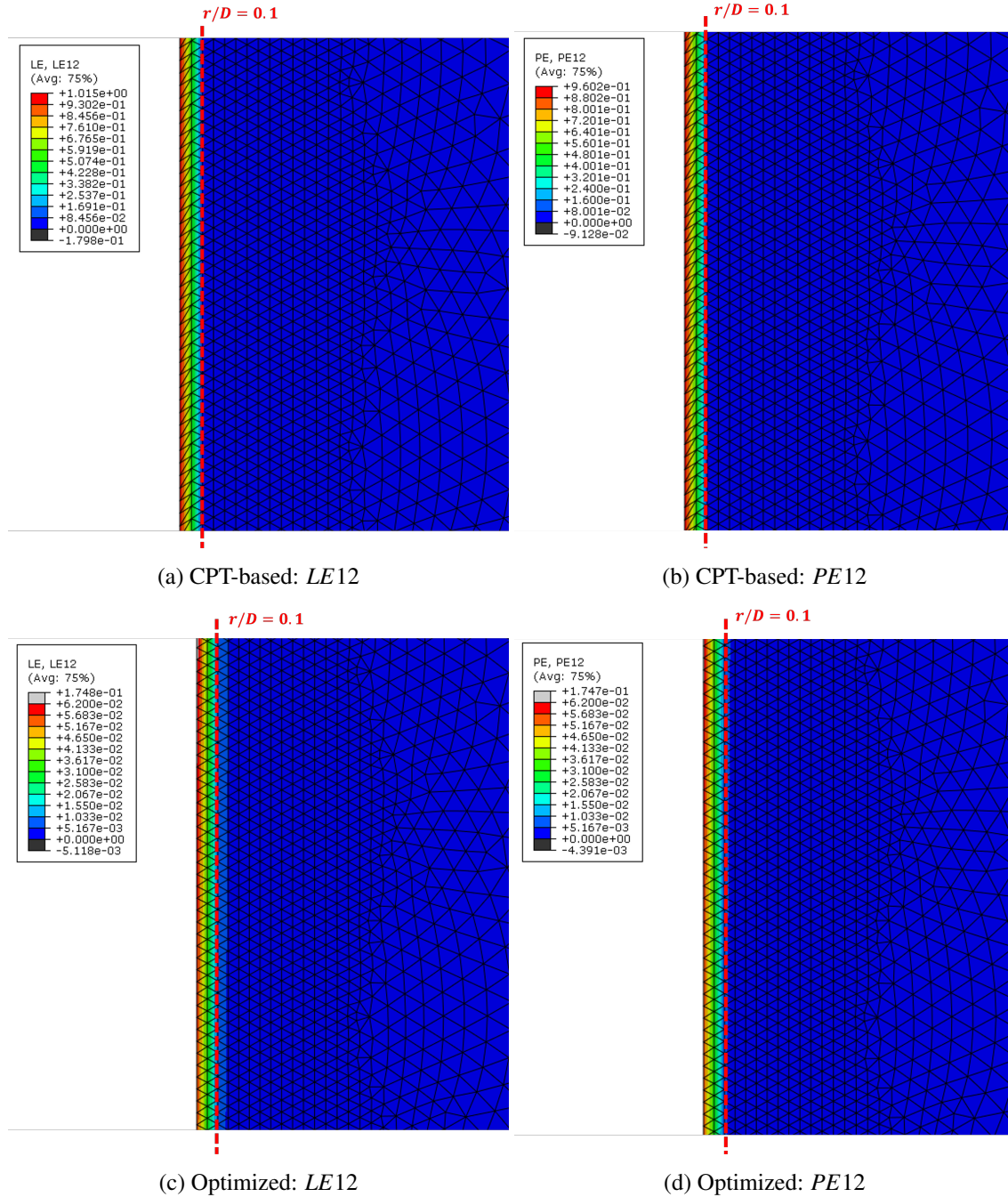


Figure 39: Shear strain ($LE12$) and plastic shear strain ($PE12$) contours in the soil adjacent to the pile for the CPT-based and optimized simulations (depth range: 21–27 ft).

6 Conclusions and Recommendations

6.1 Conclusions

This project evaluated the geotechnical resistance of CISS piles with varying remaining soil plug lengths through instrumented full-scale load tests and complementary finite-element analyses, with the objective of

assessing the suitability of the API-based design procedure required by Caltrans practice.

The main conclusions are summarized as follows.

1. **Test program.** Four nominally identical open-ended steel test piles ($D = 3$ ft, embedment 40 ft) were driven and cleaned out to target plug lengths of 0, $2D$, $4D$, and $6D$, and were tested in two series of top-down compression load tests (PLT 1 to 600 kips and PLT 2 to 1,150 kips). The DFOS instrumentation, installed in four configurations (EN, CN, S1, and S2), survived impact driving and provided high-resolution axial strain profiles throughout the program. The back-calculated t - z curves at depth were essentially independent of the cable installation method, demonstrating that DFOS is a robust and practical monitoring technique for driven pile load testing.
2. **Measured response.** Geotechnical failure was not reached in any test, and the load–displacement response remained largely linear. Within the applied load range, no systematic correlation was observed between the remaining soil plug length and either the pile head displacement or the back-calculated shaft resistance.
3. **Load transfer interpretation.** Inverse analyses using a one-dimensional beam–spring model reliably constrained the layer-wise t - z response, as the shaft springs were mobilized to 40–95% of their ultimate resistance (mean CoV $\approx 14\%$ across repeated optimizations). In contrast, the tip springs were mobilized to only 2–26%, so the fitted ultimate end bearing should be regarded as an extrapolated rather than a measured quantity. For the same reason, the plugged and unplugged idealizations were observationally indistinguishable: both reproduced the measured response by re-partitioning the same ~ 12 – 14% non-shaft contribution between base end bearing and internal plug friction.
4. **Comparison with design methods.** The back-calculated ultimate shaft capacity (mean of 1,028 kips across test conditions) exceeded the Model 3 prediction (373 kips) by a factor of approximately 2.8 and the API prediction (182 kips) by a factor of approximately 5.7. Both design methods are therefore substantially conservative for shaft resistance under the tested site conditions. The field tests did not provide a reliable basis for assessing the design end-bearing resistance because the pile tip was barely mobilized.
5. **Finite-element interpretation.** FE simulations using CPT-interpreted parameters significantly underpredicted the measured resistance. Inverse analyses required elastic moduli approximately 15–29 times the CPT-based estimates to reproduce the measured load–displacement and strain responses. Cavity expansion analysis indicates that the quasi-static installation-induced increase in confinement can account for only a 2–4 fold increase in stiffness; the remainder is attributed to driving-induced densification and to model-form limitations of the Mohr–Coulomb idealization in representing localized shear near the pile–soil interface. These results consistently indicate that shell driving substantially strengthens and stiffens the soil surrounding the pile relative to its pre-installation state.
6. **Implications for practice.** Because the mobilized soil resistance greatly exceeded the design estimates, the test loads, which were sized using the design methods, were carried almost entirely by shaft resistance at small displacements, leaving the pile tip and soil plug insufficiently mobilized to distinguish plug length effects. Within the load range applied in the tests (up to 1,150 kips), the remaining plug length (from 0 to $6D$) had no measurable influence on pile performance.

6.2 Recommendations for Future Work

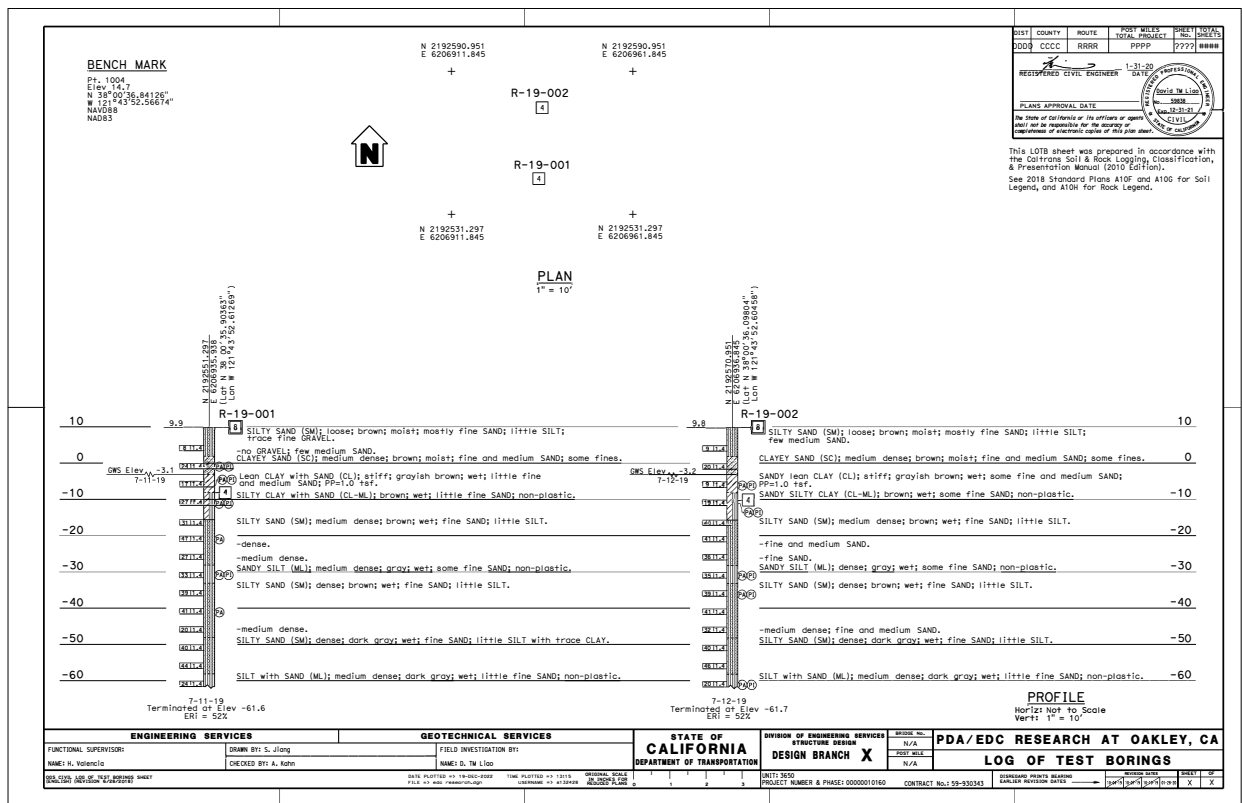
Based on the findings and limitations of this study, the following future work is recommended. First, load tests should be carried to geotechnical failure to directly measure ultimate shaft and end-bearing resistance; given the measured conservatism of the design methods, reaction systems should be sized well beyond

design-based estimates, or alternative loading schemes should be considered to mobilize the base without oversized reaction frames. Second, future tests would benefit from instrumentation or loading schemes capable of separately measuring the soil plug resistance and the tip resistance. In this study, separation of these components was attempted through inverse analyses employing the plugged and unplugged idealizations; however, because the pile tip and soil plug were only marginally mobilized under the applied loads, both idealizations reproduced the measured strain profiles equally well and could not be distinguished. Independent measurement of the plug and tip contributions would resolve this non-uniqueness directly. Third, testing of piles completed through concrete placement (Stages D–E) would quantify how the composite section and concrete–plug interaction modify the load transfer measured here at Stage C. Fourth, the finite-element framework should be extended with a layered soil profile and a constitutive model capable of representing state-dependent stiffness and shear localization, and ideally with explicit simulation of the driving process, to reduce the reliance on effective calibrated parameters. Finally, comparable instrumented tests at additional sites and soil conditions would establish whether the degree of conservatism observed here is systematic, which would support the development of updated resistance factors or site-specific design guidance for CISS piles.

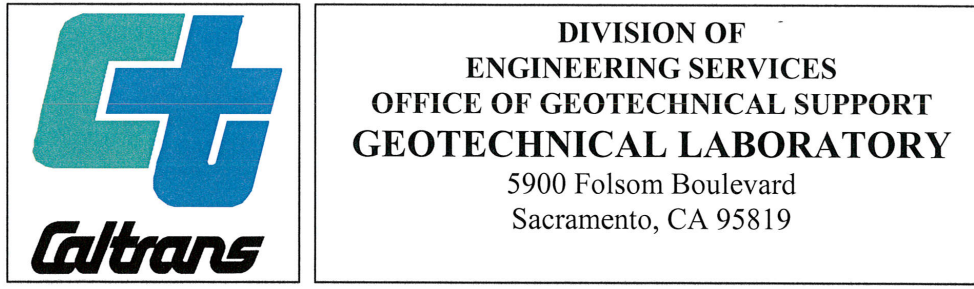
References

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Appendix A Log of Test Borings



Appendix B Boring Sample Report



Date: 9/9/2019

To: TM Liao / GDPP

From: Lilibeth C. Purta / (916) 227-5239

RE: Laboratory Test Report -- EA: 59-930342
Project: 0000001016
GL 19-045

Final test results.

Note: All remaining test specimens will be disposed of in 30 calendar days from the release date of the final test results.





CALIFORNIA DEPARTMENT OF TRANSPORTATION
GEOTECHNICAL LABORATORY

GL TRACKING NO : 19-045
Dist - EA: 59-930343
Report Date: September 9, 2019
Page: 1/1

CLASSIFICATION TEST SUMMARY

SAMPLE ID	% FINER THAN														ATTERBERG LIMITS		AS RECEIVED		Gs			
	3"	2 1/2"	2"	1 1/2"	1"	3/4"	1/2"	3/8"	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200	5µ	1µ	LL		PI	Yd (pcf)	%m
R-19-001_11											100	97	86	59	43	23	11	27	7		17.1	
R-19-001_16											100	99	98	96	88	32	15	39	18		31.5	
R-19-001_21											100	99	99	96	76	22	9	29	7		30.7	
R-19-001_31												100	81	27	14	6	1				33.4	
R-19-001_41													100	89	57	33	10	31	7		43.8	
R-19-001_51													100	40	20	11	5				35.8	
R-19-001_71													100	98	81	37	17	39	12		51.0	
R-19-002_16											100	99	99	96	90	32	14	40	18		30.5	
R-19-002_21											100	99	99	95	61	17	8	30	8		32.9	
R-19-002_41													100	95	53	28	13	29	4		40.4	
R-19-002_46													100	69	20	10	7	34	11		42.3	
R-19-002_71													100	97	74	35	17	43	17		52.8	

9/9/2019

PRESENTATION OF SITE INVESTIGATION RESULTS

CISS Pile Soil Plug

Prepared for:

UC Berkeley

ConeTec Job No: 25-56-29464

Project Start Date: 2025-04-28

Project End Date: 2025-04-28

Release Date: 2025-04-30

Report prepared by:

ConeTec, Inc.

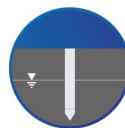
506 De Carlo Ave, Richmond, CA 94801

Tel: (510) 357-3677

ConeTecCA@conetec.com

www.conetec.com

www.conetecdataservices.com



ABOUT THIS REPORT

The attached report presents the findings of the site investigation program.

At the request of: UC Berkeley.

Conducted by: ConeTec, Inc.

Please be advised that this report, along with all associated data, is subject to the Third-Party Disclaimer and the Client Disclaimer contained in the 'Limitations' section of this report. For further reference, please consult the list of attached documents following the main body of the report.

PROJECT	
Client Name	UC Berkeley
Project Name	CISS Pile Soil Plug
Test Types	CPTu
ConeTec Project Number	25-56-29464
Additional Comments	None

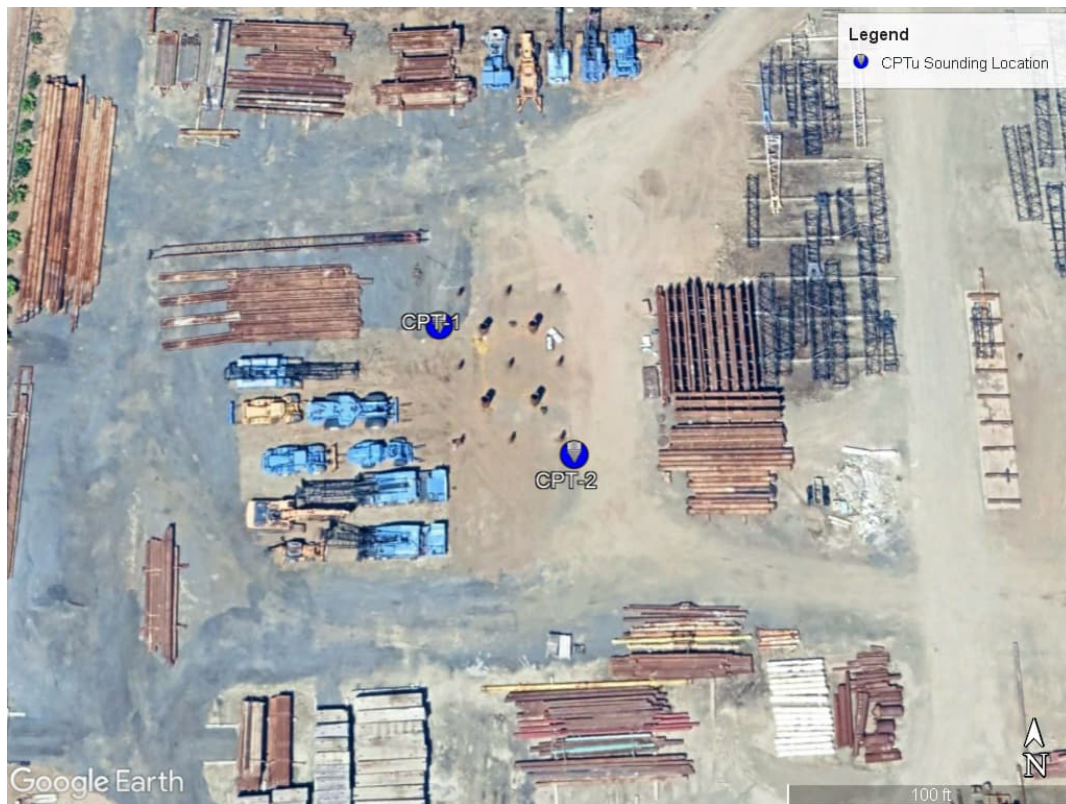
CONTENTS

The following are included in the body of the report:

- Site Map
- Limitations and Closure
- Project Information
- Test Summaries and Plots
- Supporting Documents and Materials



SITE MAP



All locations are approximate unless otherwise stated in the body of the report.

ConeTec Job Number: 25-56-29464

Client: UC Berkeley

Project: CISS Pile Soil Plug

Date: 2025-04-30



LIMITATIONS

Third-Party Disclaimer

'Report' refers to this document titled: CISS Pile Soil Plug

The Report was prepared by ConeTec for: UC Berkeley

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Client Disclaimer

ConeTec was retained by: UC Berkeley

'Report' refers to this document titled: CISS Pile Soil Plug

ConeTec was retained to collect and provide the raw data ('Data') which is included in the Report.

ConeTec has collected and reported the Data in accordance with current industry standards. No other warranties, either expressed or implied, with respect to the Data is made by ConeTec. To fully understand the Data included in the Report, reference must be made to the supporting documents and other sources referenced in the Report in their entirety. Other than the Data, the contents of the Report, including any Interpretations, should not be relied upon in any fashion without independent verification. ConeTec is in no way responsible for any loss, damage or expense resulting from the use of, and/or reliance on, such material by any party.

Closure

Thank you for the opportunity to contribute to this project. The equipment utilized, as well as the field procedures followed, fully complied with currently accepted best practice standards.

Report prepared by: Alan Sweeney, Jesse Martinez



PROJECT INFORMATION

Rig Utilized		
Description	Deployment System	Test Type
C02-015 CPT Truck Rig	Twin mounted cylinders	CPTu

Coordinates			
Test Type	Number of Locations	GPS Collection Method	EPSG Number
CPTu	2	Consumer Grade GPS	32610 (WGS84 / UTM Zone 10 North)

Piezocones Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm ²)	Sleeve Area (cm ²)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (bar)
EC1061:T1500F15U35	1061	15	225	1500	15	35

Cone Penetration Test (CPTu)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 Meters. This has been accounted for in the CPT data files.
Additional Comments	None

Calculated Geotechnical Parameters	
Additional information	The Normalized Soil Behavior Type Chart based on Q_{tn} (SBT Q_{tn}) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPTu parameters have been generated and are provided in Excel format files in the release folder. The CPTu parameter calculations are based on values of corrected tip resistance (q_t) sleeve friction (f_s) and pore pressure (u_2). Effective stresses are calculated based on unit weights that have been assigned to the individual soil behavior type zones and the assumed equilibrium pore pressure profile. Soils were classified as either drained or undrained based on the Q_{tn} Normalized Soil Behavior Type Chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures (zone 4).

REPORT APPENDICES

The appendices listed below are included in the report:

- **Cone Penetration Test (CPTu) Summary and Standard CPTu Plots**
- **Advanced Cone Penetration Test Plots with I_c , $S_u(Nkt)$, Φ , and $N1(60)I_c$**
- **Soil Behavior Type (SBT) Scatter Plots**
- **Pore Pressure Dissipation Test (PPDT) Summary and PPDT Plots**
- **Supplementary Documents and Materials**

Cone Penetration Test (CPTu) Summary and Standard CPTu Plots

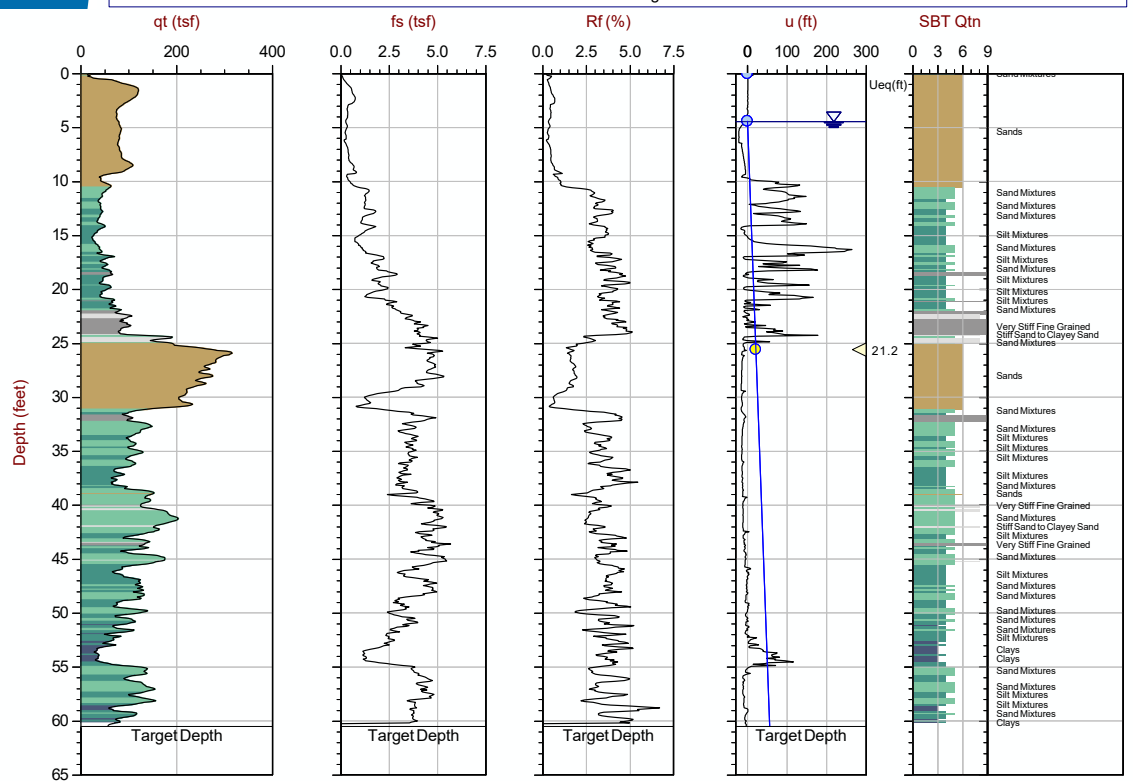




Job No: 25-56-29464
Client: UC Berkeley
Project: CISS Pile Soil Plug
Start Date: 2025-04-28
End Date: 2025-04-28

CONE PENETRATION TEST SUMMARY									
Sounding ID	File Name	Date	Cone	Cone Area (cm ²)	Assumed Phreatic Surface ¹ (ft)	Final Depth (ft)	Northing ² (m)	Easting ² (m)	Surface Elevation ³ (ft)
CPT-1	25-56-29464_CP01	2025-04-28	1061:T1500F15U35	15	4.4	60.53	4207671	611346	10
CPT-2	25-56-29464_CP02	2025-04-28	1061:T1500F15U35	15	4.4	60.53	4207656	611362	10
Totals	2 Soundings					121.06			

1. The assumed phreatic surface was based off the shallowest pore pressure dissipation tests performed within or nearest the sounding. Hydrostatic conditions were assumed for the calculated parameters.
2. The coordinates were collected using consumer grade GPS. EPSG number: 32610 (WGS84 / UTM Zone 10 North).
3. Elevations are referenced to the ground surface and were acquired from the Google Earth Elevation for the recorded coordinates.



