?1. Report No. **FHWA/LA.09/463**	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle **Estimating Setup of Driven Piles into Louisiana Clayey Soils**	5. Report Date 11/15/2009 6. Performing Organization Code **LTRC Project Number: 04-1GT** **State Project Number: 736-99-1359**	
7. Author(s) **Jay Wang, Neha Verma, and Eric Steward**	8. Performing Organization Report No.	
9. Performing Organization Name and Address **Department of Civil Engineering and Construction Engineering Technology** **Louisiana Tech University** **Ruston, LA 71272**	10. Work Unit No. 11. Contract or Grant No. **32-4116-40907**	
12. Sponsoring Agency Name and Address **Louisiana Department of Transportation and Development** **P.O. Box 94245** **Baton Rouge, LA 70804-9245**	13. Type of Report and Period Covered **Final report, 05/15/08-11/15/09** 14. Sponsoring Agency Code	

15. Supplementary Notes

Conducted in Cooperation with the U.S. Department of Transportation, Federal Highway Administration

16. Abstract

Two types of mathematical models for pile setup prediction, the Skov-Denver model and the newly developed rate-based model, have been established from all the dynamic and static testing data, including restrikes of the production piles, restrikes, static, and statnamic tests of the test piles at the LA-1 relocation project. Pile testing data from other sites, such as Mo-Pac- Railroad Overpass, Bayou Liberty, and Calcasieu River etc., have been used for model verification.

Twenty-one out of the 115 restrike records of the production piles and three load testing records from the nine tested piles were obtained at or longer than two weeks after pile installation.
The conventional Skov-Denver model is achieved with the setup parameter *A* equal to 0.57, and the normalized ultimate shaft capacity from the rate-based model is 1.846 on the basis of the entire restrike and load testing data. Based on the rate-based model with limited amount of long-term production pile restrike data, it is predicted that the ultimate shaft capacities of the piles were about twice the measured shaft capacities at the 24-hour restrike. In general, the piles at the LA-1 relocation project reaches about 90~95 percent of the ultimate shaft capacities within two weeks after installation.

Preliminary verification and prediction work has indicated that capacities of those piles at two-week or longer-time restrike or load testing were mostly under predicted if the entire database was used for the model prediction. The setup parameter and the normalized ultimate shaft capacity have turned to 0.65 and 1.985, respectively. Selected piles with restrike or load testing at or more than 200 hours after the end of driving have given the setup parameter of 0.65 and the normalized ultimate shaft capacity of 1.985. Predictions with the new rate-based model are improved. It demonstrates that long-term restrike or long-waiting load testing data have a profound and critically important role in improving reliability and accuracy of the prediction models.

An empirical relationship, between the measured pile capacity at 24-hour restrike and the calculated pile capacity based on the Cone Penetration Test (CPT) log, has been established. It will make pile setup prediction operable without the 24-hour restrike data. As the last portion of the research project, a simple Load and Resistance Factor Design (LRFD) calibration of pile setup has been performed. Resistance factors have been achieved corresponding to different target reliability indices and dead load to live load ratios.

17. Key Words Pile setup, shaft capacity prediction, rate-based mathematical model, LA-1 highway Relocation.	18. Distribution Statement Unrestricted. This document is available through the National Technical Information Service, Springfield, VA 21161.

19. Security Classification (of this report) unclassified	20. Security Classif.(of this page) unclassified	21. No. of Pages 173	22. Price

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LTRC Project No. 04-1GT
State Project No. 736-99-1359

conducted for

Louisiana Department of Transportation and Development
Louisiana Transportation Research Center

November 2009

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ACKNOWLEDGMENTS

The investigators appreciate the Louisiana Transportation Research Center (LTRC) and the Louisiana Department of Transportation and Development (LADOTD) for funding this project. The investigators would also like to acknowledge the guidance and the significant amount of administrative work provided by Dr. Zhongjie (Doc) Zhang. They would also like to acknowledge the help offered by Dr. Ching Tsai. He provided extensive information regarding the pile restrike and load testing. The principal investigator (PI) is grateful for those valuable and fruitful discussions made with Dr. Tsai and Dr. Zhang. The PI would like to express his gratitude toward Gavin Gautreau, Dr. Ching Tsai, and Dr. Doc Zhang for providing their careful corrections for the draft final report. In addition, the PI would like to acknowledge Harold (Skip) Paul for his first-round deliveries of the questionnaire of the pile setup survey to the Department of Transportation of each state in the US and each province in Canada. Also, the investigators would like to acknowledge the support of the remaining PRC members: Arturo Aguirre, Murad Abu-Farsakh, Kim Garlington, Gavin Gautreau, Ed Tavera, and Mark Morvant for those questions, comments, and suggestions they made during the project presentations.

IMPLEMENTATION STATEMENT

Reliable mathematical models are provided to predict pile capacities at different elapsed times after initial driving by collecting and analyzing available pile restrike and load testing data. Great attention is paid to make prediction models effectively workable with or without 24-hour restrike data. The analysis of pile setup is integrated into the LRFD design. The research may lead to a more cost effective pile design considering long-term pile setup effect in the future, which will eventually be integrated into relevant specifications. Recommendations are made on the beneficial use of pile setup based on the research results. During the dynamic testing for construction quality control, the research achievements will provide engineers new ways to estimate the pile setup effect. In the foundation design or construction practice, the models could be used as an additional tool for estimating pile capacity by considering setup effect. The completed research will provide a solid basis to make guidelines to take into account pile setup for the development of the *Louisiana Pile Foundation Design And Construction Manual*. The established mathematical models could be applied to pile foundation practice of other states. The predictions will be compared to the field testing measurements, and then the models will be validated, corrected, and improved. With growing engineering experience integrated in predictions models, they will be become more and more robust for pile foundation design. In order to improve the models, detailed recommendations on the future research effort of pile setup are made.

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INTRODUCTION

In geotechnical engineering practice, engineers have reported for many years that the axial capacity of a driven pile in clayey soils may increase over time after the end of installation, which is usually referred to as pile setup or freeze. Setup has long been recognized, and can contribute dramatically to long-term pile capacity. Significant amounts of field data have been achieved since the middle of last century, as shown in Figure 1. The ultimate capacity of a driven pile can be much greater than the initial capacity obtained immediately after installation, as reported by case histories in the literature and local field testing data in Louisiana. For instance, shaft capacities of the driven piles at LA-1 relocation project site increased by 30 to 100 percent during the first week after the end of driving. Capacities of some piles had significant growth even after 3 to 7 months. The incorporation of pile setup into pile design can offer substantial benefits. If it is possible to predict the setup effect during design, it may be possible to reduce pile lengths, or/and pile sections, or use smaller-diameter or thinner-wall pipe piles, or smaller-section H-piles, or reduce the size of driving equipment by using smaller hammers and/or cranes *[1]*.

LADOTD spends millions of dollars annually on the construction of driven pile foundations. The current design practice in LADOTD for driven piles is based on pile resistance at 14 days after initial driving without considering the long term development of pile capacities due to the lack of a systematic approach to handle the issue. This has led to a conservative pile design for many projects. Therefore, there is a need for developing a reliable design methodology that will account for the benefit of pile setup phenomenon in pile foundation design so that a more cost effective pile design may be used in the future. The research identifies the conditions where pile setup may be considered in design, magnitude, and rate of pile setup; reliability associated with the setup estimation; and resistance factors to be used in LRFD.

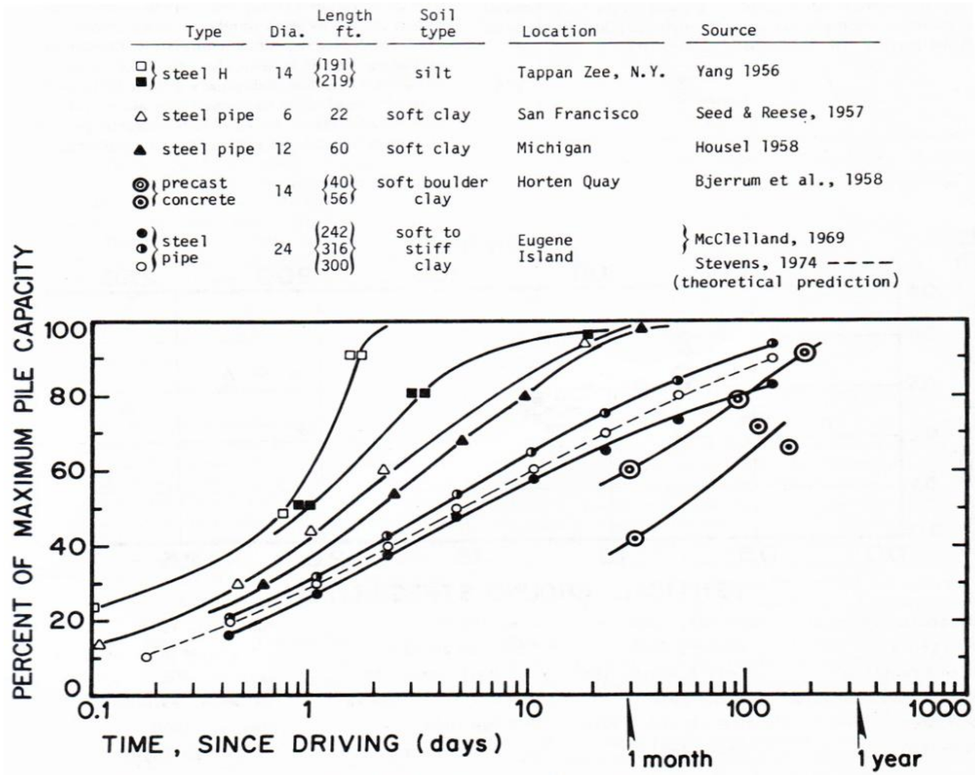


Figure 1
Field data on increase of pile capacity with time *[2]*

The pile setup mechanism has been studied by many researchers and engineers. During pile installation, soils around the pile are significantly disturbed and remolded. Excessive pore pressures are generated in saturated clays. The excessive pore pressure then dissipates and the pile regains its capacity, which could be used to explain short-term capacity increase *[2], [3], [4], [5], [6], [7], [8]*. Cases have been reported where the shaft resistance of piles driven in clayey soils kept increasing over a period of time much longer than the duration of soil reconsolidation. The capacity increase of piles driven into soft clays tends to be greater than that of piles driven into stiff clays *[9]*. Thus, the long-term capacity increase may result from other causes. Examples were presented by Schmertmann with regard to the time-strength changes in different types of soils *[10], [11]*. It is "mechanical aging" that causes the increases in the drained friction angle. Karlsrud and Haugen conducted axial tension tests on more than 20 piles in overconsolidated clay and found that pile capacities continued to increase another 22 percent within the next 30 days after the excess pore pressure dissipation for 6 days after end of driving *[12]*.

Komurka et al. illustrated a three-phase pile setup, as shown in Figure 2. Kehoe indicated that setup occurs primarily in the shaft capacity, and found that the capacity of square pre-stressed concrete piles increased an average of 58 percent at one and 200 percent at the other 11 days after the piles were driven in mixed clayey soils *[1], [13]*. More literature review has been summarized in Appendix A as part of work as tasks one and two.

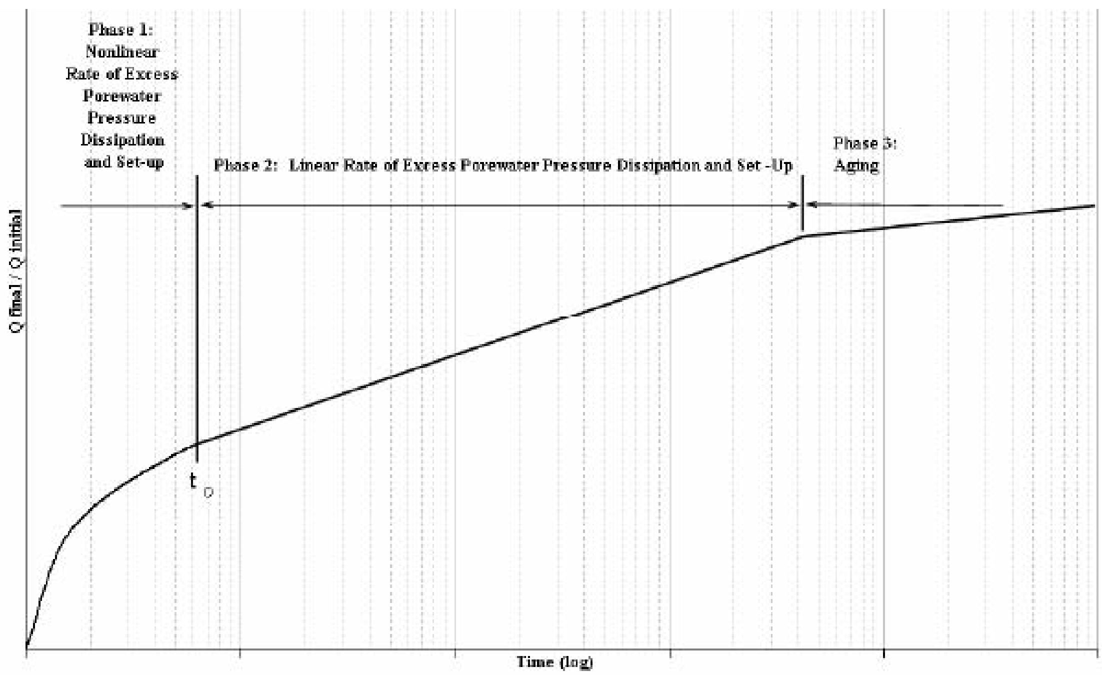


Figure 2
Idealized schematic of setup phases *[1]*

After a comprehensive literature review, researchers concluded that: (1) the semi-logarithmic relationship proposed by Skov and Denver has been widely used to predict pile setup; (2) the pile setup parameter *A* and reference time t_0 are different for different types of soils (sandy or clayey soils); (3) different pile setup parameters should be used for different types of piles (concrete piles, steel piles and timber piles, etc.); (4) in different geological regions, different pile setup parameters should be employed; and (5) the pile setup data are available for soils similar to those of Louisiana soils, such as the “Bay Mud” in the Bay area of San Francisco and the stiff, highly plastic sandy clay in the Coastal area of Charlestown, South Carolina *[14], [15]*.

The semi-logarithmic relationship between pile capacity and time, proposed by Skov and Denver and employed by many other researchers and engineers, is written as follows *[16]*:

$$\frac{Q}{Q_0} = A\log_{10}\left(\frac{t}{t_0}\right) + 1 \tag{1}$$

where, *A* is the dimensionless setup factor, and t_0, the reference time, is the time elapsed since the end of initial driving. *A* and t_0 are the parameters used to characterize piles and soils that piles are driven in. Q and Q_0 are either the total or shaft pile capacity at time t and the capacity corresponding to time t_0, respectively. The empirical relationship has been widely studied by many researchers and engineers for different soils and different sites. They found that *A* and t_0 are related to soil types; t_0 is not independent of *A* and hard to determine. Different *A* and t_0 results are outlined in Tables 1 and 2.

Table 1
A summary of pile setup factors and reference time

	Skov and Denver *[16]*		Svinkin et al. *[17]*	Axelsson *[18]*	Camp III and Parmar *[14]*
	Sand	Clay	Clayey and Sandy soils	Non-cohesive soils	Stiff, highly plastic sandy clay or sandy silt (Cooper Marl)
A	0.5	0.2	0.36~1.07	0.2~0.8	0.37~1.31
t_0 (day)	0.2	0.6	1 or 2	N/A	2
Pile type and location	Concrete piles; Alborg, Denmark		Pre-stressed concrete piles and H-piles; Ohio	N/A	Square pre-stressed concrete piles, H-piles; Coastal area, Charleston, South Carolina

Table 2
A summary of pile setup factors and reference time (continued)

	Bullock et al. (2005)	Yang and Liang (2006)
	Dense fine sand and soft to medium stiff silty clay	Clayey soils
A	0.1	0.5
t_0 (day)	1	1
Pile type and location	Square, pre-stressed concrete piles; coastal area, North Florida	Pipe, HP, concrete, pre-stress concrete, timber, etc.; different areas

After screening the existing LADOTD data of pile-testing information, researchers have seen that the log-linear relationship is appropriate. A correlation study for such a relatively simple empirical relationship does not require sophisticated field testing.

OBJECTIVE

The objective of this research was to provide LADOTD engineers with a simple, rational, and accurate method for predicting the capacities of piles over time after driving in various soil conditions and to identify the factors governing the setup, relate the setup magnitude, rate to pile and soil types, and make recommendations on the beneficial use of pile setup based on the research results.

SCOPE

The project began with a literature review in an effort to search for the practical methodologies that are being used by engineers around the world. Pile testing data collection, specifically the testing data of piles driven into Louisiana soils, was the second job. A pile setup survey was conducted among the states and provinces in the US and Canada to see how the pile setup effect is taken into account and how the benefit of pile setup is utilized in foundation design and construction practice in different places. Based on the gathered pile load testing and restrike data, mathematical models were developed for pile setup predictions, and the resistance factors were calculated to implement the pile setup effect in the LRFD design. Through the entire research project, no in-situ field tests or laboratory experiments were performed, with research efforts focused on data analyses and mathematical model development.

METHODOLOGY

The approach selected to solve the problem was based on a combination of a review of existing knowledge, collection of field testing data, survey of pile setup practice in pile foundation design in different states, development of pile setup model, verification of the newly established models against available pile testing data, the application of the models to LADOTD pile foundation design, and the application of the LRFD method incorporating pile setup to pile foundation design.

State-of-the-art pile setup prediction methods were examined and reviewed in the first step. This includes a literature search of previous and on-going nationwide research projects and case studies on the subject. Then, a survey was conducted with regard to various state highway and other agencies nationwide that have geological conditions similar to Louisiana to examine and review the state of practice on the pile foundation designs that have considered the benefit of pile setup.

As the second step of the research project, the available pile restrike and field testing data and associated geotechnical data were collected from LADOTD and other possible sources. Example projects include but not limited to: LA-1 Improvement - Golden Meadow to Fouchon, I-10 over Lake Ponchartrain (Twin Span), Tensas River - Tensas Parish, US 90 - Bayou Beouf, and five other small projects.

The data collected were screened and assessed with respect to the requirements of new methodology discussed in the introduction of the request for proposal (RFP). An interim report was due six months after project initiation to the PRC for review and approval. The report summarized the findings, data collected, and direction of future efforts.

After analyzing the existing data and discussing with LADOTD colleagues, the research team has followed the traditional way by assuming that the pile setup effect only applies to the shaft capacity (*[13], [16], [19], [20]*). Tip resistance does not display a dramatic growth after pile installation. The predicted total capacity is equal to the predicted shaft capacity plus the tip resistance measured at around 24-hour restrike or at the end of driving, or the first available restrike after end of driving. From all the data analysis and model evaluation and prediction, it is found that the assumption is appropriate.

The semi-logarithmic pile setup equation of the Skov-Denver model to Louisiana soils was achieved mainly based on the pile restrike data of the production piles and test piles, and static load testing data of the test piles at the LA-1 relocation project, which were driven into typical Louisiana soft clays. The pile capacity growth rate-based model was established as well to predict the ultimate pile capacity and the elapsed time after the end of driving (EOD) until desired pile capacity was reached. In the case of the absence of pile restrike data at the reference time (e.g. 24 hour after the EOD), empirical equations between the 24-hour restrike pile capacity and the calculated CPT data-based static pile capacity were provided to make pile setup prediction possible. Reliability analysis of the pile setup was performed, and pile setup at different elapsed time was incorporated in the LRFD method corresponding to different setup time. The prediction model validation was based on comparisons between calculated pile capacities and actual field measurements. The two mathematical models were applied to other three pile foundation sites where pile restrike or/and pile testing data are available. In this research, an attempt to increase the weight of the long-term restrike and load test data was made by picking up those piles with 200 or longer than 200-hour restrike or/and load test records. Independent mathematical models will also be established based on the selected data to see how the long-term restrike or pile testing data affect the predictions.

Application of the Skov-Denver Method

Average Unit Skin Friction and Reference Time

The 24-hour restrike records were used to evaluate parameters Q_0 and t_0 in equation (1). Due to the slight variation in the restrike time on the records, the reference time for each pile was selected based on the actual time of restrike that is closest to 24 hours. If a record at around 24-hour restrike was not available for a selected pile, an appropriate unit skin friction value at the reference time would be obtained from another pile in adjacent area. If no 24-hour restrike was available near the studied pile, then it was conservatively assumed that no setup has occurred from the 24-hour elapsed time until its next available restrike time. The normalized unit skin friction (s/s_0) was obtained by taking measured average unit skin friction at restrike time divided by the average unit skin friction at the reference time. The normalized time (t/t_0) was defined as the ratio of the restrike time to the reference time.

Parameter Estimate

Suppose that each unit skin friction resistance measurement s(i) is characterized by some measure of uncertainty that is estimated by the expected standard deviation σ_i of the correct one. The least-square criterion was followed by minimizing the χ^2 function to find the best-fit coefficients for the Skov-Denver and the rate-based models:

$$\chi^2 = \sum_{i=0}^{n} \frac{\left(s(i) - q(i)\right)^2}{\sigma_i^2} \quad (2)$$

where, q(i) is the prediction of the "correct" unit skin friction resistance at the ith restrike. It is assumed that all the capacity measurements have the same uncertainty $\sigma_i = \sigma$. The sum of the squared residuals (SSR) was defined as:

$$\sigma^2\chi^2 = \sum_{i=0}^{n}\left(s(i) - q(i)\right)^2 \quad (3)$$

In the model development, the curve fitting is completed by minimizing parameter $\sigma^2\chi^2$ to get those model coefficients.

Selection of the Reference Time for the Pile Setup Parameter

In processing the existing pile testing data to get the pile setup factor A, as many other researchers did, a common reference time, t_0 = 1 day, was chosen for the model development. However, the restrike, static, and statnamic testing data of the nine load test piles was analyzed, and other restrike times were studied for an appropriate reference time. Then, as the second stage of research, a parametric study was performed to find a best fitting t_0 values for the Louisiana soils by employing different restrike times.

Dynamic monitoring during restrike testing at different time with subsequent Case Pile Wave Analysis Program (CAPWAP) analysis after initial driving provides total and shaft resistances. The shaft resistance distribution along the pile length might also be determined. Setup parameter *A* is determined as the slopes of the linear portion of the normalized capacity $Q(t)/Q_0$ versus $\log_{10}(t)$, as given in equation (4). Observations have been made that the end bearing appears to have little setup as compared with the shaft capacity. Thus, the setup parameter *A* might be determined from the shaft resistance, as given by equation (5).

$$A(\text{time}) = \frac{\frac{Q(t)}{Q_0} - 1}{\log\left(\frac{t}{t_0}\right)} \quad \text{(whole pile)} \quad (4)$$

If the shaft resistance distributions are available for each restrike and s(t) and s_0 are the reliable unit side shear resistances at time t or t_0, respectively, the setup parameter *A* at any point on the pile wall will be:

$$A(\text{time}) = \frac{\frac{s(t)}{s_0} - 1}{\log\left(\frac{t}{t_0}\right)} \text{ (pile segment)} \qquad (5)$$

One *A* value is obtained from equation (4) at each restrike, and several *A*s are obtained from equation (5) if multiple unit skin frictions are available from the strain gauges mounted on the wall of the pile. Different piles and pile testing cases at each single site will be grouped to back calculate the setup factors. Among the piles with different lengths and diameters, or different materials, average unit shaft capacities were used for the calculations of setup parameter A. Pile testing data from different sites were employed separately. The back-computed *A* values were different from site to site. Although the amount of existing quality data are limited, such a correlation study may provide more project-specific values than is possible by using published values. In a summary, at this stage, the research effort will be focused on the following research activities:

1. Survey the existing LADOTD pile-testing data.
2. Achieve specific *A* and t_0 for different pile testing sites.
3. Choose appropriate t_0 to get the correlated A, based on the available restrike bearing capacities at different time, such as t = 2, 4…24, and 48 hours, etc.
4. Compare *A* values from different sources at different time, such as those *A* values from total bearing or shaft friction, at different restrikes, and on different piles.
5. Draw histograms and frequency distributions of the *A* values (normal, log-normal, or something else), as shown in Figure 3.
6. Achieve *A* values corresponding to different restrikes.
7. Compare the predicted bearing capacity from the Skov-Denver based model with the 14-day restrike capacity.
8. Work together with LADOTD engineers to find out appropriate *A*.
9. Predict pile setups at different elapsed times.