Max Depth: 18.450 m / 60.53 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 25-56-29464 CP01.COR
Unit Wt: SBTQtn(PKR2009)

SBT: Robertson, 2009 and 2010
Coords: (UTM Zone 10 North) N: 4207671m E: 611346m

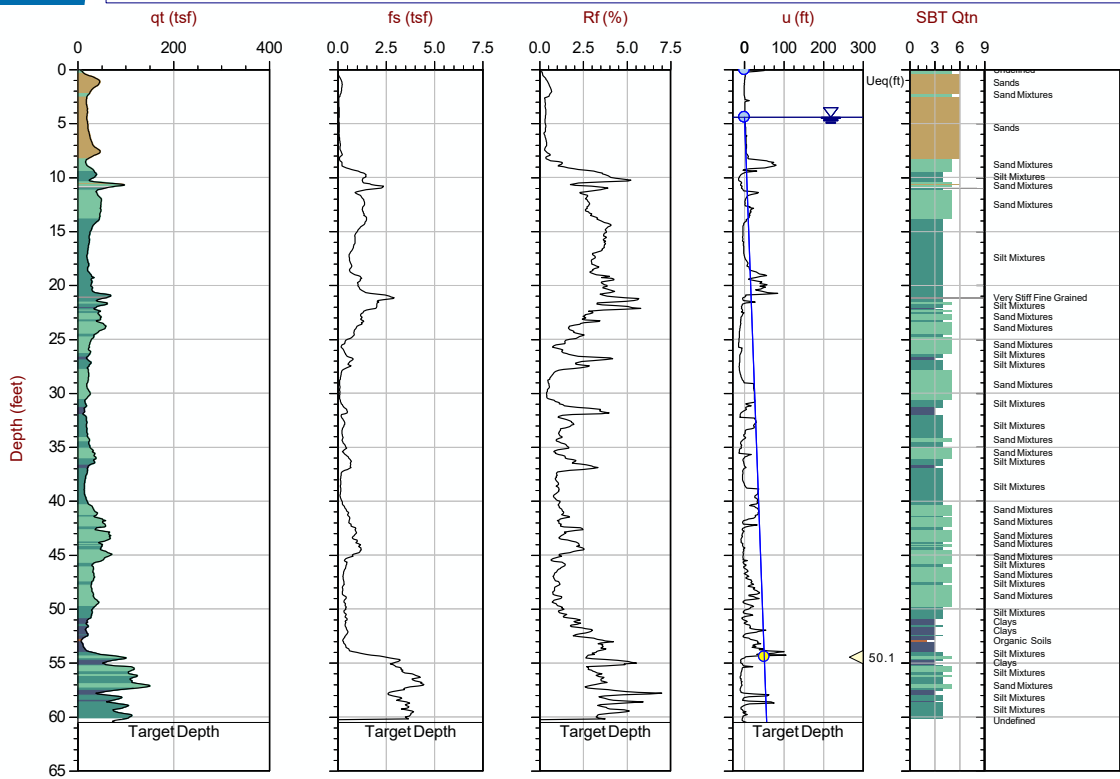
Overplot Item: ● Ueq ○ Assumed Ueq ◀ Dissipation, Ueq achieved ◄ Dissipation, Ueq not achieved ◈ Dissipation, Ueq assumed — Hydrostatic Line



UC Berkeley

Job No: 25-56-29464
Date: 2025-04-28 09:44
Site: CISS Pile Soil Plug

Sounding: CPT-2
Cone: 1061:T1500F15U35 Area=15 cm²



**Advanced Cone Penetration Test Plots with I_c , $S_u(N_{kt})$,
 Φ , and $N1(60)I_c$**

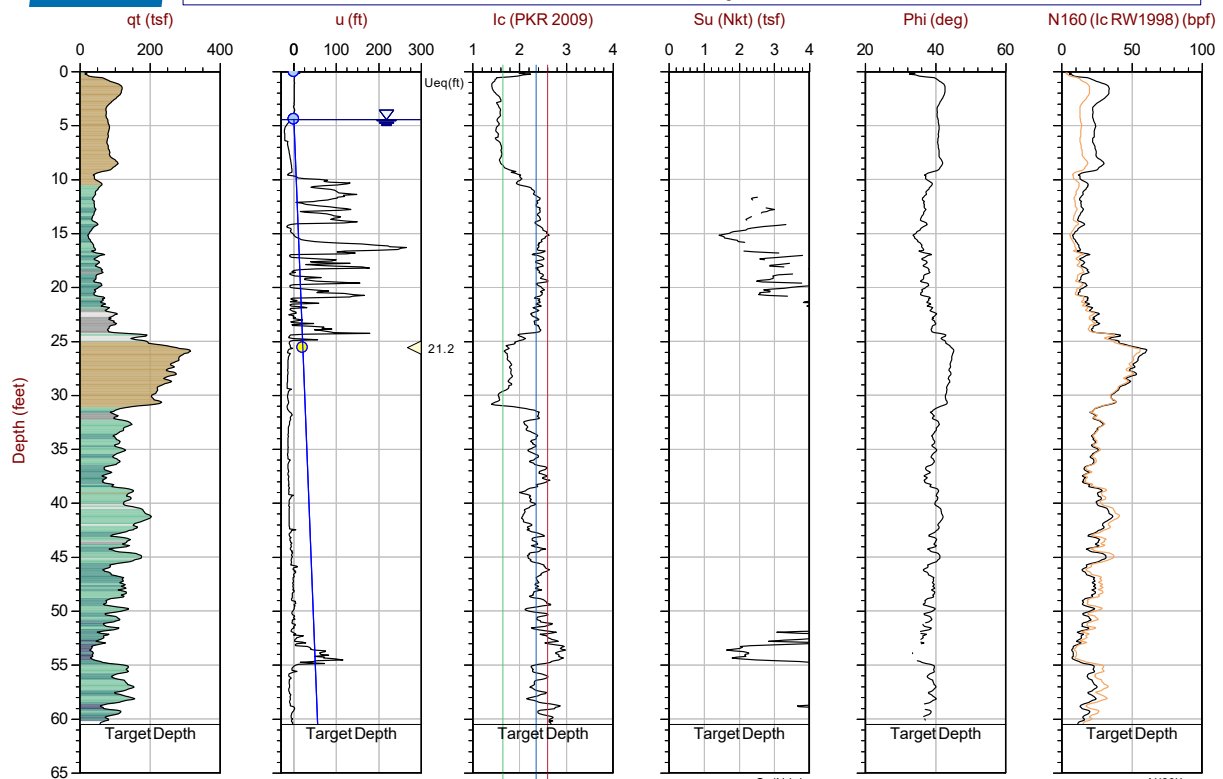




UC Berkeley

Job No: 25-56-29464
Date: 2025-04-28 08:37
Site: CISS Pile Soil Plug

Sounding: CPT-1
Cone: 1061:T1500F15U35 Area=15 cm²



Max Depth: 18.450 m / 60.53 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point
File: 25-56-29464_CP01.COR
Unit Wt: SBTQn(PKR2009)
Su Nkt/Ndu: 15.0 / 6.0
SBT: Robertson, 2009 and 2010
Coords: (UTM Zone 10 North) N: 4207671m E: 611346m

Overplot Item: ● Ueq ● Assumed Ueq < Dissipation, Ueq achieved < Dissipation, Ueq not achieved < Dissipation, Ueq assumed — Hydrostatic Line

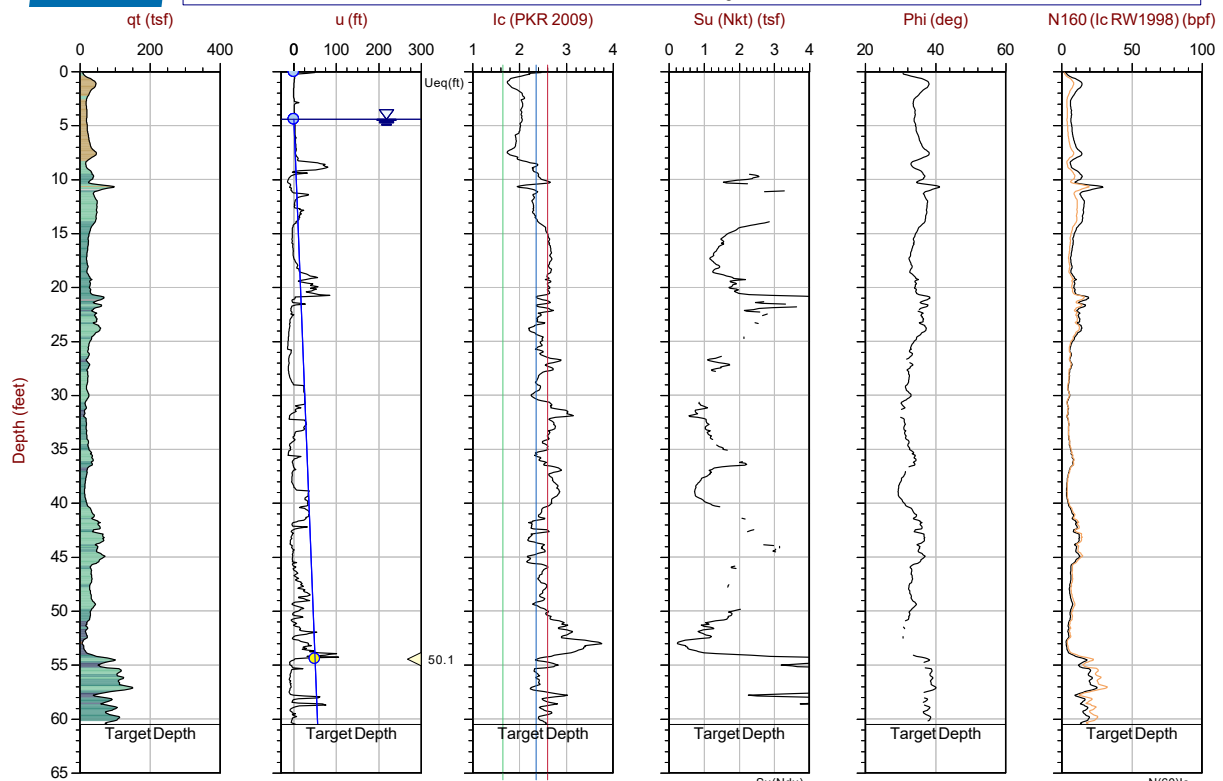
The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.



UC Berkeley

Job No: 25-56-29464
Date: 2025-04-28 09:44
Site: CISS Pile Soil Plug

Sounding: CPT-2
Cone: 1061:T1500F15U35 Area=15 cm²



Max Depth: 18.450 m / 60.53 ft File: 25-56-29464_CP02.COR SBT: Robertson, 2009 and 2010
Depth Inc: 0.025 m / 0.082 ft Unit Wt: SBTQn(PKR2009) Coords: (UTM Zone 10 North) N: 4207656m E: 611362m
Avg Int: Every Point Su Nkt/Ndu: 15.0 / 6.0

Overplot Item: ● Ueq ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved ◁ Dissipation, Ueq assumed — Hydrostatic Line

The reported coordinates were acquired from consumer grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

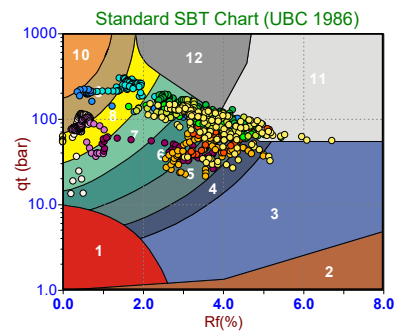
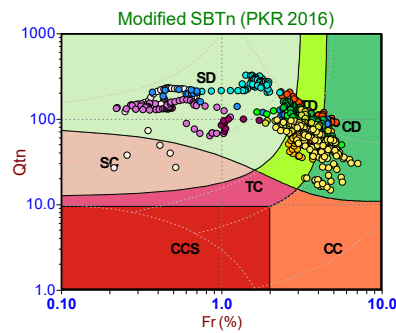
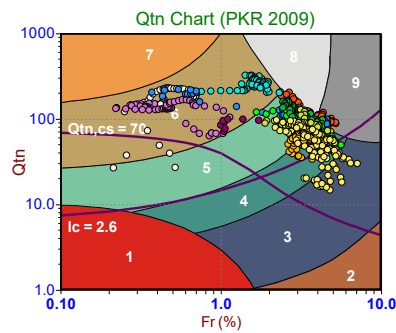
Soil Behavior Type (SBT) Scatter Plots



UC Berkeley

Job No: 25-56-29464
Date: 2025-04-28 08:37
Site: CISS Pile Soil Plug

Sounding: CPT-1
Cone: 1061:T1500F15U35 Area=15 cm²



- Depth Ranges
- >0.0 to 5.0 ft
 - >5.0 to 10.0 ft
 - >10.0 to 15.0 ft
 - >15.0 to 20.0 ft
 - >20.0 to 25.0 ft
 - >25.0 to 30.0 ft
 - >30.0 to 35.0 ft
 - >35.0 to 40.0 ft
 - >40.0 to 45.0 ft
 - >45.0 to 50.0 ft
 - >50.0 ft

- Legend
- Sensitive, Fine Grained
 - Organic Soils
 - Clays
 - Silt Mixtures
 - Sand Mixtures
 - Sands
 - Gravelly Sand to Sand
 - Stiff Sand to Clayey Sand
 - Very Stiff Fine Grained

- Legend
- CCS (Cont. sensitive clay like)
 - CC (Cont. clay like)
 - TC (Cont. transitional)
 - SC (Cont. sand like)
 - CD (Dil. clay like)
 - TD (Dil. transitional)
 - SD (Dil. sand like)

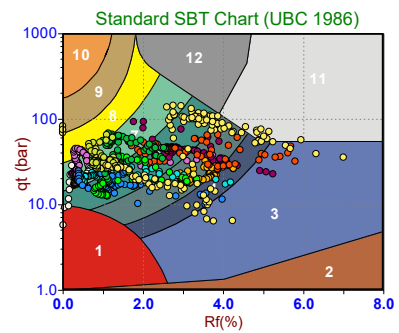
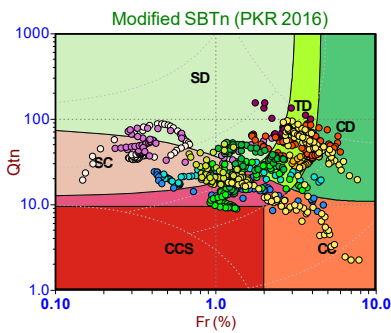
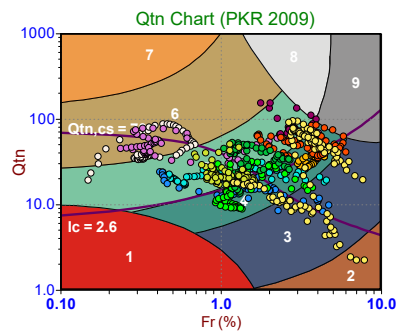
- Legend
- Sensitive Fines
 - Organic Soil
 - Clay
 - Silty Clay
 - Clayey Silt
 - Silt
 - Sandy Silt
 - Silty Sand/Sand
 - Sand
 - Gravelly Sand
 - Stiff Fine Grained
 - Cemented Sand



UC Berkeley

Job No: 25-56-29464
Date: 2025-04-28 09:44
Site: CISS Pile Soil Plug

Sounding: CPT-2
Cone: 1061:T1500F15U35 Area=15 cm²



- Depth Ranges
- >0.0 to 5.0 ft
 - >5.0 to 10.0 ft
 - >10.0 to 15.0 ft
 - >15.0 to 20.0 ft
 - >20.0 to 25.0 ft
 - >25.0 to 30.0 ft
 - >30.0 to 35.0 ft
 - >35.0 to 40.0 ft
 - >40.0 to 45.0 ft
 - >45.0 to 50.0 ft
 - >50.0 ft

- Legend
- Sensitive, Fine Grained
 - Organic Soils
 - Clays
 - Silt Mixtures
 - Sand Mixtures
 - Sands
 - Gravelly Sand to Sand
 - Stiff Sand to Clayey Sand
 - Very Stiff Fine Grained

- Legend
- CCS (Cont. sensitive clay like)
 - CC (Cont. clay like)
 - TC (Cont. transitional)
 - SC (Cont. sand like)
 - CD (Dil. clay like)
 - TD (Dil. transitional)
 - SD (Dil. sand like)

- Legend
- Sensitive Fines
 - Organic Soil
 - Clay
 - Silty Clay
 - Clayey Silt
 - Silt
 - Sandy Silt
 - Silty Sand/Sand
 - Sand
 - Gravelly Sand
 - Stiff Fine Grained
 - Cemented Sand

Pore Pressure Dissipation Test (PPD) Summary and PPD Plots





Job No: 25-56-29464
Client: UC Berkeley
Project: CISS Pile Soil Plug
Start Date: 2025-04-28
End Date: 2025-04-28

CPT_u PORE PRESSURE DISSIPATION SUMMARY

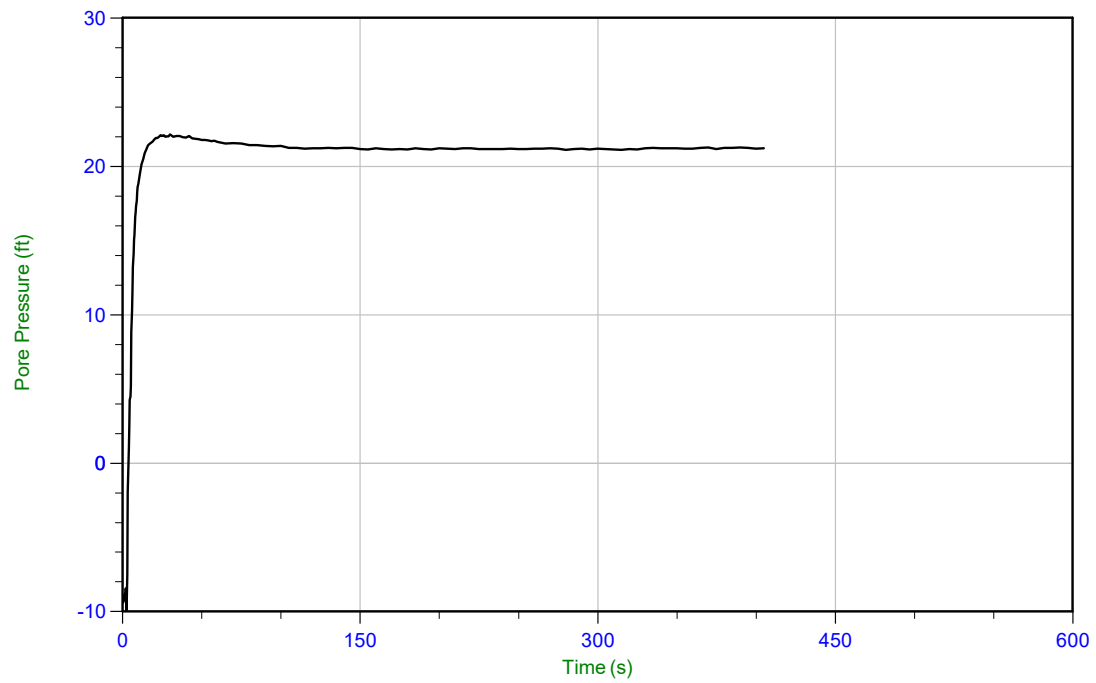
Sounding ID	File Name	Cone Area (cm ²)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure U _{eq} (ft)	Calculated Phreatic Surface (ft)
CPT-1	25-56-29464_CP01	15	405	25.59	21.2	4.4
CPT-2	25-56-29464_CP02	15	325	54.46	50.1	4.4
Totals			12 min			



UC Berkeley

Job No: 25-56-29464
Date: 2025-04-28 08:37
Site: CISS Pile Soil Plug

Sounding: CPT-1
Cone: 1061:T1500F15U35 Area=15 cm²



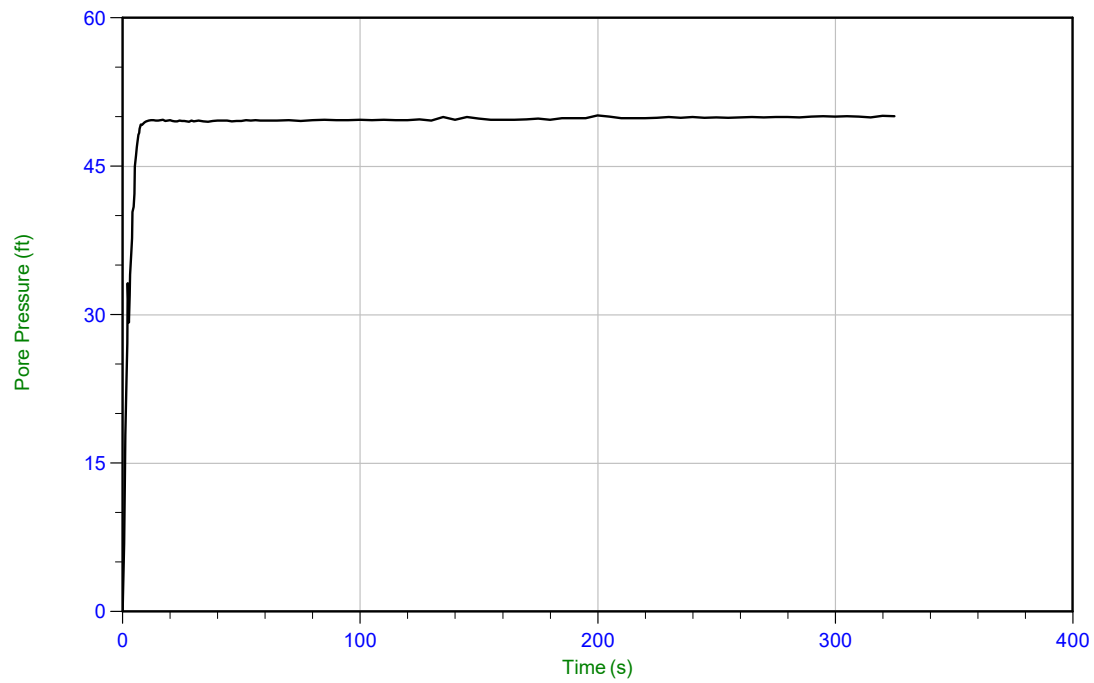
Trace Summary: Filename: 25-56-29464_CP01.PPF2 u Min: -10.0 ft WT: 1.4 m / 4.4 ft
 Depth: 7.800 m / 25.590 ft u Max: 22.1 ft Ueq: 21.2 ft
 Duration: 405.0 s u Final: 21.2 ft



UC Berkeley

Job No: 25-56-29464
Date: 2025-04-28 09:44
Site: CISS Pile Soil Plug

Sounding: CPT-2
Cone: 1061:T1500F15U35 Area=15 cm²



Trace Summary: Filename: 25-56-29464_CP02.PPF2 u Min: -0.4 ft WT: 1.3 m / 4.4 ft
 Depth: 16.600 m / 54.461 ft u Max: 50.1 ft Ueq: 50.1 ft
 Duration: 325.0 s u Final: 50.1 ft

Supporting Documents and Materials



SUPPORTING DOCUMENTS AND MATERIALS

The documents and materials listed below are included in the report:

- **Methodology Statements**
- **Cone Penetration Digital File Formats**
- **Description of Methods for Calculated CPTu Geotechnical Parameters**
- **Description of Methods for Calculated DMT Geotechnical Parameters**
- **Calibration Records**

Methodology Statements

METHODOLOGY STATEMENTS



CONE PENETRATION TEST (CPTu) - eSeries

Cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd., a subsidiary of ConeTec.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and two geophone sensors for recording seismic signals. All signals are amplified and measured with minimum sixteen-bit resolution down hole within the cone body, and the signals are sent to the surface using a high bandwidth, error corrected digital interface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm² and 15 cm² tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table. The 15 cm² penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm² piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 millimeters diameter over a length of 32 millimeters with tapered leading and trailing edges) located at a distance of 585 millimeters above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u₂" position (ASTM Type 2). The filter is six millimeters thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meets or exceeds those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.



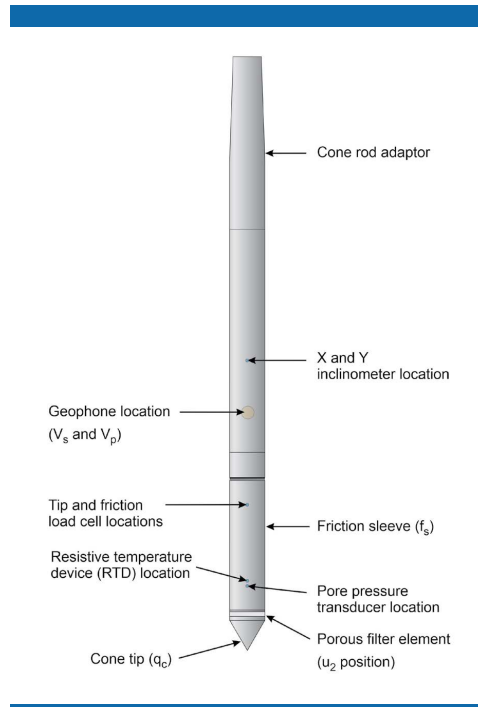


Figure CPTu. Piezocone Penetrometer (15 cm²)

The ConeTec data acquisition system consists of a Windows based computer, signal interface box, and power supply. The signal interface combines depth increment signals, seismic trigger signals and the downhole digital data. This combined data is then sent to the Windows based computer for collection and presentation. The data is recorded at fixed depth increments using a depth encoder that is either portable or integrated into the rig. The typical recording interval is 2.5 centimeters; custom recording intervals are possible.

The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance (q_c)
- Sleeve friction (f_s)
- Dynamic pore pressure (u)
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPTu operating procedures which are in general accordance with the current [ASTM D5778](#) standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of two centimeters per second, within acceptable tolerances. Typically one meter length rods with an outer diameter of 1.5 inches are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil under vacuum pressure prior to use
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with [ASTM](#) standards

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by [Robertson, P.K., 2010](#). The Soil Behavior Type (SBT) classification chart developed by [Robertson, P.K., 2010](#) is presented in [Figure SBT](#). It should be noted that it is not always possible to accurately identify a soil behavior type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behavior type.

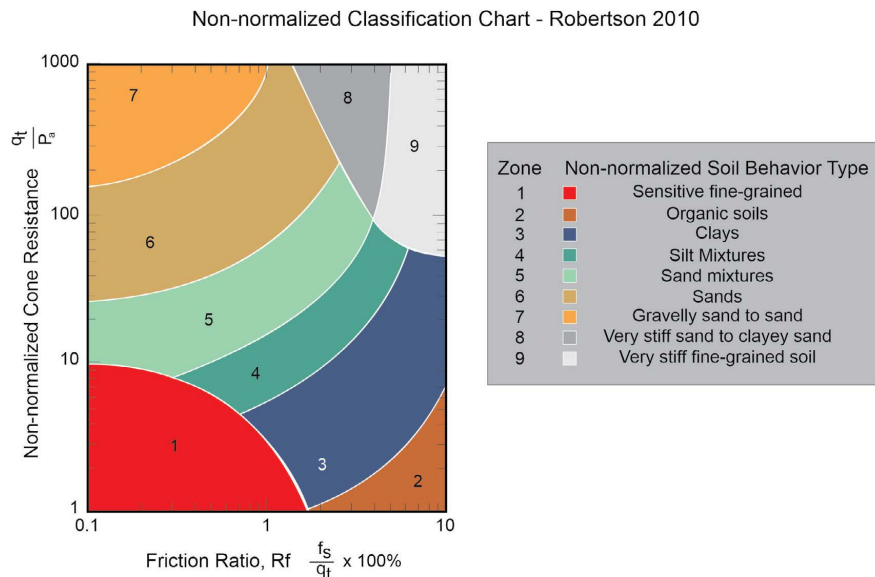


Figure SBT. Non-Normalized Soil Behavior Type Classification Chart (SBT)



The recorded tip resistance (q_c) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance (q_t) according to the following expression presented in [Robertson et al. \(1986\)](#):

$$q_t = q_c + (1-a) \cdot u_2$$

where: q_t is the corrected tip resistance

q_c is the recorded tip resistance

u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)

a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction (f_s) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio (R_f) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

For additional information on CPTu interpretations and calculated geotechnical parameters, refer to [Robertson et al. \(1986\)](#), [Lunne et al. \(1997\)](#), [Robertson \(2009\)](#), [Mayne \(2013, 2014\)](#) and [Mayne and Peuchen \(2012\)](#).

REFERENCES

ASTM D5778-20, 2020, "Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", ASTM International, West Conshohocken, PA. DOI: [10.1520/D5778-20](#).

Lunne, T., Robertson, P.K. and Powell, J. J. M., 1997, "Cone Penetration Testing in Geotechnical Practice", Blackie Academic and Professional.

Mayne, P.W., 2013, "Evaluating yield stress of soils from laboratory consolidation and in-situ cone penetration tests", Sound Geotechnical Research to Practice (Holtz Volume) GSP 230, ASCE, Reston/VA: 406-420. DOI: [10.1061/9780784412770.027](#).

Mayne, P.W. and Peuchen, J., 2012, "Unit weight trends with cone resistance in soft to firm clays", Geotechnical and Geophysical Site Characterization 4, Vol. 1 (Proc. ISC-4, Pernambuco), CRC Press, London: 903-910.

Mayne, P.W., 2014, "Interpretation of geotechnical parameters from seismic piezocone tests", CPT'14 Keynote Address, Las Vegas, NV, May 2014.

Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, "Use of Piezometer Cone Data", Proceedings of In Situ 86, ASCE Specialty Conference, Blacksburg, Virginia.

Robertson, P.K., 2009, "Interpretation of cone penetration tests – a unified approach", Canadian Geotechnical Journal, Volume 46: 1337-1355. DOI: [10.1139/T09-065](#).

Robertson, P.K., 2010. Soil behavior type from the CPT: an update. 2nd International Symposium on Cone Penetration Testing, CPT'10, Huntington Beach, CA, USA





PORE PRESSURE DISSIPATION TEST

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in [Figure PPD-1](#). For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure (u) with time (t).

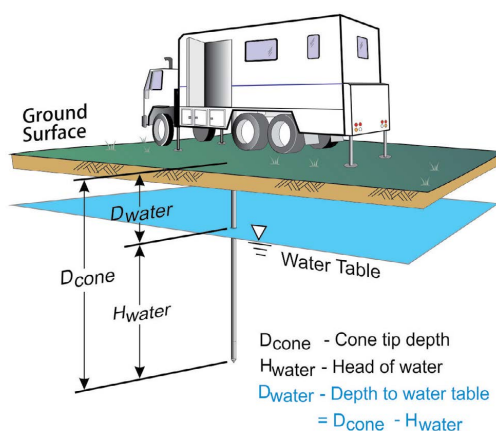


Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behavior.

The typical shapes of dissipation curves shown in [Figure PPD-2](#) are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

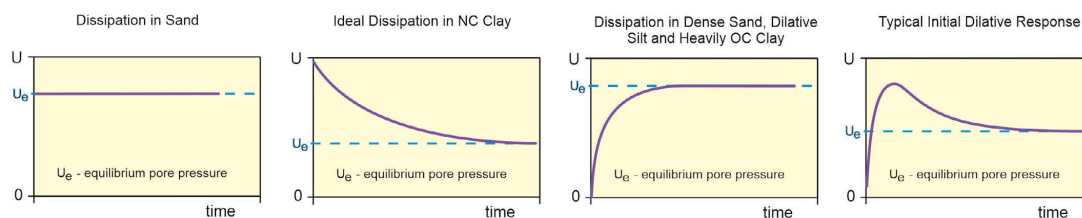


Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure (u_{eq}) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve in [Figure PPD-2](#).

Cone Penetration Digital File Formats





CONE PENETRATION DIGITAL FILE FORMATS - eSeries

CPT Data Files (COR Extension)

ConeTec CPT data files are stored in ASCII text files that are readable by almost any text editor. ConeTec file names start with the job number (which includes the two digit year number) an underscore as a separating character, followed by two letters based on the type of test and the sounding ID. The last character position is reserved for an identifier letter (such as b, c, d etc) used to uniquely distinguish multiple soundings at the same location. The CPT sounding file has the extension COR. As an example, for job number 21-02-00001 the first CPT sounding will have file name 21-02-00001_CP01.COR

The sounding (COR) file consists of the following components:

1. Two lines of header information
2. Data records
3. End of data marker
4. Units information

Header Lines

Line 1: Columns 1-6 may be blank or may indicate the version number of the recording software

Columns 7-21 contain the sounding Date and Time (Date is MM:DD:YY)

Columns 23-38 contain the sounding Operator

Columns 51-100 contain extended Job Location information

Line 2: Columns 1-16 contain the Job Location

Columns 17-32 contain the Cone ID

Columns 33-47 contain the sounding number

Columns 51-100 may contain extended sounding ID information

Data Records

The data records contain 4 or more columns of data in floating point format. A comma and spaces separate each data item:

Column 1: Sounding Depth (meters)

Column 2: Tip (q_c), recorded in units selected by the operator

Column 3: Sleeve (f_s), recorded in units selected by the operator

Column 4: Dynamic pore pressure (u), recorded in units selected by the operator

Column 5: Empty or may contain other requested data such as Gamma, Resistivity or UVIF data

End of Data Marker

After the last line of data there is a line containing an ASCII 26 (CTL-Z) character (small rectangular shaped character) followed by a newline (carriage return / line feed). This is used to mark the end of data.



Units Information

The last section of the file contains information about the units that were selected for the sounding. A separator bar makes up the first line. The second line contains the type of units used for depth, q_c , f_s and u . The third line contains the conversion values required for ConeTec's software to convert the recorded data to an internal set of base units (bar for q_c , bar for f_s and meters for u). Additional lines intended for internal ConeTec use may appear following the conversion values.

CPT Data Files (XLS Extension)

Excel format files of ConeTec CPT data are also generated from corresponding COR files. The XLS files have the same base file name as the COR file with a -BSC suffix. The information in the file is presented in table format and contains additional information about the sounding such as coordinate information, and tip net area ratio.

The BSCI suffix is given to XLS files which are enhanced versions of the BSC files and include the same data records in addition to inclination data collected for each sounding.

CPT Dissipation Files (XLS Extension)

Pore pressure dissipation files are provided in Excel format and contain each dissipation trace that exceeds a minimum duration (selected during post-processing) formatted column wise within the spreadsheet. The first column (Column A) contains the time in seconds and the second column (Column B) contains the time in minutes. Subsequent columns contain the dissipation trace data. The columns extend to the longest trace of the data set.

Detailed header information is provided at the top of the worksheet. The test depth in meters and feet, the number of points in the trace and the particular units are all presented at the top of each trace column.

CPT Dissipation files have the same naming convention as the CPT sounding files with a "-PPD" suffix.

Data Records

Each file will contain dissipation traces that exceed a minimum duration (selected during post-processing) in a particular column. The dissipation pore pressure values are typically recorded at varying time intervals throughout the trace; rapidly to start and increasing as the duration of the test lengthens. The test depth in meters and feet, the number of points in the trace and the trace number are identified at the top of each trace column.

Cone Type Designations

Cone ID	Cone Description	Tip Cross Sect. Area (cm ²)	Tip Capacity (bar)	Sleeve Area (cm ²)**	Sleeve Capacity (bar)	Pore Pressure Capacity (bar)
EC###	A15T1500F15U35	15	1500	225	15	35
EC###	A15T375F10U35	15	375	225	10	35
EC###	A10T1000F10U35	10	1000	150	10	35

refers to the Cone ID number

**Outer Cylindrical Area



Description of Methods for Calculated CPT Geotechnical Parameters

CALCULATED CPT GEOTECHNICAL PARAMETERS

A Detailed Description of the Methods Used in ConeTec's CPT Geotechnical Parameter Calculation and Plotting Software



Revision SZW-Rev 18

Revised February 10, 2023

Prepared by Jim Greig, M.A.Sc, P.Eng (BC, AB, ON)



Limitations

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates. For this project, ConeTec has provided site investigation services, prepared factual data reporting and produced geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

To understand the calculations that have been performed and to be able to reproduce the calculated parameters the user is directed to the basic descriptions for the methods in this document and the detailed descriptions and their associated limitations and appropriateness in the technical references cited for each parameter.

BETTER INFORMATION, **BETTER DECISIONS**

ConeTec's Calculated CPT Geotechnical Parameters as of February 10, 2023.

ConeTec's CPT parameter calculation and plotting routine provides a tabular output of geotechnical parameters based on current published CPT correlations and is subject to change to reflect the current state of practice. Due to drainage conditions and the basic assumptions and limitations of the correlations, not all geotechnical parameters provided are considered applicable for all soil types. The results are presented only as a guide for geotechnical use and should be carefully examined for consideration in any geotechnical design. Reference to current literature is strongly recommended. ConeTec does not warranty the correctness or the applicability of any of the geotechnical parameters calculated by the program and does not assume liability for any use of the results in any design or review. For verification purposes we recommend that representative hand calculations be done for any parameter that is critical for design purposes. The end user of the parameter output should also be fully aware of the techniques and the limitations of any method used by the program. The purpose of this document is to inform the user as to which methods were used and to direct the end user to the appropriate technical papers and/or publications for further reference.

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates.

The CPT calculations are based on values of tip resistance, sleeve friction and pore pressures considered at each data point or averaged over a user specified layer thickness (e.g., 0.20 m). Note that q_t is the tip resistance corrected for pore pressure effects and q_c is the recorded tip resistance. The corrected tip resistance (corrected using u_2 pore pressure values) is used for all calculations. Since all ConeTec cones have equal end area friction sleeves pore pressure corrections to sleeve friction, f_s , are not performed.

Corrected tip resistance: $q_t = q_c + (1-a) \cdot u_2$ (consistent units are required)

where: q_t is the corrected tip resistance

q_c is the recorded tip resistance

u_2 is the recorded dynamic pore pressure from behind the tip (u_2 position)

a is the Net Area Ratio for the cone (typically 0.80 for ConeTec cones)

The total stress calculations are based on soil unit weight values that have been assigned to the Soil Behavior Type (SBT) zones, from a user defined unit weight profile, by using a single uniform value throughout the profile, through unit weight estimation techniques described in various technical papers or from a combination of these methods. The parameter output files indicate the method(s) used.

Effective vertical overburden stresses are calculated using the total stress and equilibrium pore pressure (u_{eq} or u_0) values derived from an assumed hydrostatic distribution of pore pressures below the water table or from a user defined equilibrium pore pressure profile (typically obtained from CPT dissipation tests) or a combination of the two. For over water projects the stress effects of the column of water above the mudline are taken into account as is the appropriate unit weight of water. How this is done depends on where the instruments are zeroed (i.e. on deck or at the mudline). The parameter output files indicate the method(s) used.

A majority of parameter calculations are derived from or driven by results based on material types as determined by the various soil behavior type charts depicted in Figures 1 through 6. The parameter output files indicate the method(s) used.

The Soil Behavior Type classification chart shown in Figure 1 is the classic non-normalized SBT Chart developed at the University of British Columbia and reported in Robertson, Campanella, Gillespie and Greig (1986). Figure 2 shows the original normalized (linear method) SBTn chart developed by Robertson (1990). The Bq classification charts



shown in Figures 3a and 3b incorporate pore pressures into the SBT classification and are based on the methods described in Robertson (1990). Many of these charts have been summarized in Lunne, Robertson and Powell (1997). The Jefferies and Davies SBT chart shown in Figure 3c is based on the techniques discussed in Jefferies and Davies (1993) which introduced the concept of the Soil Behavior Type Index parameter, I_c . Take note that the I_c parameter developed by Robertson and Fear (1995) and Robertson and Wride (1998) is similar in concept but uses a slightly different calculation method than that defined by Jefferies and Davies (1993) as the latter incorporates pore pressure in their technique through the use of the B_q parameter. The normalized Q_{tn} SBT chart shown in Figure 4 is based on the work by Robertson (2009) utilizing a variable stress ratio exponent, n , for normalization based on a slightly modified redefinition and iterative approach for I_c . The boundary curves drawn on the chart are based on the work described in Robertson (2010).

Figure 5 shows a revised 1986 SBT Chart presented to CPT'10 by Robertson (2010b). It is known as the Updated non-normalized Soil Behavior Chart (also referred to as the Rev SBT Chart (PKR2010) in our output files). This chart was produced to be more in line with all post-1986 Robertson charts having the same 9 soil type zones, a $\log_{10}$ axis for friction ratio, R_f in this case, and a unitless tip resistance axis.

Figure 6 shows a revised behavior based chart by Robertson (2016) depicting contractive-dilative zones. As the zones represent material behavior rather than soil gradation ConeTec has chosen a set of zone colors that are less likely to be confused with material type colors from previous SBT charts. These colors differ from those used by Dr. Robertson. A green palette was selected for the dilative (desirable) side of the chart and a red palette for the contractive side of the chart.

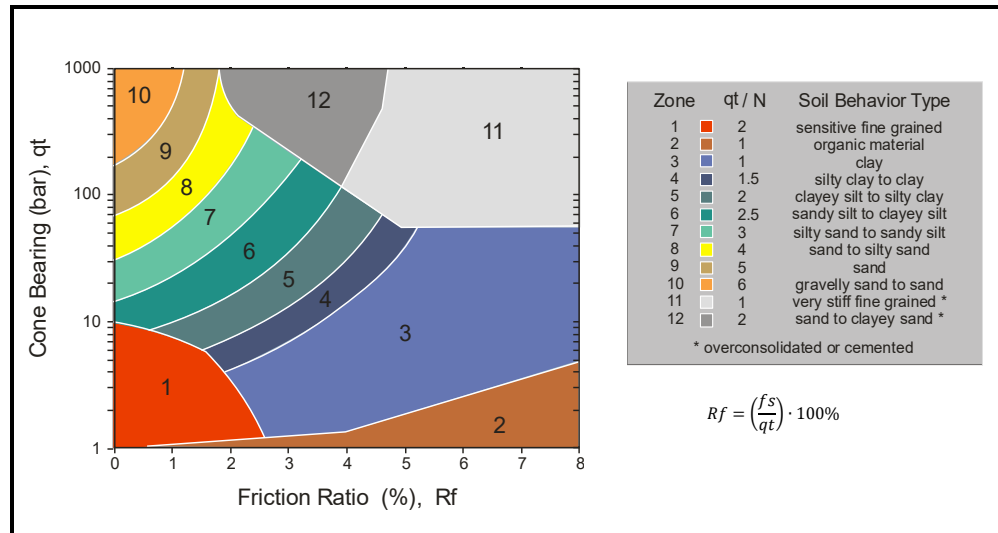


Figure 1. Non-normalized Soil Behavior Type Classification Chart (SBT)

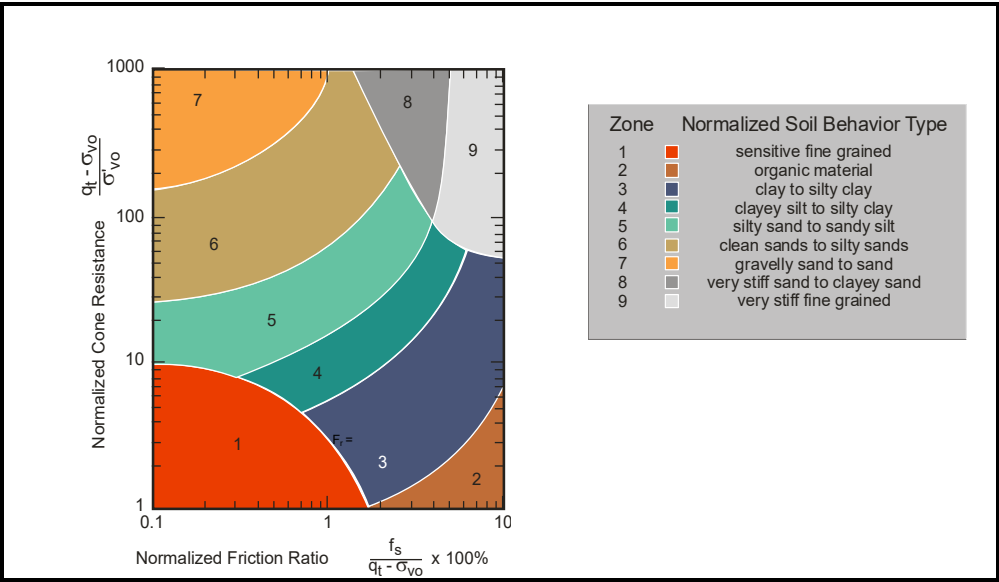


Figure 2. Normalized Soil Behavior Type Classification Chart (SBTn)