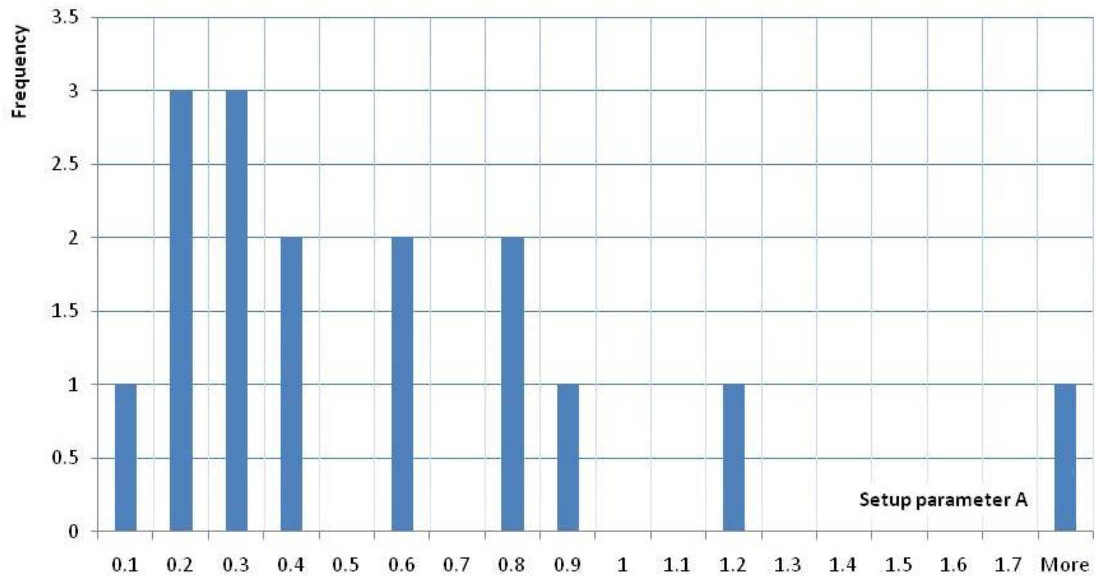


Figure 3
Histograms and frequency distributions of the *A* values

Other Published Correlations

Beside the simple empirical equation for the setup that increases linearly with logarithmic increase of time, and its improvement, the following empirical equations that have been developed by other researchers were employed for pile setup prediction in the research in an attempt to have a best-fit prediction model. There is a lack of restrike data at the end of driving from the construction site. Therefore, equations that were developed by Huang, Guang-Yu, Svinkin, and Svinkin and Skov were hardly implemented in the research *[21]*, *[22]*, *[23]*, *[24]*. In addition, prediction of pile capacity at 14 days after initial driving requires the sensitivity S_t of the Louisiana clayey soils, which is not readily available. Therefore, those models that were listed in Table 3 are not recommended to use in Louisiana based on the preliminary research work on the models.

Table 3
Other setup prediction models *[9]*

Authors	Equation	Comments
Huang *[21]*	$Q_t = Q_{EOD} + 0.263(1+\log(t)(Q_{max} - Q_{EOD}))$	Q_t is the total pile capacity at time t (days) after the initial driving; Q_{EOD} = Pile capacity at the end of driving; Q_{MAX} = maximum pile capacity
Guang-Yu *[22]*	$Q_{14} = (0.375S_t + 1)Q_{EOD}$	Q_{14} is the total pile capacity at 14 days after the initial driving
Svinkin *[23]*	$Q_t = 1.4Q_{EOD}t^{0.1}$ $Q_t = 1.025Q_{EOD}t^{0.1}$	Upper bound Lower bound
Svinkin and Skov *[24]*	$Q_u(t)/Q_{EOD} - 1 = B[\log_{10}(t) + 1]$	t_0 = 0.1 day, B is similar to *A* in Skov and Denver *[16]*

Development of the Pile Capacity Growth Rate-based Model

An ideal pile setup prediction model should be able to predict the ultimate capacity and the time it takes to achieve the pile resistance designers intend to use. The Skov-Denver model cannot predict ultimate pile resistances. Motivated by the expectation, the pile setup data were re-examined and a rate-based model was proposed and developed.

There are a total of six restrike records from the piles at Bent NC29 of LA-1 and nine pile capacity records from the cylinder test pile at test site No. 3. Restrike time, skin friction resistance, or the average unit skin friction S(t) and the unit growth rate of the skin friction q(t) are presented in Tables 4 and 5, respectively. The unit growth rate is defined as:

$$q(t) = \frac{1}{S(t)}\frac{dS(t)}{dt} \cong \frac{1}{S(t)}\frac{(S(t+\Delta t) - S(t-\Delta t))}{2\Delta t} \qquad (6a)$$

$$\cong \frac{1}{S(t)}\frac{(S(t+1) - S(t-1))}{T/T_0(t+1) - T/T_0(t-1)} \qquad (6b)$$

The growth rates are calculated and presented in the last column of each table, with equation (6b) for Table 5 and equation (6a) for Table 4, respectively. It can be observed that the unit skin friction growth rate is the largest immediately after the pile installation. It reduces with the increase in the skin friction resistance. The resistance must eventually stop growing after a certain period of time, which indicates that a theoretical ultimate shaft capacity may be reached.

Based on the observation, it is reasonable to assume that the unit skin friction growth rate is a function of the initial growth rate r and the magnitude of the unit skin friction and that the growth rate gets smaller and smaller with the increase in the skin friction. It is written as *[25]*:

$$\frac{1}{S(t)}\frac{dS(t)}{dt} = r\left(1 - \frac{S(t)}{S(\infty)}\right) \tag{7}$$

where, $S(\infty)$ is the ultimate unit skin friction based on the data displayed in Table 4. A similar equation can be established for the shaft resistances presented in Table 5, and then $S(\infty)$ is the ultimate skin friction. In equation (7), S(t) might represent the unit skin friction, or shaft resistance, and t can be replaced by a dimensionless time factor t/t_0 (or T/T_0). Solving differential equation (7), the closed-form solution is:

$$S(t) = \frac{S(\infty)S(0)}{S(0)+(S(\infty)-S(0)e^{r(t-t_0)}} \tag{8a}$$

where, S(0) is the bearing capacity at the reference time of t_0 = 24 hours. The two parameters [$S(\infty)$ and r] are usually achieved from the least squared method. As it was done for the Skov-Denver model, based on the normalized unit skin friction, the rate-based models were first established, respectively for the four segments of the LA-1 site, and then a synthetic model was developed for the combined restrike data. Another rate-based model was established from the combined data of the test piles. Outcomes of the model development are discussed in next section (Discussion of Results). All the model parameters resulting from different data sources will be given in tables for comparisons and discussion. The measured and predicted skin frictions will be plotted in figures. As an example of the rate-based model, the prediction equation for pile shaft capacity from the gathered restrike data of all the production piles at the site of NC-1B, one of the construction segments of LA-1 relocation project, is established as follows:

$$S(t) = \frac{1.865S(t_0)}{1+(1.865-1)e^{-0.213(t/24-1)}} \text{ (kips, tons, kN, etc.)} \tag{8b}$$

Table 4
Restrike records and skin friction growth rates for Bent NC29

Pile	Time	T/T_0	R_{skin}	Unit skin friction S(t)	$\Delta S/\Delta(T/T_0)/S$
	hrs		kN / kips	kN/m^2 / $kips/ft^2$	
NC29-03	24	1	947.4 / 213	11.13114 / 0.23248	
NC29-03	144	6	1,205.4 / 271	14.19402 / 0.29645	0.030110630
NC29-03	672	28	1,925.9 / 433	22.67070 / 0.47349	0.007546516
NC29-02	744	31	1,570.1 / 353	18.47163 / 0.38579	0.001178809
NC29-03	1728	72	2,006.0 / 451	23.62878 / 0.49350	0.005258606
NC29-02	1728	72	2,001.6 / 450	23.56606 / 0.49219	

Table 5
The unit skin friction growth rates for the 54-in. cylinder pile at testing site No. 3

Event	t (hours)	Ru (kips)	S(t) (kips)	**$\Delta S/\Delta(t)/S$**
End of Driving	0.0	378	287	
Restrike 2 hrs	2.0	696	596	0.1733781
Restrike 4 hrs	3.9	798	690	0.0185149
Restrike 24 hrs	24.7	1027	886	0.0078698
Restrike 48 hrs	44.2	1112	971	0.0030227
Restrike 72 hrs	72.4	1169	1026	0.0017709
Restrike5 days	117.4	1247	1104	0.0007025
Restrike 12 days	287.7	1337	1193	0.0006005
Load Test	384.0	1395	1295	

The Developed Correlation between 24-hour Restrike Shaft Capacity and Calculated Cone Penetration Test (CPT) Data-Based Shaft Capacity (Skin Friction)

Usually, pile capacity, with the setup effect taken into account, is predicted on the basis of the pile capacity measured at a reference time, for instance, the pile capacity at the 24-hour restrike. However, the 24-hour restrike data are not usually available. The Project Review Committee suggested finding an empirical relationship between the measured pile capacity at 24-hour resstrike and relevant soil material properties. In the research, an empirical equation was established, which relates the measured 24-hour shaft capacity to the calculated pile capacity based on the CPT log. The calculated pile skin frictions are from the French Central Bridge and

Pavement Laboratory method (LCPC), the Schmertmann method, and the de Ruiter and Beringen method, respectively, by running software PileConeAnalysis developed by LTRC. Relationships between the measured 24-hour capacity and the average skin frictions from the three methods were also established.

The relationship between the measured 24-hour skin friction and the calculated skin friction are presented in several ways: (1) ratio of the measured skin friction to the calculated skin friction versus the calculated skin friction; (2) quad root of the skin friction ratio versus the calculated skin friction; and (3) the measured skin friction versus the calculated skin friction.

LRFD Calibration of the Pile Capacity Accounting for Pile Setup Effect

The reliability analysis of the pile setup at different elapsed time was performed. Due to the availability of pile setup data in this project, reliability analysis was only conducted on one category: concrete square pile in Louisiana coastal area. In this research, pile capacity was only predicted using the Skov-Denver and rate-based models. Reliability of the static pile capacity analysis and field test methods, such as Alpha, CPT, and Norlund methods was evaluated later. Other static analysis methods provided in software DRIVEN was evaluated as well. The reliability indices were calculated using the MVFOSM. The research employed the load statistics and the load factors from the latest AASHTO LRFD specifications to make the pile foundation design consistent with the bridge superstructure design *[26]*. As an example, in this report, the load combination of dead load (QD) and live load (QL) for the Strength I Case is chosen for the reliability analysis and the subsequent calibration of the resistance factors. In the conducted research, two random variables, the load (Q) and the resistance (R), verified with data provided by LADOTD was assumed to be lognormally distributed. As specified in the AASHTO LRFD specifications, the load factors used in the reliability analysis are 1.25 for dead load and 1.75 for live load. The reliability index for MVFOSM is given as follows:

$$\beta = \frac{\ln\left[\dfrac{\lambda_R FS(QD/QL+1)}{\lambda_{QD} QD/QL + \lambda_{QL}}\sqrt{\left(1+COV_{QD}^2+COV_{QL}^2\right)/\left(1+COV_R^2\right)}\right]}{\sqrt{\ln\left[\left(1+COV_R^2\right)\left(1+COV_{QD}^2+COV_{QL}^2\right)\right]}} \quad (9)$$

where, FS is the factor of safety, and COV_R and COV_Q are coefficients of variation of R and Q, respectively. Values λ_{QD}, λ_{QL}, and λ_R are the bias factors for dead load, live load, and resistance, respectively. It was seen that the reliability indices of pile setup capacities vary widely among the different design methods at different elapsed times.

In the LRFD-based pile foundation design, a constant target reliability index should be used in the calibration of the resistance factors. Tentatively, target reliability indices of 2.0, 2.5 and 3.0, corresponding to the probability of failure of approximately 10 percent, 1 percent, and 0.1 percent are suggested in the research. For redundant piles, which is usually defined as five or more piles per pile cap, a failure probability equal to 1 percent is recommended, which corresponds to the target reliability index of approximately 2.33. For non-redundant piles, which are considered as four or fewer piles per pile cap, it is recommended to use a failure probability p_f = 0.1 percent, corresponding to a reliability index of $\beta = 3.0$ *[26]*.

Calibration of the Resistance Factors Considering Pile Setup

If values of the load modifiers are taken as ones, the basic requirements for LRFD can be expressed as:

$$R_r = \phi R_n \geq \sum \gamma_i Q_i \;(\eta_i = 1.0) \tag{10}$$

where, ϕ is the resistance factor, γ_i is the the load factor, Q_i is the nominal load, and R_n is the nominal resistance, which is predicted using the established mathematical models. Corresponding to the AASHTO Strength I case, in which dead and live loads are involved only, the fundamental resistance factor can be calculated as:

$$\phi = \frac{\lambda_R \left(\gamma_{QD} \dfrac{QD}{QL} + \gamma_{QL} \right) \sqrt{\dfrac{1 + COV_{QD}^2 + COV_{QL}^2}{1 + COV_R^2}}}{\left(\lambda_{QD} \dfrac{QD}{QL} + \lambda_{QL} \right) \exp\left\{ \beta_T \sqrt{\ln\left[\left(1 + COV_R^2\right)\left(1 + COV_{QD}^2 + COV_{QL}^2\right) \right]} \right\}} \tag{11}$$

DISCUSSION OF RESULTS

Analysis of the Pile Setup Survey Results

A survey questionnaire was sent to every state in the United States and all the provinces of Canada. Eventually, 36 completed surveys returned. It has been found from the returned surveys that most of the responded states/provinces held the same opinion that pile setup is important, and some have considered pile setup to some extent in their pile foundation designs and constructions. However, no state/province has ever taken into account more than 14 days of pile setup. Many states thought that lack of a reliable prediction model is the reason why they did not seriously account for pile setup in their pile foundation practice. The returned information in the survey was summarized as given in the tables in Appendix E.

General Soil Information

Clay or silt dominates at the LA-1 relocation site. Gray and gray and tan clay with silt were found to a depth of about 200 feet with occasional sand or silty sand layers, with a very soft to stiff consistency. The upper 25 feet of the soils include some peats and organic rich clays. The mudline is about 1~3 feet below the water table. Within the depth of 10 feet, soil moisture contents are between 30 and 50, very close to the liquid limits. The liquidity indexes differ between 20 and 40 within the depth of 70 feet. Compressive strengths from the unconsolidated undrained tests range from 0.1 to 0.5 tsf. Some fundamental soil data were obtained from the combinations of boring logs and CPT logs. They are presented as follows in Figures 4 through 6. A typical soil profile from CPT data log and two boring logs is shown in Figure 7.

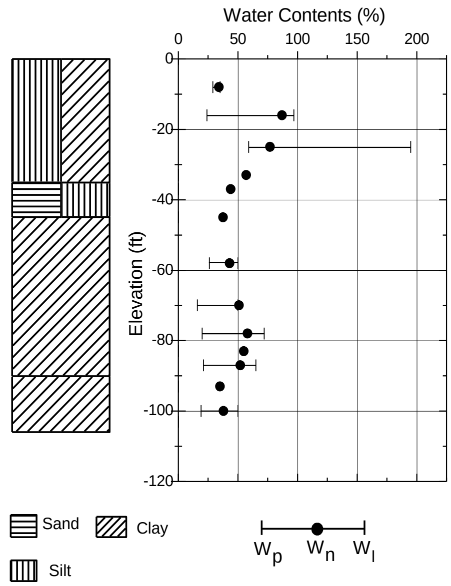


Figure 4
Combined soil data from boring log B187 and CPT log CPT 188

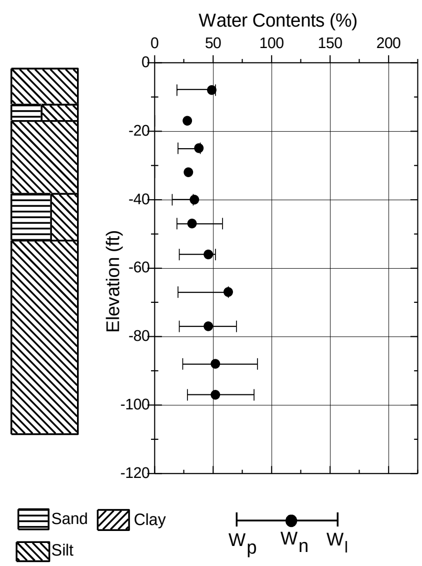


Figure 5
Combined soil data from B-189 and CPT 15+97

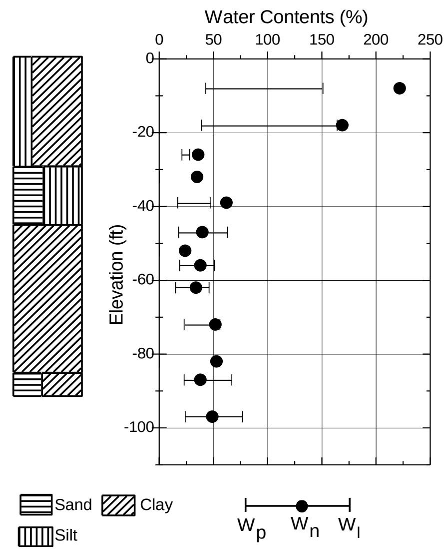


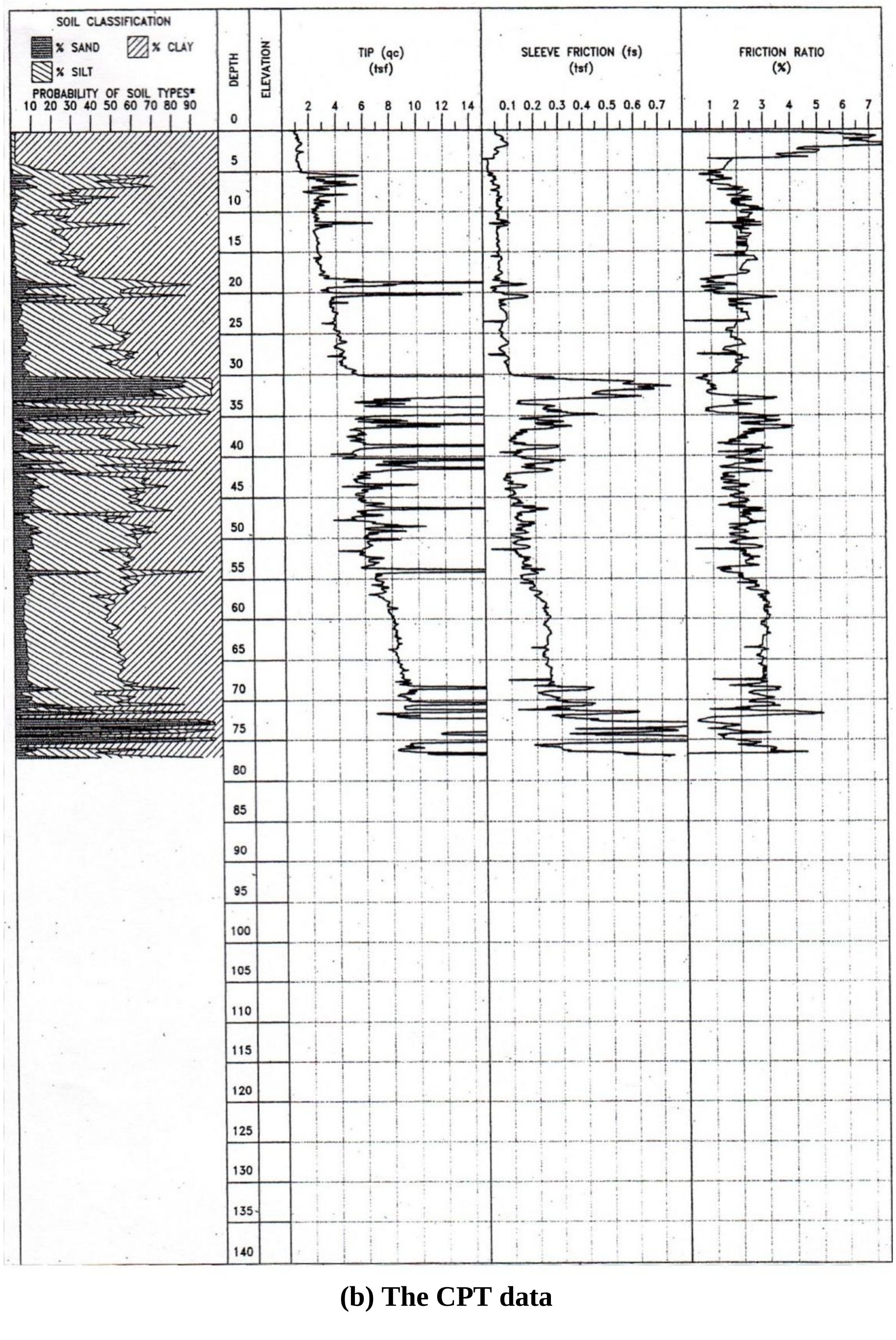
Figure 6
Combined soil data from B-191 and CPT 191

SOIL TYPE⊗ AND COLOR	WET DENSITY	MOISTURE CONTENT	LIQUID LIMIT	PLASTICITY INDEX	qu	SPT or UU	FAILURE MODE	SAMPLE NUMBER	DEPTH	ELEVATION	WATER TABLE
									0		
WATER									5		
BK. PT. w/TRA. CL.		801						C-1			
	67	553	495	406		0.14 ●5.6	YLD.	C-2			
GR. ORG. CL. w/TRA. PT.		193						C-3	10		
GR. CL.	105	75	64	38			SL.	C-4			
GR. SI. CL.	104	52	33	14		0.06 ●9.8	YLD.	C-5	15		
w/LYR. GR. SI. SA.		53						C-6	20		
GR. CL. w/SI. PKT. & SH.	100	47	82	63		0.08 ●16.8	YLD.	C-7	25		
w/SH. & SA. LYR.		42						C-8	30		
GR. SI. SA. w/TRA. CL.	102	28				0.22 ●23.8	YLD.	C-9	35		
GR. CL. w/SH. & SI. PKT.		56						C-10	40		
GR. CL. SI. w/SH. & PT.	104	36				0.27 ●30.8	45°S.	C-11	45		
GR. CL. w/TRA. SI.	105	58	50	31		0.30 ●34.3	YLD.	C-12	50		
w/SH.		48						C-13	55		
GR. CL.		66						C-14	60		
	104	49	50	28		0.15 ●44.8	YLD.	C-15	65		
w/GAS		56						C-16	70		
w/SI. PKT.	96	60	73	49		0.11 ●51.8	YLD.	C-17	75		
GR. CL. SI. w/TRA. SA.	102	36				7.53 ●55.3	YLD.	C-18	80		
GR. CL.		39						C-19	85		
w/SA. LYR.		41						C-20	90		
GR. SI. CL.	111	35	46	25		0.59 ●65.8	YLD.	C-21	95		
GR. CL.		39						C-22	100		
	109	42	51	28		0.38 ●72.8	YLD.	C-23	105		
									110		
									115		
									120		
									125		
									130		
									135		
									140		

TEST PILE NO.
STA.:
LOCATION:
TYPE OF PILE:
TYPE OF HAMMER:
RATED ENERGY: FT. LBS.
DATE OF DRIVING:
DRIVING RESISTANCE (TONS)
STR. NO.:
SQD. LDR.:

BORING NO.: B-54	STA.: 366+25.33
LATITUDE: 29° 13' 44.7"	LOCATION:
LONGITUDE: 90° 11' 49.1"	DATE TAKEN: 7-31-03
CONT. SECT.:	LOG MILE:

(a) The boring log data



(b) The CPT data

Figure 7
Typical CPT and boring log data at the LA-1 relocation site *[27]*

Pile Capacity Records from Restrikes and Load Tests

A Brief Introduction to the Production Pile Restrike Data that were Collected from the Site of the LA-1 Relocation Project

All production pile restrike data came from Phase 1B of the LA-1 relocation project, which consists of the construction of a 4-mile long high-level bridge with connecting ramps and interchanges. The 16-inch, 24-inch, and 30-inch prestressed concrete (PPC) piles were used extensively in the project. A total of 115 restrike records from 95 piles have been gathered from the four segments: the North Connector (NC), the South Connector (SC), the mainline-span over Bayou Lafourche (mainline), and the Ramp N1 (N1). They are summarized as shown in Table 6. As many as 63 records are from the short term restrikes of less than 50 hours after EOD, and there are only 23 long-term records of more than two weeks. These pile capacity records from restrikes were achieved from signal matching (CAPWAP) analysis.

All the data from the nine load-tested piles were collected at the four locations along the new LA-1 alignment for Phase 1B. One location was selected to represent the soil conditions of the main piers, one to represent the soil condition at the bridge approaches to the main span, and the other seven were representative of the soil conditions along the approximately 5.5-mile-long Phase 1A bridge. Of the nine test piles, six were 16-inch, 24-inch, and 30-inch PPC piles; two were 54-inch cylinder concrete piles; and one was a 30-in steel pipe pile.

Table 6
A summary of pile restrike records at the site of LA-1B relocation project

Site	Restrike records	Piles	Bents	Number of restrike records at different time after EOD (hrs)				
				0-50	51-100	101 - 335	≥ 336	Incomplete or EOD
North Connector	37	29	27	21	2	8	5	1
South Connector	19	18	18	15	—	2	1	1*
1B-mainline	45	39	34	26	5	7	6	1
Ramp N1	14	9	7	1	1	3	9	
Total	115	95	86	63	8	20	21	3

* One EOD driving record was found at the site of South Connector

As an example, results of shaft and total capacities vs. time of the productions piles at the construction segment of North Connector are given in Table 7 and are plotted in Figures 8 and 9, respectively. The remaining results are placed in Appendix B, Figures 55 through 60 and Tables 42 through 44. Restrike data and static or statnamic load testing data of the nine test piles are presented in Tables 45 through 51.

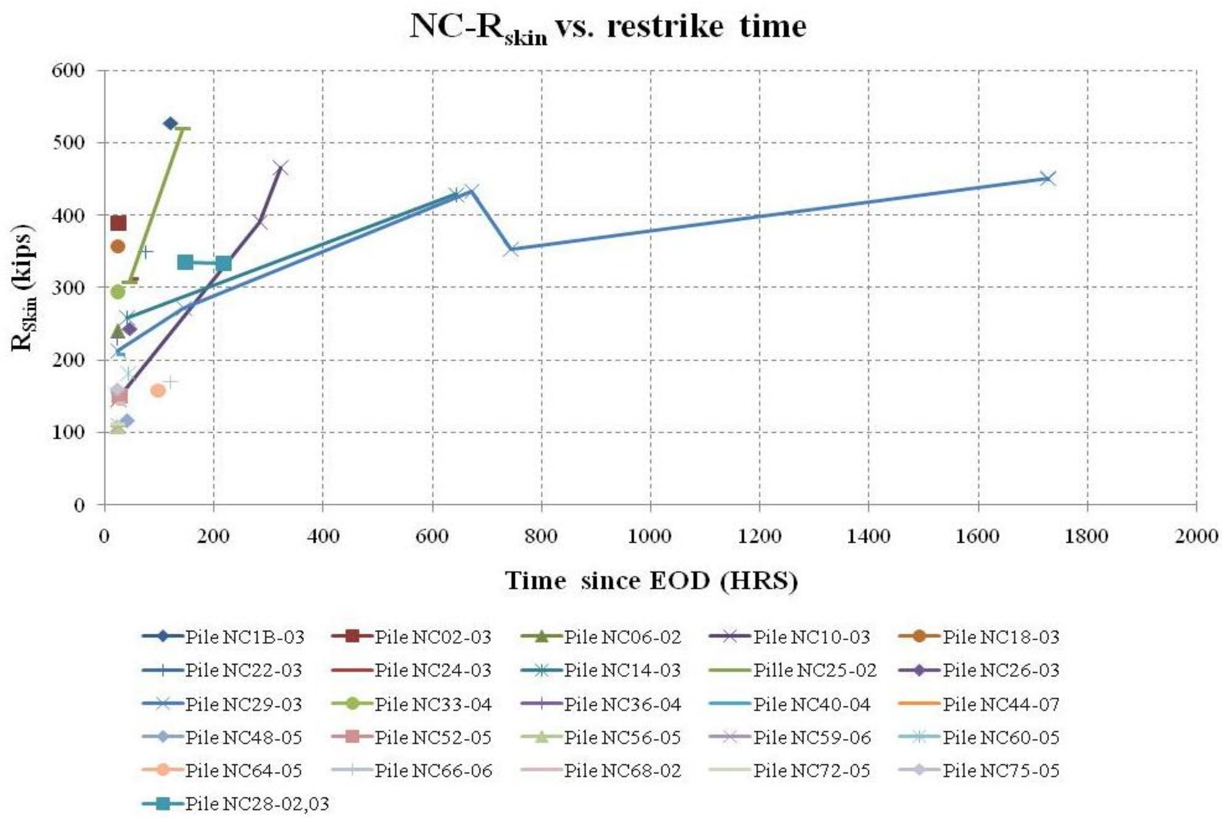


Figure 8
Pile shaft capacity change with time from the restrikes at the North Connector

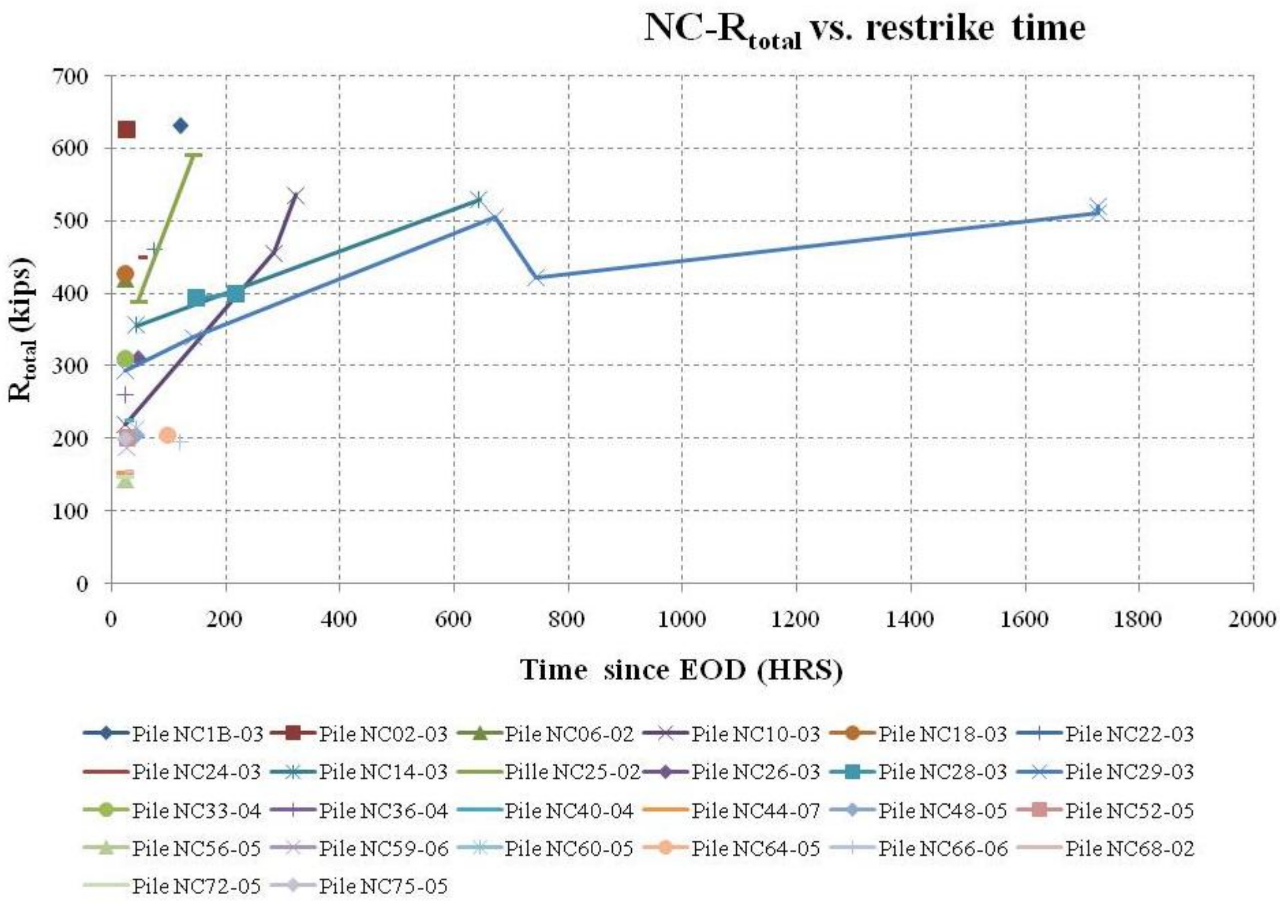


Figure 9
Pile total capacity change with time from the restrikes at the North Connector

Table 7
Pile type, capacity, soil information, and other information of the production piles at the North Connector

Pile	Pile Type	Restrike Date	Time (Hrs)	Penetration Length (ft)	Soil Type	R_{skin} (kips)	R_{tip} (kips)	R_{tot} (kips)
NC75-05	NA	6/28/2006	23	NA	NA	159	41	200
NC72-05	NA	8/4/2006	24	NA	NA	111	36	147
NC68-02	16" SQ. PPC Solid	8/10/2006	24	80.18	NA	140	15	155
NC66-06	16" SQ. PPC Solid	9/14/2006	120	80.08	NA	170	26	195
NC64-05	16" SQ. PPC Solid	9/5/2006	98	89.9	NA	158	47	205
NC60-05	16" SQ. PPC Solid	9/7/2006	43	84.3	NA	182	32	214
NC59-06	16" SQ. PPC Solid	8/29/2006	25	59.38	NA	110	77	187
NC56-05	16" SQ. PPC Solid	9/13/2006	23	83.62	NA	108	34	143
NC52-05	16" SQ. PPC Solid	9/20/2006	27	83.57	NA	151	50	201
NC48-05	16" SQ. PPC Solid	9/26/2006	42	82.77	Major clay with sand	116	89	205
NC47-06	16" SQ. PPC Solid	9/27/2006	NA	47.82	Major clay with sand	NA	NA	NA
NC44-07	16" SQ. PPC Solid	10/2/2006	24	81.37	Major clay with sand	110	43	153
NC40-04	16" SQ. PPC Solid	10/11/2006	24	81.92	Major clay with sand	208	17	225
NC36-04	16" SQ. PPC Solid	10/21/2006	24	77.08	Major clay with sand	231	29	260
NC33-04	16" SQ. PPC Solid	10/26/2006	24	71.44	Major clay with sand	294	17	310
NC29-03	24" SQ. PPC Solid	12/21/2006	24	114.31	Major clay with sand	213	82	294
NC29-03	24" SQ. PPC Solid	12/27/2006	144	114.31	Major clay with sand	271	69	340
NC29-03	24" SQ. PPC Solid	1/17/2007	672	114.31	Major clay with sand	433	72	505

(continued)

NC29-02	24" SQ. PPC Solid	1/20/2007	744	114.31	Major clay with sand	353	70	422
NC29-02	24" SQ. PPC Solid	3/2/2007	1728	114.31	Major clay with sand	451	59	510
NC29-03	24" SQ. PPC Solid	3/2/2007	1728	114.31	Major clay with sand	450	70	520
NC28-02	24" SQ. PPC Solid	1/26/2007	218	113.47	Major clay with silt	333	67	400
NC28-03	24" SQ. PPC Solid	1/23/2007	148	113.47	Major clay with silt	335	60	395
NC26-03	24" SQ. PPC Solid	2/16/2007	46	112.07	Major clay with silt	243	67	310
NC25-02	24" SQ. PPC Solid	3/29/2007	46	111.51	Major clay with silt	308	80	388
NC25-02	24" SQ. PPC Solid	4/2/2007	144	111.51	Major clay with silt	519	71	590
NC24-03	24" SQ. PPC Solid	3/21/2007	48	110.67	Major clay with silt	312	138	450
NC22-03	24" SQ. PPC Solid	4/2/2007	75	109.27	Major clay with sand	349	111	460
NC18-03	24" SQ. PPC Solid	4/26/2007	24	126.95	Major clay with shell	357	71	428
NC14-03	24" SQ. PPC Solid	5/10/2007	42	126.41	Major clay with silt	259	97	356
NC14-03	24" SQ. PPC Solid	6/4/2007	644	126.41	Major clay with silt	429	101	530
NC10-03	24" SQ. PPC Solid	5/24/2007	24	126.41	Major clay with silt	145	75	220
NC10-03	24" SQ. PPC Solid	6/4/2007	285	126.41	Major clay with silt	391	65	456
NC10-03	24" SQ. PPC Solid	6/7/2007	323	126.41	Major clay with silt	466	70	536
NC06-02	24" SQ. PPC Solid	6/14/2007	24	141.41	Major clay with sand	240	179	419
NC02-03	24" SQ. PPC Solid	6/28/2007	25	141.41	Major clay with sand	388	238	626
NC1B-03	24" SQ. PPC Solid	7/17/2007	120	147.5	Major clay with sand	527	104	631

Pile Load Testing Data Summary and the Testing Data Sample at Test Site No. 2

A total of nine instrumented test piles were driven and tested at four locations along the proposed alignment of the new LA-1 highway *[28]*. The test piles consisted of eight precast prestressed concrete (PPC) piles ranging in size from 16-inch square to 54-inch hollow spun-cast cylinder piles and a single 30-inch diameter, open-ended steel pipe pile. The piles were driven into varying embedment and were monitored during driving using the Pile Driving Analyzer (PDA). Restrikes were conducted on each pile at pre-determined intervals to assess the development of pile setup as a function of time following the end of driving and then tested at the predetermined times to correlate the PDA measurements with static pile capacity. Pile driving and load test data were recorded digitally, analyzed, and interpreted using dedicated software. The nine pile testing results were summarized in Table 8. Restrike and testing results of the two test piles at the test site 2 are presented in Tables 9 and 10, respectively. The remaining test results are given in Appendix B.

Table 8
Summary of load tests conducted for LA-1 relocation project

Pile Type	Pile Length (ft)	Test Method	Date Driven	Date Tested	Pile Tip Elevation (ft)	Pile Capacity (kips)
Test Site 2 - 29° 15' 00N, 90° 13' 03W (North approach to main span)						
54-inch Cylinder	160	Statnamic	7/9/2004	7/16/2004	-148.5	1295
16-inch Square PPC	130	Static	7/7/2004	7/14/2004	-119.4	427
Test Site 3 - 29°14' 51N, 90° 12' 34W (Support for main span)						
54-inch Cylinder	160	Statnamic	6/6/2004	6/22/2004	-148.1	1395
30-inch Square PPC	190	Static	6/4/2004	6/17/2004	-178.4	1650
30-inch Steel Pipe Pile	195	Static	6/1/2004	6/16/2004	-183.2	1597
Test Site 4 - 29° 13' 50N, 90° 11' 50W (Low level trestle)						
24-inch Square PPC	210	Static	7/27/2004	8/2/2004	-202.5	1656
24-inch Square PPC	160	Static	7/27/2004	8/2/2004	-152.7	861
Test Site 5 - 29° 13' 05N, 90° 11' 34W (low level trestle- Phase 1A)						
24-inch Square PPC	170	Static	8/9/2004	8/17/2004	-163.1	769
24-inch Square PPC	145	Static	8/9/2004	8/17/2004	-138.1	739

Skin friction distribution at the end of each pile load testing was back calculated from the strain measurements on pile reinforcements. It is assumed that the distribution pattern at load testing applies to those restrikes for each pile. As such, the skin friction distributions starting from the EOD until the end of load testing can be plotted for each pile. The skin friction growths at different elevations for the selected tested piles are given as in Figures 61 through 67 in Appendix B.

Table 9
Restrike and load test data of the 16-in. PPC pile - T2

Event	Date	Time	t (hours)	Ru (kips)	Rs (kips)	Rt (kips)
End of Driving	7/7/2004	10:08 AM	0.0	49	14	35
Restrike 2 hrs	7/7/2004	12:21 PM	2.2	178	155	23
Restrike 4 hrs	7/7/2004	2:04 PM	3.9	210	176	35
Restrike 6 hrs	7/7/2004	4:07 PM	6.0	243	205	38
Restrike 22 hrs	7/8/2004	7:45 PM	21.6	383	258	125
Restrike 55 hrs	7/9/2004	6:05 PM	56.0	434	311	122
Restrike 76 hrs	7/10/2004	3:03 PM	76.9	474	341	134
Restrike 96 hrs	7/11/2004	11:00AM	96.9	473	339	133
Load Test 168 hrs	7/14/2004	NA	168.0	427	400	27

Table 10
Restrike and load test data of the 54-in. cylinder pile - T2

Event	Date	Time	Time from EOD (hours)	Ru (kips)	Rs (kips)	Rt (kips)
End of Driving	7/9/2004	2:28 PM	0.0	303	201	102
Restrike 2 hrs	7/9/2004	4:16 PM	1.8	708	502	206
Restrike 5 hrs	7/9/2004	7:35 PM	5.1	860	643	218
Restrike 23 hrs	7/10/2004	1:40 PM	23.2	1128	788	340
Restrike 46 hrs	7/11/2004	12:53PM	46.4	1207	820	387
Restrike 70 hrs	7/12/2004	12:47PM	70.3	1279	889	390
Restrike 93 hrs	7/13/2004	11:16AM	92.8	1298	902	396
Load Test 168 hrs	7/16/2004	NA	168.0	1295	1199	96

Pile Testing Data Collected from Other Sites

The following pile testing and restrike data were collected from other three sites. They are only used for verifying the established models. The predictions and measurements were compared and discussed.

Table 11
Pile testing data collected from other sites

Pile Location	Pile Name	Pile type	Restrike Details	Skin Resistance (kips)	Tip Resistance (kips)	Total Resistance (kips)	Unit Friction (ksf)
Mo-Pac-Railroad Overpass, West Baton Rouge	TP-1	24" SQ PPC	EOD	192	334	526	0.26
		24" SQ PPC	EOD	216	343	559	0.29
		24" SQ PPC	48 Hrs	353	292	645	0.48
		24" SQ PPC	7 Days	424	224	648	0.57
	TP-3	24" SQ PPC	EOD	111	25	136	0.16
		24" SQ PPC	24 Hrs	234	38	272	0.38
		24" SQ PPC	9 Day	319	31	350	0.46
		24" SQ PPC	Static Test	NA	NA	400	NA
	TP-4	24" SQ PPC	EOD	175	339	514	0.24
		24" SQ PPC	48 Hrs	302	302	604	0.41
	TP-5	24" SQ PPC	EOD	178	328	506	0.24
		24" SQ PPC	24 Hrs	373	270	643	0.5
Bayou Liberty	TP-1	24" SQ PPC	EOD	49	31	80	NA
		24" SQ PPC	3 Days	194	37	240	NA
		24" SQ PPC	7 Days	351	58	409	NA
Calcasieu River	TP-1	NA	1 Hr	484	210	694	NA
		NA	20 Hrs	627	285	912	NA
		NA	456 Hrs	1002	238	1239	NA
		NA	432 Hrs (Static)	NA	NA	662	NA
	TP-2	NA	1 Hr	370	599	4311	NA
		NA	96 Hrs	837	532	969	NA
		NA	408 Hrs	1009	663	1671	NA
		NA	383 Hrs(Static)	NA	NA	662	NA
	Bent 17-P04	24" SQ PPC	20 Hrs	533	178	712	NA
	Bent 18-P04	24" SQ PPC	72 Hrs	571	310	881	NA

Data Analysis

Skin friction, tip resistance, and total capacity are available for each restrike event. As an example, 44 valid records of pile capacity versus restrike time were collected at the site of the LA-1 mainline, as illustrated in Figures 55 and 56. Those pile capacities were obtained from the signal matching (CAPWAP) analyses, and the data appeared quite random. There was insufficient restrike information from any individual pile to develop a mathematical model. One solution was to group several piles to achieve a combination of the restrike data. However, these piles that are not the same diameter were driven to different depths and embedded into various soil strata. As such, their pile resistances are not comparable. It would be misleading to simply bring together all the shaft or total pile resistances for the statistical analyses. A more reasonable approach was to use the average unit skin frictions as the parameter of interest, instead of the total or skin friction resistances.

Procedure Demonstration of the Model Establishments Using the Restrike Data from the Site Segment of NC-1B (LA-1)

Average Unit Skin Friction and Reference Time

Pile restrike records with the restrike time of around 24 hours are picked up, and the corresponding average unit skin frictions are calculated and presented in Table 12.

Table 12
Average pile capacity (23-25) hours after EOD

Pile Type	Restrike time (hrs)	Shaft capacity (kips)	Average unit skin friction (kips/ft^2)
16" SQ. PPC, Solid	24	140	0.33
16" SQ. PPC, Solid	25	110	0.42
16" SQ. PPC, Solid	23	108	0.24
16" SQ. PPC, Solid	24	110	0.25
16" SQ. PPC, Solid	24	208	0.47
16" SQ. PPC, Solid	24	231	0.56
24" SQ. PPC, Solid	24	357	0.35
24" SQ. PPC, Solid	24	145	0.14
24" SQ. PPC, Solid	24	240	0.21
24" SQ. PPC, Solid	25	388	0.34
Average	24.1	203.7	0.33

The average unit skin friction of the 10 records is 0.33 kips/ft^2. The average unit skin friction for all the 36 valid restrike records are also calculated. They are given in Table 13 and plotted in Figure 10. The frequency distributions at different ranges of unit skin frictions are plotted as a histogram in Figure 11. It is reasonable to assume a standard normal distribution for the average unit skin friction.