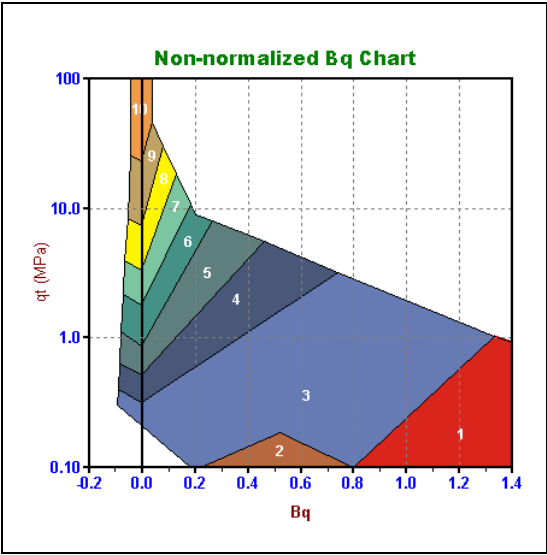


Figure 3a. Alternate Soil Behavior Type Chart (SBT Bq): qt - Bq



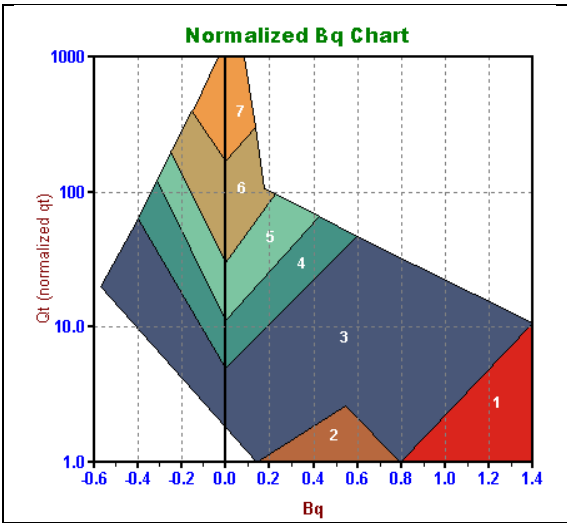


Figure 3b. Alternate Soil Behavior Type Charts (SBT Bqn): Q_t - B_q

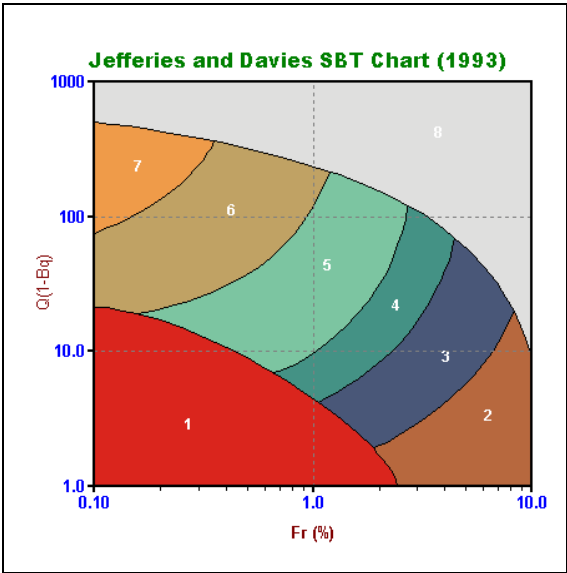


Figure 3c. Alternate Soil Behavior Type Charts: $Q(1-B_q)$ - F_r



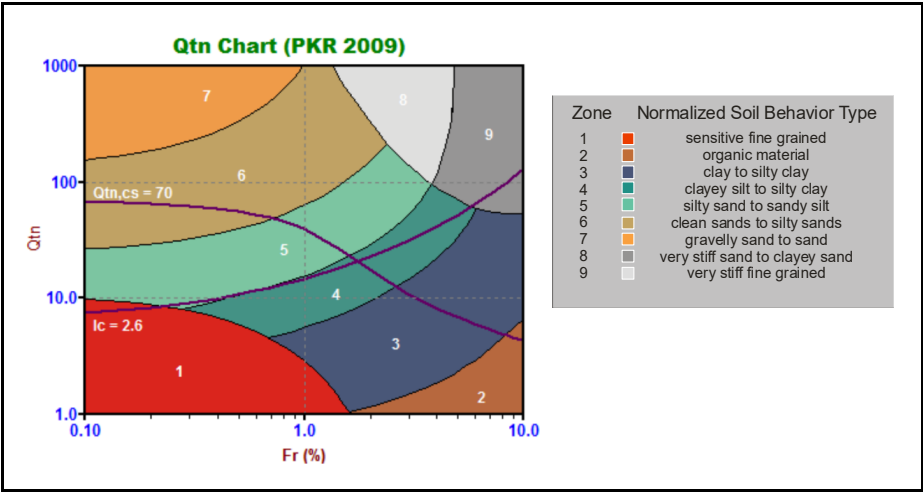


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Q_{tn})

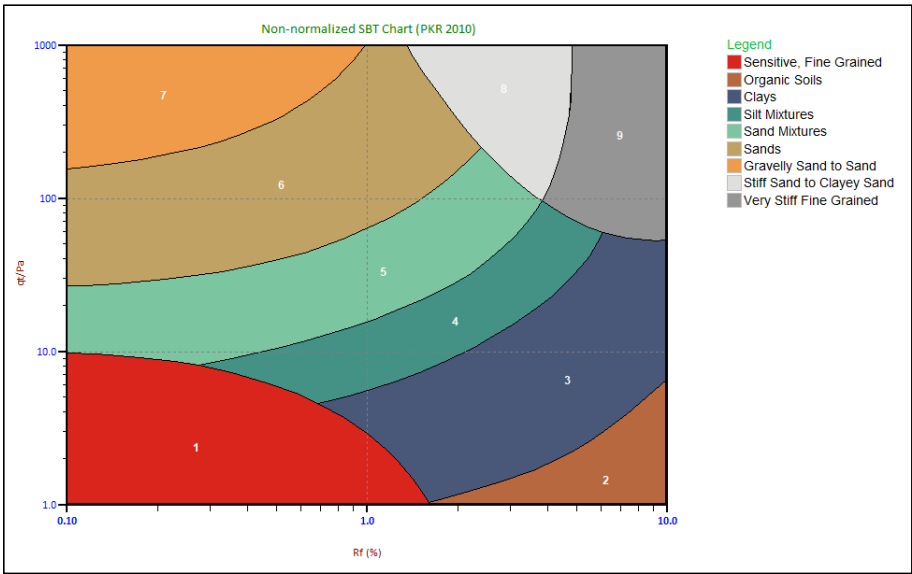


Figure 5. Non-normalized Soil Behavior Type Chart (2010)

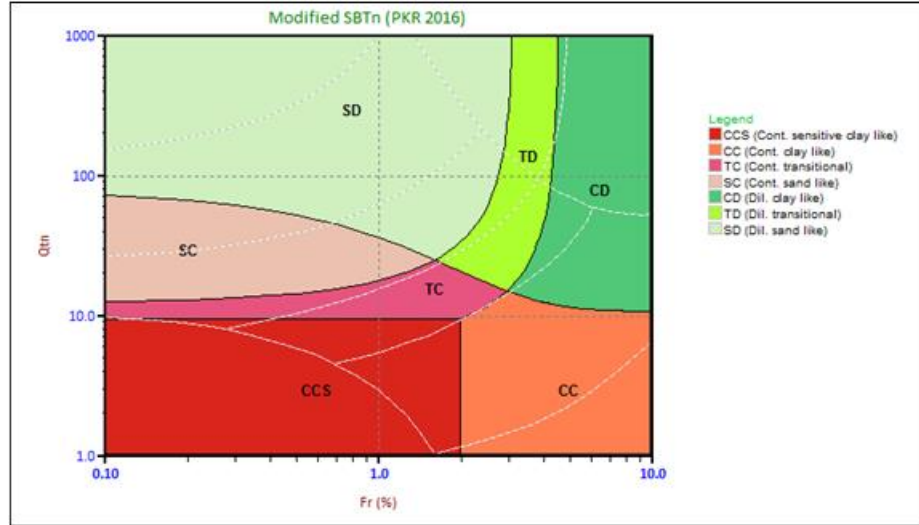


Figure 6. Modified SBTn Behavior Based Chart

Details regarding the geotechnical parameter calculations are provided in Tables 1a and 1b. The appropriate references cited are listed in Table 2. Non-liquefaction specific parameters are detailed in Table 1a and liquefaction specific parameters are detailed in Table 1b.

Where methods are based on charts or techniques that are too complex to describe in this summary, we recommend that the user refer to the cited material. Specific limitations for each method are described in the cited material.

Where the results of a calculation/correlation are deemed 'invalid' the value will be represented by the text strings "-9999", "-9999.0", the value 0.0 (Zero) or an empty cell. Invalid results will occur because of (and not limited to) one or a combination of:

1. Invalid or undefined CPT data (e.g., drilled out section or data gap).
2. Where the calculation method is inappropriate, for example, drained parameters in a material behaving in an undrained manner (and vice versa).
3. Where input values are beyond the range of the referenced charts or specified limitations of the correlation method.
4. Where pre-requisite or intermediate parameter calculations are invalid.

The parameters selected for output from the program are often specific to a particular project. As such, not all of the calculated parameters listed in Tables 1a and 1b may be included in the output files delivered with this report.

The output files are typically provided in Microsoft Excel XLS, XLSX or CSV format. The ConeTec software has several options for output depending on the number or types of calculated parameters desired or those specifically contracted for by the client. Each output file is named using the original file base name (from the .COR file) followed



by a three or four character indicator of the output set selected (e.g. BSC, TBL, NLI, NL2, IFI, IFI2, IFI3) and possibly followed by an operator selected suffix identifying the characteristics of the particular calculation run.

Table 1a. CPT Parameter Calculation Methods – Non liquefaction Parameters

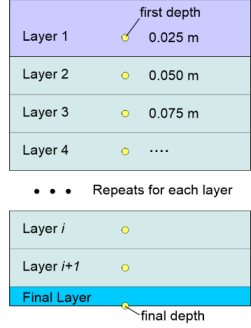
Reference Notes: CK* - Common Knowledge, U* - Unpublished

Calculated Parameter	Description	Equation	Ref
Depth	Mid Layer Depth <i>(where calculations are done at each point then Mid Layer Depth = Recorded Depth)</i>	$[Depth (Layer Top) + Depth (Layer Bottom)] / 2.0$	CK*
Elevation	Elevation of Mid Layer is based on the sounding collar elevation supplied by the client or through a site survey In Sweden a variation of elevation is used where the elevation increases with depth. We refer to this as inverse elevation.	Elevation = Collar Elevation – Depth InverseElevation = Collar Elevation + Depth	CK* N/A
Avg qc	Averaged recorded tip value (q_c)	$Avgqc = \frac{1}{n} \sum_{i=1}^n q_c$ <i>n=1 when calculations are done at each point</i>	CK*
Avg qt	Averaged corrected tip (q_t) where: $q_t = q_c + (1 - \alpha) \cdot u_2$ Averaged q_t is not calculated using the average q_c and averaged u values. Averaged q_t is based on the average of the q_t values calculated at each data point.	$Avgqt = \frac{1}{n} \sum_{i=1}^n q_t$ <i>n=1 when calculations are done at each point</i>	1
Avg fs	Averaged sleeve friction (f_s) No pore pressure corrections are applied to f_s .	$Avgfs = \frac{1}{n} \sum_{i=1}^n f_s$ <i>n=1 when calculations are done at each point</i>	CK*
Avg Rf	Averaged friction ratio (R_f) where friction ratio is defined as: $R_f = 100\% \cdot \frac{f_s}{q_t}$	$AvgRf = 100\% \cdot \frac{Avgfs}{Avgqt}$ <i>not an average of individual R_f values</i>	CK*
Avg u	Averaged dynamic pore pressure (u)	$Avgu = \frac{1}{n} \sum_{i=1}^n u_i$ <i>n=1 when calculations are done at each point</i>	CK*
Avg Res	Averaged Resistivity (this data is not always available since it is a specialized test requiring an additional module)	$AvgRes = \frac{1}{n} \sum_{i=1}^n Resistivity_i$ <i>n=1 when calculations are done at each point</i>	CK*
Avg UVIF	Averaged UVIF ultra-violet induced fluorescence (this data is not always available since it is a specialized test requiring an additional module)	$AvgUVIF = \frac{1}{n} \sum_{i=1}^n UVIF_i$ <i>n=1 when calculations are done at each point</i>	CK*
Avg Temp	Averaged Temperature (this data is not always available)	$AvgTemp = \frac{1}{n} \sum_{i=1}^n Temperature_i$ <i>n=1 when calculations are done at each point</i>	CK*
Avg Gamma	Averaged Gamma Counts (this data is not always available since it is a specialized test requiring an additional module)	$AvgGamma = \frac{1}{n} \sum_{i=1}^n Gamma_i$ <i>n=1 when calculations are done at each point</i>	CK*
SBT	Soil Behavior Type as defined by Robertson et al 1986 (often referred to as Robertson and Campanella, 1986)	See Figure 1	1, 5
SBTn	Normalized Soil Behavior Type as defined by Robertson 1990 (linear normalization using Q_t , now referred to as Q_{t1})	See Figure 2	2, 5



Calculated Parameter	Description	Equation	Ref
SBT-Bq	Non-normalized Soil Behavior type based on non-normalized tip resistance and the B_q parameter	See Figure 3a	1, 2, 5
SBT-Bqn	Normalized Soil Behavior type based on normalized tip resistance (Q_t , now called Q_{tn}) and the B_q parameter	See Figure 3b	2, 5
SBT-JandD	Soil Behavior Type as defined by Jeffries and Davies	See Figure 3c	7
SBT Qtn	Soil Behavior Type as defined by Robertson (2009) using a variable stress ratio exponent for normalization based on I_c (PKR 2009)	See Figure 4	15
Modified Non-normalized SBT Chart SBT (PKR2010)	This is a revised version of the simple 1986 non-normalized SBT chart (presented at CPT '10). The revised version has been reduced from 12 zones to 9 zones to be similar to the normalized Robertson charts. Other updates include a dimensionless tip resistance normalized to atmospheric pressure, q_t/P_a , on the vertical axis and a log scale for non-normalized friction ratio, R_f , along the horizontal axis.	See Figure 5	33
Modified SBTn (contractive /dilative)	Modified SBTn chart as defined by Robertson (2016) indicating zones of contractive/dilative behavior. Note that ConeTec displays the chart with colors different from Robertson. ConeTec's colors were chosen to avoid confusion with soil type descriptions.	See Figure 6	30
Unit Wt.	Unit Weight of soil determined from one of the following user selectable options: 1) uniform value 2) value assigned to each SBT zone 3) value assigned to each SBTn zone 4) value assigned to SBTn zone as determined from Robertson and Wride (1998) based on q_{c1n} 5) values assigned to SBT Qtn zones 6) values based on Robertson updated non-normalized Soil Behavior Type Chart (2010b) 6) Mayne f_s (sleeve friction) method 7) Robertson and Cabal 2010 method 8) user supplied unit weight profile The last option may co-exist with any of the other options.	See references	3, 5, 15, 21, 24, 29, 33

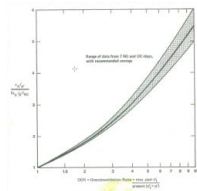


Calculated Parameter	Description	Equation	Ref
TStress σ_v	Total vertical overburden stress at Mid Layer Depth <i>A layer is defined as the averaging interval specified by the user where depths are reported at their respective mid-layer depth.</i> For data calculated at each point layers are defined using the recorded depth as the mid-point of the layer. Thus, a layer starts half-way between the previous depth and the current depth unless this is the first point in which case the layer start is at zero depth. The layer bottom is half-way from the current depth to the next depth unless it is the last data point. Defining layers affects how stresses are calculated since the unit weight attributed to a data point is used throughout the entire layer. This means that to calculate the stresses the total stress at the top and bottom of a layer are required. The stress at mid layer is determined by adding the incremental stress from the layer top to the mid-layer depth. The stress at the layer bottom becomes the stress at the top of the subsequent layer. Stresses are NOT calculated from mid-point to mid-point. For over-water work the total stress due to the column of water above the mud line is taken into account where appropriate.	$TStress = \sum_{i=1}^n \gamma_i h_i$ where γ_i is layer unit weight h_i is layer thickness ● CPT Data Point Depths  ● ● ● Repeats for each layer Layer i ● Layer i+1 ● Final Layer ● final depth	CK*
EStress σ_v'	Effective vertical overburden stress at mid-layer depth.	$\sigma_v' = \sigma_v - u_{eq}$	CK*
Equil u u_{eq} or u_0	Equilibrium pore pressures are determined from one of the following user selectable options: 1) hydrostatic below the water table 2) user supplied profile 3) combination of those above When a user supplied profile is used/provided a linear interpolation is performed between equilibrium pore pressures defined at specific depths. If the profile values start below the water table then a linear transition from zero pressure at the water table to the first defined point is used. Equilibrium pore pressures may come from dissipation tests, adjacent piezometers or other sources. Occasionally, an extra equilibrium point ("assumed value") will be provided in the profile that does not come from a recorded value to smooth out any abrupt changes or to deal with material interfaces. These "assumed" values will be indicated on our plots and in tabular summaries.	For the hydrostatic option: $u_{eq} = \gamma_w \cdot (D - D_{wt})$ where u_{eq} is equilibrium pore pressure γ_w is the unit weight of water D is the current depth D_{wt} is the depth to the water table	CK*
K_0	Coefficient of earth pressure at rest, K_0 .	$K_0 = (1 - \sin \Phi') OCR^{\sin \Phi'}$	17
C_n	Overburden stress correction factor used for $(N_1)_{eq}$ and older CPT parameters.	$C_n = (P_a / \sigma_v')^{0.5}$ where $0.0 < C_n < 2.0$ (user adjustable, typically ranging from 1.7 to 2.0) P_a is atmospheric pressure (100 kPa)	4, 12

Calculated Parameter	Description	Equation	Ref
C_q	Overburden stress normalizing factor.	$C_q = 1.8 / [0.8 + (\sigma'_v/P_a)]$ where $0.0 < C_q < 2.0$ (user adjustable) P_a is atmospheric pressure (100 kPa) <i>Robertson and Wride define C_q to be the same as C_n. The Olson definition above is used in the program.</i>	3, 12
N_{60}	SPT N value at 60% energy calculated from q_t/N ratios assigned to each SBT zone. This method has abrupt N value changes at zone boundaries.	See Figure 1	5
$(N_1)_{60}$	SPT N_{60} value corrected for overburden pressure.	$(N_1)_{60} = C_n \cdot N_{60}$	4
N_{60lc}	SPT N_{60} values based on the I_c parameter, as defined by Robertson and Wride 1998 (3), or by Robertson 2009 (15).	$(q_t/P_a)/N_{60} = 8.5 (1 - I_c/4.6)$ $(q_t/P_a)/N_{60} = 10^{(1.1268 - 0.2817I_c)}$ P_a being atmospheric pressure	3, 5 15, 31
$(N_1)_{60lc}$	SPT N_{60} value corrected for overburden pressure (using $N_{60} I_c$). User has 3 options.	1) $(N_1)_{60lc} = C_n \cdot (N_{60} I_c)$ 2) $q_{c15}/(N_1)_{60lc} = 8.5 (1 - I_c/4.6)$ 3) $(Q_{cn})/(N_1)_{60lc} = 10^{(1.1268 - 0.2817I_c)}$	4 5 15, 31
S_u or $S_u (N_{kt})$	Undrained shear strength based on q_t S_u factor N_{kt} is user selectable.	$S_u = \frac{q_t - \sigma_v}{N_{kt}}$	1, 5
S_u or $S_u (N_{du})$ or $S_u (N_{du})$	Undrained shear strength based on pore pressure S_u factor N_{du} is user selectable.	$S_u = \frac{u_z - u_{eq}}{N_{du}}$	1, 5
D_r	Relative Density determined from one of the following user selectable options: 1) Ticino Sand 2) Hokksund Sand 3) Schmertmann (1978) 4) Jamiolkowski (1985) - All Sands 5) Jamiolkowski et al (2003) (various compressibilities, K_o)	See reference (methods 1 through 4) Jamiolkowski et al (2003) reference	5 14
PHI ϕ	Friction Angle determined from one of the following user selectable options (methods 1 through 4 are for sands and method 5 is for silts and clays): 1) Campanella and Robertson 2) Durgunoglu and Mitchel 3) Janbu 4) Kulhawy and Mayne 5) NTH method (clays and silts)	See appropriate reference	5 5 5 11 23
Delta U/q_t $\Delta u/q_t$ du/q_t	Differential pore pressure ratio (older parameter used before B_q was established)	$= \frac{\Delta u}{q_t}$ where: $\Delta u = u - u_{eq}$ and u = dynamic pore pressure u_{eq} = equilibrium pore pressure	39

Calculated Parameter	Description	Equation	Ref
B _q	Pore pressure parameter	$Bq = \frac{\Delta u}{qt - \sigma_v}$ where: $\Delta u = u - u_{eq}$ and u = dynamic pore pressure u_{eq} = equilibrium pore pressure	1, 2, 5
Net q _t or qtNet	Net tip resistance (used in many subsequent correlations)	$qt - \sigma_v$	36
q _e or q _E or q _E	Effective tip resistance (using the dynamic pore pressure u ₂ and not equilibrium pore pressure)	$q_t - u_2$	36
qeNorm	Normalized effective tip resistance	$\frac{qt - u_2}{\sigma_v}$	36
Q _t or Norm: Qt or Q _{t1}	Normalized q _t for Soil Behavior Type classification as defined by Robertson (1990) using a linear stress normalization. Note this is different from Q _{tn} . This parameter was renamed to Q _{t1} in Robertson, 2009. Without normalization limits this parameter calculates to very high unrealistic values at low stresses.	$Q_t = \frac{qt - \sigma_v}{\sigma_v}$	2, 5, 15
F _r or Norm: Fr	Normalized Friction Ratio for Soil Behavior Type classification as defined by Robertson (1990)	$Fr = 100\% \cdot \frac{fs}{qt - \sigma_v}$	2, 5
Q(1-B _q) Q(1-B _q) + 1	Q(1-B _q) grouping as suggested by Jefferies and Davies for their classification chart and the establishment of their I _c parameter. Later papers added the +1 term to the equation.	$Q \cdot (1 - Bq)$ $Q \cdot (1 - Bq) + 1$ where Bq is defined as above and Q is the same as the normalized tip resistance, Q_{t1}, defined above	6, 7, 34
q _{c1}	Normalized tip resistance, q _{c1} , using a fixed stress ratio exponent, n (this method has stress units)	$q_{c1} = q_t \cdot (Pa / \sigma_v')^{0.5}$ where: P_a = atmospheric pressure	21
q _{c1} (0.5)	Normalized tip resistance, q _{c1} , using a fixed stress ratio exponent, n (this method is unit-less)	$q_{c1} (0.5) = (q_t / P_a) \cdot (Pa / \sigma_v')^{0.5}$ where: P_a = atmospheric pressure	5
q _{c1} (C _n)	Normalized tip resistance, q _{c1} , based on C _n (this method has stress units)	$q_{c1}(Cn) = C_n \cdot q_t$	5, 12
q _{c1} (C _q)	Normalized tip resistance, q _{c1} , based on C _q (this method has stress units)	$q_{c1}(Cq) = C_q \cdot q_t$ (some papers use q _c)	5, 12
q _{c1n}	normalized tip resistance, q _{c1n} , using a variable stress ratio exponent, n (where n=0.0, 0.70, or 1.0) (this method is unit-less)	$q_{c1n} = (q_t / P_a) (P_a / \sigma_v')^n$ where: P_a = atm. Pressure and n varies as described below	3

Calculated Parameter	Description	Equation	Ref
I_c or I_c (RW1998)	Soil Behavior Type Index as defined by Robertson and Wride (1997, 1998) for estimating grain size characteristics and providing smooth gradational changes across the SBTn chart. I_c (RW1998) is different from that of Jefferies and Davies (7) and is different from I_c (PKR2009).	$I_c = [(3.47 - \log_{10} Q)^2 + (\log_{10} Fr + 1.22)^2]^{0.5}$ Where: $Q = \left(\frac{qt - \sigma_v}{P_a} \right) \left(\frac{P_a}{\sigma_v} \right)^n$ Or $Q = q_{clin} = \left(\frac{qt}{P_a} \right) \left(\frac{P_a}{\sigma_v} \right)^n$ depending on the iteration in determining I_c And Fr is in percent P_a = atmospheric pressure n has the following distinct values: 0.5, 0.75 and 1.0 and is determined in an iterative manner based on the resulting I_c in each iteration Note that NCEER replaced 0.75 with 0.70	3, 4, 5 10
I_c (PKR 2009)	Soil Behavior Type Index, I_c (PKR 2009) is based on a variable stress ratio exponent n , which itself is based on I_c (PKR 2009). An iterative calculation is required to determine I_c (PKR 2009) and its corresponding n (PKR 2009).	$I_c \text{ (PKR 2009)} = [(3.47 - \log_{10} Q_{tn})^2 + (1.22 + \log_{10} F_r)^2]^{0.5}$	15
n (PKR 2009)	Stress ratio exponent n , based on I_c (PKR 2009). An iterative calculation is required to determine n (PKR 2009) and its corresponding I_c (PKR 2009).	$n \text{ (PKR 2009)} = 0.381 (I_c) + 0.05 (\sigma'_v / P_a) - 0.15$	15
Q_{tn} (PKR 2009)	Normalized tip resistance using a variable stress ratio exponent based on I_c (PKR 2009) and n (PKR 2009). An iterative calculation is required to determine Q_{tn} (PKR 2009).	$Q_{tn} = [(qt - \sigma_v) / P_a] (P_a / \sigma'_v)^n$ where P_a = atmospheric pressure (100 kPa) n = stress ratio exponent described above	15
FC	Apparent fines content (%)	$FC = 1.75 (I_c^{3.25}) - 3.7$ $FC = 100 \text{ for } I_c > 3.5$ $FC = 0 \text{ for } I_c < 1.26$ $FC = 5\% \text{ if } 1.64 < I_c < 2.6 \text{ AND } F_r < 0.5$	3
I_c Zone	This parameter is the Soil Behavior Type zone based on the I_c parameter (valid for zones 2 through 7 on SBTn or SBT Qtn charts)	$I_c < 1.31$ Zone = 7 $1.31 < I_c < 2.05$ Zone = 6 $2.05 < I_c < 2.60$ Zone = 5 $2.60 < I_c < 2.95$ Zone = 4 $2.95 < I_c < 3.60$ Zone = 3 $I_c > 3.60$ Zone = 2	3
CD	The contractive / dilative boundary on Robertson's Modified SBTn (contractive/dilative) Chart shown in Figure 6 above. The boundary is marked as CD = 70 on the chart in the relevant paper. Similar to the $Q_{tn,cs} = 70$ line in Figure 4.	$CD = 70 = (Q_{tn} - 11) (1 + 0.06F_r)^{17}$ lower bound of CD = 60: $CD = 60 = (Q_{tn} - 9.5) (1 + 0.06F_r)^{17}$	30