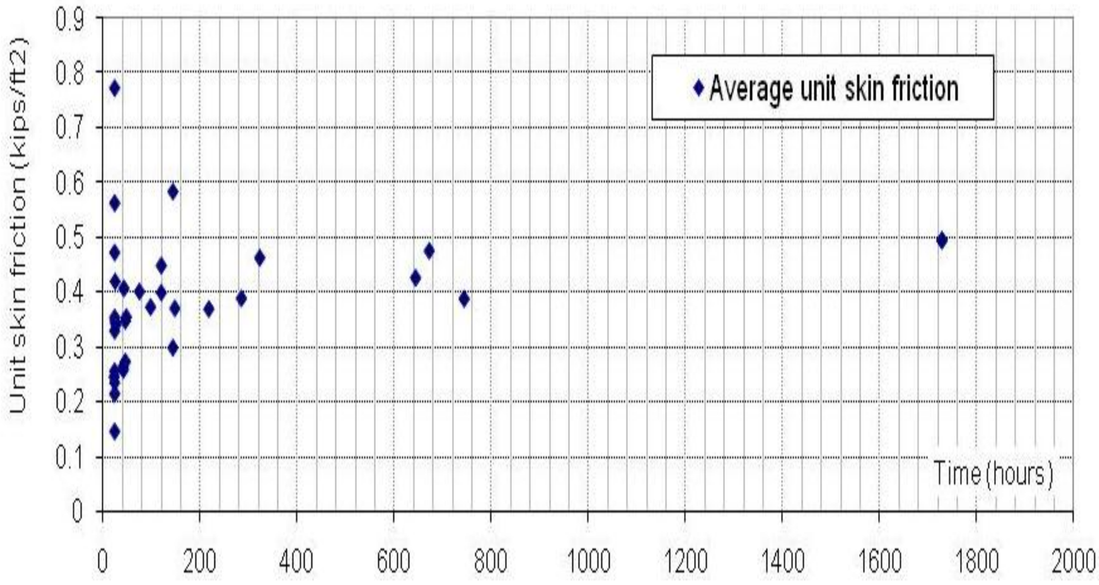


Figure 10
Average unit skin friction versus time (1B-North Connector)

Table 13
Average unit skin friction, time ratios (time/reference time), and skin friction ratios at NC-1B

Pile	Date	Time hrs	R_{skin} kips	R_{ult} kips	Average unit skin friction (s) kips/ft^2	t/t_0	s/s_0
NC75-05	6/28/2006	23	159	182	0.37	1	1
NC72-05	8/4/2006	24	111	135	0.26	1	1
NC68-02	8/10/2006	24	140	164	0.33	1	1
NC66-06	9/14/2006	120	170	290	0.40	5	1.21
NC64-05	9/5/2006	98	158	256	0.37	4.08	1.13
NC60-05	9/7/2006	43	182	225	0.40	1.72	0.97
NC59-06	8/29/2006	25	110	135	0.42	1	1
NC56-05	9/13/2006	23	108	131	0.24	1	1
NC52-05	9/20/2006	27	151	178	0.34	1	1
NC48-05	9/26/2006	42	116	158	0.26	1.56	0.78
NC47-06	9/27/2006	NA	NA	NA	NA	NA	NA
NC44-07	10/2/2006	24	110	134	0.25	1	1
NC40-04	10/11/2006	24	208	232	0.47	1	1
NC36-04	10/21/2006	24	231	255	0.56	1	1
NC33-04	10/26/2006	24	294	318	0.77	1	1
NC29-02	1/20/2007	744	353	1097	0.39	31	1.66

(continued)

NC29-02	3/2/2007	1728	451	2179	0.49	72	2.12
NC29-03	12/21/2006	24	213	237	0.23	1	1
NC29-03	12/27/2006	144	271	415	0.30	6	1.29
NC29-03	1/17/2007	672	433	1105	0.47	28	2.04
NC29-03	3/2/2007	1728	450	2178	0.49	72	2.12
NC28-02	1/26/2007	218	333	551	0.37	9.08	1.58
NC28-03	1/23/2007	148	335	483	0.37	6.17	1.59
NC26-03	2/16/2007	46	243	289	0.27	1.91	1.17
NC25-02	3/29/2007	46	308	354	0.35	1	1
NC25-02	4/2/2007	144	519	663	0.58	3.13	1.68
NC24-03	3/21/2007	48	312	360	0.35	2	1.00
NC22-03	4/2/2007	75	349	424	0.40	3.13	1.14
NC18-03	4/26/2007	24	357	381	0.35	1	1
NC14-03	5/10/2007	42	259	301	0.26	1	1
NC14-03	6/4/2007	644	429	1073	0.42	15.33	1.66
NC10-03	5/24/2007	24	145	169	0.14	1	1
NC10-03	6/4/2007	285	391	676	0.39	11.88	2.70
NC10-03	6/7/2007	323	466	789	0.46	13.46	3.21
NC06-02	6/14/2007	24	240	419	0.21	1	1
NC02-03	6/28/2007	25	388	626	0.34	1	1
NC1B-03	7/17/2007	120	527	631	0.45	4.8	1.30

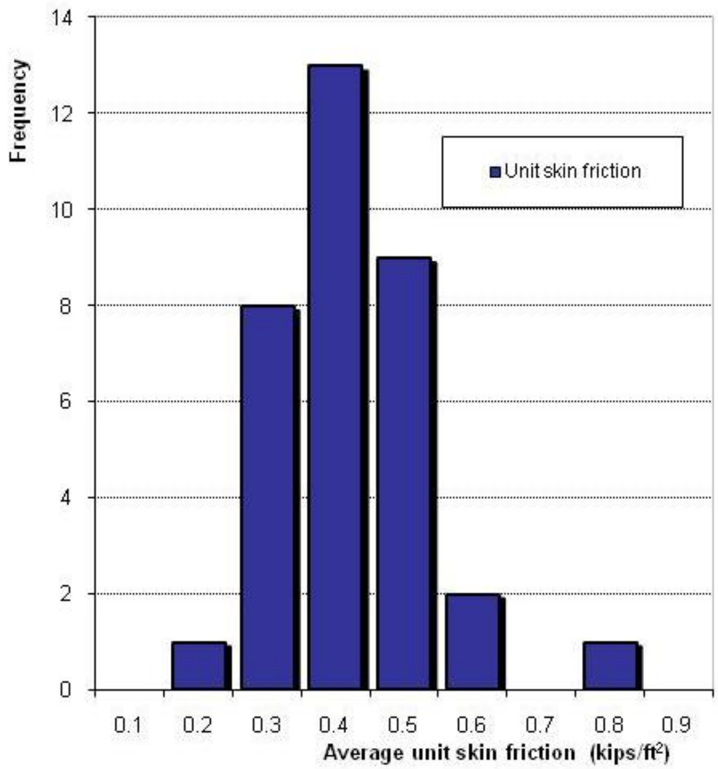


Figure 11
Histogram of the average unit skin friction (NC-1B)

In Table 13, column 6 displays the average unit skin friction for all the restrike records. Originally, the average time of 24.1 hours given in Table 12 was selected as the reference time. However, the statistical analysis on the Skov-Denver model did not result in good results. In the research, the reference times selected for different piles at the site of NC-1B were in the range of 23-48 hours. If a reference time in this range was not available for a pile, an appropriate reference time was obtained for the pile from an adjacent pile. The normalized unit skin friction (s/s_0) was obtained by taking the measured average unit skin friction divided by the average unit skin friction at the reference time as given in column 8 of Table 13.

Establishment of the Skov-Denver Model

The popular Skov-Denver model was established using the restrike data from the site of NC-1B. Different ways were taken to select the reference time. The setup parameters *A* and their distributions were studied. Lower bound and upper bound *A* values were achieved. Significant information was provided to pick up *A* values for pile setup calculations. Based on the least-square method, the prediction model was established and presented in Table 14 with the setup parameter *A* equal to 0.717.

Upper and Lower Bounds of Setup Parameter *A*

Columns 7 (t/t_0) and 8 (s/s_0) from Table 13 are plotted in Figure 12 with logarithmic scale for t/t_0 and arithmetic scale for s/s_0, respectively. Figure 12 indicates a lower bound of $A = 0.2$ and an upper bound of $A = 1.8$ cover most of the data from the site of NC-1B with an average value $A = 0.7$. Extensive literature review and these results may suggest the practical use of $A = 0.2$ for the skin friction prediction if a conservative pile foundation design is performed.

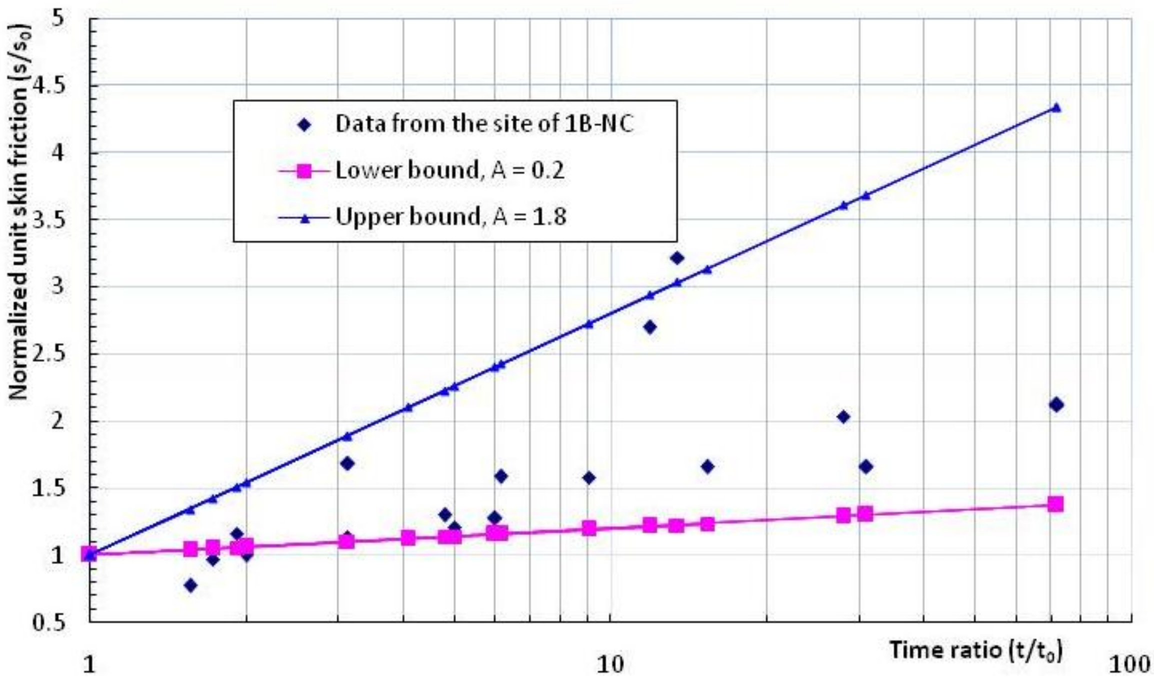


Figure 12
Upper and lower bounds of setup parameter *A* for the site of North Connector

Distributions of Setup Parameter *A*

A histogram was plotted in Figure 13 for setup parameters *A*. It is shown that setup parameters *A* might follow a log-normal distribution. As an example, the distribution tells how dispersive the parameter is. The plot has been expanded and analyzed by performing similar calculations from other construction segments and the nine test piles.

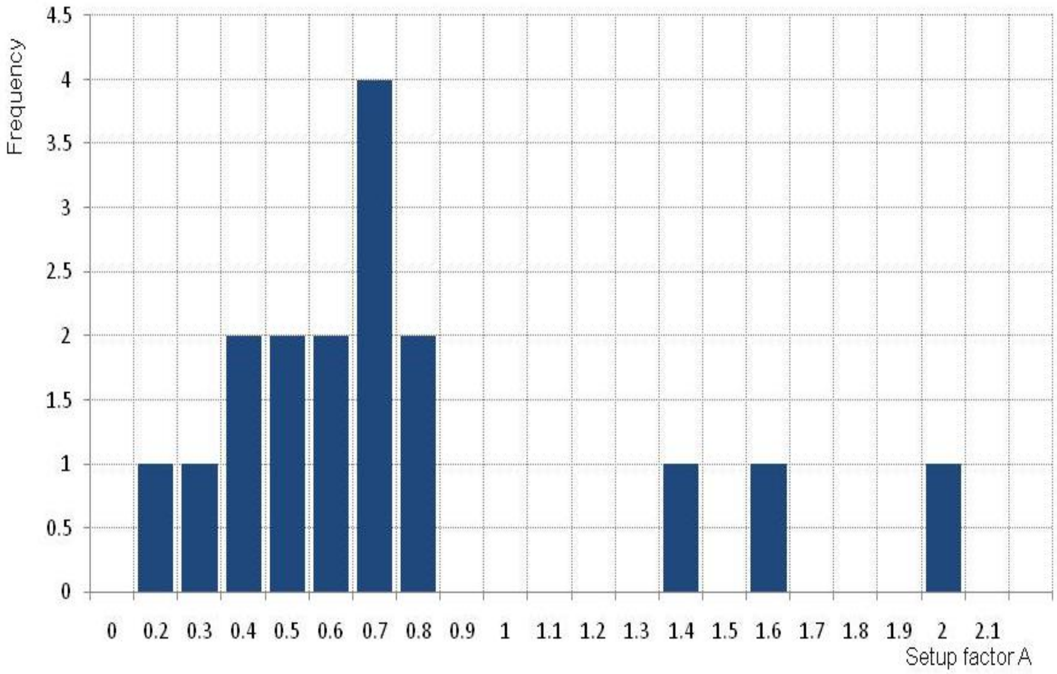


Figure 13
Histogram of setup parameter *A* for the NC-1B

Establishment of the Growth Rate-Based Model

Using the restrike data collected from the NC-1B and following the strategy and procedures addressed in the Methodology section, the normalized time (t/t_0) and the normalized unit skin friction (s/s_0) presented in Table 13 were used for the model development. The model equation is also given in Table 14 with two model parameters, the initial normalized skin friction growth rate r_0, and the ultimate normalized skin friction $s(\infty)/s(0)$, equal to 0.238 and 2.161, respectively. The initial unit skin friction growth rate r_0 is interpreted as $ds/dt/s_0 = 0.238$, and the ultimate normalized skin friction indicates that with the increase in elapsed time after the end of driving, the ultimate skin friction of each individual PPC driven pile would be 2.161 times as large as the skin friction measured at the 24-hour restrike of the same pile. Like the established Skov-Denver model, the rate-based model has been modified by employing restrike and load testing data from other construction sites and the nine load test piles.

Table 14
Established models for the piles at the site of North Connector

Mathematical model for pile setup prediction	Parameters			Equations
Skov-Denver method	Setup factor *A*		Reference time t_0 (hrs)	$\boldsymbol{S(t) = S_0 * (0.717 * Log\left(\frac{t}{t_0}\right) + 1)}$
	0.717		24	
Rate-based method	Initial pile setup growth rate r_0	Ultimate normalized unit skin friction $q(\infty)$	Reference time t_0 (hrs)	$\boldsymbol{q(t) = \frac{2.161q_0}{(q_0 + (2.161 - q_0) * e^{-0.238(\frac{t}{t0}-1)})}}$
	0.238	2.161	24	Note: q = $\frac{S(t)}{S_0}$, q_0=1

Analyses of the Rate-Based Model and the Skov-Denver Model and their Comparisons

The established Skov-Denver and the rate-based models presented in Table 14 were used for predictions. The measured and predicted normalized unit skin frictions were given in Table 15, where the predicted ones were obtained from both the proposed rate-based model and the Skov-Denver model. The total residual is defined as the sum of the squared residuals (SSRs) between the normalized measured capacities and predicted ones, as shown at the bottom of Table 15. The SSRs were calculated for the rate-based model and the Skov-Denver model, respectively. As a result, the rate-based model was established with an SSR of 2.9853, and the Skov-Denver model gave a SSR of 3.8008. The less the SSR is, the smaller the discrepancy between the predicted capacity and measured one would be, and the more accurate the predictions would be. The predictions and measurements are plotted in Figures 14 and 15, respectively. For the sake of comparison, the two models were also plotted in the same figure for the same scale, as shown in Figure 16.

Table 15
Predicted and measured normalized average unit skin friction

Restriking time after EOD (hrs)	Time ratio t/t_0	Measured from restriking s/s_0	Predicted unit skin friction ratio s/s_0 Rate-based method	Skov-Denver method
24	1	1	1	1
42	1.56	0.78	1.07	1.14
43	1.72	0.97	1.09	1.14
46	1.92	1.17	1.12	1.20
48	2.00	1.00	1.13	1.22
75	3.13	1.14	1.27	1.35
98	3.13	1.68	1.27	1.36
120	4.08	1.13	1.39	1.44
120	4.80	1.30	1.47	1.49
144	5.00	1.21	1.49	1.50
144	6.00	1.28	1.60	1.56
148	6.17	1.59	1.61	1.57
218	9.08	1.58	1.85	1.69
285	11.88	2.70	1.99	1.77
323	13.46	3.21	2.04	1.81
644	15.33	1.66	2.08	1.85
672	28.00	2.04	2.16	2.04
744	31.00	1.66	2.16	2.07
1728	72.00	2.12	2.16	2.33
1728	72.00	2.12	2.16	2.33
Sum of squared residuals (SSR): $\sum\left(\left(\frac{S}{S_0}\right)_{measured}-\left(\frac{S}{S_0}\right)_{predicted}\right)^2$			2.9853	3.8008

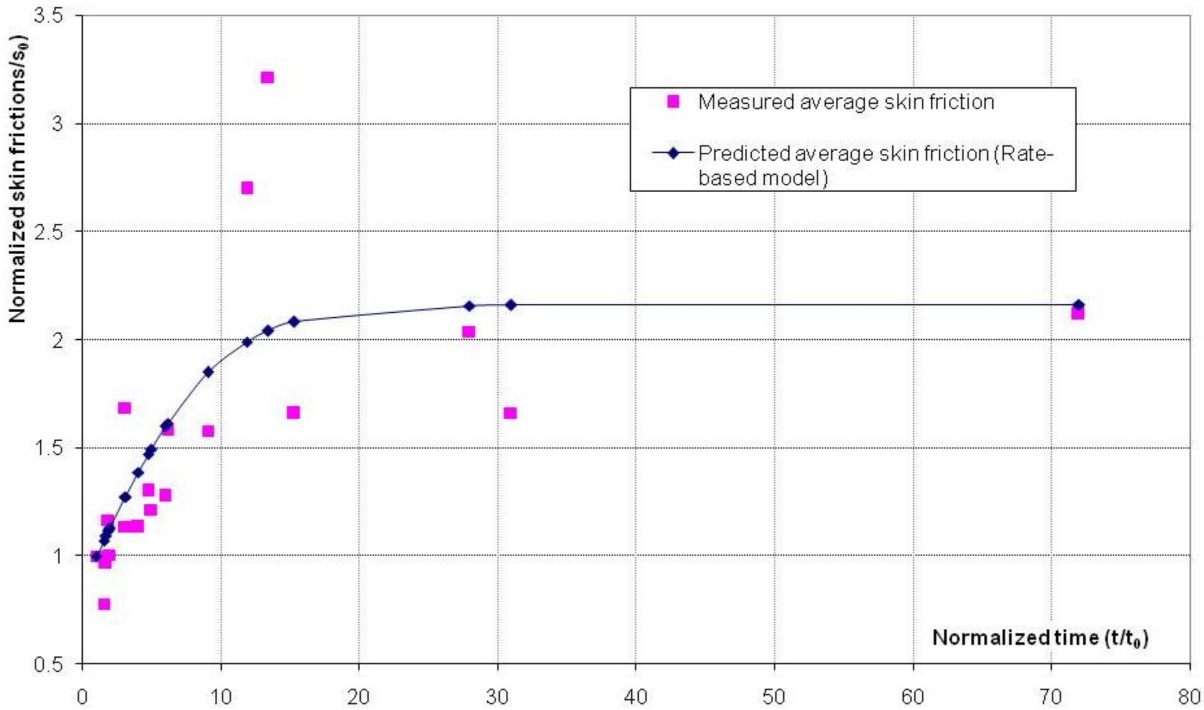


Figure 14
Measured and predicted normalized skin friction (rate-based)

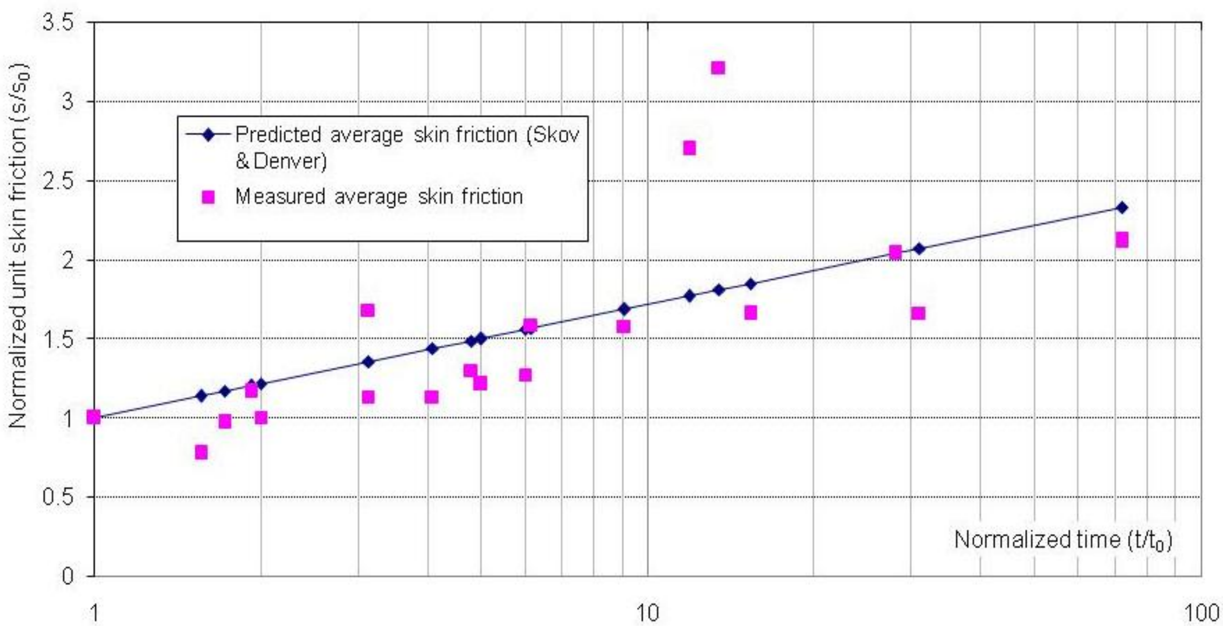


Figure 15
Measured and predicted normalized skin friction (Skov-Denver method)

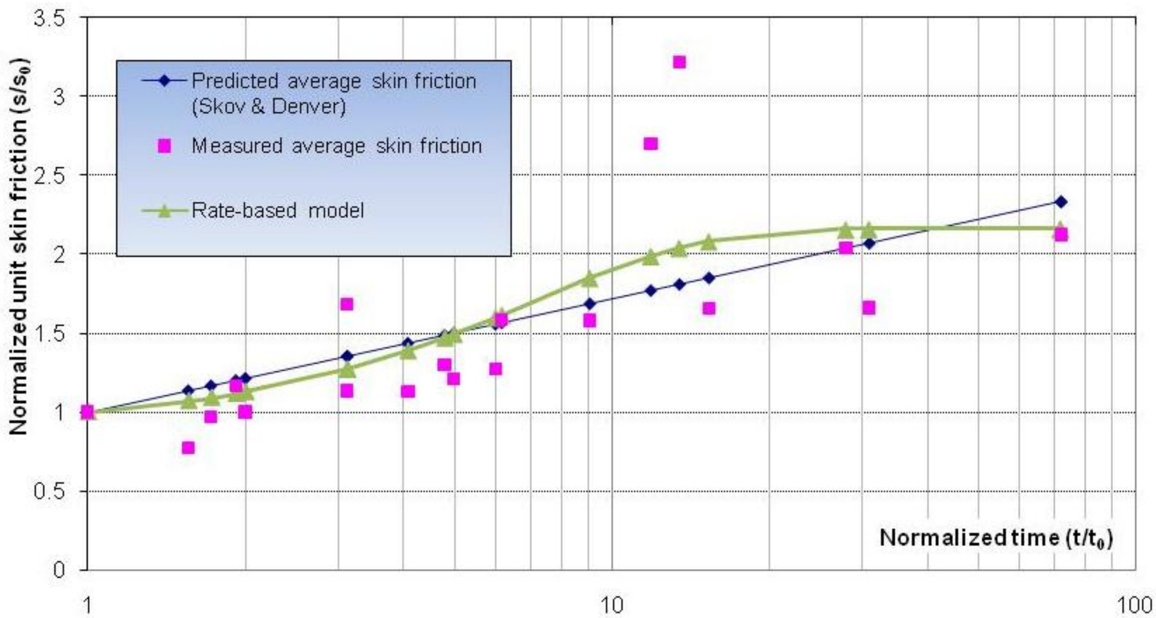


Figure 16
Measured and predicted normalized unit skin friction from the Skov-Denver and rate-based models (North Connector)

As described before, the Skov-Denver method is unable to provide the ultimate pile setup, but the proposed rate-based model has the capability. It can also give the amount of elapsed time after the EOD for the expected pile setup. At the site of the NC-1B, the ultimate average skin friction is about twice the average unit skin friction that was observed at the restriking time of around 24 hours after the EOD. Similar work was carried out for other sites and similar results such as the ultimate bearing capacity ratio was suggested for Louisiana soils.

The Skov-Denver Models from the Production Pile Restrikes at Different Construction Segments

As an example, procedures of the least-square method have been demonstrated to get the setup parameter *A* by minimizing the SSR between measured and predicted normalized unit resistances for construction segment NC-1B. In the research, the Skov-Denver model was established for each of the four segments of the LA-1 relocation project. Then, the restrike data from the four segments were compiled to establish a synthetic model for the entire LA-1 relocation project from the production piles. The four individual segment models and the synthetic model were plotted in Figure 17. Setup parameters *A*s are 0.362, 0.521, 0.547, and 0.717 for the four segments, respectively, and 0.547 for all the production piles of the entire project. These *A* values do not vary drastically, which indicates that the synthetic model could provide a reasonably reliable prediction for the project. It is inferred that a similar *A* value may be used for PPC piles installed in similar soil conditions as previously described.

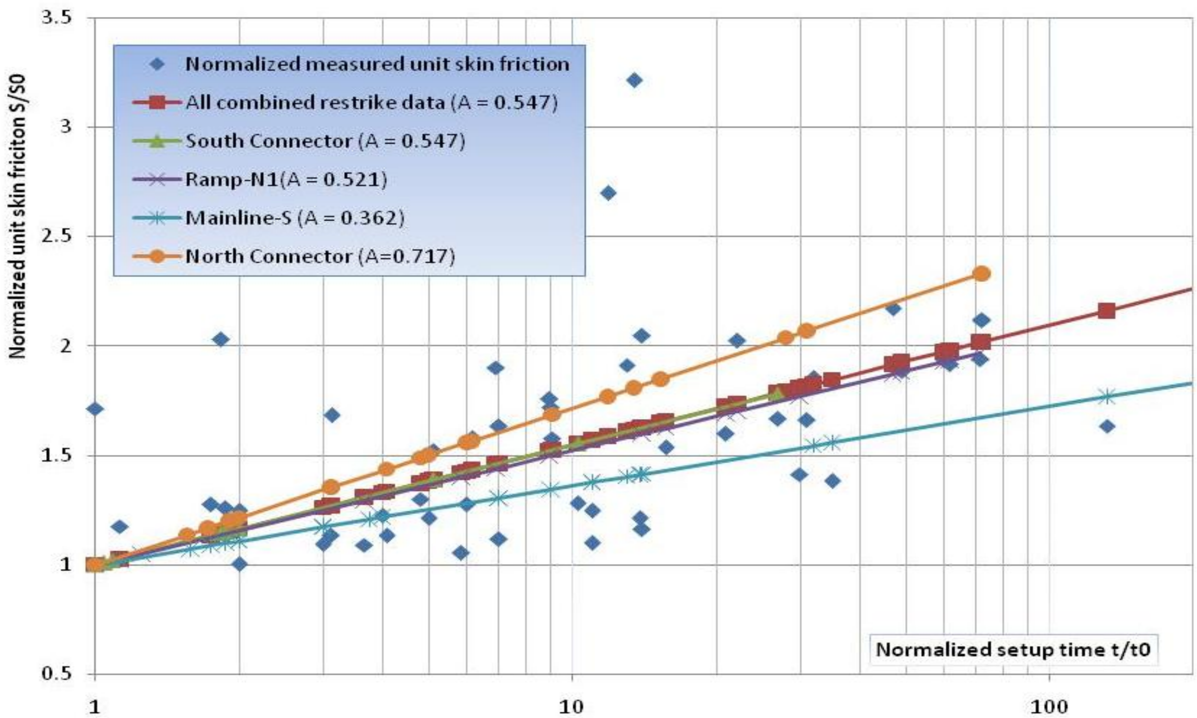


Figure 17
Normalized measured unit skin frictions and their Skov-Denver predictions from the restrike data of the production piles

The Skov-Denver Model from the Nine Test Piles

The nine instrumented test piles were installed and tested along the new LA-1 highway at four sites, typically reflecting the subsurface conditions at the site. Those records of the restrikes that were conducted over one or two weeks before the static or statnamic load tests, together with the load testing data, were employed to develop the Skov-Denver model for the pile setup prediction.

As there were usually five or six pile capacity records for each test pile, an independent Skov-Denver model was developed accordingly for each pile. Those normalized skin resistances were used in the model development. The selected reference time and setup parameters *A* were determined and given in the third and four lines of Table 16, respectively for the nine piles. Additionally, a synthetic Skov-Denver model was developed by combining all the test pile data, and it is presented in the last column. After comparing the setup parameter *A* values in Table 16 with those in Figure 17, it was found that those *A* values in Table 16 disperse more severely than in Figure 17. Investigation has revealed that there were one or two capacity measurements that were inconsistent with the remaning measurements for the 30-inch PPC pile at test site three (T3), the 24-inch PPC pile at T4, and the two 24-inch PPC piles at T5, respectively. They resulted in inconsistent high *A* values in the prediction models. Due to the small size of data, several errors or deviations will result in a misleading conclusion. However, when combining all test piles, the difference between the parameter *A* from test piles (0.67, SSR = 3.32) is not significant with that of the production piles (0.55, SSR = 7.65). Overall, the pile setup parameters range mainly from 0.5 to 0.7 for the four construction segments and the nine test pile, respectively, as shown in Table 17.

Table 16
Skov-Denver models for the nine test piles

Data source	T2		T3			T4		T5		Data from all the nine test piles
	16″ PPC	54″ Cylin.	30″ PPC	30″ Pipe	54″ Cylin.	24″ PPC	24″ PPC	24″ PPC	24″ PPC	
Reference time t_0	21.6	23.2	23.6	24.1	24.7	20.6	23.7	21.7	23.6	20.6~24.7
Setup parameter *A*	0.565	0.428	1.177	0.358	0.359	0.618	0.907	1.059	0.993	0.670

Table 17
Skov-Denver models from the restrike data of the production piles and the nine test piles

Data Source	Mainline	North Connector	South Connector	Ramp-N1	Combined data of all the production piles	The nine test piles
Parameter A	0.362	0.717	0.547	0.521	0.547	0.670
Reference time t_0 = 24 hours for the production piles, and 20.6~24.7 for the test piles.						

Application of the Skov-Denver Model to All the Production and Test Piles

The least-square method was conducted using all the pile testing data (restrikes of the production piles, restrikes, static and statnamic tests of the test piles) at the LA-1 relocation project. The Skov-Denver model is developed as follows:

$$S(t) = S(t_0)(0.570 \log\left(\frac{t}{t_0}\right) + 1) \tag{12}$$

This model was used for all the verifications and predictions in the following sections. It was suggested that LADOTD use this model for pile capacity predictions to take into account pile setup effect in their pile foundation practice for PPC piles driven in typical south Louisiana soft clayey soils. Examples of pile capacity prediction using equation (12) are presented in Appendix D.

Effect of the Reference Time on the Setup Parameter *A*

Total and shaft resistances were provided by the dynamic monitoring during a restrike, together with subsequent CAPWAP analysis. The setup parameter *A* is the slope of the linear portion of the normalized capacity Q/Q_0 versus $\log_{10}(t/t_0)$, as given in equation (4) *[19]*. It is re-written as equation (13). Observations indicate that the end bearing appears to be constant or has an insignificant setup effect as compared with the shaft capacity. Thus, the setup parameter *A* could also be determined from the shaft resistance by replacing the total resistance Q_t with the shaft resistance Q_s.

$$A(\text{time}) = \frac{\dfrac{Q_t}{Q_0} - 1}{\log\left(\dfrac{t}{t_0}\right)} \tag{13}$$

If the shaft resistance distributions are available for each restrike, then the setup parameter *A* can be found by replacing Q_t and Q_0 with the unit skin frictions S and S_0, respectively.

Skov and Denver pointed out that reference time t_0 is a function of soil type *[16]*. During a brief period right after pile installation, pile capacity increases because of increases in effective vertical and horizontal stresses with a mechanism that has not been well understood *[29]*. Prediction of bearing capacities using the measurements from the end of driving or restrikes performed at a time $t<t_0$ seems unreliable. They recommended t_0 of 1 day for clays and 0.5 day for sands. At the LA-1 relocation site, a large number of restrikes were performed on the test piles within 24 hours after pile installation, which offers the chance to study the selection of the reference time.

After installation of the test piles, restrikes were usually performed at around 2, 4, 6, and 24 hours after the initial driving. Different restrike times were selected as the reference time t_0 and the corresponding setup parameter *A* values were calculated for each individual strike of each test pile using equation (13) with all the available restrike data. Each restrike ended up with one independent value of A. As a sample presentation of the calculations, Tables 18 and 19 give those *A* values at different restrike times corresponding to different t_0 for the representative test piles at sites of T2 and T3.

Table 18
Setup parameter *A* values for the 16-inch square PPC pile at site T2

Restrike time (hrs)	Setup parameter *A* values corresponding to different reference time (hrs)						
	t0=2.2	t0=3.9	t0=6.0	t0=21.6	t0=56.0	t0=76.9	t0=98.9
2.2	—						
3.9	0.54	—					
6.0	0.74	0.88	—				
21.6	0.67	0.63	0.46	—			
56.0	0.72	0.66	0.53	0.50	—		
76.9	0.78	0.72	0.60	0.58	0.70	—	
98.9	0.72	0.66	0.54	0.48	0.38	0.06	—
168 (Load Test)	0.84	0.78	0.66	0.62	0.60	0.51	0.75

Table 19
Setup parameter *A* values for the 30-inch pipe pile at site T3

Restrike time (hrs)	*A* values corresponding to different reference time (hrs)					
	t0 = 2.3	t0 = 4.1	t0 =24.1	t0 =48.9	t0 = 76.3	t0 =172.5
2.3	—					
4.1	1.93	—				
24.1	0.70	0.20	—			
48.9	0.62	0.21	0.195	—		
76.3	0.57	0.20	0.177	0.14	—	
172.5	0.52	0.20	0.174	0.15	0.16	—
360.0 (Load Test)	0.79	0.43	0.500	0.57	0.68	1.19

Table 18 displays those *A*s for the 16-inch PPC pile at load test site T2. It shows that the *A* values fall in a narrow range for each reference time, for example, in the range between 0.54 and 0.84 for t_0 = 2.2 hours, between 0.63 and 0.88 for t_0 = 3.9 hours, and between 0.46 and 0.66 for t_0 = 6.0 hours. If t_0 is taken as 21.6 hours, the *A* values will range between 0.50 and 0.62. The results indicate that a small reference time does not cause a large statistical variation of *A* values, which implies a good agreement between the measured bearing capacities and predicted ones following the Skov-Denver model, even though a very small reference time such as a t_0 of 2.2 hours is used. The results indicate that selection of the reference time is not critical in the setup prediction. Consistent *A* values corresponding to a small reference time demonstrate that pile resistances in the early stages after initial driving are as predictable as those capacities corresponding to a certain period of time after initial driving. Similar results were observed from the test piles at T2, T3, and T4, which were presented in Appendix C. Notable exceptions are the

30-inch pipe pile at T3 and the 160-foot long PPC pile at T4. Table 19 shows the calculated *A*s for the pipe pile at T3. However, further analyses showed that large variations of *A* values corresponded to all the reference times employed. Therefore, it is reasonable to believe that for the individual piles, the high variations may not originate from the selection of a small reference time. It is suspected that the high variation was a result of poor CAPWAP analyses.

Based on the restrike data that have been analyzed, it is seen that pile capacities at very early restrikes that were usually performed within less than one day are also valuable in establishing those prediction models. It is noted that the *A* values corresponding to a small reference time are largely different from those corresponding to the 24-hour restrike or the restrikes at a larger reference time. Engineers must exercise their cautions in selecting an appropriate *A* for prediction when different reference times are used. The research based on the limited amount of data has seemingly endorsed the selection of a small reference time for pile capacity prediction. However, the restrikes within 24 hours after initial driving were rarely performed. The reference time t_0 of 24 hours has been mostly reported by other researchers and engineers. Therefore, the 24-hour reference time is used in the research in order to make a compatible comparison.

Distribution of the Setup Parameter *A*

Based on equation (13), the setup parameters *A* were computed for all restrikes. Two types of *A* values have been obtained from the production pile restrikes and the load test piles, and they are presented in Figures 18 and 19, respectively. Also, those *A* values are plotted as histograms in Figures 20 and 21, respectively.

As described in the beginning of the report, only one or two restrikes were performed for each of the production piles. The setup parameters *A* from the production piles had to be computed by grouping restrike data from multiple piles in adjacent area from the same bent or adjacent bents. However, in contrast, there were generally five or six restrikes on each individual test pile before it was tested to failure. As a result, those *A* values from the pile load tests exhibited less dispersions than those *A*s from the production pile restrikes. The average values of *A*, standard deviations, and coefficients of variation for the two scenarios (the production pile restrikes and the pile load tests) are 0.68, 0.71, and 1.04 and 0.59, 0.39, and 0.67, respectively. Apparently, the test piles provided more reliable results than the production piles. If several restrike records are available for a single pile, it suggests that a model be established using that data to predict its capacity. Pile setup prediction from the model based on the restrike data from the pile itself is usually more accurate than the models with the restrike data of other piles involved.

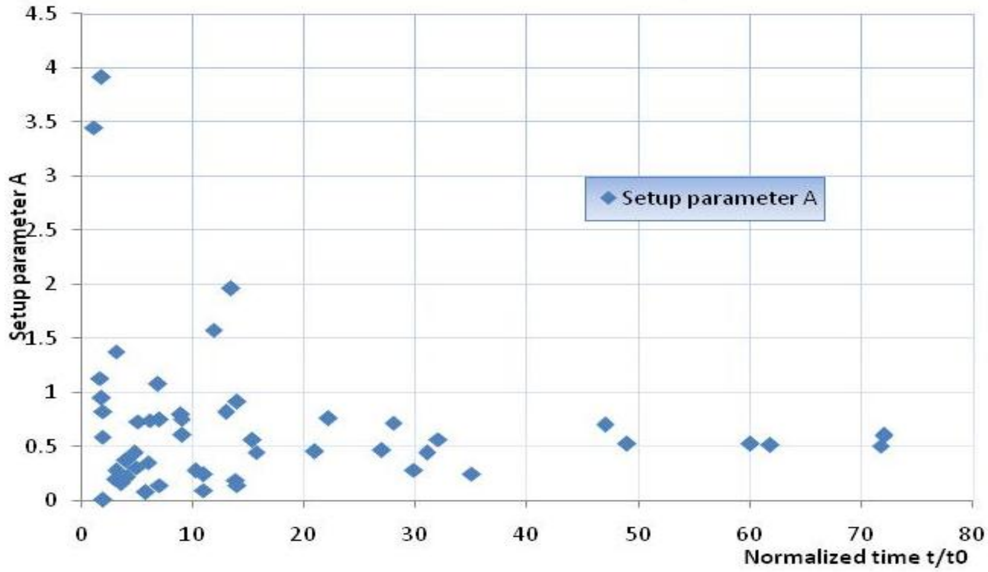


Figure 18
Variations of setup parameter *A* with time (production pile restrike data)

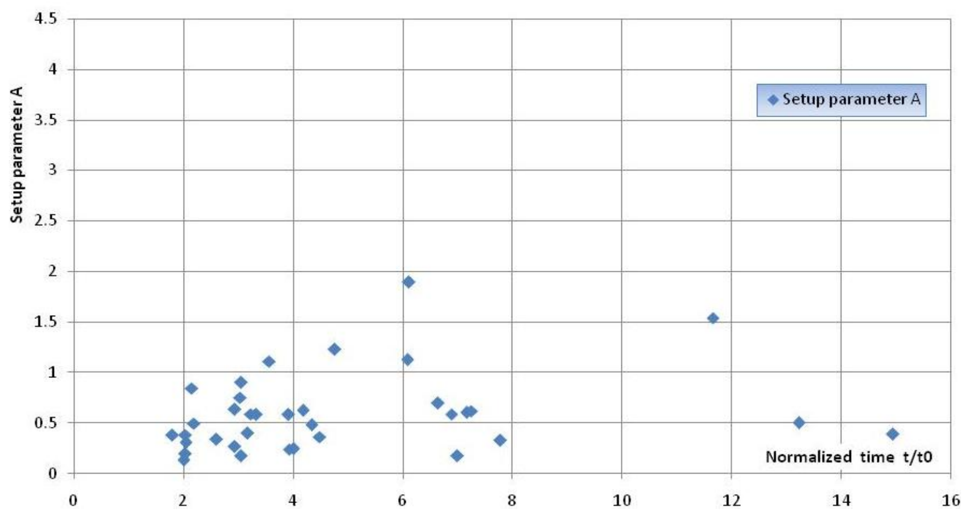


Figure 19
Variations of setup parameter *A* with time (pile load testing data)

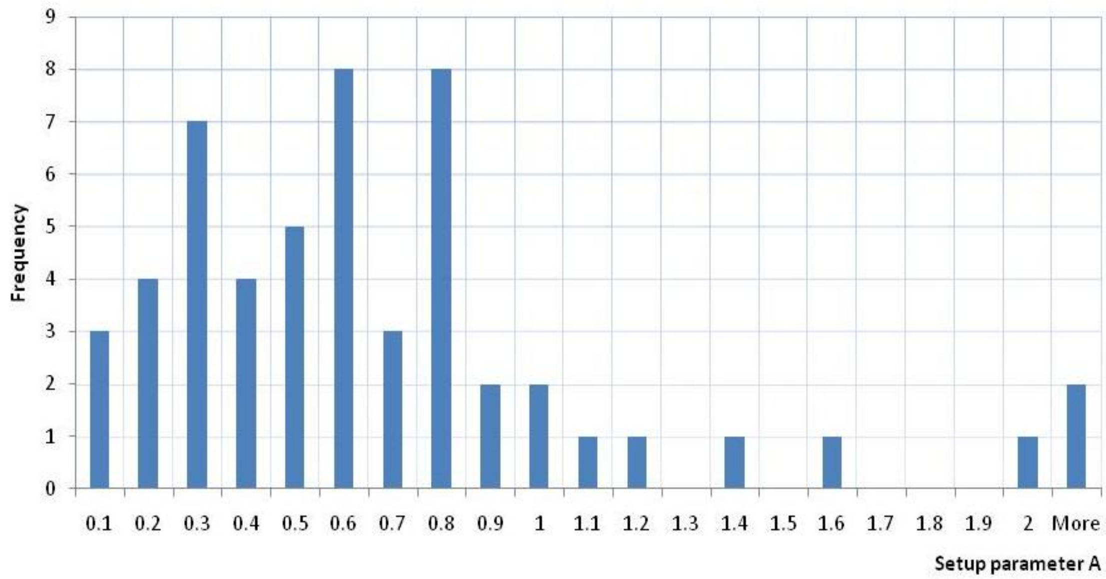


Figure 20
Distribution of the setup parameter *A* (all the combined production pile restrikes)

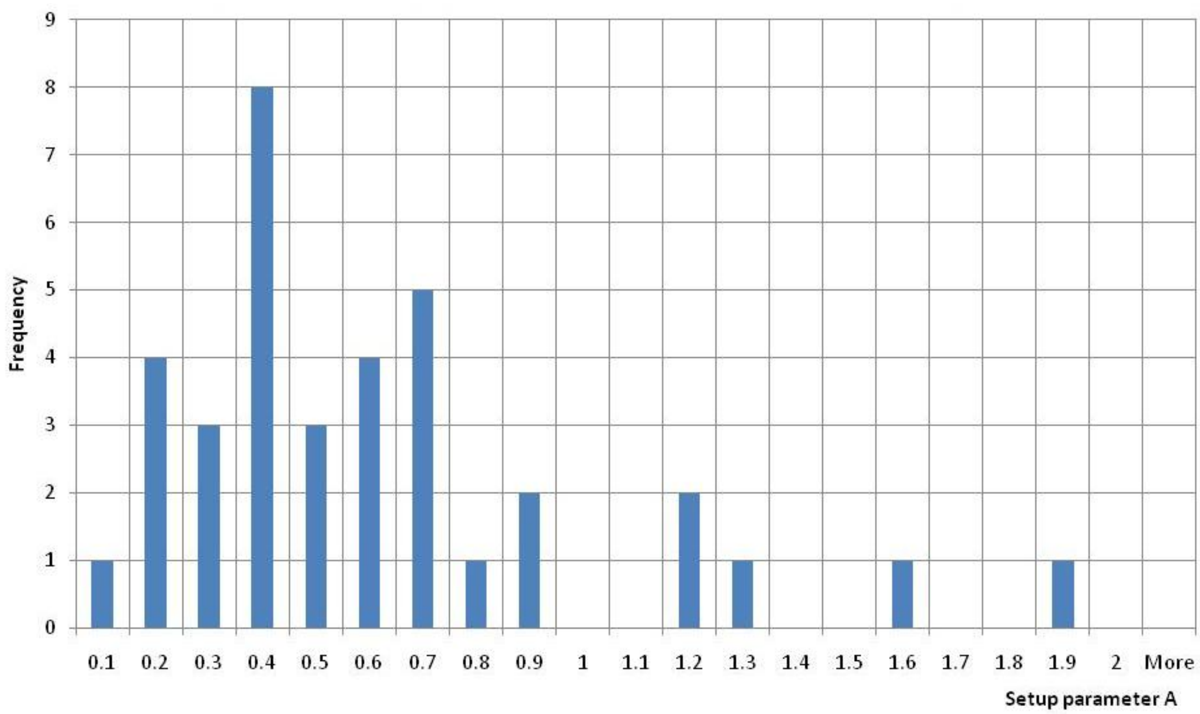


Figure 21
Distribution of the setup parameter *A* (all the nine test piles)

The Growth Rate-based Models

In addition to the pile capacity growth rate-based model for the site of North Connector, which was presented for model development demonstration before, the growth rate-based model was also developed for the other three segments of the LA-1 site and the combined data of the nine test piles, respectively. All the model parameters are given in Table 20.

Table 20
The two-parameter rate-based models from the different data sources

Data source	North Connector	South Connector	Main line-S	Ramp N1	LA-1 Relocation (combined data)	The nine tested piles
Initial growth rate of the normalized unit skin friction r_0	0.238	4.131	0.069	0.221	0.213	0.308
Ultimate normalized unit skin friction S_∞/S_0	2.161	1.414	1.857	1.872	1.865	1.840

As the last step in developing the rate-based models, a synthetic model from all the integrated data of production and test piles was developed as presented as equation (14). Using all the collected data of the normalized unit skin friction or normalized skin friction on which the Skov-Denver model was established, the rate-based model was developed. The equation is written as:

$$S(t) = \frac{1.846S(t_0)}{1+0.846e^{-0.261(\frac{t}{t_0}-1)}} \tag{14}$$

where, S(t) is the predicted skin friction at time t, and $S(t_0)$ is the measured skin friction at reference time t_0.

Prediction of the Ultimate Skin Frictions

As shown in Table 20, the initial unit growth rates ($ds/dt/s_0$) of the normalized unit skin friction ranges normally between 0.221 (Ramp N1) and 0.308 (combination of the nine test piles), except the extremely large rate of 4.131 at the South Connector and the extremely small rate of 0.069 at the mainline. However, the ultimate normalized unit skin frictions do not vary drastically, with the smallest 1.4 at the South Connector and the largest 2.2 at the North Connector. It implies that the ultimate skin friction was around twice as much as the skin friction measured at the 24-hour restrike. Based on the model presented as equation (14), at the site of the LA-1 relocation project, the shaft capacity generally gained 90~95 percent of the ultimate shaft capacity two weeks after the pile installations. Examples of pile capacity prediction using equation (14) are presented in Appendix D.

As indicated before, there are only 21 restrike records of two or more than two weeks available from the production piles. The conclusion made in this research regarding the ultimate pile capacity prediction needs to be validated in the future engineering practice. A reliable prediction of the ultimate pile capacity depends largely on the availability of large volume of long-term pile restrike or load testing data. Outcomes of the research project indicate that more research efforts must be made before long term predictions can be used in engineering practice.

Comparison of the Prediction Models from Different Data Sources

The Skov-Denver and the rate-based prediction models were developed based on the same data sources. The measured unit skin frictions, the predicted skin frictions from the Skov-Denver and the rate-based models, respectively, were plotted in Figure 16 for the North Connector and Figures 22 through 25 for the South Connector, mainline, Ramp-N1, and the nine tested piles, respectively. For the sake of comparison, the model parameters and the SSR (sums of the squared residuals) are presented in Table 21. It appears that the predictions from the rate-based model are slightly more accurate than those given by the Skov-Denver model. Figure 26 has presented the measurements and model predictions from all the data of the production piles at the LA-1 relocation project site, and Figure 27 shows similar results from all the collected data from the production and test piles.

Table 21
Summary of the SSRs between the Skov-Denver model and the rate-based model

Data Source	North Connector	South Connector	Mainline	Ramp N1	LA-1 Relocation (combined data)	The nine tested piles
Skov-Denver	3.80	1.03	1.39	1.50	7.65	3.32
Rate-based	2.99	1.96	1.43	1.40	7.59	3.11

As described before, the Skov-Denver model cannot predict the ultimate pile resistance, while the proposed rate-based model can be used to get the job done. In pile foundation practice, one could combine these two models for pile resistance predictions if a limited amount of restrike data is available. Parameter *A* of the Skov-Denver model is determined first, which will generate data for the rate-based model. Finally, the ultimate pile resistance will be achieved from the rate-based model.