Calculated Parameter	Description	Equation	Ref
I_b	Hyperbolic fit defining the boundary between SBT soil types proposed by Schneider as a better fit than the I_c circles. $I_b = 32$ represents the boundary for most sand like soils. $I_b = 22$ represents the upper boundary for most clay like soils. The region between $I_b=22$ and $I_b=32$ is the “transitional soil” zone.	$I_b = 100 (Q_{tn} + 10) / (70 + Q_{tn} F_r)$	30
State Param or State Parameter or ψ	The state parameter index, ψ , is defined as the difference between the current void ratio, e , and the critical void ratio, e_c . Positive ψ - contractive soil Negative ψ - dilative soil This is based on the work by Been and Jefferies (1985) and Plewes, Davies and Jefferies (1992) This method uses mean normal stresses based on a uniform value of K_0 or a calculated K_0 using methods described elsewhere in this document	See reference	6, 8
Yield Stress σ_p'	Yield stress is calculated using the following methods 1) General method 2) 1 st order approximation using q_t Net (clays) 3) 1 st order approximation using Δu_2 (clays) 4) 1 st order approximation using q_e (clays) 5) Based on Vs	All stresses in kPa 1) $\sigma_p' = 0.33 \cdot (q_t - \sigma_v)'^{m'} (\sigma_{atm}/100)^{1-m'}$ where $m' = 1 - \frac{0.28}{1 + (I_c / 2.65)^{25}}$ 2) $\sigma_p' = 0.33 \cdot (q_t - \sigma_v)$ 3) $\sigma_p' = 0.54 \cdot (\Delta u_2)$ $\Delta u_2 = u_2 - u_0$ 4) $\sigma_p' = 0.60 \cdot (q_t - u_2)$ 5) $\sigma_p' = (Vs/4.59)^{1.47}$	19 20 20 20 18
OCR OCR(JS1978) YSR(Mayne2014) YSR (qtNet) YSR (deltaU) YSR (qe) YSR (Vs) OCR (PKR2015)	Over Consolidation Ratio based on 1) Schmertmann (1978) method involving a plot plot of $S_u/\sigma_v' / (S_u/\sigma_v')_{NC}$ and OCR  2) based on Yield stresses described above 3) approximate version based on qtNet 4) approximate version based on Δu 5) approximate version based on effective tip, q_e 6) approximate version based on shear wave velocity, V_s and σ_v' 7) based on Qt	1) requires a user defined value for NC S_u/P_c' ratio 2 through 5) based on yield stresses 6) YSR (Vs) = $\sigma_p' (Vs) / \sigma_v'$ 7) OCR = $0.25 \cdot (Qt)^{1.25}$	9 19 20 20 20 18 32
Es/qt	Intermediate parameter for calculating Young's Modulus, E, in sands. It is the Y axis of the reference chart. Note that Figure 5.59 from reference 5, Lunne, Robertson and Powell, (LRP) has an error. The X axis values are too high by a factor of 10. The plot is based on Baldi's (not Bellotti as cited in	Based on Figure 5.59 in the reference	5, 37

Calculated Parameter	Description	Equation	Ref
	LRP) original Figure 3 where the X axis is: $\frac{q_c}{\sqrt{\sigma'_v}}$ (both in kPa) with a range of 200 to 3000. Figure 5.59 from LRP shows a dimensionless form of the equation, q_{c1}, displaying the same range of values. Figure 5.59's X axis uses $q_{c1} = \left(\frac{q_c}{P_a}\right) \left(\frac{P_a}{\sigma'_v}\right)^{0.5}$ The two expressions are not the same: they differ by a factor of $\frac{\sqrt{P_a}}{P_a}$. With P_a taken to be 100 kPa the factor is 1/10. Substituting typical values of 200 bar (20000 kPa) for q_c and 225 kPa for σ'_v one gets: $20000 / 15 = 1333.33$ for Bellotti's axis and $(200/1)(100/225)^{0.5} = 200 * (10/15) = 133.3$ for LRP's axis (noting that $P_a = 1$ bar) showing a factor of 10 difference.		
Es or Es Young's Modulus E	Young's Modulus based on the work done in Italy. There are three types of sands considered in this technique. The user selects the appropriate type for the site from: a) OC Sands b) Aged NC Sands c) Recent NC Sands Each sand type has a family of curves that depend on mean normal stress. The program calculates mean normal stress and linearly interpolates between the two extremes provided in the E_s/q_t chart. E_s is evaluated for an axial strain of 0.1%.	Mean normal stress is evaluated from: $\sigma'_m = \frac{1}{3}(\sigma'_v + \sigma'_h + \sigma'_h)$ where σ'_v = vertical effective stress σ'_h = horizontal effective stress and $\sigma_h = K_o \cdot \sigma'_v$ with K_o assumed to be 0.5	5
Delta U/TStress $\Delta u / \sigma_v$	Differential pore pressure ratio with respect to total stress	$= \frac{\Delta u}{\sigma_v}$ where: $\Delta u = u - u_{eq}$	39
Delta U/ESTress, P Value, Excess Pore Pressure Ratio $\Delta u / \sigma'_v$	Differential pore pressure ratio with respect to effective stress. Key parameter (P, Normalized Pore Pressure Parameter, Excess Pore Pressure Ratio) in the Winckler et. al. static liquefaction method.	$= \frac{\Delta u}{\sigma'_v}$ where: $\Delta u = u - u_{eq}$	25, 25a
Su/ESTress S_u / σ'_v	Undrained shear strength ratio with respect to vertical effective overburden stress using the $S_u (N_{kt})$ method	$= S_u (N_{kt}) / \sigma'_v$	9, 23
Vs or Vs	Recorded shear wave velocities (not estimated). The shear wave velocities are typically collected over 1 m depth intervals. Each data point over the relevant depth range is assigned the same V_s value.	recorded data	27
Vp or Vp	Recorded compression wave (or P wave) velocities (not estimated). The P wave velocities are typically collected over 1 m depth intervals. Each data point over the relevant depth range is assigned the same V_p value.	recorded data	27

Calculated Parameter	Description	Equation	Ref
V_{s30} V_{s100}	The average shear wave velocity of the near surface materials to a depth of 30 m (100 ft). It is based on the sum of all travel times through all layers in the top 30m (100 ft). V_{s100} is the same calculation as V_{s30} except down to a depth of 100 feet.	$V_{s30} = \frac{\text{total thickness of all layers to 30 m}}{\sum \left(\frac{\text{layer thickness}}{\text{layer shear wave velocity}} \right)}$ $V_{s100} = \frac{\text{total thickness of all layers to 30 m}}{\sum (\text{layer travel times})}$	38
G_{max}	G_{max} determined from SCPT shear wave velocities (not estimated values). Note that seismic data (V_s) is collected over set depth intervals (typically 1 meter). Each data point over the test segment is assigned the same V_s value. Since soil density changes with depth, slightly different G_{max} values may be calculated over the test depth interval.	$G_{max} = \rho V_s^2$ where ρ is the mass density of the soil determined from the estimated unit weights at each test depth	27
qtNet/ G_{max}	Net tip resistance ratio with respect to the small strain modulus G_{max} determined from SCPT shear wave velocities (not estimated values)	$= (q_t - \sigma_v) / G_{max}$ where $G_{max} = \rho V_s^2$ and ρ is the mass density of the soil determined from the estimated unit weights at each test depth	15, 28, 30
qUlt	A site specific and client specific parameter for estimating the limiting stress for “crane walk” accessibility	$q_{ult} = CraneWalkFactor \cdot S_u$ Where: <i>CraneWalkFactor</i> is client provided	U*
Estimated G_o	Estimated value for small strain shear modulus	$G_o = 0.0188 [10^{(0.55I_c + 1.68)}] (q_t - \sigma_v)$	15
Estimated E_{25}	Estimated value for Young’s Modulus, E, at a 25% working load	$E_{25} = \alpha_E (qtNet)$ where $\alpha_E = 0.015 [10^{(0.55I_c + 1.68)}]$	15
k_{SBT}	Estimated soil permeability derived from Soil Behavior Type (SBT) Chart I_c values.	For $1.0 < I_c \leq 3.27$: $k = 10^{(0.952 - 3.04I_c)}$ in m/s For $3.27 < I_c < 4.0$: $k = 10^{(-4.52 - 1.37I_c)}$ in m/s	35
M or D' Constrained Modulus	Constrained Modulus based on 1) Robertson, M 2) Mayne, D'	1) Robertson $M = \alpha_M (q_t - \sigma_v)$ $I_c > 2.2$ (fine grained) $\alpha_M = q_t$ when $q_t < 14$ $\alpha_M = 14$ when $q_t > 14$ $I_c < 2.2$ (coarse grained) $\alpha_M = 0.0188 [10^{(0.55I_c + 1.68)}]$ $D' = \alpha_D (q_t - \sigma_v)$ where $\alpha_D = 5$	32 23

Table 1b. CPT Parameter Calculation Methods – Liquefaction Parameters

Calculated Parameter	Description	Equation	Ref
K_{SPT} or K_s	Equivalent clean sand factor for $(N_1)_{60}$	$K_{SPT} = 1 + ((0.75/30) \cdot (FC - 5))$	10
K_{CPT} or K_c (RW1998)	Equivalent clean sand correction for q_{c1n}	$K_{cpt} = 1.0$ for $l_c \leq 1.64$ $K_{cpt} = f(l_c)$ for $l_c > 1.64$ (see reference) $K_c = -0.403 l_c^4 + 5.581 l_c^3 - 21.63 l_c^2 + 33.75 l_c - 17.88$	3, 10
K_c (PKR 2010)	Clean sand equivalent factor to be applied to Q_{tn}	$K_c = 1.0$ for $l_c \leq 1.64$ $K_c = -0.403 l_c^4 + 5.581 l_c^3 - 21.63 l_c^2 + 33.75 l_c - 17.88$ for $l_c > 1.64$	16
$(N_1)_{60cs} l_c$	Clean sand equivalent SPT $(N_1)_{60cs}$. User has 3 options.	1) $(N_1)_{60cs} l_c = \alpha + \theta((N_1)_{60cs})$ 2) $(N_1)_{60cs} l_c = K_{SPT} \cdot ((N_1)_{60cs})$ 3) $(q_{c1n}) / ((N_1)_{60cs}) l_c = 8.5 (1 - l_c/4.6)$ FC $\leq$ 5%: $\alpha = 0, \theta = 1.0$ FC $\geq$ 35%: $\alpha = 5.0, \theta = 1.2$ 5% < FC < 35%: $\alpha = \exp[1.76 - (190/FC^2)]$ $\theta = [0.99 + (FC^{1.5}/1000)]$	10 10 5
q_{c1nCS}	Clean sand equivalent q_{c1n}	$q_{c1nCS} = q_{c1n} \cdot K_{cpt}$	3
$Q_{tn,CS}$ (PKR 2010)	Clean sand equivalent for Q_{tn} described above - Q_{tn} being the normalized tip resistance based on a variable stress exponent as defined by Robertson (2009)	$Q_{tn,CS} = Q_{tn} \cdot K_c$ (PKR 2016)	16
$S_u(Liq)/ES_v$ or $S_u(Liq)/\sigma'_v$	Liquefied shear strength ratio as defined by Olson and Stark	$\frac{S_u(Liq)}{\sigma'_v} = 0.03 + 0.0143(q_{c1})$ Note: σ'_v and s'_v are synonymous	13
$S_u(Liq)/ES_v$ or $S_u(Liq)/\sigma'_v$ (PKR 2010)	Liquefied shear strength ratio as defined by Robertson (2010)	$\frac{S_u(Liq)}{\sigma'_v}$ Based on a function involving $Q_{tn,CS}$	16
$S_u(Liq)$ (PKR 2010)	Liquefied shear strength derived from the liquefied shear strength ratio and effective overburden stress	$S_u(Liq) = \sigma'_v \cdot \left(\frac{S_u(Liq)}{\sigma'_v} \right)$	16
Cont/Dilat Tip	Contractive / Dilative q_{c1} Boundary based on $(N_1)_{60}$	$(\sigma'_v)_{boundary} = 9.58 \times 10^{-4} [(N_1)_{60}]^{4.79}$ q_{c1} is calculated from specified q_t (MPa)/N ratio	13
CRR	Cyclic Resistance Ratio (for Magnitude 7.5)	$q_{c1nCS} < 50$: $CRR_{7.5} = 0.833 [q_{c1nCS}/1000] + 0.05$ $50 \leq q_{c1nCS} < 160$: $CRR_{7.5} = 93 [q_{c1nCS}/1000]^3 + 0.08$	10
K_g or K_g	Small strain Stiffness Ratio Factor, K_g	$[G_{max}/q_t]/[q_{c1n}^m]$ m = empirical exponent, typically 0.75	26



Calculated Parameter	Description	Equation	Ref
K_g^*	Revised K_g factor extended to fine grained soils (Robertson).	$K_g^* = (G_o / q_n)(Q_{tn})^{0.75}$ where q_n is the net tip resistance = $q_t - \sigma_v$	30
SP Distance	State Parameter Distance, Winckler static liquefaction method	Perpendicular distance on Q_{tn} chart from plotted point to state parameter $\Psi = -0.05$ curve	25
URS NP Fr	Normalized friction ratio point on $\Psi = -0.05$ curve used in SP distance calculation		25
URS NP Q_{tn}	Normalized tip resistance (Q_{tn}) point on $\Psi = -0.05$ curve used in SP Distance calculation		25



Table 2. References

No.	Reference
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No.	Reference
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Calibration Records





CERTIFICATE OF CALIBRATION

Calibration Information			
Cone Serial Number	EC1061	Model	A15 T1500 F15 U35
Date	2024-05-08	Signature	
Technician Performing Calibration	Richard Chen		
Calibration Approved By	Vishrut Khunt		

Lab Condition	As Found	As Left		
Lab Temperature	N/A	23°C		
Lab Humidity	N/A	27%	Reason for Calibration	Repair

Cone Information				
Tip Stress Limit	1500	bar	Tip End Area	15 cm ²
Friction Stress Limit	15	bar	Friction Surface Area	225 cm ²
Pressure Limit	35	bar	RTD Location	Pressure Carrier
X-Inclinometer Limit	30	degrees	Geophone	X and Z
Y-Inclinometer Limit	30	degrees	Temperature Range	-20°C to 60°C

Baseline Summary: (For Reference Only)

Channel	Units	As Found	As Left
Tip	bar	-3.622	-1.064
Sleeve	bar	0.059	-0.023
Pressure	bar	1.235	1.026
X-Inclinometer	degrees	-0.350	-0.013
Y-Inclinometer	degrees	-0.100	0.006
Temperature	°C	23.630	23.734

Classified in accordance with ISO 22476-1:2012 Class 1

Classified in accordance with ISO 22476-1:2012 Class 2

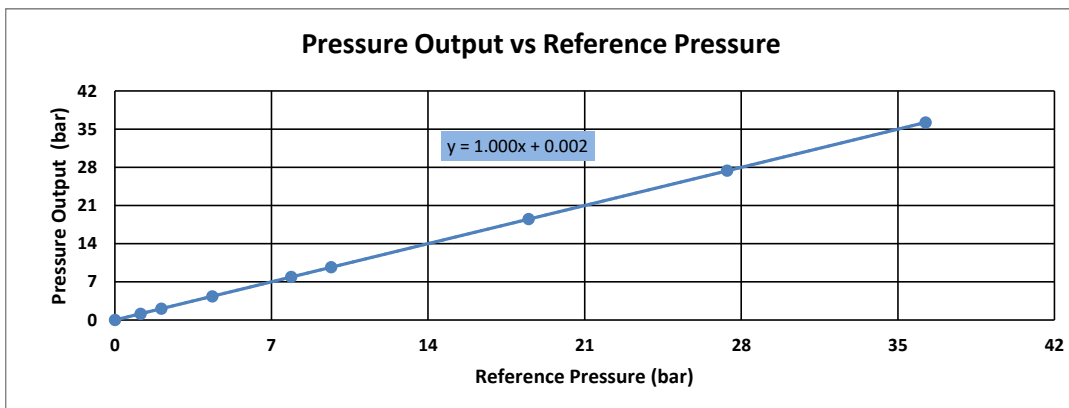
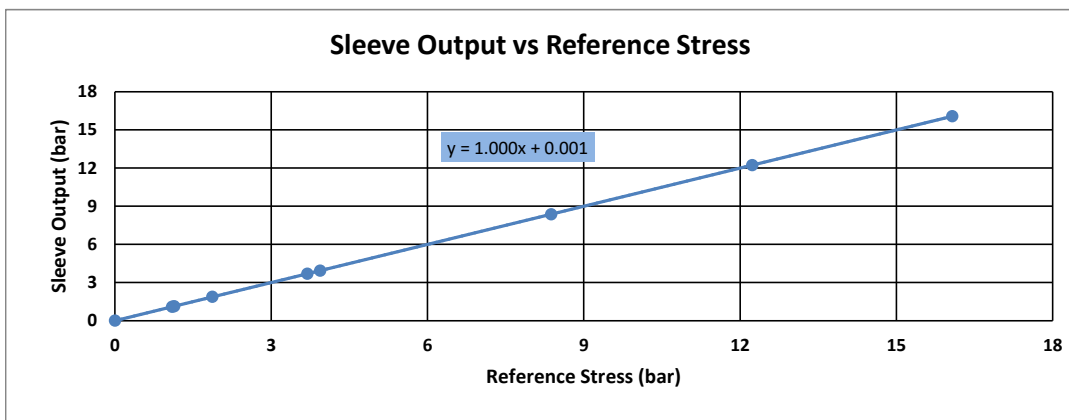
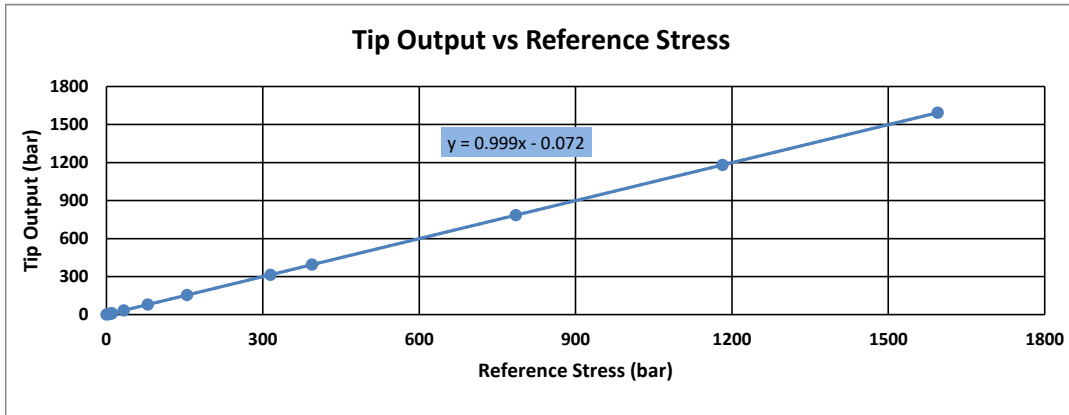
Calibrated in general accordance with the ASTM D5778-20 and D7400-08 standards

Calibrated with Adara calibration procedure EC_CPTCAL-2.2

Collective uncertainty of the measurement standards conforms to a test uncertainty ratio (TUR) of 3:1 for tip and sleeve measurement and 4:1 for pressure measurement with a confidence level k=2



Cone Output vs Reference Stress/Pressure Plots





Calibration Results

Tip Calibration					
As Found			As Left		
Max. Non Linearity	0.08%	PASS	Max. Non Linearity	0.10%	PASS
Calibration Error	0.07%	PASS	Calibration Error	0.19%	PASS

Sleeve Calibration					
As Found			As Left		
Max. Non Linearity	0.34%	PASS	Max. Non Linearity	0.01%	PASS
Calibration Error	0.35%	PASS	Calibration Error	0.05%	PASS

Pressure Calibration					
As Found			As Left		
Max. Non Linearity	0.03%	PASS	Max. Non Linearity	0.04%	PASS
Calibration Error	0.09%	PASS	Calibration Error	0.06%	PASS

X-Inclinometer Calibration					
As Found			As Left		
Max. Non Linearity	N/A	N/A	Max. Non Linearity	0.04%	PASS
Calibration Error	N/A	N/A	Calibration Error	-0.08%	PASS

Y-Inclinometer Calibration					
As Found			As Left		
Max. Non Linearity	N/A	N/A	Max. Non Linearity	-0.33%	PASS
Calibration Error	N/A	N/A	Calibration Error	0.67%	PASS

Seismic Calibration					
As Found			As Left		
Trigger Delay Error	N/A	N/A	Trigger Delay Error	0.02%	PASS

Temperature Calibration					
Full Scale Error	0.23%	PASS			

Channel	Cold	Room	Hot	Units
Ref_Temp	3.3	22.2	43.4	°C
Tip	-4.317	-0.915	2.823	bar
Sleeve	-0.061	-0.022	0.021	bar
Pressure	1.048	1.063	1.064	bar
Temperature	3.281	22.037	43.378	°C

Tip Temperature Coefficient	0.178 bar/°C	PASS
Sleeve Temperature Coefficient	0.002 bar/°C	PASS
Pressure Temperature Coefficient	0.000 bar/°C	PASS



Adara Systems Ltd.
201-8327 Eastlake Drive, Burnaby, BC, Canada V5A4W2
Tel: 1-604-273-6013

Testing Equipment Details				
Testing Machines	Model Number	Serial Number	Calibration Number	Due Date
Tip Load Cell	Precision	P-10289	100490	2025-09-18
Sleeve Load Cell	Precision	P-10868	100579	2025-10-01
Digital Loadcell Indicator	4215	62140	100490	2024-07-18
Fluke Reference Pressure Monitor	RPM4 A10Ms	3910	100835	2024-12-12
Tektronix Function Generator	AFG3021B	C030955	100751	2024-10-20
Thermometer	THS-222-555	D23255834	100410	2024-07-11
Thermometer	THS-222-555	D23255829	100410	2024-07-11
Thermometer	THS-222-555	D20345575	100565	2024-07-14

Adara Error Definitions

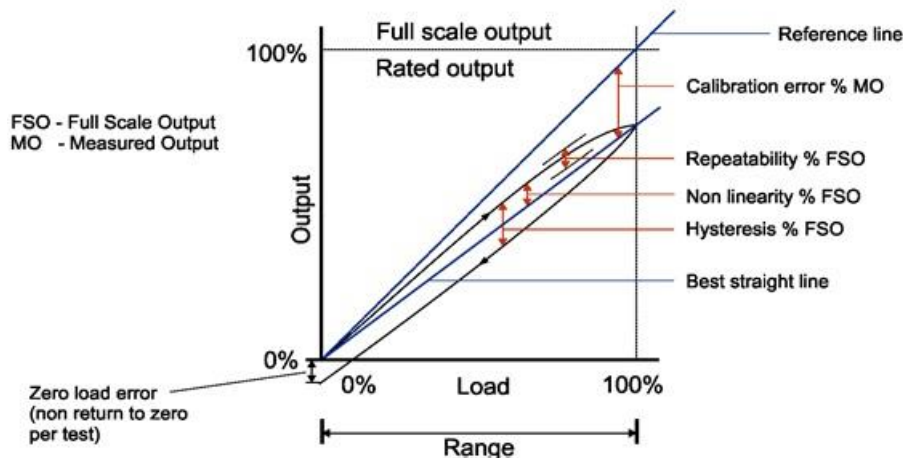


Figure 1: Definition of Calibration Terms for Load Cell and Transducers (Adapted from [1])

Actual Sensitivity	The slope of the best fit line through all data points starting at zero load.
Slope Error	The error in the best fit line compared to the ideal linear calibration in % . $\text{Slope Error} = (\text{Best Fit Slope} - \text{Ideal Slope}) / \text{Ideal Slope}$
Maximum Non Linearity	This value represents the maximum error (absolute value) relative to the best fit line considering each calibration point starting at loads greater than approximately 10% of FSO. The reported errors are a percent error of FSO. Adara's Pass/Fail criteria is 0.5% of FSO (ASTM is 0.5% of FSO at loads > 20% FSO).
Calibration Error	This value represents the maximum error (absolute value) in the recorded load value as compared to the actual load value for each calibration point for loads greater than approximately 10% of FSO. Adara's Pass/Fail criteria for the tip and sleeve is 0.5% of MO and 1.0% of MO for the pore pressure (ASTM for the tip and sleeve is 1.5% and 1.0% of MO respectively at loads greater than 20% of FSO)

Temperature Check Passing Criteria

Tip Temperature Coefficient	<0.200 bar/°C
Sleeve Temperature Coefficient	<0.005 bar/°C
Pressure Temperature Coefficient	<0.0196 bar/°C



ASTM D5778-20 Annex A Summary [1]

A1.4 Force Transducer Calibration Requirements

A1.4.1 states the following limits:

Non Linearity	Tip	$\leq +0.5\%$ of FSO
	Sleeve	$\leq +1.0\%$ of FSO
Calibration Error	Tip	$\leq +1.5\%$ of MO at loads $> 20\%$ FSO
	Sleeve	$\leq +1.0\%$ of MO at loads $> 20\%$ FSO

A1.5 Pressure Transducer Calibrations

A1.5.1 limits:

Non Linearity	Pore Pressure	$\leq +1.0\%$ of FSO
Calibration Error	Pore Pressure	not specified

ISO 22476 -1:2012 Summary [2]

Section 5.2 states the following allowable minimum accuracy

Class 1	Cone Resistance	35 kPa or 5%
	Sleeve Friction	5 kPa or 10%
	Pore Pressure	10 kPa or 2%
Class 2	Cone Resistance	100 kPa or 5%
	Sleeve Friction	15 kPa or 15%
	Pore Pressure	25 kPa or 3%

Note: ISO Compliance is based on low end calibration only.