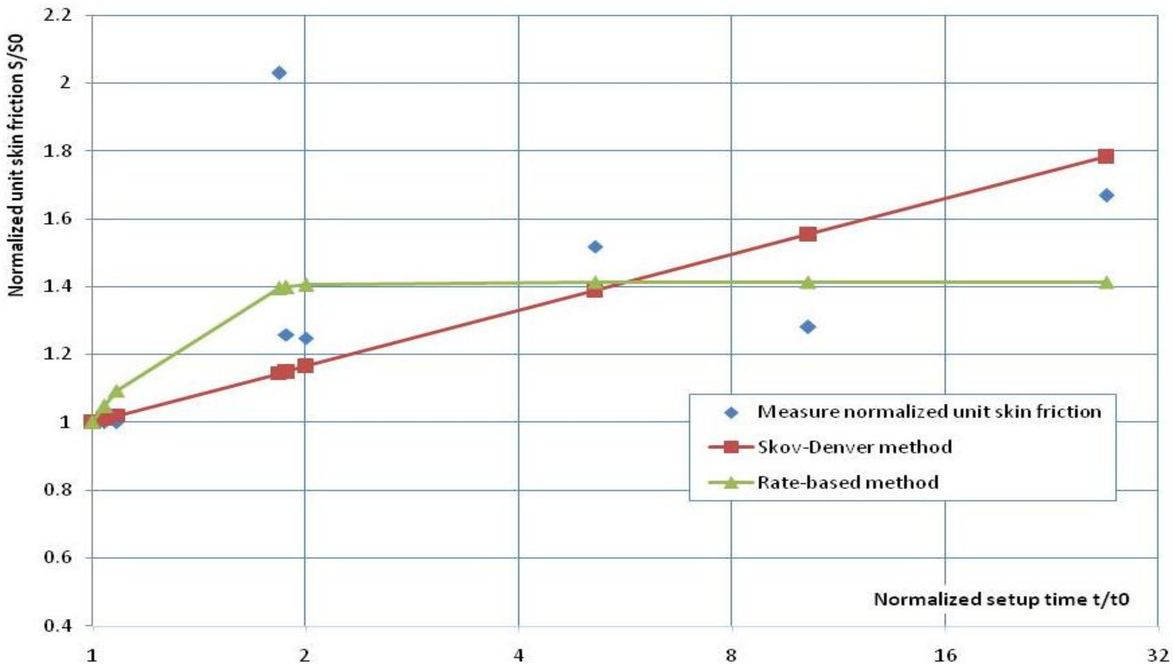


Figure 22
Normalized measured unit skin frictions and their predictions (South Connector)

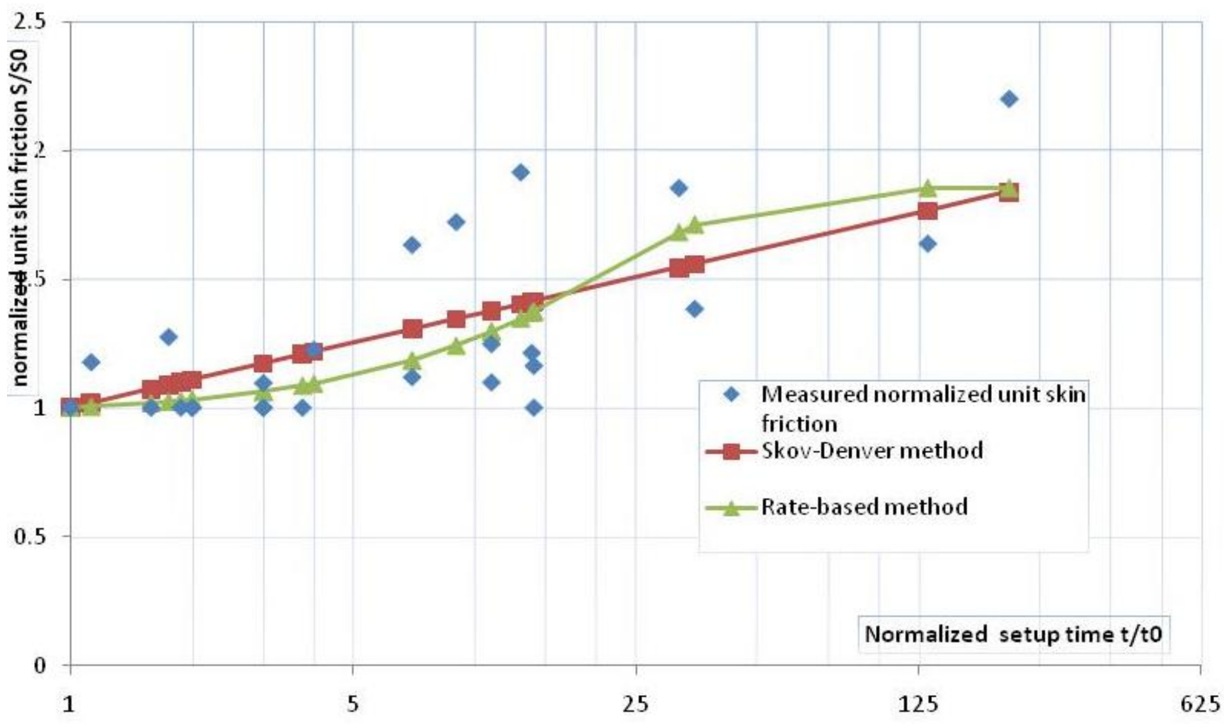


Figure 23
Normalized measured unit skin frictions and their predictions (mainline)

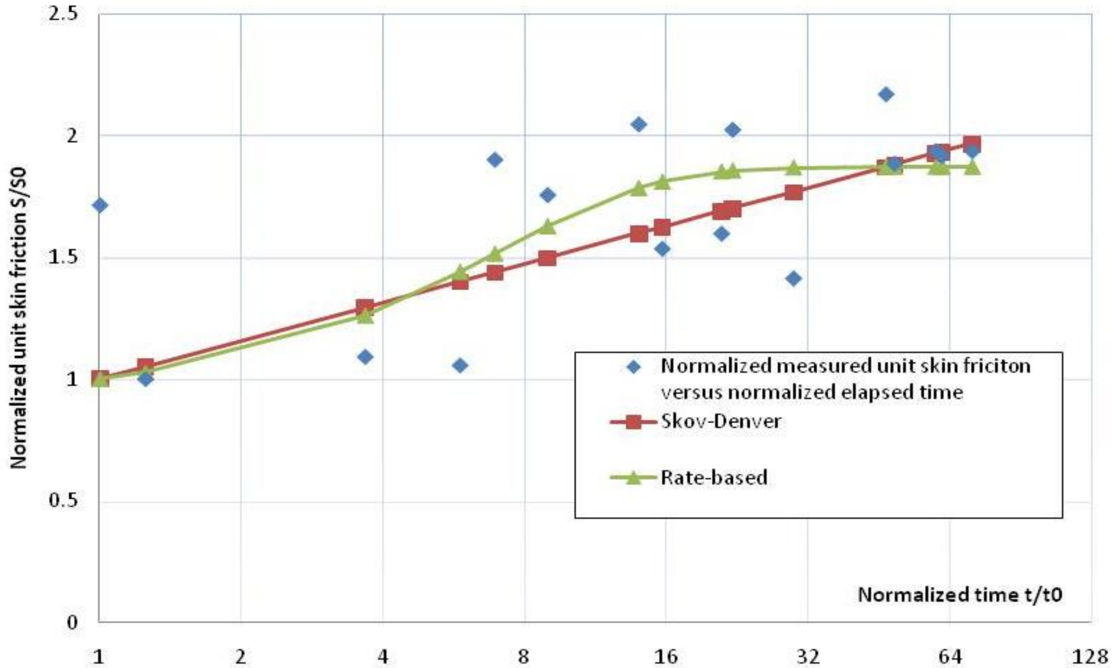


Figure 24
Normalized measured unit skin frictions and their predictions (ramp-N1)

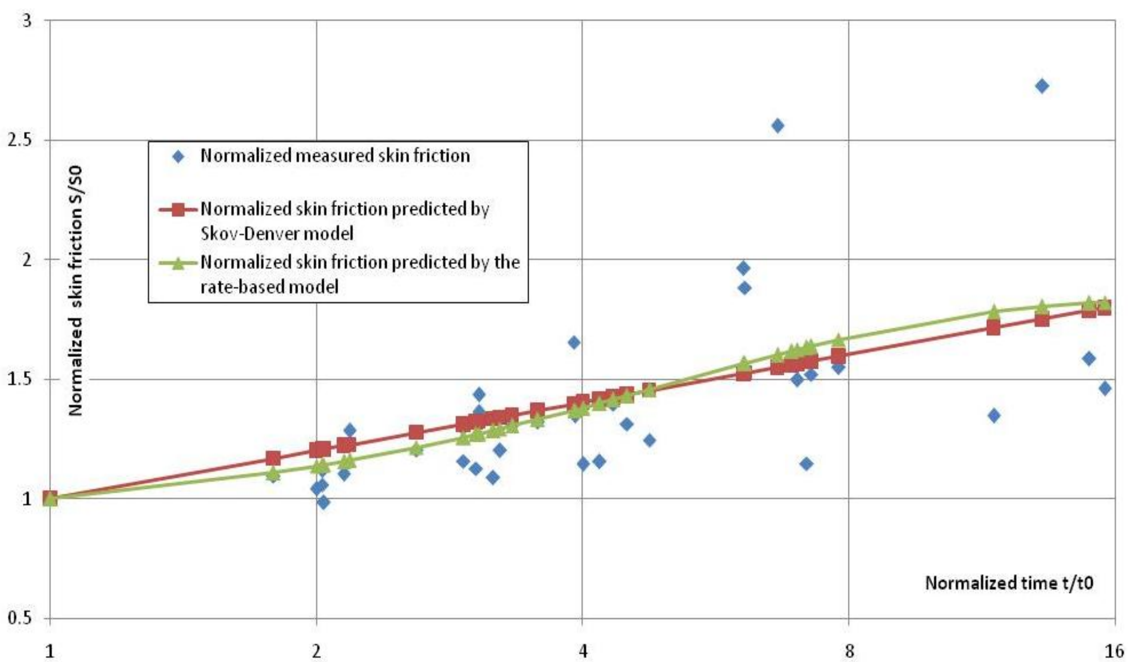


Figure 25
Normalized measured unit skin frictions and their predictions (test piles)

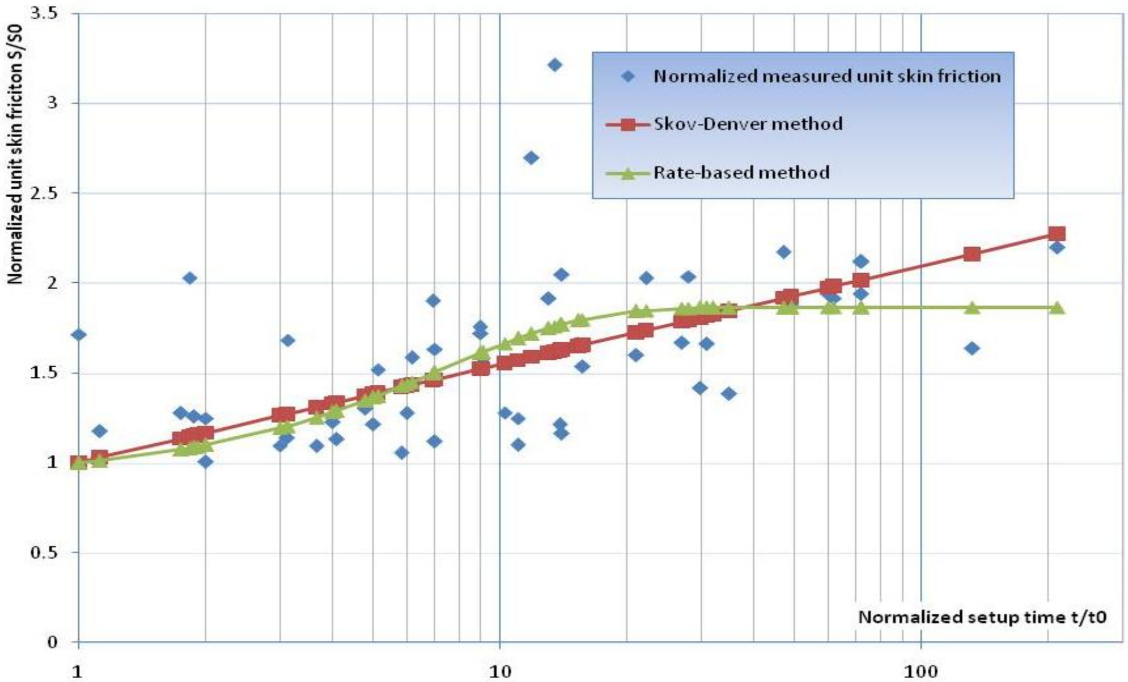


Figure 26
Normalized measured unit skin frictions their predictions (all the production piles)

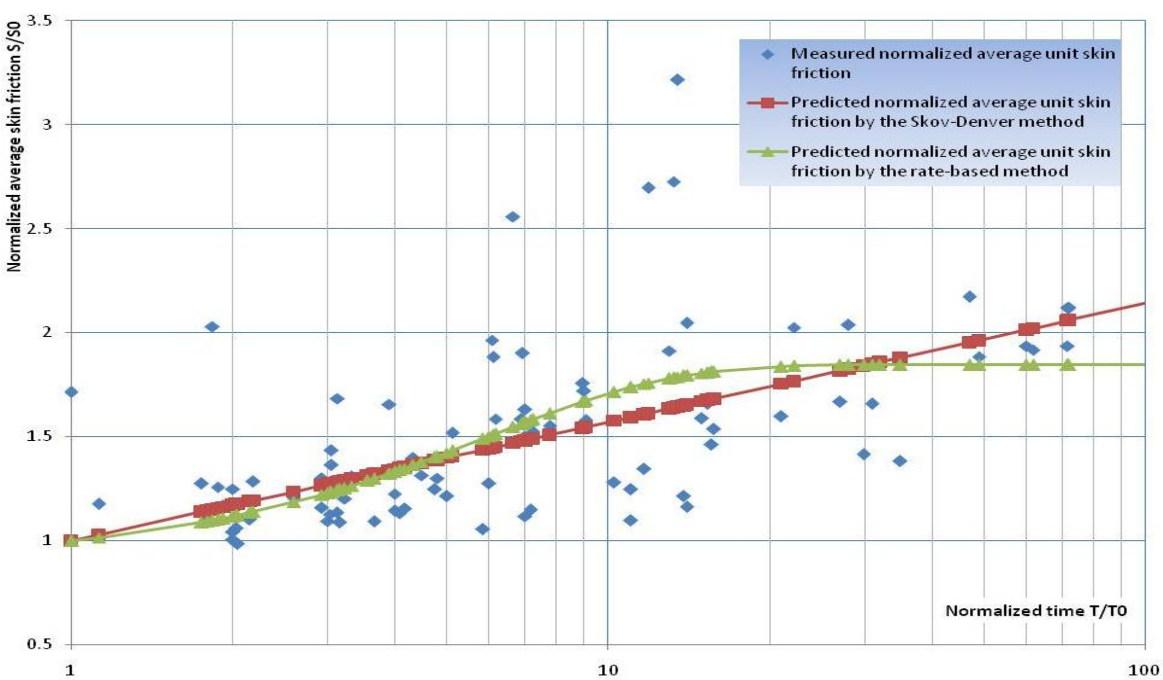


Figure 27
Normalized measured unit skin frictions and their predictions (data from all the production and test piles)

Prediction Models Development Based on the Selected Restrike and Load Testing Records

In the mathematical model development, most of the pile testing records came from short term restrikes of the production piles and the nine test piles at the LA-1 site. Seventy-one out of 115 restrike records of the production piles were achieved within 100 hours after pile installation. Therefore, the short-term pile testing records are over weighted in the model development. Subsequently, the long-term prediction of pile capacity, specifically, reliability of the ultimate pile capacity prediction would be questionable. In order to improve the accuracy and reliability of the prediction models, a large volume of long-term restrike or pile testing data is required.

In this research, an attempt to increase the weight of the long-term restrike and load test data was made by picking up those piles with 200 or longer than 200-hour restrike or/and load test records. Table 22 presents the selected pile capacity measurements from the three test piles at the test site No. 3. Each of the three piles held restrike or/and test records longer than 200 hours after the end of driving. In Table 23, many production piles present records longer than 200 hours. However, some piles are short of the 24-hour restrike records. For the same reason as described before, a pile capacity record at the reference time of around 24 hours is necessary for model development. As such, restrike records of around 24 hours from piles in adjacent area were selected and listed in the table.

Table 22
Selected pile testing data from the test piles

Pile Type	Striking Time (hrs)	R_{tip} (kips)	R_{skin} (kips)	R_u (kips)
T3--30-inch PPC	23.6	650	414	1065
T3--30-inch PPC	69.2	649	537	1187
T3--30-inch PPC	162.4	641	655	1297
T3--30-inch PPC	312.0	521	1129	1650
T3-30-inch Pipe	24.1	101	733	834
T3-30-inch Pipe	48.9	108	777	885
T3-30-inch Pipe	76.3	110	798	907
T3-30-inch Pipe	172.5	115	842	958
T3-30-inch Pipe	360.0	434	1163	1597
T3-54-inch Cyln.	24.7	141	886	1027
T3-54-inch Cyln.	44.2	141	971	1112
T3-54-inch Cyln.	72.4	143	1026	1169
T3-54-inch Cyln.	117.4	143	1104	1247
T3-54-inch Cyln.	287.7	144	1193	1337
T3-54-inch Cyln.	384.0	100	1295	1395

Table 23
Selected pile restrike records of the production piles

Pile	Type of Pile	Date	Time	R_{skin}	R_{tip}	R_{ult}
NC29-03	24" SQ. PPC, Hollow	12/21/2006	24	213	82	294
NC29-03	24" SQ. PPC, Hollow	12/27/2006	144	271	69	340
NC29-03	24" SQ. PPC, Hollow	1/17/2007	672	433	72	505
NC29-02	24" SQ. PPC, Hollow	1/20/2007	744	353	70	422
NC29-02	24" SQ. PPC, Hollow	3/2/2007	1728	451	59	510
NC29-03	24" SQ. PPC, Hollow	3/2/2007	1728	450	70	520
NC14-03	24" SQ. PPC, Solid	5/10/2007	42	259	97	356
NC14-03	24" SQ. PPC, Solid	6/4/2007	644	429	101	530
NC10-03	24" SQ. PPC, Solid	5/24/2007	24	145	75	220
NC10-03	24" SQ. PPC, Solid	6/4/2007	285	391	65	456
NC10-03	24" SQ. PPC, Solid	6/7/2007	323	466	70	536
41S-03	30" SQ. PPC Solid	3/30/2007	24	364	76	440
41S-03	30" SQ. PPC Solid	4/2/2007	96	446	68	514
41S-03	30" SQ. PPC Solid	10/26/2007	5040	800	100	900
65S-03	24" SQ. PPC Solid	01/09/08	24	193	82	275
65S-03	24" SQ. PPC Solid	01/21/08	312	369	60	430
84S-15	24" SQ. PPC Solid	03/07/08	24	165	67	232
84S-15	24" SQ. PPC Solid	03/15/08	216	284	40	324
106S-13	24" SQ. PPC Solid	02/28/08	840	603	228	831
106S-22	24" SQ. PPC Solid	02/28/08	336	506	122	628
117S-06	24" SQ. PPC Solid	04/16/08	3168	299	76	375
123S-03	24" SQ. PPC Solid	11/16/07	768	530	131	661
N1-24-02	24" SQ. PPC Solid	7/24/2007	88	186	66	252
N1-24-02	24" SQ. PPC Solid	8/23/2007	717	241	39	280
N1-24-03	24" SQ. PPC Solid	9/6/2007	1128	370	50	420
N1-17-02	24" SQ. PPC Solid	8/23/2007	30	120	87	207
N1-17-03	24" SQ. PPC Solid	9/6/2007	377	262	54	316
N1-17-02	24" SQ. PPC Solid	11/1/2007	1721	331	74	405
N1-14-02	24" SQ. PPC Solid	9/6/2007	140	180	60	240
N1-14-02	24" SQ. PPC Solid	11/1/2007	1484	327	72	399
N1-12-02	24" SQ. PPC Solid	9/17/2007	166	324	81	405
N1-12-02	24" SQ. PPC Solid	10/3/2007	532	345	100	445
N1-09-03	24" SQ. PPC Solid	10/3/2007	336	347	74	420
N1-02-03	24" SQ. PPC Solid	11/1/2007	504	272	69	341
N1-02-03	24" SQ. PPC Solid	11/29/2007	1176	321	80	401
SC45-02	24" SQ. PPC Solid	12/14/2006	24	317	69	386
SC54-03	30" SQ. PPC Solid	5/17/2007	648	739	211	950
SC59-03	30" SQ. PPC Solid	6/27/2007	24	345	190	535
SC61-04	30" SQ. PPC Solid	7/17/2007	246	405	149	554

Using the selected pile capacity data presented in Tables 22 and 23, the Skov-Denver model and the rate-based model were developed with the model parameters given in Table 24 and measurements and corresponding predictions are plotted in Figure 28. It shows that, with the weight of long-term pile capacity measurements enhanced, the pile setup parameter *A* has increased to 0.648, and the ultimate normalized skin friction has increased to 1.985. The rate-based prediction model implies that 90 percent of the ultimate skin friction has been gained at 14 days after the end of driving. The new models based on the selected data indicate that long-term pile capacity measurements play a vital role for those prediction models. A reliable and accurate prediction model depends largely on the availability of a large volume of long-term pile restrike data. Achievements of long-term data should be the focus of pile setup research at the next stage.

Table 24
Established models from the selected pile restrike and testing data

<table>
<tr><td>Mathematical model for pile setup prediction</td><td colspan="3">Parameters</td><td>Equations</td></tr>
<tr><td rowspan="2">Skov-Denver method</td><td colspan="2">Setup factor A</td><td>Reference time t_0 (hrs)</td><td rowspan="2">$\boldsymbol{S(t) = S_0 * \left(0.648 * Log\left(\frac{t}{t_0}\right) + 1\right)}$</td></tr>
<tr><td colspan="2">0.648</td><td>24</td></tr>
<tr><td rowspan="2">Rate-based method</td><td>Initial pile setup growth rate r_0</td><td>Ultimate normalized unit skin friction $q(\infty)$</td><td>Reference time t_0 (hrs)</td><td rowspan="2">$\boldsymbol{q(t) = \frac{1.985 q_0}{\left(q_0 + (1.985 - q_0) * e^{-0.172\left(\frac{t}{t_0} - 1\right)}\right)}}$
Note: q = $\frac{S(t)}{S_0}$, q_0=1</td></tr>
<tr><td>0.172</td><td>1.985</td><td>24</td></tr>
</table>

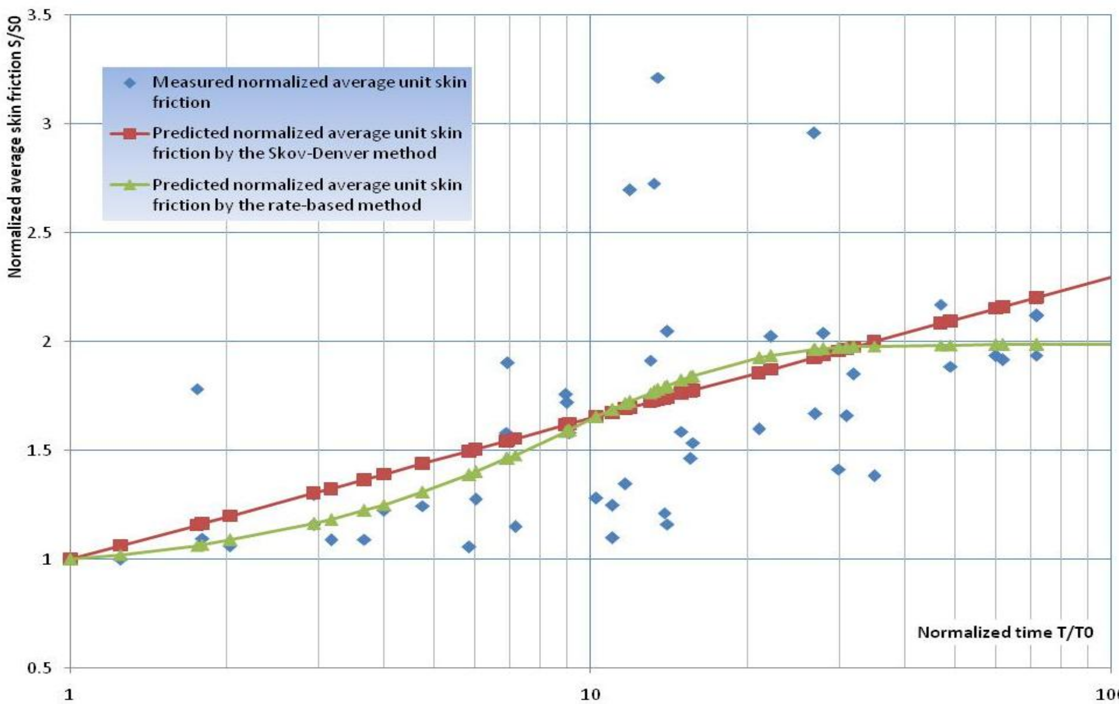


Figure 28
Normalized measured skin frictions and their predictions based on the selected pile restrike and load testing data

The Developed Correlation between the Measured Shaft Capacity at 24-hour Restrike and the CPT-based Computed Shaft Capacity

It has been known that pile capacity, with the setup effect taken into account, is able to be predicted on the basis of the pile capacity measured at a reference time, for instance, the pile capacity at the 24-hour restrike if the Skov-Denver model or the rate-based model is employed. However, the 24-hour restrike data are sometimes not available in pile foundation practice. The Project Review Committee suggested establishing an empirical relationship between the measured 24-hour pile capacity and relevant soil properties. In the research, the empirical equations involving the calculated pile capacity based on the developed CPT data. The digital data of partial CPT log at the LA-1 site was provided by Rauser *[30]*. In the pile foundation design practice of LADOTD, pile capacity is usually computed from the LCPC method, the Schmertmann method, or the de Ruiter and Beringen method, if CPT data is available. Following the assumption that pile tip resistance usually does not show a strong setup effect, the empirical relationships between measured skin friction at 24-hour restrike and the calculated skin friction were only established using the three methods, respectively. For comparison, an additional relationship between the measured 24-hour skin friction and the average skin friction from the three CPT methods was also presented.