References

[1] ASTM D5778-20. "Standard Test Method for Electronic Friction Cone and Piezocone Penetration Testing of Soils". ASTM, West Conshohocken, PA, USA.

[2] ISO 22476-1:2012. "Geotechnical investigation and testing - Field Testing - Part 1: Electrical cone and piezocone penetration test". ISO, Geneva, Switzerland.

ASTM D7400-08. "Standard Test Methods for Downhole Seismic Testing". ASTM, West Conshohocken, PA, USA.

Appendix D GRLWeap Analysis Report



Job No. 2024CA00138

Report on: Preconstruction Wave Equation Analysis
CISS Pile Load Testing – UC Berkeley Project
Oakley, CA

Prepared for: Foundation Constructors, Inc.
By: Camilo Alvarez, PE & William Chambers CPEng
April 12, 2024

www.GRLengineers.com

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April 12, 2024

Dhruv Patel
Foundation Constructors, Inc
PO Box 97
Oakley, CA 94561

Re: CISS Pile Load Testing – UC Berkeley Project
Oakley, CA
Driving System Submittal

GRL Job No. 2024CA00138

Dear Mr. Dhruv Patel:

We are pleased to submit our report on results obtained from our wave equation analyses. We based our study on background material given to us by Foundation Constructors on April 11, 2024. Using the submitted boring logs, we attempted to determine the suitability of the proposed hammer system to install Cast in Steel Shell (CISS) 36-inch OD open ended pipe piles for the University of Californian Berkeley project in Oakley, CA.

Calculations were done with our GRLWEAP 2014 program. A description of this program is included in Appendix A. Appendix B contains the GRLWEAP program basic output results for each analysis. Soil information is presented in Appendix C.

The following is a summary of the analysis details.

Objectives

Determine the suitability of the proposed driving systems to install CISS 36-inch OD pipe piles at this site.

Piles

The piles to be installed at this project are CISS 36-inch diameter by 0.5-inch wall thickness steel pipe piles. Reported lengths are 40 ft. GRL used soil information provided by Foundation Constructors, Inc., and modeled soil profiles of the piles to be installed.

The pipe piles are *assumed* to have a minimum yield strength of **45** ksi.

Pile details are included in Table 1 below:

Table 1. Pile Data Table

Pile Type	Hammer Model	Pile Length (ft)	Soil Boring
CISS 36-inch Diameter	Delmag D36	40.0	R-19-001
	Delmag D46	40.0	R-19-002

Hammers

The proposed driving hammer analyzed was the Delmag D36 single acting (open ended) diesel hammer. Ram weight is reported to be 7.9 kips. The D36 has a maximum geometric stroke of 10.6 ft with a rated energy of 83.8.1 kip-ft.

The proposed driving hammer analyzed was the Delmag D46 single acting (open ended) diesel hammer. Ram weight is reported to be 10.1 kips. The D46 has a maximum geometric stroke of 10.6 ft with a rated energy of 107.1 kip-ft.

The driving systems were analyzed at maximum fuel setting to determine the maximum stress levels at which the piles will be subjected to, and Appendix D includes the hammer data sheet with all driving system information.

Soil

The soil profile consists of multiple layers of silt, sand, and clay. Boring logs were submitted to GRL. GRL used borings R-19-001 and R-19-002 and designated the soil resistance in which higher stress values will be encountered to ensure the worst-case scenario during pile driving. Soil information is presented in Appendix C. Soil parameters, including unit shaft friction, end bearing, and soil damping and quakes are presented below in Tables 2a and 2b.

Table 2a. Soil Model for Borehole R-19-001

Depth (ft)	Unit Shaft Resist (ksf)	Unit Toe Resist (ksf)	Skin Quake (in)	Toe Quake (in)	Skin Damping (s/ft)	Toe Damping (s/ft)
0.0	0.0	33.4	0.10	0.10	0.05	0.15
7.9	0.4	33.4	0.10	0.10	0.05	0.15
7.9	1.0	30.5	0.10	0.10	0.20	0.15
24.9	2.8	30.5	0.10	0.10	0.20	0.15
24.9	0.8	196.3	0.10	0.10	0.05	0.15
37.9	1.2	196.3	0.10	0.10	0.05	0.15
37.9	3.4	125.3	0.10	0.10	0.10	0.15
41.9	3.7	125.3	0.10	0.10	0.10	0.15
41.9	1.3	183.8	0.10	0.10	0.05	0.15
45.0	1.4	183.8	0.10	0.10	0.05	0.15

Table 2b. Soil Model for Borehole R-19-002

Depth (ft)	Unit Shaft Resist (ksf)	Unit Toe Resist (ksf)	Skin Quake (in)	Toe Quake (in)	Skin Damping (s/ft)	Toe Damping (s/ft)
0.0	0.0	37.6	0.10	0.10	0.05	0.15
7.8	0.4	37.6	0.10	0.10	0.05	0.15
7.8	0.9	22.6	0.10	0.10	0.20	0.15
12.8	1.5	22.6	0.10	0.10	0.20	0.15
12.8	1.5	21.4	0.10	0.10	0.20	0.15
24.8	2.3	21.4	0.10	0.10	0.20	0.15
24.8	0.9	171.3	0.10	0.10	0.05	0.15
37.8	1.3	171.3	0.10	0.10	0.05	0.15
37.8	3.3	125.3	0.10	0.10	0.10	0.15
42.8	3.6	125.3	0.10	0.10	0.10	0.15
42.8	1.3	192.1	0.10	0.10	0.05	0.15
45.0	1.3	192.1	0.10	0.10	0.05	0.15

The unit shaft resistance calculated from the soil data is thought to occur long term (i.e. some amount of time after pile driving has ceased). The inverse of soil setup factor is used to calculate the SRD (Static Resistance to Driving) of the shaft resistance in the various soil layers and are calculated from:

$$\text{SRD} = (\text{Long Term Static Resistance}) / (\text{Setup Factor})$$

For example, for the sandy silts, we assumed a setup factor of 2.0, and therefore only 50% of the long-term shaft resistance will act on the shaft during impact driving. The assumption is that after pile driving is finished and a sufficiently long waiting period has passed, soil setup will bring the capacity back to its long-term value. For the end bearing, we did not assume any loss or gain of resistance during driving. Gain/loss factors of 0.5/1.0 and 1.0/1.0 are presented for each impact drive analysis.

Discussion of Results

Soil Boring R-19-001 - Delmag D36

The proposed D36 single acting (open ended) diesel hammer system should be able to install the proposed CISS piles to the desired penetration of 40 ft below existing ground level. The D36 was analyzed at 100% energy setting (Fuel # 4) to final penetration of 40 ft. This was done to limit the maximum stress and the driving capabilities of the driving system. Lower fuel settings may be used, if required.

Maximum compression stress levels were calculated to reach **24.8** ksi and tension stress levels were estimated to be **4.9** ksi. The compression and tension stresses are below the recommended compression stress level of 90% of F_y (**40.5** ksi). Based on the soil model, the end of drive blow counts is expected to be close to **23** bpf at a **7.3** ft stroke height. The long-term static soil resistance at the expected penetration is expected to be close to **560** kips.

This long-term static resistance will vary depending on soil conditions. Slight changes on the bearing soils could result in lower resistance values than those specified. Estimated driving time of this pile is expected to be close to **8** minutes. This is just an estimate to have a gross idea of the installation time.

Soil Boring R-19-002 - Delmag D36

The proposed D36 single acting (open ended) diesel hammer system should be able to install the proposed CISS piles to the desired penetration of 40 ft below existing ground level. The D36 was analyzed at 100% energy setting (Fuel # 4) to final penetration of 40 ft. This was done to limit the maximum stress and the driving capabilities of the driving system. Lower fuel settings may be used, if required.

Maximum compression stress levels were calculated to reach **24.6** ksi and tension stress levels were estimated to be **4.8** ksi. The compression and tension stresses are below the recommended compression stress level of 90% of F_y (**40.5** ksi). Based on the soil model, the end of drive blow counts is expected to be close to **22** bpf at a **7.3** ft stroke height. The long-term static soil resistance at the expected penetration is expected to be close to **540** kips.

This long-term static resistance will vary depending on soil conditions. Slight changes on the bearing soils could result in lower resistance values than those specified. Estimated driving time of this pile is expected to be close to **7** minutes. This is just an estimate to have a gross idea of the installation time.

Soil Boring R-19-001 - Delmag D46

The proposed D46 single acting (open ended) diesel hammer system should be able to install the proposed CISS piles to the desired penetration of 40 ft below existing ground level. The D46 was analyzed at 100% energy setting (Fuel # 4) to final penetration of 40 ft. This was done to limit the maximum stress and the driving capabilities of the driving system. Lower fuel settings may be used, if required.

Maximum compression stress levels were calculated to reach **27.2** ksi and tension stress levels were estimated to be **5.7** ksi. The compression and tension stresses are below the recommended compression stress level of 90% of F_y (**40.5** ksi). Based on the soil model, the end of drive blow counts is expected to be close to **20** bpf at an **8.0** ft stroke height. The long-term static soil resistance at the expected penetration is expected to be close to **560** kips.

This long-term static resistance will vary depending on soil conditions. Slight changes on the bearing soils could result in lower resistance values than those specified. Estimated driving time of this pile is expected to be close to **8** minutes. This is just an estimate to have a gross idea of the installation time.

Soil Boring R-19-002 - Delmag D46

The proposed D46 single acting (open ended) diesel hammer system should be able to install the proposed CISS piles to the desired penetration of 40 ft below existing ground level. The D46 was analyzed at 100% energy setting (Fuel # 4) to final penetration of 40 ft. This was done to limit the maximum stress and the driving capabilities of the driving system. Lower fuel settings may be used, if required.

Maximum compression stress levels were calculated to reach **27.0** ksi and tension stress levels were estimated to be **5.8** ksi. The compression and tension stresses are below the recommended compression stress level of 90% of F_y (**40.5** ksi). Based on the soil model, the end of drive blow counts is expected to be close to **19** bpf at a **7.9** ft stroke height. The long-term static soil resistance at the expected penetration is expected to be close to **540** kips.

This long-term static resistance will vary depending on soil conditions. Slight changes on the bearing soils could result in lower resistance values than those specified. Estimated driving time of this pile is expected to be close to **7** minutes. This is just an estimate to have a gross idea of the installation time.

Conclusions

The Delmag D36 and D46 single acting diesel hammer should be capable of driving the CISS 36-inch diameter pipe piles to the required penetrations of 40 ft at the proposed site. For these piles, blow counts of vary from **19** to **22** bpf with hammer strokes ranging from **7.3** to **8.0** ft and are expected at final pile penetration of 40.0 ft below existing ground level (Soil boring levels).

Driving criteria recommendations that will include different fuel settings, if necessary, can be provided after the test pile program. Initially, a maximum hammer setting was assumed to be used during installation as no major indications of high levels of stress were evidenced. Those considerations will be adjusted after testing.

Additional Considerations

The stresses computed by Wave Equation are considered average stresses. Stresses caused by bending, and/or non-uniform soil resistance effects can be additive and may exceed the pile material yield strength. Inclined piles may be subjected to additional bending stresses unless well supported by the leads and not forced to maintain the batter within a close tolerance when obstructions cause the piles to move out of the planned direction.

Please note that although the wave equation analysis is a valuable tool for the prediction of pile driving behavior, its results can only be as realistic as the input data and the validity of the basic mathematical models. Although we have made every effort to provide you with an accurate analysis, we know that there are limitations to this technology. Please refer to the disclaimers printed in Appendix A for further details. Any consideration due to the pile proximity is not included herein and is the responsibility of the geotechnical engineer of record.

We appreciate the opportunity to be of assistance to you on this project. Please contact our office should you have any questions regarding this submittal, or if we may be of further service.

Respectfully,
GRL Engineers, Inc.



Camilo Alvarez, P.E.
Senior Engineer



William Chambers, MIEAust CPEng
Senior Engineer

Appendix A
DESCRIPTION OF GRLWEAP
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Background

Since the early 1970's wave equation analysis of pile driving has become a standard tool for the preparation of pile installations by impact driving. The original concept had been developed by E.A.L. Smith of Raymond International during and after WWII. Basically, the analysis simulates what is happening in hammer, pile and soil during and immediately after the ram impact. It does this by modeling the system's components with masses, springs and dashpots and calculating the displacements and velocities of the masses and the forces in the springs. Stresses are determined from forces divided by cross sectional area at points that are roughly 1 m apart. This method of calculating the pile movements and stresses is an accurate solution of the wave equation, a differential equation that mathematicians enjoy working with.

If Smith would have only prepared a numerical solution for a mathematical problem, nobody would have paid attention. However, because of his intimate knowledge of pile driving, he was able to prepare a whole system including realistic recommendations for the various necessary input parameters, analysis procedure and output preparation.

GRLWEAP 2010 is the latest version of the wave equation program originally developed by GRL Engineers, Inc. for the Federal Highway Administration under the name WEAP. This code is now being maintained and distributed by Pile Dynamics, Inc. GRLWEAP 2010 builds on the successes of earlier versions and adds several enhancements and improvements.

Input Preparation

A realistic model of the hammer-pile-soil system has to represent all components participating in the impact pile driving event. Important components of the hammer are all of its striking parts (piston, piston rod, ram, ram point, etc.). The ram strikes the striker plate and hammer cushion (if present) which in turn pushes against the helmet and its inserts and then the pile top. For concrete piles a pile cushion transfers the forces to the pile top and the pile acts against the soil.

The cushion materials and the soil are energy absorbing and therefore are modeled non-elastically. All other parts of the system are simply modeled with linear springs. An exception are mechanical splices which also may be somewhat energy absorbing. Cushions and splices are therefore represented by non-linear springs.

The soil model consists of an elasto-plastic spring which represents soil stiffness and **ultimate capacity** of normally 1 m long soil segments. Actually, instead of entering a soil stiffness value, wave equation practice requires the **quake** (or the ultimate capacity value divided by the stiffness) to be entered. The quake is that pile displacement at which the soil resistance changes from elastic to plastic. The quake is typically 2.5 mm along the shaft of the pile; however, for displacement piles, the quake at the pile toe can take very large values such as 25 mm or more. Being a function of the pile penetration, the elasto-plastic soil spring represents the static soil resistance component. Following

hammer impact, the pile moves rapidly through the soil and for that reason there is also a dynamic component which is represented by a dashpot, i.e., a device that resists a faster pile motion with a higher resistance than a slower motion. Multiplying the pile velocity with an appropriate **damping factor** yields the dynamic or damping resistance of the soil.

In summary, the soil resistance at all shaft segments and at the pile toe are represented by stiffness (quake), ultimate capacity and a damping constant. While the user has to define the relative magnitude of the ultimate capacity values at various shaft segments, the quake and damping factors are automatically distributed with capacity proportional magnitudes.

Analysis Options

Smith's approach aimed at producing a relationship between blow count (also called driving resistance) and ultimate bearing capacity and pile stresses. This relationship would be valid for one particular depth and it is called the **Bearing Graph** (1st analysis option; see Fig. 1).

Construction control would be simplified if a driving criterion could be established as a function of hammer stroke, for all those hammer types which do not have a fixed stroke (diesel hammers, hydraulic hammers). This can be accomplished with the **Inspector's Chart** which relates blow count to stroke for a required ultimate capacity value (2nd analysis option; see Fig. 2).

A solution should also be available which shows calculated blow count vs. depth. To make this meaningful, the soil resistance must be input as a function of depth. Thus, while the bearing graph yields capacity from blow count (or blow count from capacity) at one depth, the

so-called **Driveability Analysis** (3rd analysis option; see Fig. 3) calculates blow count and stresses as a function of depth. To simplify the input preparation, GRLWEAP offers different static soil analysis routines. They estimate the static soil resistance from inputs like soil type (**ST** analysis), SPT blow counts (**SA**), Cone Penetration resistances (**CPT**) or friction angle and undrained shear strength (**API**). Each one of these four methods produces a unit shaft resistance (in ksf or kPa) and a unit end bearing (same units) as a function of depth. The unit shaft resistance is multiplied by the program with the pile perimeter and segment length to yield the ultimate capacity at a segment. The end bearing is multiplied by the pile toe area to yield ultimate pile toe capacity. Further modification of these capacity values is possible by multiplication with the so-called Gain-Loss factors which change the statically calculated long term capacity values to static resistance to driving (SRD); this process is further described below. Please note that the wave equation analyst sometimes modifies the results from the static soil analysis for conservatism.

Numerical Analysis

The program calculates the motions of the pile mass points and the forces in the springs for a large number of time steps. The time increment is roughly equal to $\frac{1}{2}$ of the time that the stress wave requires to travel through a pile segment (typically 0.1 ms). Numerical difficulties would only be expected if the time increment is chosen too large.

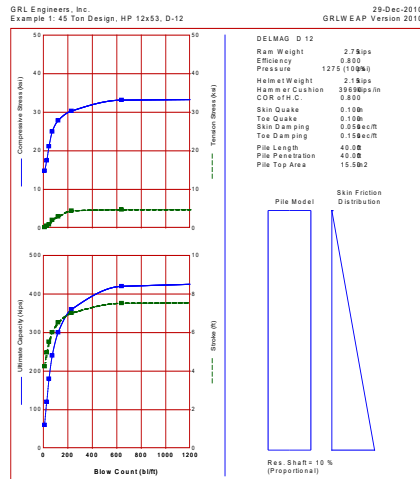


Fig. 1: Example of a Bearing Graph

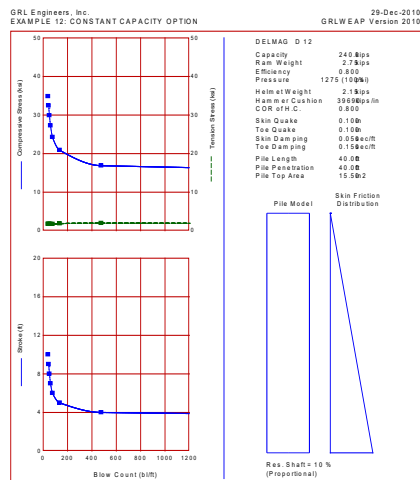


Fig. 2: Example of an Inspector's Chart

Hammer Models

GRLWEAP provides hammer data in its data file for most commonly used hammers. Additionally, where hammer manufacturers have provided information on driving systems, this information is also available in an organized fashion.

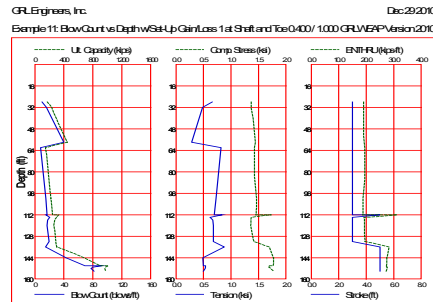


Fig. 3: Example of a Driveability Result

One of the most important hammer model inputs is the **hammer efficiency** which reduces the theoretically available hammer energy to a more realistic one. GRL has collected a large number of transferred energy values for a variety of hammers and has compared this measured information with the results of the wave equation analysis. While the data collected on construction sites shows a fair amount of scatter, using the hammer file data and standard efficiency values, GRLWEAP tends to calculate an average performance.

Time Dependency of Soil Resistance

(A) Soil Setup

Because excess positive pore pressures often develop during pile driving in fine grained soils (clays, silts) and even in fine sands, the capacity of a pile at the time of driving may often be less than the long term pile capacity. These pore pressures reduce the effective stress acting on the pile thereby reducing the soil resistance to pile penetration, and thus the pile capacity at the time of driving. As these pore pressures dissipate, the soil resistance acting on the pile increases as does the axial pile capacity. This phenomena is called soil setup or soil freeze. There are numerous other reasons for soil setup such as remolding and then realignment of clay particles, arching around the pile that reduces effective stresses during

pile installation in very dense sands, soil fatigue in over-consolidated clays etc.

(B) Relaxation

Relaxation, i.e., capacity reduction with time occurring primarily at the pile toe has been observed for piles driven into weathered shale. Relaxation may take several days after installation to fully develop. Where relaxation occurs, pile capacity estimates based upon the blow count obtained during initial driving or short term restrike tests can significantly overpredict long term pile capacity. Therefore, piles driven into shale should be tested after a minimum one week wait either statically or dynamically with particular emphasis on the first few blows. Relaxation has also been observed for displacement piles driven into dense saturated silts or fine sands due to a negative pore pressure effect at the pile toe. In general, relaxation occurs at the pile toe and is therefore relevant for end bearing piles. Restrike tests should be performed and compared with the records from early restrike blows in order to avoid dangerous over-prediction of capacity.