In Table 25, the measured shaft capacities at 24-hour restrikes were picked up from all the production piles and the nine test piles at the LA-1 relocation project site. In Table 26, the quad root ratios of the measured skin friction to the calculated skin friction are presented for the three methods and the average result of the three methods. In the research, the relationship between the measured 24-hour skin friction and the calculated skin friction were presented as: (1) ratio of the measured skin friction to the calculated skin friction versus the calculated skin friction, (2) quad root of the skin friction ratio versus the calculated skin friction, and (3) the measured skin friction versus the calculated skin friction. Only results from the second and third correlations are plotted. They are presented in Figures 29 through 38.

Table 25
Measured shaft capacity at 24-hour restrike and the calculated shaft capacity using the CPT data log

Pile	Restrike Time (hrs)	Measured shaft capacity R_m (tons)	Calculated shaft capacity R_n (tons)			
			LCPC Method	Schmertmann Method	de Ruiter and Beringen method	Average values of the three methods
NC75-05	23	79.65	111	140	91	114
NC72-05	24	55.5	110	140	88	113
NC68-02	24	70	110	140	88	113
NC56-05	23	54.2	119	154	98	124
NC44-07	24	55	116	148	93	119
NC40-04	24	104	118	148	94	120
NC36-04	24	115.3	111	140	88	113
NC33-04	24	146.9	103	131	84	106
NC29-03	24	106.3	254	270	210	245
NC18-03	24	178.5	242	350	213	268
NC10-03	24	72.5	246	345	210	267
NC06-02	24	120	279	390	240	303
NC02-03	25	194.2	279	390	240	303
SC02-02	16	37.9	81	130	70	94
SC05-02	24	71.6	75	120	69	88
SC10-02	23	61.5	72	120	60	84
SC13-02	26	65	69	115	58	81
SC17-03	23	160.45	255	370	218	281

(continued)

SC21-03	25	120.5	287	410	247	315
SC25-02	24	192.5	250	370	218	279
SC29-03	24	177.5	250	365	217	277
SC33-03	24	175.25	250	365	217	277
SC37-03	24	179	250	365	217	277
SC45-02	24	158.5	211	345	176	244
SC59-03	24	172.5	241	390	200	277
20S-02	24	171.5	116	122	104	114
34S-02	24	163	210	210	180	200
37S-03	24	263.15	900	920	420	747
41S-03	24	182	190	210	150	183
53S-02	24	221.65	410	370	420	400
58S-03	24	149.15	226	260	210	232
61S-03	24	146.8	255	294	232	260
65S-03	24	96.5	295	325	261	294
69S-03	24	130	256	295	230	260
73S-02	24	148.5	261	290	234	262
82S-02	24	257.5	430	500	460	463
84S-15	24	82.4	201	200	180	194
87S-18	27	158.5	205	200	183	196
89S-21	24	187.05	346	375	315	345
105S-05	24	249.45	880	550	550	660
105S-22	24	186.15	880	550	550	660
109S-03	24	390.05	395	335	365	365
N1-21-03	24	146.35	182	205	210	199
T2--16-in. PPC	21.6	129	147	160	143	150
T2--54-in. Cylin.	23.2	394	505	590	530	542
T3-30-in. PPC	23.6	207	620	710	585	638
T3-54-in. Cylin.	24.7	443	720	830	630	727
T4-24-in. PPC-160	20.6	259	422	480	360	421
T4-24-in. PPC-210	23.7	333.5	600	660	565	608
T5-24-in. PPC-145	21.7	136	301	345	319	322
T5-24-in. PPC-170	23.6	180.5	470	500	480	483

Table 26
Quad root ratio of the measured 24-hour shaft capacity to the calculated shaft capacity from different methods

Pile	Quad root of the ratio of the measured capacity to the calculated capacity ($\sqrt[4]{Rm/Rn}$)			
	LCPC Method	Schmertmann Method	de Ruiter and Beringen method	Average of all methods
NC75-05	0.9204	0.8685	0.9672	0.9143
NC72-05	0.8428	0.7935	0.8912	0.8378
NC68-02	0.8932	0.8409	0.9444	0.8878
NC56-05	0.8215	0.7702	0.8624	0.8136
NC44-07	0.8298	0.7808	0.8769	0.8245
NC40-04	0.9689	0.9156	1.0256	0.9649
NC36-04	1.0095	0.9526	1.0699	1.0051
NC33-04	1.0928	1.0291	1.1500	1.0850
NC29-03	0.8043	0.7921	0.8435	0.8119
NC18-03	0.9267	0.8451	0.9568	0.9031
NC10-03	0.7368	0.6771	0.7665	0.7219
NC06-02	0.8098	0.7448	0.8409	0.7933
NC02-03	0.9134	0.8400	0.9484	0.8947
SC02-02	0.8271	0.7348	0.8578	0.7976
SC05-02	0.9885	0.8789	1.0093	0.9497
SC10-02	0.9614	0.8461	1.0062	0.9250
SC13-02	0.9852	0.8671	1.0289	0.9474
SC17-03	0.8906	0.8115	0.9262	0.8693
SC21-03	0.8050	0.7363	0.8357	0.7867
SC25-02	0.9367	0.8493	0.9694	0.9111
SC29-03	0.9179	0.8351	0.9510	0.8944
SC33-03	0.9150	0.8324	0.9480	0.8916
SC37-03	0.9199	0.8368	0.9530	0.8963
SC45-02	0.9310	0.8233	0.9742	0.8978
SC59-03	0.9198	0.8155	0.9637	0.8883
20S-02	1.1027	1.0889	1.1332	1.1075
34S-02	0.9386	0.9386	0.9755	0.9501
37S-03	0.7353	0.7313	0.8897	0.7705
41S-03	0.9893	0.9649	1.0495	0.9982
53S-02	0.8575	0.8798	0.8523	0.8628
58S-03	0.9013	0.8703	0.9180	0.8954
61S-03	0.8711	0.8406	0.8919	0.8666

(continued)

65S-03	0.7563	0.7382	0.7798	0.7571
69S-03	0.8442	0.8148	0.8671	0.8406
73S-02	0.8685	0.8459	0.8925	0.8679
82S-02	0.8797	0.8471	0.8650	0.8634
84S-15	0.8002	0.8012	0.8226	0.8076
87S-18	0.9377	0.9435	0.9647	0.9483
89S-21	0.8575	0.8404	0.8778	0.8579
105S-05	0.7297	0.8206	0.8206	0.7841
105S-22	0.6782	0.7627	0.7627	0.7288
109S-03	0.9969	1.0388	1.0167	1.0167
N1-21-03	0.9470	0.9192	0.9137	0.9261
T2--16-in PPC	0.9679	0.9476	0.9746	0.9630
T2--54-in Cylin.	0.9398	0.9040	0.9285	0.9235
T3-30-in PPC	0.7601	0.7348	0.7713	0.7546
T3-54-in Cylin.	0.8857	0.8547	0.9157	0.8836
T4-24-inch PPC-160	0.8851	0.8571	0.9210	0.8858
T4-24-inch PPC-210	0.8634	0.8431	0.8765	0.8605
T5-24-inch PPC-145	0.8199	0.7924	0.8080	0.8064
T5-24-inch PPC-170	0.7872	0.7751	0.7831	0.7817

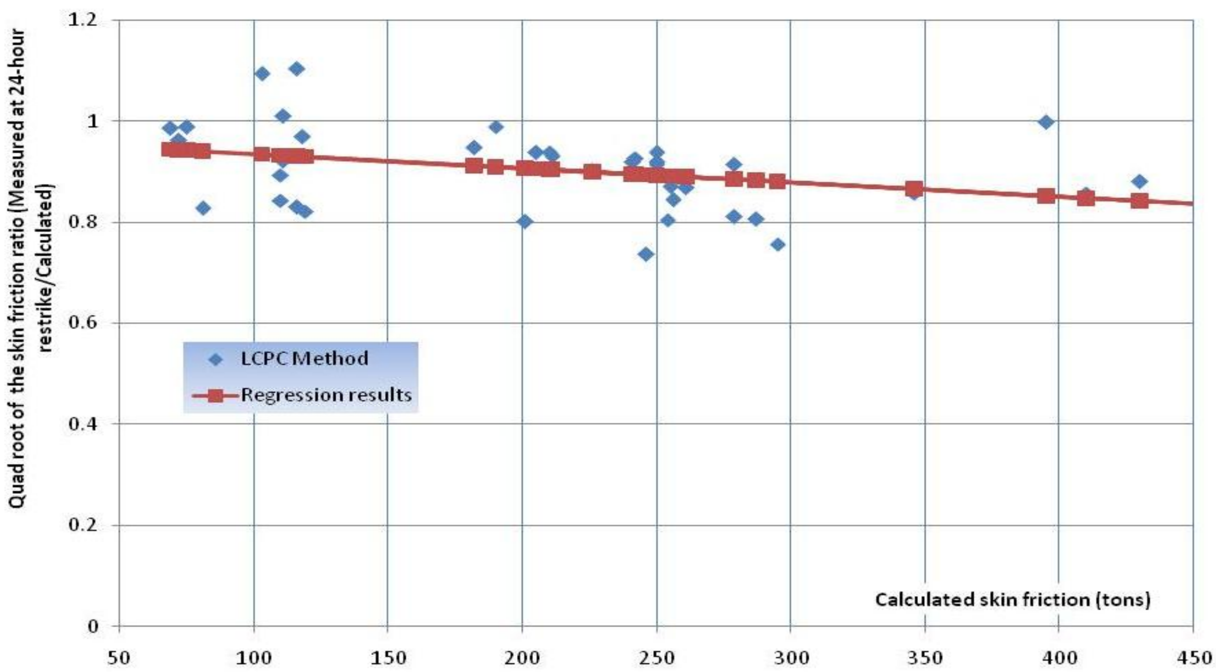


Figure 29
Quad ratio of the measured 24-hour skin friction and the calculated skin friction versus the calculated skin friction (LCPC method)

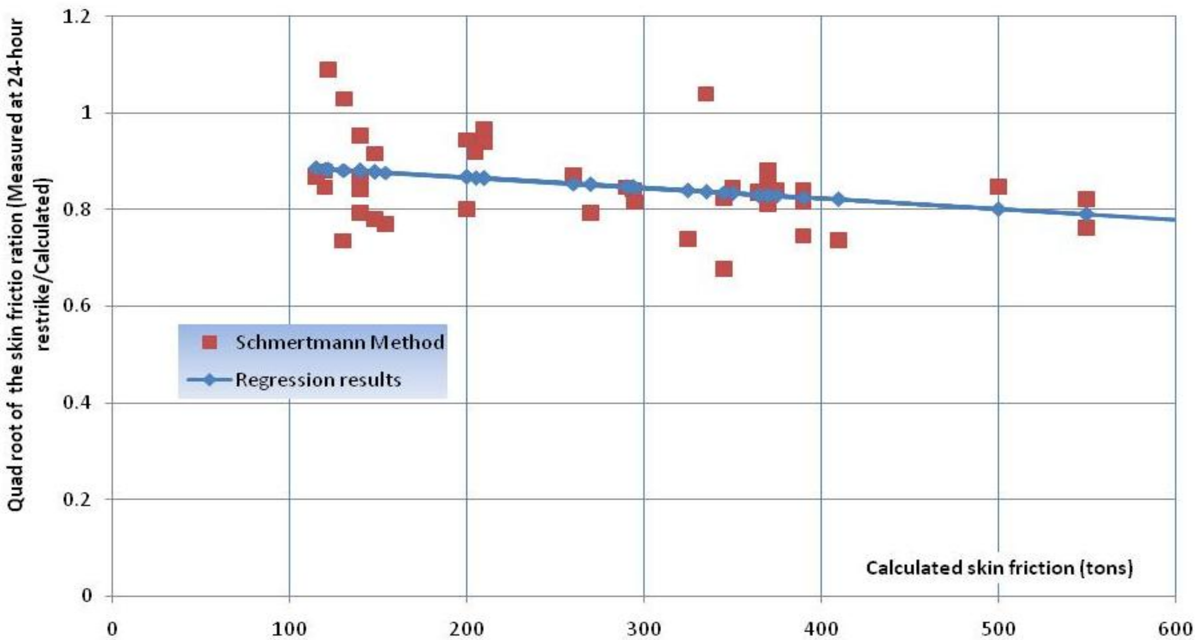


Figure 30
Quad ratio of the measured 24-hour skin friction and the calculated skin friction versus the calculated skin friction (Schmertmann method)

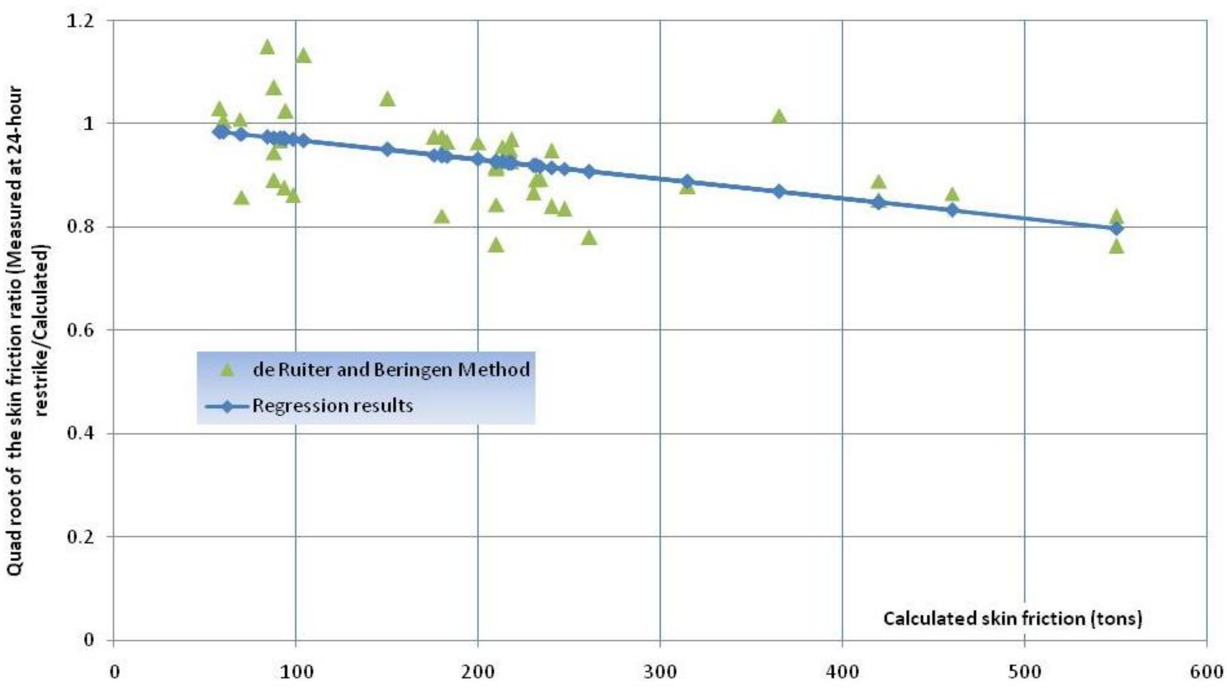


Figure 31
Quad ratio of the measured 24-hour skin friction and the calculated skin friction versus the calculated skin friction (de Ruiter and Berlingen method)

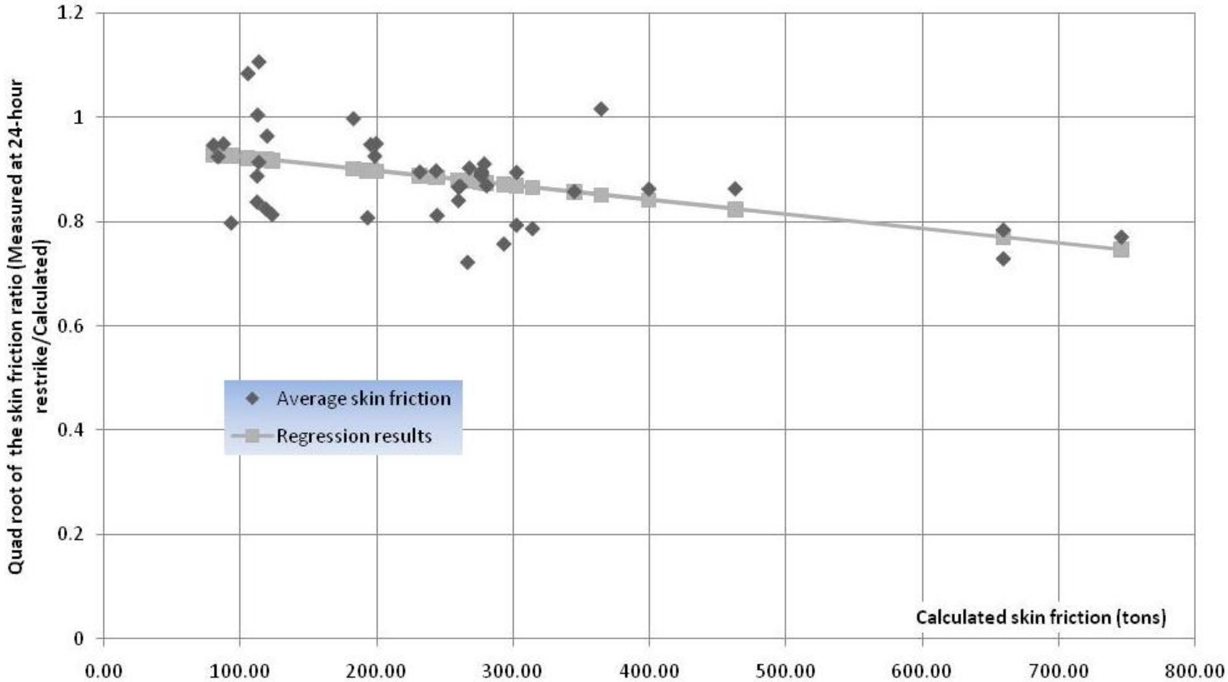


Figure 32
Quad ratio of the measured 24-hour skin friction and the calculated skin friction versus the calculated skin friction (average results from the three methods)

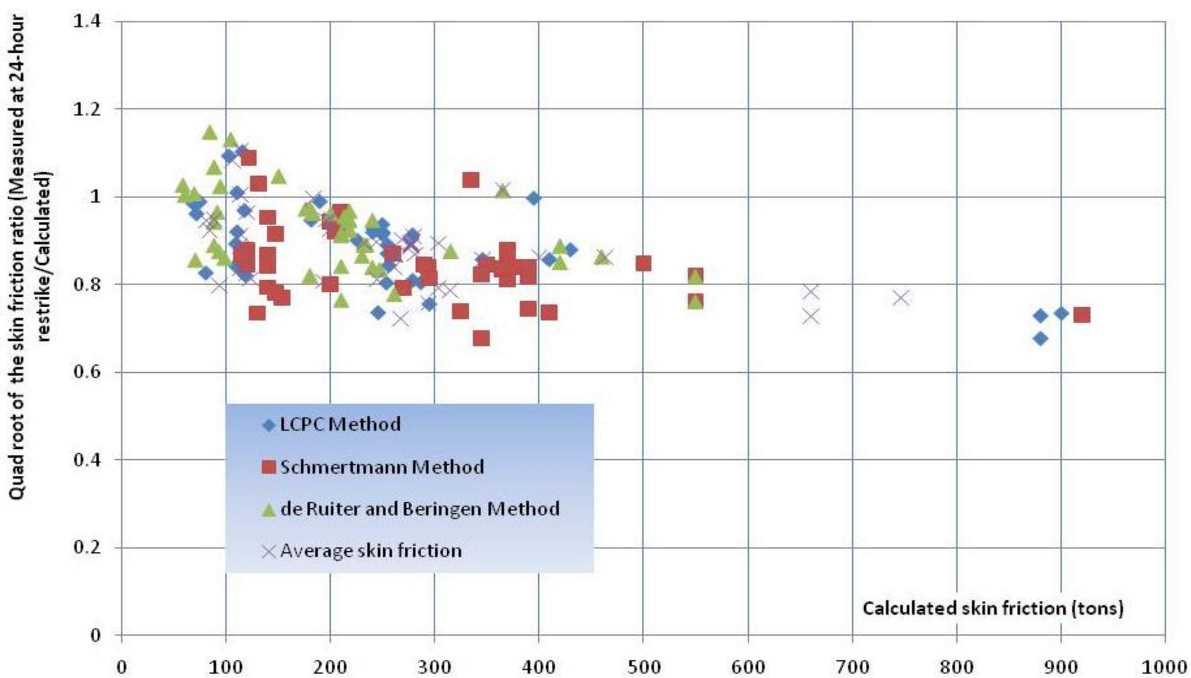


Figure 33
Quad ratio of the measured 24-hour skin friction and the calculated skin friction versus the calculated skin friction (mixed results)

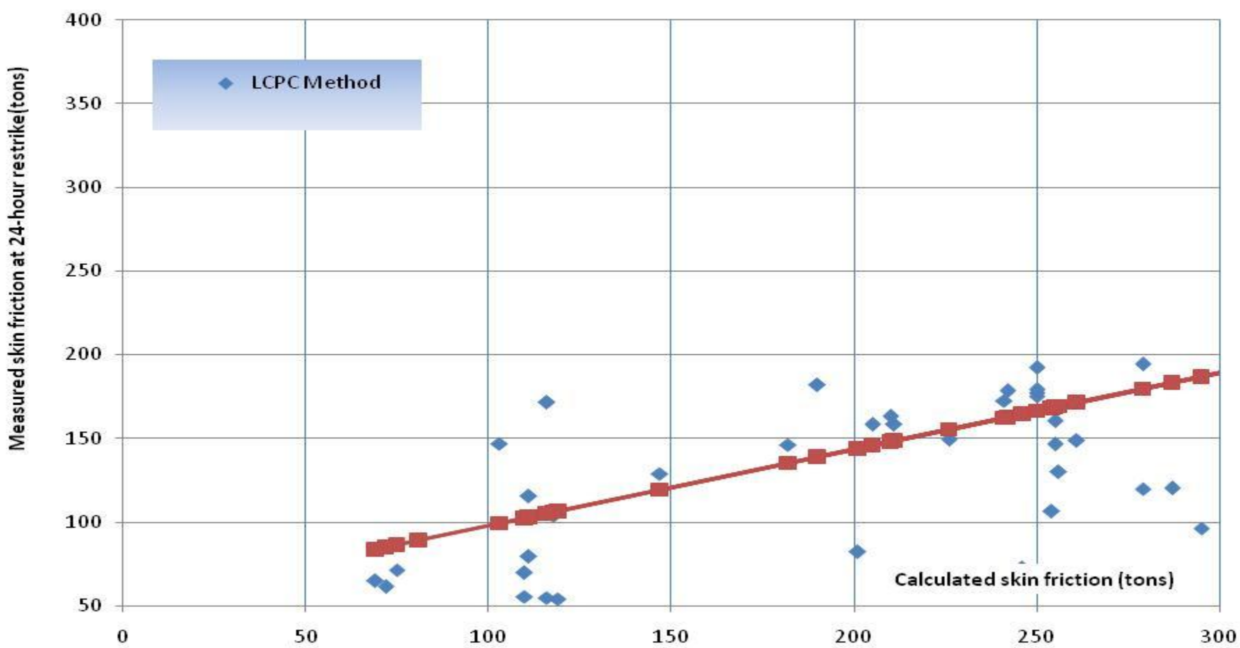


Figure 34
Measured 24-hour skin friction versus the calculated skin friction (LCPC method)