Static Resistance to Driving

When performing a driveability analysis, i.e. a prediction of blow count vs. depth, the first step is to calculate the long term bearing capacity of the pile by means of standard geotechnical methods (see input above for four different approaches provided by GRLWEAP). This analysis step yields the pile capacity that is expected to be present after setup and/or relaxation effects have taken place, i.e., some time after pile installation. Next, an estimate has to be made as to the change of capacity due to the pile driving effects. In general, one would expect the Static Resistance to Driving (SRD) to be less along the shaft than the long term shaft resistance and, in the case of relaxation, somewhat greater at the pile toe.

For the shaft resistance, the various layers through which the pile moves will have different setup factors and will therefore lose different percentages of shaft resistance due to driving. GRLWEAP therefore accepts **setup factors** for the various layers as an input. The program can then analyze either full loss of setup (continuous driving), partial setup (short driving interruption) or full setup (long driving interruption) depending on the so-called **gain-loss** factor which specifies the fraction of resistance with which the most sensitive layer is analyzed. The resistance of the other layers is adjusted under consideration of the setup factors. For example, a Gain-Loss Factor couple of 0.4/1.0 will analyze the most sensitive layer with only 40% of the long term shaft resistance and with 100% of the long term end bearing.

Analysis of Open Pile Profiles

Open ended pipe piles or H-piles which do not bear on rock may behave differently under dynamic and static loading conditions. Under dynamic loads the soil inside the pile or between its flanges may slip and produce internal friction while under static loads the plug may move with the pile, thereby creating end bearing over the full pile cross section. As a result both friction and end bearing components may be different under static and dynamic conditions. In wave equation practice it is common to analyze H-piles and pipe pile of modest diameters (say less than 500 mm diameter) as though they were plugged during driving which means that their end bearing is assumed to act on the gross area of the pile.

Stresses

GRLWEAP calculated stresses are average values both over the cross section and over a segment length.

Additional allowance has to be made for bending or for non-uniform contact stresses at the pile toe. Also, where changes of cross section occur over short distances it may be necessary to manually calculate stresses based on the wave equation calculated forces. To prevent damage it is therefore important to maintain good hammer-pile alignment, consider the effect of the hammer weight on the unsupported length of piles driven on a batter and to protect the pile toes using appropriate devices or an increased cross sectional area when driving into hard layers.

In the United States it has become generally acceptable to limit the dynamic installation stresses, averaged over the cross section, of driven piles to the following levels:

- 90% of yield strength for steel piles
- 85% of the concrete compressive strength - after subtraction of the effective prestress - for concrete piles in compression
- 100% of effective prestress plus $\frac{1}{2}$ of the concrete's tension strength for prestressed piles in tension
- 70% of the reinforcement strength for regularly reinforced concrete piles in tension
- 300% of the static design allowable stress for timber

The theory on which GRLWEAP is based is the one-dimensional wave equation. For that reason, stress predictions by the wave equation analysis can only be averages over the pile cross section. Thus, as aforementioned, bending stresses or stress concentrations due to non-uniform impact or uneven soil or rock resistance are not considered in these results. Stress maxima calculated by the wave equation are usually subjected to the same limits as those measured directly or calculated from

measurements by the Pile Driving Analyzer® ("PDA"). Note also that the dynamic stresses calculated by GRLWEAP and displayed in its final summary table may occur anywhere along the pile. When comparing these stresses with measured values, it must be assured that the stresses were calculated for the measurement location. The GRLWEAP Extrema Tables provide that information.

Factors of Safety

The wave equation works exclusively with the **ultimate pile bearing capacity**, R_{ult} or as it is called in AASHTO the **nominal resistance**. If this failure load were applied to the pile, then excessive settlements would occur. Therefore, it is absolutely necessary that the actual applied load, also called the design load, R_d (or working load or safe load), is less than R_{ult} . Thus, a factor of safety, F.S., has to be applied to the ultimate pile bearing capacity. Depending on site variability, uncertainty of loads and the number and type of pile tests performed, a variety of Factors of Safety have been proposed.

For example, for highway related projects and based on AASHTO specifications, the Federal Highway Administration proposed that a resistance factor of 0.5 be applied to the nominal resistance determined from a wave equation calculated bearing graph with the field observed blow count. Considering a typical combined load factor of 1.4 for bridges, the overall factor of safety would be $1.4/0.5 = 2.8$ when only the wave equation analysis in combination with the observed blow count is used. The resistance factor could be increased (the overall safety factor decreased) if additional dynamic or static testing were performed.

It should be mentioned that all capacity determination methods should be

combined with soil exploration and static pile analysis. Also, specifications are occasionally updated and therefore the latest version should be consulted for the appropriate factors of safety. Codes, among them IBC, PDCA, ASCE, or specifications issued by State Departments of Transportation specify different factors of safety.

Applications

(A) Capacity Determination

The GRLWEAP program calculates a relationship between bearing capacity, pile stress and blow count. As mentioned earlier, this relationship is often called the "bearing graph." Once the blow count is known from pile installation logs, the bearing graph yields the bearing capacity. This approach requires no measurements other than blow count and, of course, the assumption that the driving system and soil performed as analyzed. The wave equation is particularly useful during the design stage of a project for the selection of hammer, cushion and pile size.

(B) Determination of Driving Criterion

Given a required ultimate capacity (design load times Factor of Safety), the bearing graph will indicate the associated blow count. Thus, driving the pile to this blow count will, according to the wave equation, provide for sufficient capacity at the time of testing. The bearing graph will also indicate the maximum tension and compression stresses occurring at that same time. Note that the length of pile, depth of penetration and many other factors have to be reasonably close to the analyzed situation for a reliable prediction of the blow count and stresses.

(C) Prediction of Driveability

In general, not only the final blow count and pile stresses are of interest, but also those occurring during the installation at various other depths. This can be conveniently calculated with

GRLWEAP's driveability option. Thus, should a soft or hard layer cause excessive tension or compression stresses or unacceptable blow counts, then the driveability analysis option allows the analyst to modify cushion stiffness, hammer stroke and/or other driving system parameters to produce a safe and optimal prescription for pile installation.

Advanced Options

Under certain circumstances and/or for greater realism, the GRLWEAP analyst may choose to perform the calculations with a variety of special options such as the following.

- (a) Residual Stress Analysis is required for Monotube[®] piles and is recommended for flexible friction piles in general. These types of piles remain in a compressed state between hammer blows a phenomenon which alters their driving behavior. Comparison analyses generally indicate slightly lower blow counts and higher stresses than standard Smith type analyses.
- (b) Bearing graph with constant shaft resistance or end bearing options, in contrast to the standard procedure which increases shaft resistance and end bearing proportionally.
- (c) Variable pressure diesel hammer analyses calculate iteratively the combustion pressure needed to maintain a fixed stroke. This is of interest when the stroke has been observed in the field to substantially differ from the standard value. (Caution has to be exercised with this option when the observed stroke is higher than indicated by a standard analysis. Should the higher stroke be due to pre-ignition which lowers energy

- transfer while increasing the stroke then a non-conservative calculated blow count will result.
- (d) Single stroke diesel hammer analysis will not attempt to match upstroke with downstroke; this is a possible option for Inspector Chart analyses.
 - (e) Bending analysis (offshore wave version only) superimposes static bending due to hammer weight on inclined piles with dynamic stresses. The calculated bending stresses take the place of the tension stresses for output and, therefore may not be suitable for concrete piles.
 - (f) Fatigue table option (offshore wave version only) produces tables of stress ranges and number of occurrences based on driveability analyses.
 - (g) Hammer position options (offshore wave version only) for intermediate or pile bottom hammer location.
 - (h) Two pile option is for situations where the stress propagates in two branches as it happens in mandrel driven piles.

Additional Design Considerations

Numerous factors have to be considered in pile foundation design. Some of these considerations include

- loss of shaft resistance due to scour or other effects; the situation analyzed for pile driving would include all of the resistance that later may be lost due to scour.
- loss of structural pile strength due to additional bending loads, buckling (the dynamic loads generally do not cause buckling even though they may exceed the buckling strength of the pile section), corrosion, etc.
- lateral and uplift loading requirements
- settlements
- additional pile loading from settling soil layers causing downdrag or negative skin friction
- effective stress changes (due to changes in water table, excavations, fills or other changes in overburden)
- selecting an appropriate static analysis method for a given pile site. The static analysis methods built-in to GRLWEAP should be correlated to specific soil types or regions with static and dynamic load tests.

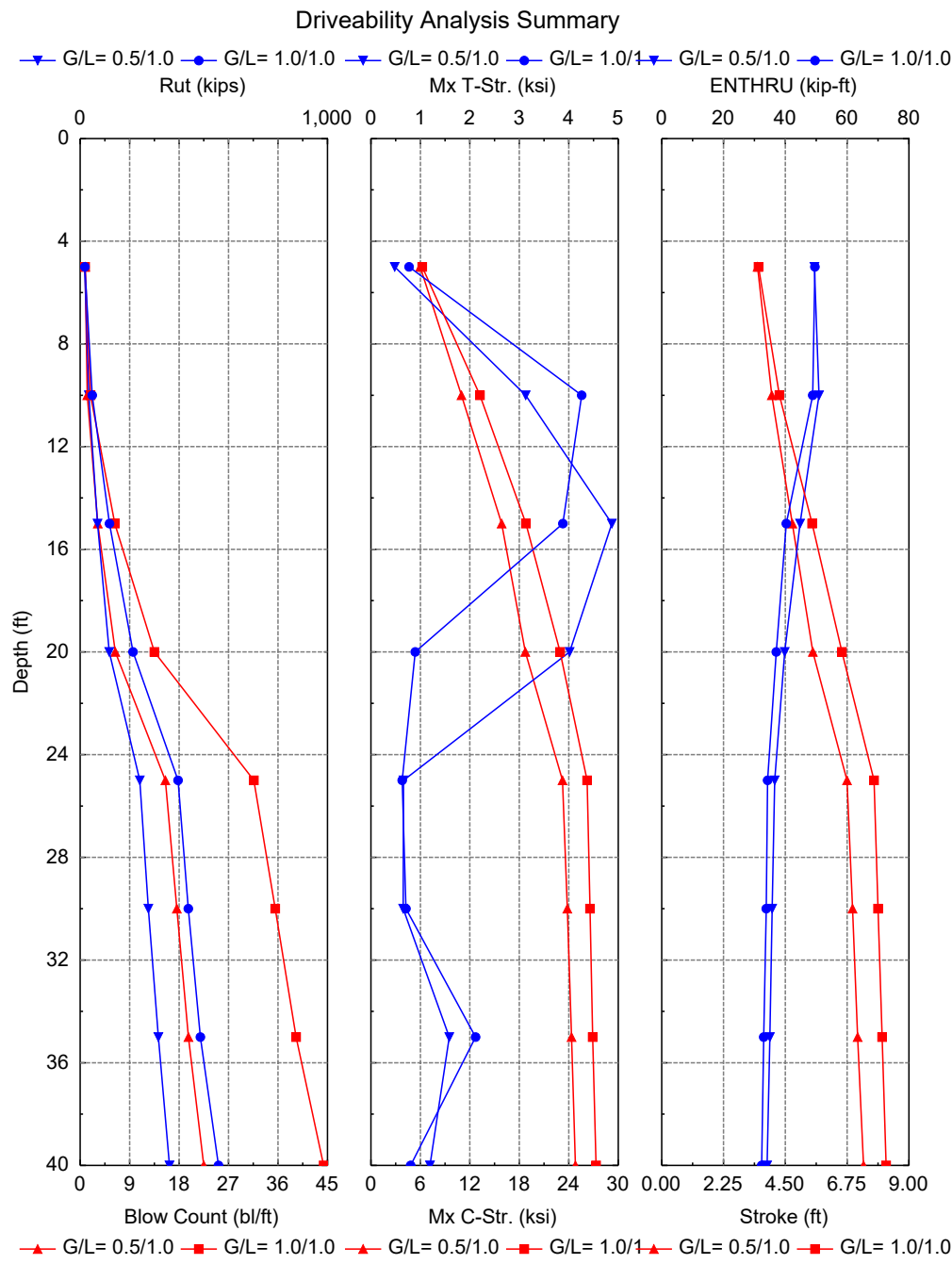
These and other factors cannot be evaluated as part of the wave equation analysis. The foundation designer should determine if these or any other considerations are applicable to this project and the foundation design.

Reliability of Wave Equation Analysis Results

The results calculated by the wave equation analysis program depend on a variety of hammer, pile and soil input parameters. Although attempts have been made to base the analysis on the best available information, actual field conditions may vary and therefore stresses and blow counts may differ from the predictions reported. Capacity predictions derived from wave equation analyses should use restrike information. However, because of the uncertainties associated with restrike blow counts and restrike hammer energies, correlations of GRLWEAP capacity results from restrike blow counts with static test capacities have often displayed considerable scatter.

Appendix B

GRLWEAP Analyses Results



Gain/Loss Factor at Shaft/Toe = 0.500/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow bl/ft	CtMx ksi	C-StrMx ksi	T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	17.9	5.0	12.9	0.9	5.980	0.482	3.49	49.4	D 36	
10.0	35.2	23.4	11.8	1.3	10.979	3.130	4.01	50.9	D 36	
15.0	70.3	58.4	11.8	3.2	15.864	4.871	4.75	44.7	D 36	
20.0	117.8	106.0	11.8	6.4	18.735	4.018	5.50	39.8	D 36	
25.0	241.3	165.2	76.0	15.5	23.237	0.659	6.75	36.6	D 36	
30.0	275.8	199.8	76.0	17.6	23.827	0.653	6.95	35.8	D 36	
35.0	316.4	240.4	76.0	19.7	24.340	1.584	7.13	35.0	D 36	
40.0	361.1	312.6	48.5	22.5	24.801	1.194	7.34	34.1	D 36	

Total driving time: 8 minutes; Total Number of Blows: 377 (starting at penetration 5.0 ft)

Gain/Loss Factor at Shaft/Toe = 1.000/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow bl/ft	CtMx ksi	C-StrMx ksi	T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	18.9	6.0	12.9	0.9	6.223	0.772	3.52	49.6	D 36	
10.0	48.7	36.9	11.8	1.9	13.223	4.260	4.28	48.9	D 36	
15.0	118.8	107.0	11.8	6.3	18.810	3.880	5.47	40.3	D 36	
20.0	213.8	202.0	11.8	13.5	22.945	0.896	6.56	37.1	D 36	
25.0	396.1	320.1	76.0	31.6	26.226	0.633	7.73	34.2	D 36	
30.0	437.6	361.5	76.0	35.5	26.575	0.709	7.87	33.8	D 36	
35.0	486.3	410.3	76.0	39.3	26.900	2.117	8.02	33.0	D 36	
40.0	559.2	510.7	48.5	44.3	27.314	0.804	8.17	32.3	D 36	

Total driving time: 18 minutes; Total Number of Blows: 753 (starting at penetration 5.0 ft)

GRLWEAP: Wave Equation Analysis of Pile Foundations

36-inch x 0.5 x 40 ft D36 + R-19-001

4/12/2024

GRL ENGINEERS, INC

GRLWEAP 14.1.20.1

ABOUT THE WAVE EQUATION ANALYSIS RESULTS

The GRLWEAP program simulates the behavior of a preformed pile driven by either an impact hammer or a vibratory hammer. The program is based on mathematical models, which describe motion and forces of hammer, driving system, pile and soil under the hammer action. Under certain conditions, the models only crudely approximate, often complex, dynamic situations.

A wave equation analysis generally relies on input data, which represents normal situations. In particular, the hammer data file supplied with the program assumes that the hammer is in good working order. All of the input data selected by the user may be the best available information at the time when the analysis is performed. However, input data and therefore results may significantly differ from actual field conditions.

Therefore, the program authors recommend prudent use of the GRLWEAP results. Soil response and hammer performance should be verified by static and/or dynamic testing and measurements. Estimates of bending or other local stresses (e.g., helmet or clamp contact, uneven rock surfaces etc.), prestress effects and others must also be accounted for by the user.

The calculated capacity-blow count relationship, i.e. the bearing graph, should be used in conjunction with observed blow counts for the capacity assessment of a driven pile. Soil setup occurring after pile installation may produce bearing capacity values that differ substantially from those expected from a wave equation analysis due to soil setup or relaxation. This is particularly true for pile driven with vibratory hammers. The GRLWEAP user must estimate such effects and should also use proper care when applying blow counts from restrike because of the variability of hammer energy, soil resistance and blow count during early restriking.

Finally, the GRLWEAP capacities are ultimate values. They **MUST** be reduced by means of an appropriate factor of safety to yield a design or working load. The selection of a factor of safety should consider the quality of the construction control, the variability of the site conditions, uncertainties in the loads, the importance of structure and other factors.

PILE INPUT

Uniform Pile		Pile Type:	Open-End Pipe
Pile Length: (ft)	40.000	Pile Penetration: (ft)	40.000
Pile Size: (ft)	3.00	Toe Area: (in ²)	55.76

Pile Profile

Lb Top ft	X-Area in ²	E-Modulus ksi	Spec. Wt lb/ft ³	Perim. ft	Crit. Index -
0.0	55.8	30,000.0	492.0	9.4	0
40.0	55.8	30,000.0	492.0	9.4	0

HAMMER INPUT

ID	16	Made By:	DELMAG
Model	D 36	Type:	OED

Hammer Data

ID	Ram Wt kips	Ram L. in	Ram Ar. in ²	Rtd. Stk ft	Effic. -	Rtd. Energy kip-ft
16	7.930	106.7	302.0	10.6	0.80	83.8

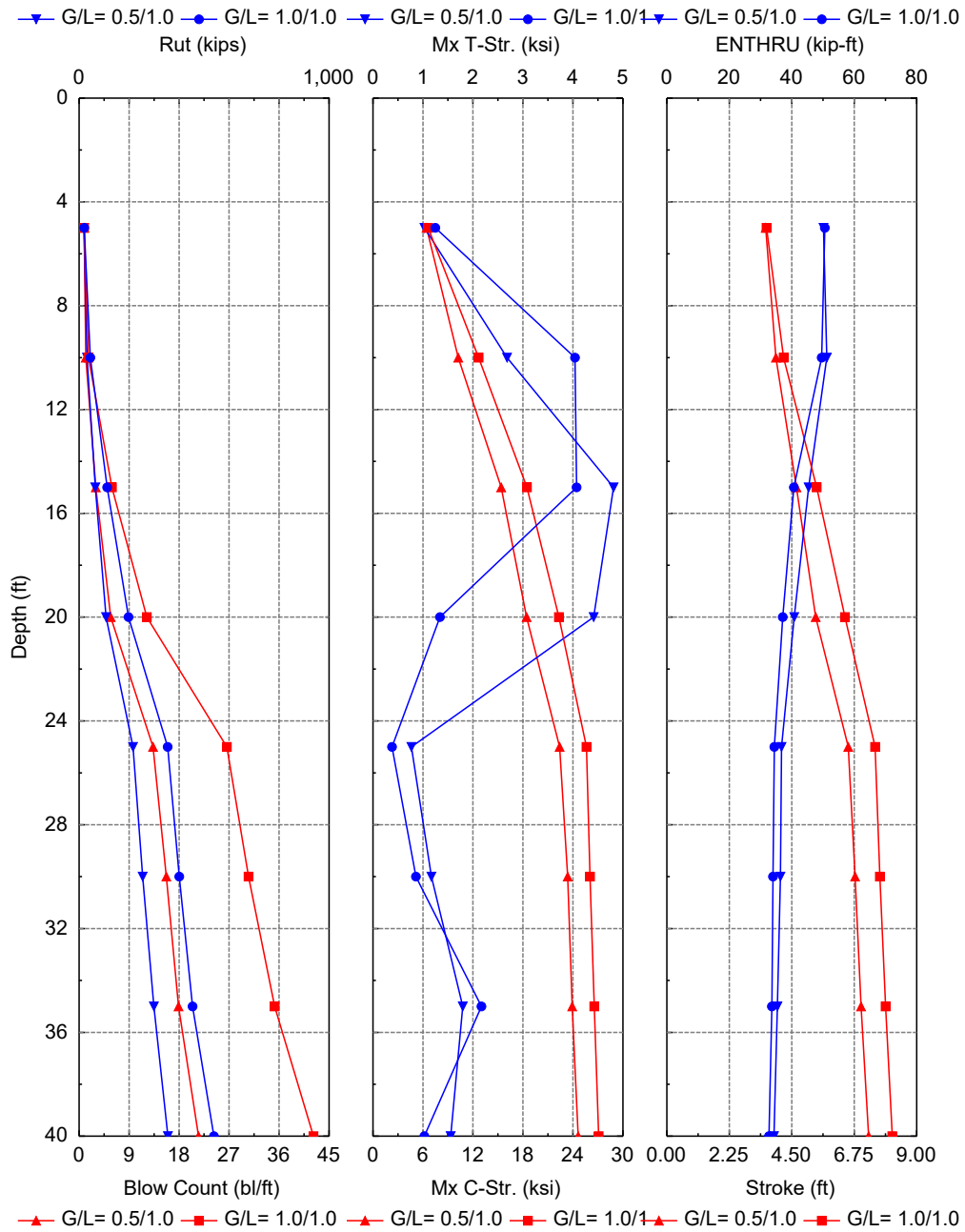
DRIVE SYSTEM FOR DELMAG D 36-OED

Type	X-Area in ²	E-Modulus ksi	Thickness in	COR	Round-out in	Stiffness kips/in
Hammer C.	415.000	470.000	3.500	0.800	0.120	55729.085
Helmet Wt.	3.800	kips				

SOIL RESISTANCE DISTRIBUTION

Depth ft	Unit Rs ksf	Unit Rt ksf	Qs in	Qt in	Js s/ft	Jt s/ft	Set. F. -	Limit D. ft	Set. T. Hours	EB Area in ²
0.0	0.0	33.4	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8
7.9	0.4	33.4	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8
7.9	1.0	30.5	0.10	0.10	0.20	0.15	2.0	6.6	168.0	55.8
24.9	2.8	30.5	0.10	0.10	0.20	0.15	2.0	6.6	168.0	55.8
24.9	0.8	196.3	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8
37.9	1.2	196.3	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8
37.9	3.4	125.3	0.10	0.10	0.10	0.15	1.5	6.6	24.0	55.8
41.9	3.7	125.3	0.10	0.10	0.10	0.15	1.5	6.6	24.0	55.8
41.9	1.3	183.8	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8
45.0	1.4	183.8	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8

Driveability Analysis Summary



Gain/Loss Factor at Shaft/Toe = 0.500/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow bl/ft	CtMx ksi	C-StrMx ksi	T-Str. ft	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	19.6	5.0	14.6	0.9	6.431	1.031	3.56	50.2	D 36	
10.0	31.7	23.0	8.8	1.2	10.242	2.683	3.93	51.2	D 36	
15.0	65.1	56.8	8.3	3.0	15.392	4.814	4.67	45.4	D 36	
20.0	107.9	99.6	8.3	5.7	18.444	4.415	5.36	40.8	D 36	
25.0	215.7	149.4	66.3	13.3	22.404	0.769	6.53	36.7	D 36	
30.0	254.3	188.0	66.3	15.7	23.378	1.169	6.78	36.4	D 36	
35.0	299.0	232.6	66.3	17.9	23.910	1.796	6.99	35.4	D 36	
40.0	355.3	306.8	48.5	21.5	24.638	1.555	7.27	34.3	D 36	

Total driving time: 7 minutes; Total Number of Blows: 340 (starting at penetration 5.0 ft)

Gain/Loss Factor at Shaft/Toe = 1.000/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow bl/ft	CtMx ksi	C-StrMx ksi	T-Str. ft	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	20.6	6.0	14.6	0.9	6.619	1.247	3.59	50.6	D 36	
10.0	44.9	36.1	8.8	1.7	12.689	4.041	4.20	49.6	D 36	
15.0	112.2	103.9	8.3	5.9	18.480	4.074	5.39	40.7	D 36	
20.0	197.6	189.3	8.3	12.2	22.311	1.341	6.41	37.1	D 36	
25.0	354.2	287.8	66.3	26.6	25.626	0.382	7.50	34.4	D 36	
30.0	400.5	334.2	66.3	30.5	26.057	0.860	7.67	34.0	D 36	
35.0	454.1	387.7	66.3	35.2	26.565	2.171	7.88	33.6	D 36	
40.0	539.2	490.7	48.5	42.2	27.086	1.027	8.12	32.7	D 36	

Total driving time: 15 minutes; Total Number of Blows: 668 (starting at penetration 5.0 ft)

GRLWEAP: Wave Equation Analysis of Pile Foundations

36-inch x 0.5 x 40 ft D36 + R-19-002

4/12/2024

GRL ENGINEERS, INC

GRLWEAP 14.1.20.1

ABOUT THE WAVE EQUATION ANALYSIS RESULTS

The GRLWEAP program simulates the behavior of a preformed pile driven by either an impact hammer or a vibratory hammer. The program is based on mathematical models, which describe motion and forces of hammer, driving system, pile and soil under the hammer action. Under certain conditions, the models only crudely approximate, often complex, dynamic situations.

A wave equation analysis generally relies on input data, which represents normal situations. In particular, the hammer data file supplied with the program assumes that the hammer is in good working order. All of the input data selected by the user may be the best available information at the time when the analysis is performed. However, input data and therefore results may significantly differ from actual field conditions.

Therefore, the program authors recommend prudent use of the GRLWEAP results. Soil response and hammer performance should be verified by static and/or dynamic testing and measurements. Estimates of bending or other local stresses (e.g., helmet or clamp contact, uneven rock surfaces etc.), prestress effects and others must also be accounted for by the user.

The calculated capacity-blow count relationship, i.e. the bearing graph, should be used in conjunction with observed blow counts for the capacity assessment of a driven pile. Soil setup occurring after pile installation may produce bearing capacity values that differ substantially from those expected from a wave equation analysis due to soil setup or relaxation. This is particularly true for pile driven with vibratory hammers. The GRLWEAP user must estimate such effects and should also use proper care when applying blow counts from restrike because of the variability of hammer energy, soil resistance and blow count during early restriking.

Finally, the GRLWEAP capacities are ultimate values. They **MUST** be reduced by means of an appropriate factor of safety to yield a design or working load. The selection of a factor of safety should consider the quality of the construction control, the variability of the site conditions, uncertainties in the loads, the importance of structure and other factors.

PILE INPUT

Uniform Pile		Pile Type:	Open-End Pipe
Pile Length: (ft)	40.000	Pile Penetration: (ft)	40.000
Pile Size: (ft)	3.00	Toe Area: (in ²)	55.76

Pile Profile

Lb Top ft	X-Area in ²	E-Modulus ksi	Spec. Wt lb/ft ³	Perim. ft	Crit. Index -
0.0	55.8	30,000.0	492.0	9.4	0
40.0	55.8	30,000.0	492.0	9.4	0

HAMMER INPUT

ID	16	Made By:	DELMAG
Model	D 36	Type:	OED

Hammer Data

ID	Ram Wt kips	Ram L. in	Ram Ar. in ²	Rtd. Stk ft	Effic. -	Rtd. Energy kip-ft
16	7.930	106.7	302.0	10.6	0.80	83.8

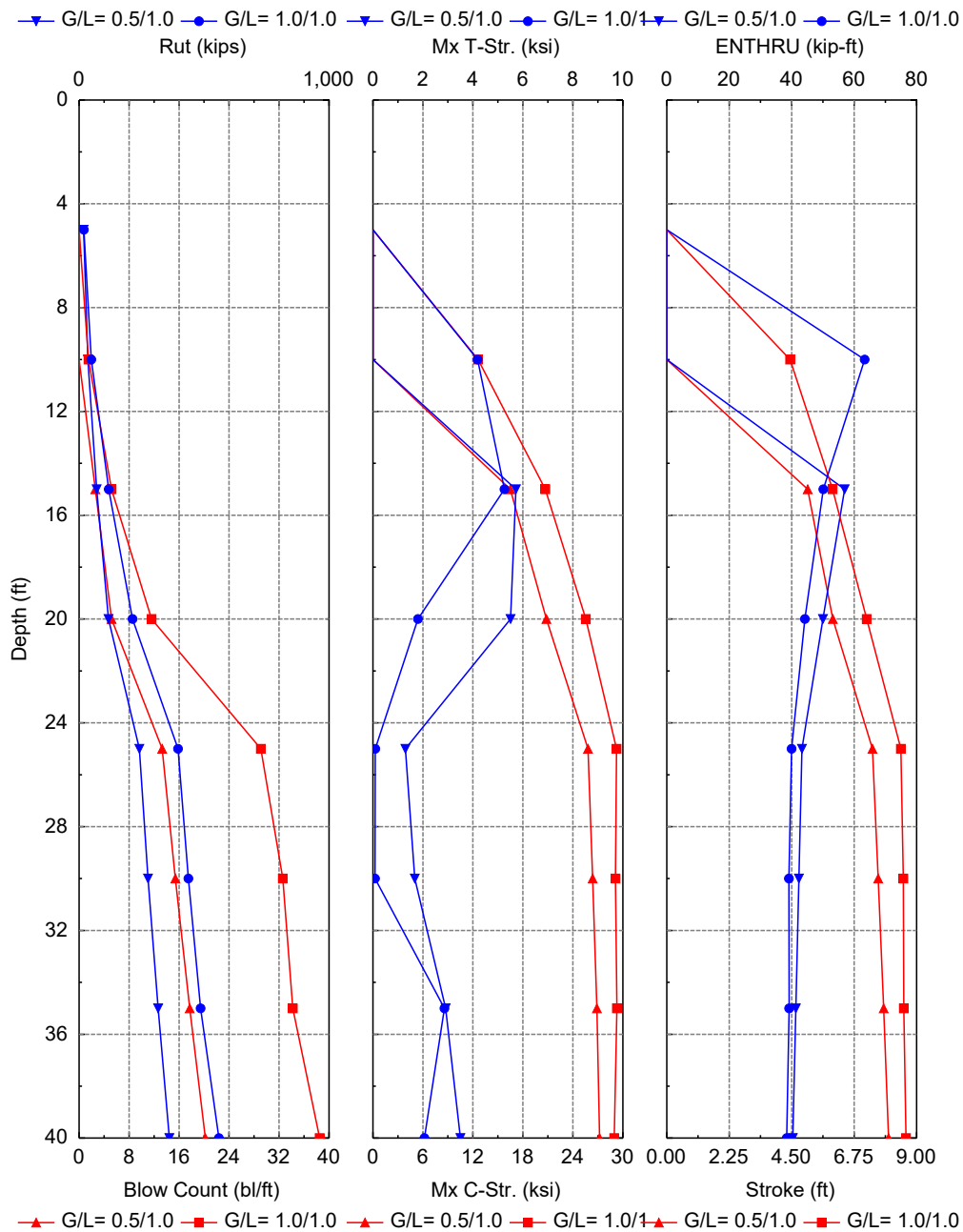
DRIVE SYSTEM FOR DELMAG D 36-OED

Type	X-Area in ²	E-Modulus ksi	Thickness in	COR	Round-out in	Stiffness kips/in
-				-		
Hammer C.	415.000	470.000	3.500	0.800	0.120	55729.085
Helmet Wt.	3.800	kips				

SOIL RESISTANCE DISTRIBUTION

Depth ft	Unit Rs ksf	Unit Rt ksf	Qs in	Qt in	Js s/ft	Jt s/ft	Set. F. -	Limit D. ft	Set. T. Hours	EB Area in ²
0.0	0.0	37.6	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8
7.8	0.4	37.6	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8
7.8	0.9	22.6	0.10	0.10	0.20	0.15	2.0	6.6	168.0	55.8
12.8	1.5	22.6	0.10	0.10	0.20	0.15	2.0	6.6	168.0	55.8
12.8	1.5	21.4	0.10	0.10	0.20	0.15	2.0	6.6	168.0	55.8
24.8	2.3	21.4	0.10	0.10	0.20	0.15	2.0	6.6	168.0	55.8
24.8	0.9	171.3	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8
37.8	1.3	171.3	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8
37.8	3.3	125.3	0.10	0.10	0.10	0.15	1.5	6.6	24.0	55.8
42.8	3.6	125.3	0.10	0.10	0.10	0.15	1.5	6.6	24.0	55.8
42.8	1.3	192.1	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8
45.0	1.3	192.1	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8

Driveability Analysis Summary



Gain/Loss Factor at Shaft/Toe = 0.500/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow bl/ft	CtMx ksi	C-StrMx ksi	T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	17.9	5.0	12.9	0.0	0.000	0.000	0.00	0.0	0.0	D 46
10.0	35.2	23.4	11.8	0.0	0.000	0.000	0.00	0.0	0.0	D 46
15.0	70.3	58.4	11.8	2.6	16.485	5.721	5.07	57.0	57.0	D 46
20.0	117.8	106.0	11.8	5.2	20.824	5.506	5.98	50.0	50.0	D 46
25.0	241.3	165.2	76.0	13.3	25.816	1.297	7.41	43.3	43.3	D 46
30.0	275.8	199.8	76.0	15.4	26.373	1.673	7.61	42.3	42.3	D 46
35.0	316.4	240.4	76.0	17.7	26.885	2.910	7.81	41.3	41.3	D 46
40.0	361.1	312.6	48.5	20.2	27.212	3.501	7.99	40.4	40.4	D 46

Total driving time: 8 minutes; Total Number of Blows: 321 (starting at penetration 5.0 ft)

Gain/Loss Factor at Shaft/Toe = 1.000/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow bl/ft	CtMx ksi	C-StrMx ksi	T-Str. ksi	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	18.9	6.0	12.9	0.0	0.000	0.000	0.00	0.0	0.0	D 46
10.0	48.7	36.9	11.8	1.5	12.630	4.189	4.44	63.3	63.3	D 46
15.0	118.8	107.0	11.8	5.2	20.688	5.261	5.97	50.1	50.1	D 46
20.0	213.8	202.0	11.8	11.6	25.549	1.802	7.20	44.2	44.2	D 46
25.0	396.1	320.1	76.0	29.1	29.245	0.096	8.43	39.9	39.9	D 46
30.0	437.6	361.5	76.0	32.6	29.107	0.082	8.52	39.1	39.1	D 46
35.0	486.3	410.3	76.0	34.2	29.289	2.862	8.54	39.2	39.2	D 46
40.0	559.2	510.7	48.5	38.5	28.968	2.059	8.62	38.4	38.4	D 46

Total driving time: 16 minutes; Total Number of Blows: 666 (starting at penetration 5.0 ft)

GRLWEAP: Wave Equation Analysis of Pile Foundations

36-inch x 0.5 x 40 ft D46 + R-19-001

4/12/2024

GRL ENGINEERS, INC

GRLWEAP 14.1.20.1

ABOUT THE WAVE EQUATION ANALYSIS RESULTS

The GRLWEAP program simulates the behavior of a preformed pile driven by either an impact hammer or a vibratory hammer. The program is based on mathematical models, which describe motion and forces of hammer, driving system, pile and soil under the hammer action. Under certain conditions, the models only crudely approximate, often complex, dynamic situations.

A wave equation analysis generally relies on input data, which represents normal situations. In particular, the hammer data file supplied with the program assumes that the hammer is in good working order. All of the input data selected by the user may be the best available information at the time when the analysis is performed. However, input data and therefore results may significantly differ from actual field conditions.

Therefore, the program authors recommend prudent use of the GRLWEAP results. Soil response and hammer performance should be verified by static and/or dynamic testing and measurements. Estimates of bending or other local stresses (e.g., helmet or clamp contact, uneven rock surfaces etc.), prestress effects and others must also be accounted for by the user.

The calculated capacity-blow count relationship, i.e. the bearing graph, should be used in conjunction with observed blow counts for the capacity assessment of a driven pile. Soil setup occurring after pile installation may produce bearing capacity values that differ substantially from those expected from a wave equation analysis due to soil setup or relaxation. This is particularly true for pile driven with vibratory hammers. The GRLWEAP user must estimate such effects and should also use proper care when applying blow counts from restrike because of the variability of hammer energy, soil resistance and blow count during early restriking.

Finally, the GRLWEAP capacities are ultimate values. They **MUST** be reduced by means of an appropriate factor of safety to yield a design or working load. The selection of a factor of safety should consider the quality of the construction control, the variability of the site conditions, uncertainties in the loads, the importance of structure and other factors.

PILE INPUT

Uniform Pile		Pile Type:	Open-End Pipe
Pile Length: (ft)	40.000	Pile Penetration: (ft)	40.000
Pile Size: (ft)	3.00	Toe Area: (in ²)	55.76

Pile Profile

Lb Top ft	X-Area in ²	E-Modulus ksi	Spec. Wt lb/ft ³	Perim. ft	Crit. Index -
0.0	55.8	30,000.0	492.0	9.4	0
40.0	55.8	30,000.0	492.0	9.4	0

HAMMER INPUT

ID	22	Made By:	DELMAG
Model	D 46	Type:	OED

Hammer Data

ID	Ram Wt kips	Ram L. in	Ram Ar. in ²	Rtd. Stk ft	Effic. -	Rtd. Energy kip-ft
22	10.140	137.8	302.0	10.6	0.80	107.1

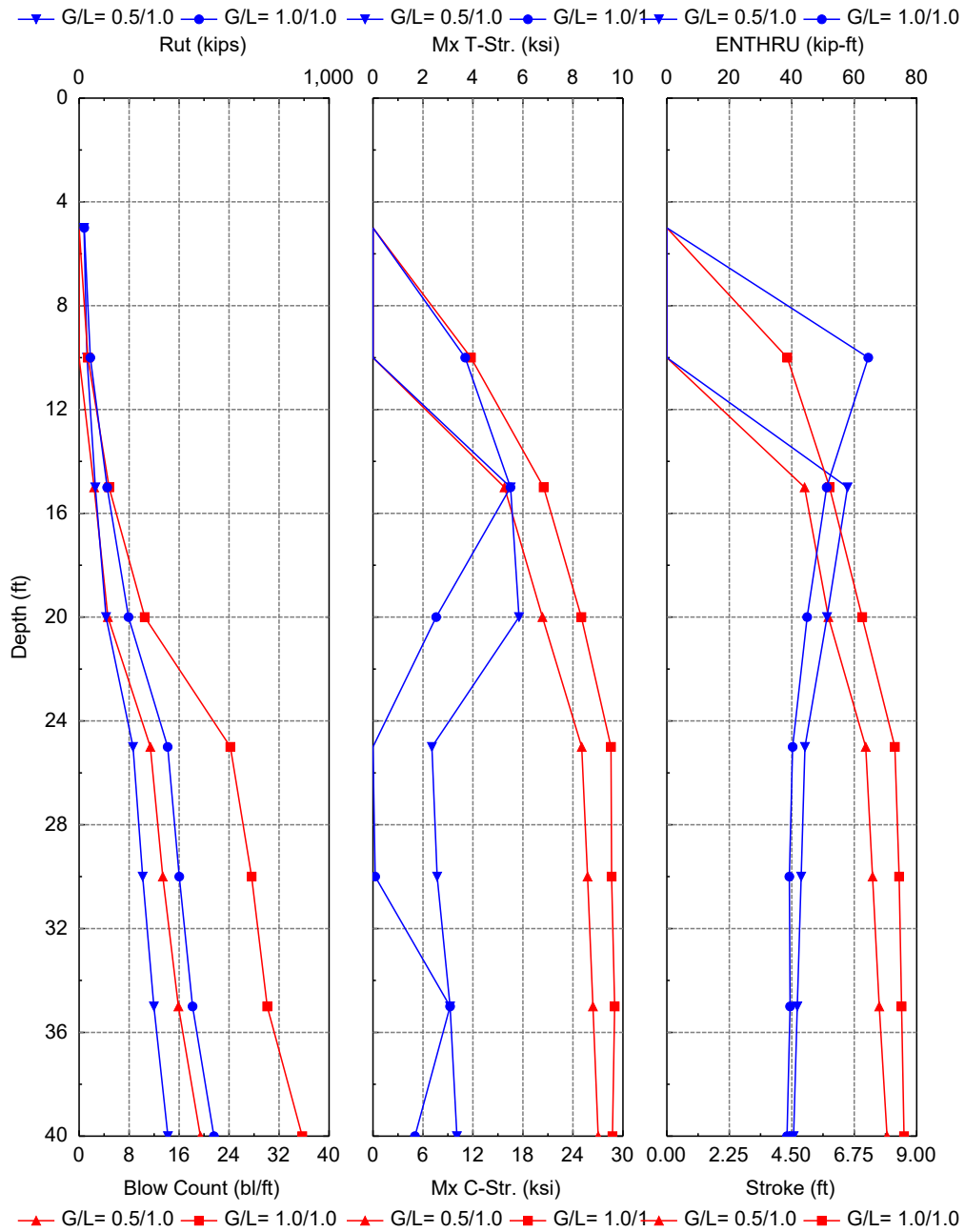
DRIVE SYSTEM FOR DELMAG D 46-OED

Type	X-Area in ²	E-Modulus ksi	Thickness in	COR	Round-out in	Stiffness kips/in
Hammer C.	415.000	470.000	3.500	0.800	0.120	55729.085
Helmet Wt.	3.800	kips				

SOIL RESISTANCE DISTRIBUTION

Depth ft	Unit Rs ksf	Unit Rt ksf	Qs in	Qt in	Js s/ft	Jt s/ft	Set. F. -	Limit D. ft	Set. T. Hours	EB Area in ²
0.0	0.0	33.4	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8
7.9	0.4	33.4	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8
7.9	1.0	30.5	0.10	0.10	0.20	0.15	2.0	6.6	168.0	55.8
24.9	2.8	30.5	0.10	0.10	0.20	0.15	2.0	6.6	168.0	55.8
24.9	0.8	196.3	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8
37.9	1.2	196.3	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8
37.9	3.4	125.3	0.10	0.10	0.10	0.15	1.5	6.6	24.0	55.8
41.9	3.7	125.3	0.10	0.10	0.10	0.15	1.5	6.6	24.0	55.8
41.9	1.3	183.8	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8
45.0	1.4	183.8	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8

Driveability Analysis Summary



Gain/Loss Factor at Shaft/Toe = 0.500/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow bl/ft	CtMx ksi	C-StrMx ksi	T-Str. ft	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	19.6	5.0	14.6	0.0	0.000	0.000	0.00	0.0	0.0	D 46
10.0	31.7	23.0	8.8	0.0	0.000	0.000	0.00	0.0	0.0	D 46
15.0	65.1	56.8	8.3	2.4	15.783	5.506	4.96	57.8	57.8	D 46
20.0	107.9	99.6	8.3	4.6	20.340	5.842	5.82	51.3	51.3	D 46
25.0	215.7	149.4	66.3	11.4	25.056	2.358	7.16	44.2	44.2	D 46
30.0	254.3	188.0	66.3	13.4	25.754	2.567	7.41	43.0	43.0	D 46
35.0	299.0	232.6	66.3	15.9	26.397	3.086	7.65	41.7	41.7	D 46
40.0	355.3	306.8	48.5	19.4	27.037	3.360	7.93	40.6	40.6	D 46

Total driving time: 7 minutes; Total Number of Blows: 286 (starting at penetration 5.0 ft)

Gain/Loss Factor at Shaft/Toe = 1.000/1.000

Depth ft	Rut kips	Rshaft kips	Rtoe kips	Blow bl/ft	CtMx ksi	C-StrMx ksi	T-Str. ft	Stroke ft	ENTHRU kip-ft	Hammer -
5.0	20.6	6.0	14.6	0.0	0.000	0.000	0.00	0.0	0.0	D 46
10.0	44.9	36.1	8.8	1.4	11.757	3.691	4.34	64.5	64.5	D 46
15.0	112.2	103.9	8.3	4.8	20.486	5.504	5.86	51.2	51.2	D 46
20.0	197.6	189.3	8.3	10.5	25.007	2.528	7.04	44.9	44.9	D 46
25.0	354.2	287.8	66.3	24.2	28.571	0.000	8.21	40.3	40.3	D 46
30.0	400.5	334.2	66.3	27.6	28.641	0.091	8.37	39.2	39.2	D 46
35.0	454.1	387.7	66.3	30.1	29.012	3.087	8.45	39.5	39.5	D 46
40.0	539.2	490.7	48.5	35.7	28.753	1.680	8.54	38.5	38.5	D 46

Total driving time: 14 minutes; Total Number of Blows: 582 (starting at penetration 5.0 ft)

GRLWEAP: Wave Equation Analysis of Pile Foundations

36-inch x 0.5 x 40 ft D46 + R-19-002
GRL ENGINEERS, INC

4/12/2024
GRLWEAP 14.1.20.1

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The calculated capacity-blow count relationship, i.e. the bearing graph, should be used in conjunction with observed blow counts for the capacity assessment of a driven pile. Soil setup occurring after pile installation may produce bearing capacity values that differ substantially from those expected from a wave equation analysis due to soil setup or relaxation. This is particularly true for pile driven with vibratory hammers. The GRLWEAP user must estimate such effects and should also use proper care when applying blow counts from restrike because of the variability of hammer energy, soil resistance and blow count during early restriking.

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PILE INPUT

Uniform Pile		Pile Type:	Open-End Pipe
Pile Length: (ft)	40.000	Pile Penetration: (ft)	40.000
Pile Size: (ft)	3.00	Toe Area: (in ²)	55.76

Pile Profile

Lb Top ft	X-Area in ²	E-Modulus ksi	Spec. Wt lb/ft ³	Perim. ft	Crit. Index -
0.0	55.8	30,000.0	492.0	9.4	0
40.0	55.8	30,000.0	492.0	9.4	0

HAMMER INPUT

ID	22	Made By:	DELMAG
Model	D 46	Type:	OED

Hammer Data

ID	Ram Wt kips	Ram L. in	Ram Ar. in ²	Rtd. Stk ft	Effic. -	Rtd. Energy kip-ft
22	10.140	137.8	302.0	10.6	0.80	107.1

DRIVE SYSTEM FOR DELMAG D 46-OED

Type	X-Area in ²	E-Modulus ksi	Thickness in	COR	Round-out in	Stiffness kips/in
Hammer C.	415.000	470.000	3.500	0.800	0.120	55729.085
Helmet Wt.	3.800	kips				

SOIL RESISTANCE DISTRIBUTION

Depth ft	Unit Rs ksf	Unit Rt ksf	Qs in	Qt in	Js s/ft	Jt s/ft	Set. F. -	Limit D. ft	Set. T. Hours	EB Area in ²
0.0	0.0	37.6	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8
7.8	0.4	37.6	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8
7.8	0.9	22.6	0.10	0.10	0.20	0.15	2.0	6.6	168.0	55.8
12.8	1.5	22.6	0.10	0.10	0.20	0.15	2.0	6.6	168.0	55.8
12.8	1.5	21.4	0.10	0.10	0.20	0.15	2.0	6.6	168.0	55.8
24.8	2.3	21.4	0.10	0.10	0.20	0.15	2.0	6.6	168.0	55.8
24.8	0.9	171.3	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8
37.8	1.3	171.3	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8
37.8	3.3	125.3	0.10	0.10	0.10	0.15	1.5	6.6	24.0	55.8
42.8	3.6	125.3	0.10	0.10	0.10	0.15	1.5	6.6	24.0	55.8
42.8	1.3	192.1	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8
45.0	1.3	192.1	0.10	0.10	0.05	0.15	1.2	6.6	1.0	55.8

4/12/2024

4/5

GRLWEAP 14.1.20.1

Appendix C

Additional Information