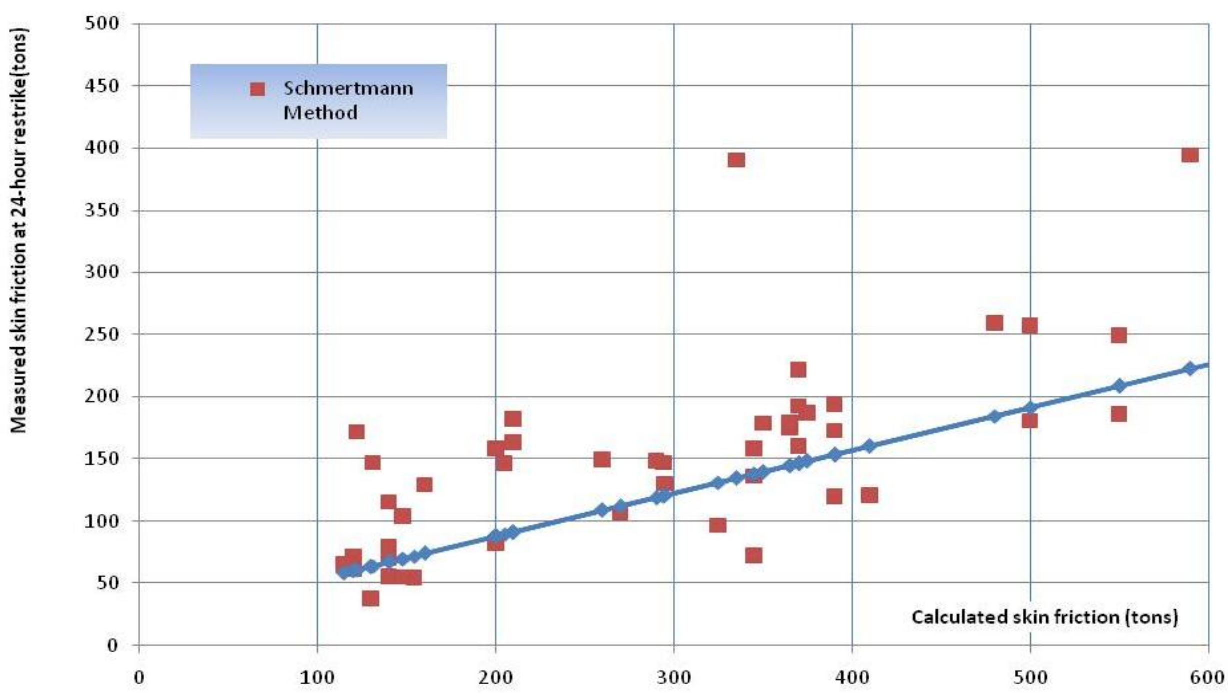


Figure 35
Measured 24-hour skin friction versus the calculated skin friction (Schmertmann method)

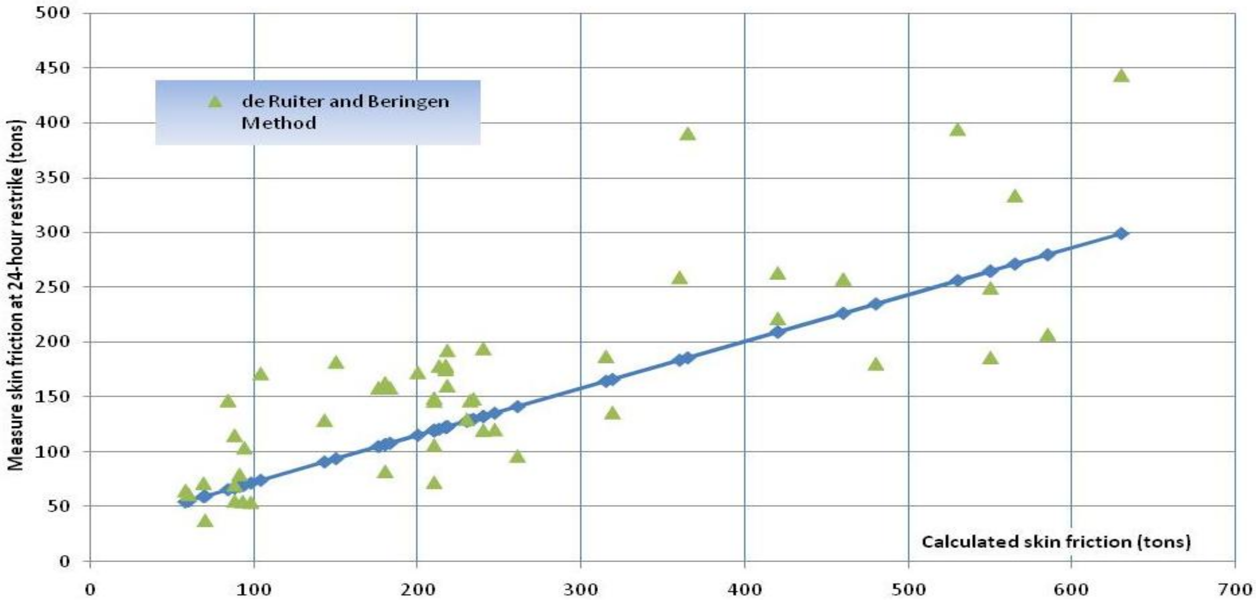


Figure 36
Measured 24-hour skin friction versus the calculated skin friction (de Ruiter and Berlingen method)

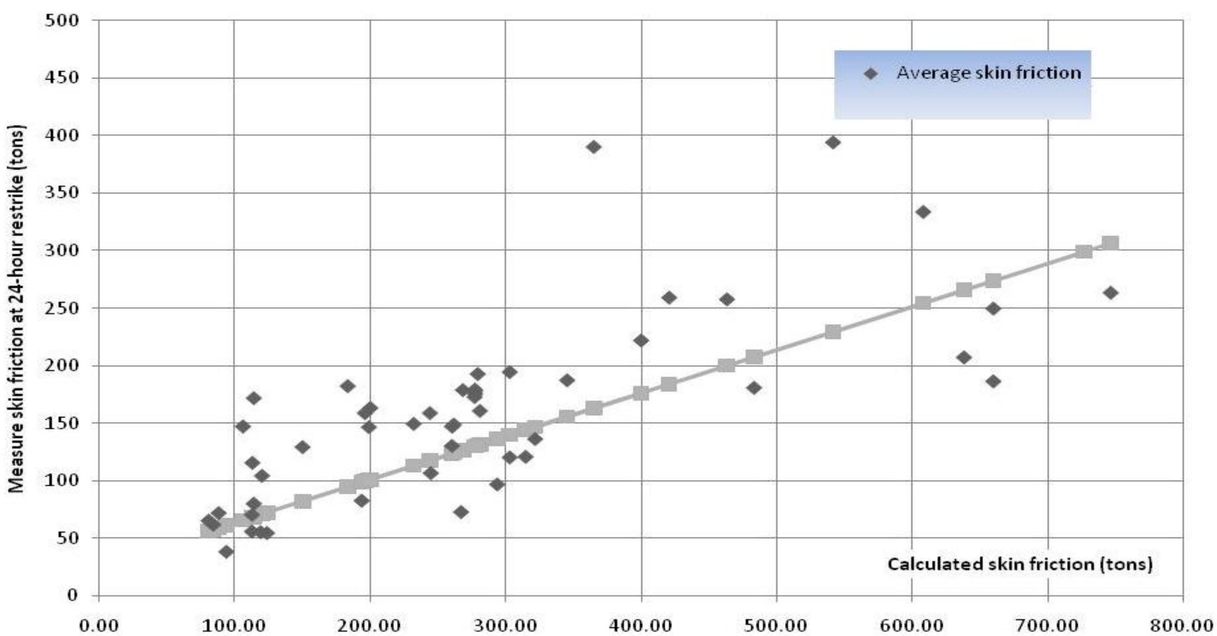


Figure 37
Measured 24-hour skin friction versus the calculated skin friction (average value of the three methods)

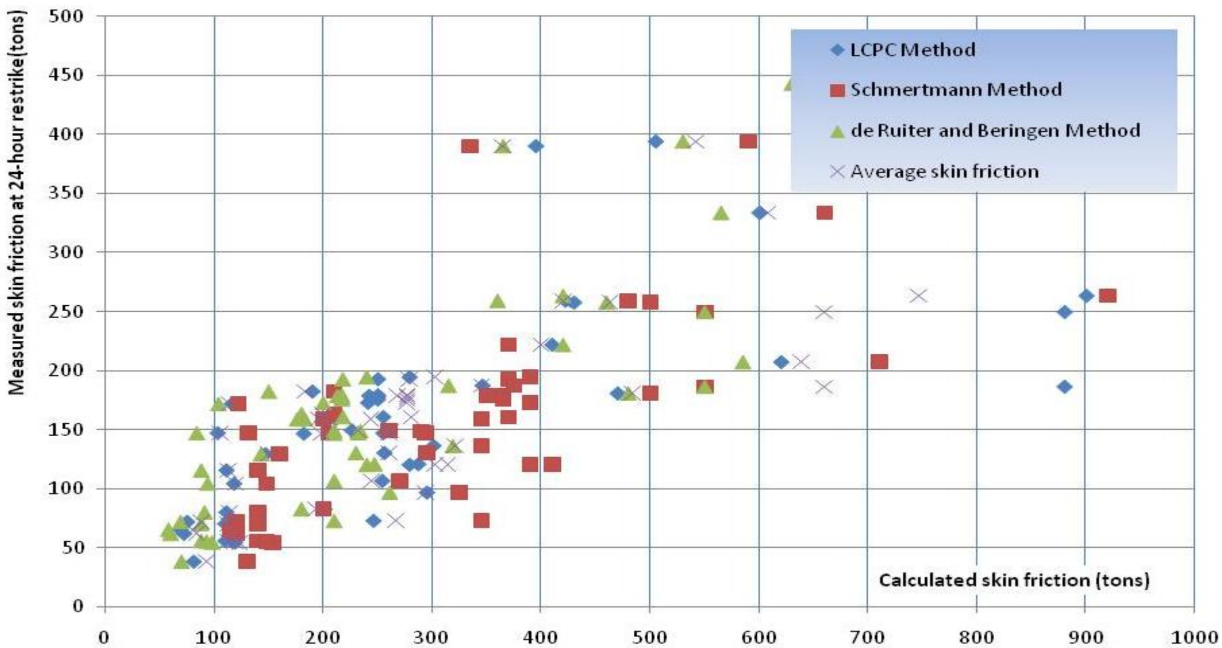


Figure 38
Measured 24-hour skin friction versus the calculated skin friction (mixed results)

The least square method was applied for the above plotted measurements and calculated results, and empirical equations for predicting the 24-hour skin friction were developed for the four methods, respectively. The dimensionless parameters, A and B, are shown in Table 27. As plotted in previous figures, the first type of equations was developed to establish the relationship between the quad root of the ratio of the measured 24-hour skin friction to the calculated skin friction and the calculated skin friction. SSR is presented in Table 27 as SSR_1. The second type of equations gave the straight linear relationship between measured 24-hour skin friction and calculated skin friction. Sum of the squared residuals of the second type of correlation was normalized by dividing up the SSR by the squared predicted 24-hour skin friction, and the subsequent normalized SSRs are presented in Table 27 as SSR_2. Differences between SSR_1 and SSR_2 indicate that the relationship of the quad ratio versus the calculated skin friction displays less variation. Therefore, the empirical relationship is recommended for 24-hour restrike skin frictions, if the measured one is not available.

Table 27
Empirical equations for the predicted 24-hour skin friction

Empirical equation	CPT method	LCPC	Schmertmann	De Ruiter and Beringen	Average
$\sqrt[4]{\frac{R_m^{24-hour}}{R_n}} = AR_n + B$	A	-2.4 × 10^{-4}	-1.60 × 10^{-4}	-3.00 × 10^{-4}	-2.20 × 10^{-4}
	B	0.96	0.90	0.99	0.94
	SSR_1	0.2664	0.2940	0.2874	0.2720
$R_m^{24-hour} = AR_n + B$	A	0.46	0.35	0.43	0.38
	B	52.19	18.71	30.24	25.95
	SSR_2	5.0654	5.5993	4.9970	5.1973
$SSR_1 = \left[\sqrt[4]{\frac{R_m^{24-hour}}{R_n}} - (AR_n + B)\right]^2$; $SSR_2 = [R_m^{24-hour} - (AR_n + B)]^2/(AR_n + B)^2$					

Resistance Factors of Pile Setup for the LRFD Calibration

In the calibration of LRFD method, the resistance factor ϕ was calculated following equation (11). In the process of calculating resistance factors, shaft capacity of each pile at different elapsed times was predicted using the Skov-Denver model and the rate-based model based on its measured capacity at the reference time of 24 hours, respectively. The bias factor was computed by dividing measured pile capacity by the corresponding predicted pile capacity.

Bias Factor Calculation

After pile installation, pile capacities corresponding to different elapsed times, e.g., two weeks, or one month after the end of driving are different. Thus, different resistance factors should be used for the predicted pile capacities at different times. However, because there is limited amount of pile testing data available, only one resistance factor is tentatively determined for all the predicted pile capacities at different setup times. In the LRFD implementation, the bias factor is defined as the ratio of measured capacity to predicted one. It is assumed that the bias factors of pile resistances at different setup times are independent of the setup time, thus all the bias factors at different setup times are combined to calculate the average bias factor λ_R. After the analyses, the bias factor corresponding to the Skov-Denver model is 0.997 and 0.991 for the rate-based model.

The Coefficient of Variation COV_R

Because of variation of pile capacity at different setup times, the coefficient of variation COV_R should be evaluated based on the pile capacities that were measured at the same time. Coefficients of variation COV_R at different setup times might be different. Most of the measured pile capacities came from the restrikes at around 24 hours after the end of driving based on the available data. Therefore, COV_R is calculated using the measured shaft capacity at around 24-hour restrikes, which is assumed to apply to the measured capacities at any other setup time. The calculated COV_R turns out to be 0.371.

Factors of Loads and Selected Target Reliability Indices

The dead and live loads are assumed as lognormal distributions. All these load factors and coefficients of variations were achieved from page 13 of the NCHRP Report 507 *[26]*. They are listed as:

$\gamma_{QL} = 1.75$,

$\gamma_{QD} = 1.25$,

$\lambda_{QL} = 1.15$,

$\lambda_{QD} = 1.05$,

$COV_{QL} = 0.2$, and

$COV_{QD} = 0.1$.

In the pile foundation design, the level of safety should be consistent with the LRFD-based pile foundation design manual issued by AASHTO, and a constant target reliability index should be used. Four target reliability indices (β_T) of 2.0, 2.33, 2.5, and 3.0 were selected in the research, and the ratio QD/QL (or Q_{DL}) was taken as 1.0, 2.0, 2.5, 3.0, and 4.0, respectively.

The Calibrated Resistance Factors for the Skov-Denver Model and the Rate-Based Model

The calibrated resistance factors are calculated as follows:

Table 28
Resistance factors for the two prediction models

	Q_{DL} =1.0 β_T=2.0	Q_{DL} =2.0 β_T=2.5	Q_{DL} =3.0 β_T=3.0	Q_{DL} =4.0 β_T=2.33	Q_{DL} =2.5 β_T=2.33
Skov-Denver	0.63	0.50	0.41	0.51	0.53
Rate-based	0.62	0.50	0.40	0.51	0.52

It is worthwhile to note that these resistance factors are paired up with the nominal resistances that are predicted by the Skov-Denver and the rate-based models, respectively. For instance, if the 14-day pile resistance is used in bridge foundation design, and the 14-day restrike resistance is not available, then engineers are able to use the developed Skov-Denver model [equation (12) or the equation in Table 24] or the rate-based model [equation (14) or the equation in Table 24] to predict the nominal 14-day pile resistance. As the last step, the predicted nominal resistance is multiplied by a selected resistance factor from Table 28, depending on the prediction model and the required target reliability index, to find the factored resistance.

Future Work to Do

Preliminary research has been done for the calibration of the LRFD to account for pile setup. In the future, more data of the measured pile capacities at different setup times and at different sites should be collected. The bias factors of the measured pile resistances should be based on the total pile resistances, and different coefficients of variations of the pile resistances should be applied to calibrate the resistance factors at different times, respectively. Resistance factors based on the two prediction models of pile setup need to be improved by employing more data that will be achieved from different places. Resistance factors considering pile setup with respect to different static calculation methods, such as the Alpha method and the different CPT methods, will be calibrated.

Application of the Rate-based Model: Ultimate Pile Prediction Capacity

Predictions at the Site of LA-1

The rate-based model is able to be employed for ultimate pile capacity prediction. In this session, predictions will be made and evaluated based on those static load testing or long-term restrike data. Table 29 presents some selected piles with restrikes or load tests conducted at or longer than 336 hours (two weeks) after the end of driving. Data in the column of “Restrike 1” represent the skin friction and tip resistance of a pile achieved at around 24-hour restrike or the first group of available restrike data after the end of driving. Data in the column of “Restrike 2” represent the shaft and tip resistances at the last restrike or the static load testing. The capacity ratios in the last column present the ratio of the shaft capacity of “Restrike 2” to that of “Restrike 1” and the ratio of the two total capacities, respectively. These ratios must be theoretically less than the predicted normalized ultimate shaft capacities by the rate-based model, which is defined as $S(\infty)/S(t_0)$ (t_0 = 24 hours) if the prediction is reasonable and accurate. If the predicted ratio is less than the measured one, it indicates that the measured capacity is under predicted. If massive predictions are under predicted, then the prediction model needs to be improved. Based on the established rate-based model, predictions were made, as shown in Table 30 for the selected piles.

Table 29
Selected long-term retrike data or load testing data

	Restrike 1			Restrike 2 (or load test)			Capacity ratio	
Pile	Time (hrs)	R_{skin}	R_{tip}	Time (hrs)	R_{skin} (kips)	R_{tip} (kips)	R_{skin}	Total
NC29-03	24	213	82	1728	450	70	2.11	1.76
NC14-03	42	259	97	644	429	101	1.66	1.49
NC10-03	24	145	75	323	466	70	3.21	2.44
41S-03	24	364	76	5040	800	100	2.20	2.05
65S-03	24	193	82	312	369	60	1.91	1.56
84S-15	24	165	67	216	284	40	1.72	1.40
106S-22	336	506	122	840	603	228	1.19	1.00
117S-06	3168	299	76	NA	NA	NA	NA	NA
123S-03	768	530	131	NA	NA	NA	NA	NA
N1-24-02	88	186	66	1128	370	50	1.99	1.67
N1-17-02	30	120	87	1721	331	74	2.76	1.96
N1-14-02	140	180	60	1484	327	72	1.82	1.66
N1-12-02	166	324	81	532	345	100	1.06	1.10
N1-09-03	336	347	74	NA	NA	NA	NA	NA
N1-02-03	504	272	69	1176	321	80	1.18	1.18
SC-54-3	648	739	211	NA	NA	NA	NA	NA
30’’- PPC Pile - T3	23.6	414	650	312	1129	521	2.73	1.55
30’’ Pipe Pile - T3	24.1	733	101	360	1163	434	1.59	1.91
54’’ Cylinder Pile - T3	24.7	886	141	384	1295	100	1.46	1.36

Table 30
Ultimate pile capacity prediction

Pile	Measured capacity at the last restrike or load test				Predicted ultimate capacity based on the model established from the entire database			Predicted ultimate capacity based on the model established from the selected data		
					$R(t_0) = 0.261$, and $s(\infty)/s(t_0) = 1.846$			$R(t_0) = 0.172$, and $s(\infty)/s(t_0) = 1.985$		
	Time (hrs)	R_{skin} (kips)	R_{tip} (kips)	R_{tot} (kips)	R_{skin} (kips)	R_{tot} (kips)	Under predicted ?	R_{skin} (kips)	R_{tot} (kips)	Under predicted ?
NC29-03	1728	450	70	520	393	475	Yes	423	505	Yes
NC14-03	644	429	101	530	420	517	Yes	452	549	---
NC10-03	323	466	70	536	268	343	Yes	288	363	Yes
41S-03	5040	800	100	900	672	748	Yes	723	799	Yes
65S-03	312	369	60	429	356	438	---	383	465	---
84S-15	216	284	40	324	305	372	---	328	395	---
106S-22	840	603	228	831	565	687	Yes	608	730	Yes
117S-06	3168	299	76	275	337	413	---	362	438	---
123S-03	768	530	131	661	528	659	Yes	568	699	---
N1-24-02	1128	370	50	420	260	326	Yes	280	346	Yes
N1-17-02	1721	331	74	404	210	297	Yes	226	313	Yes
N1-14-02	1484	327	72	399	231	291	Yes	248	308	Yes
N1-12-02	532	345	100	445	404	485	---	434	515	---
N1-09-03	336	347	74	421	312	386	Yes	335	409	Yes
N1-02-03	1176	321	80	401	286	355	Yes	308	377	---
SC-54-3	648	739	211	950	816	1027	---	877	1088	---
30''- PPC Pile - T3	312	1129	521	1650	764	1414	Yes	822	1472	Yes
30'' Pipe Pile - T3	360	1163	434	1597	1353	1454	Yes	1455	1556	Yes
54'' Cylinder Pile – T3	384	1295	100	1395	1636	1777	---	1759	1900	---

In Table 30, the measured shaft capacities and tip resistances at the last restrike or load test are listed on the left part for those piles with the last pile capacity records collected at or longer than two weeks after the end of driving. The ultimate capacity predictions were made, and the results were presented in the middle part and right side of the table. If the predicted ultimate total capacity is less than any measured total capacity, it indicates that the ultimate capacity is under predicted. The ultimate shaft capacities were predicted by the established rate-based model. Based on the assumption that there is no pile setup effect on the tip resistance, the predicted total capacity is the sum of the predicted shaft capacity and the measured tip resistance at the 24-hour restrike or the first available restrike record after the end of driving. As shown in Table 30, two predictions were made. The first one was made using the rate-based model on the entire restrike and load testing data. As described before, the majority of the pile capacity data came from the short-term restrikes within 100 hours after the end of driving, which has made the short-term data over weighted, or over represented, in the data volume. It was found that 13 of the 19 long-term measured capacities were under predicted. The second prediction was completed using the second rate-based model that was established on the selected pile restrike and load testing data in which those piles with the last restrike or load testing record conducted within 200 hours after end of driving were ruled out. With the weighted compensation in the long term records, the predictions have been improved (with 10 of 19 pile records under predicted). If more long-term measurements are available, the prediction models will be enhanced, and the prediction results will be improved.

Prediction Implementation at Other Sites

Predictions were also made using the established the Skov-Denver model (A = 0.570) and the rate-based model [$S(\infty)/S(t_0)$ = 1.846; r_0 = 0.260] for pile restrike and load testing data that were gathered at other sites. Measurements and predictions are presented in Table 31 (a), (b), and (c). Measured pile capacities are given in Table 31 (a), predicted capacities based on the Skov-Denver model are given in Table 31 (b), and the rate-based model predictions are in Table 31 (c). Overall, the mathematical models give reasonable predictions. In the predictions, only skin frictions were predicted using the models. To make the assumption consistent, it is assumed that tip resistance of any pile does not demonstrate any setup effect and is constant after the end of driving. Tip resistance at any prediction time takes the measured value at the 24-hour restrike. If the measured 24-hour tip resistance is not available, then the measurement at the restrike time closest to the 24- hour is assumed to be the tip resistance at any prediction time.

Table 31
Measured and predicted pile capacities at other project sites

(a)

Pile Location	Pile Name	Pile Dimension	Restrike or load testing time	Measured Pile Capacity		
				Skin Resistance (kips)	Tip Resistance (kips)	Total Resistance (kips)
Mo-Pac-Railroad Overpass, West Baton Rouge	TP-1	24" SQ PPC	48 Hrs	353	292	645
		24" SQ PPC	7 Days	424	224	648
	TP-3	24" SQ PPC	24 Hrs	234	38	272
		24" SQ PPC	9 Day	319	31	350
Bayou Liberty	TP-1	24" SQ PPC	3 Days	194	37	240
		24" SQ PPC	7 Days	351	58	409
Calcasieu River	TP-1	NA	1 Hr	484	210	694
		NA	20 Hrs	627	285	912
		NA	456 Hrs	1002	238	1239
		NA	432 Hrs (Static)	NA	NA	662
	TP-2	NA	1 Hr	370	599	969
		NA	96 Hrs	837	532	1369
		NA	408 Hrs	1009	663	1671
		NA	383 Hrs(Static)	NA	NA	662
Bogue Chitto Bridge # 1	NA	NA	2.4 Hrs	300	170	470
		NA	1 Day (24 Hrs)	350	230	580
		NA	14 Days (336 Hrs)	750	230	980
Bogue Chitto Bridge # 2	NA	NA	2.4 Hrs	200	120	320
		NA	1 Day (24 Hrs)	250	160	410
		NA	14 Days (336 Hrs)	590	160	750
Bogue Chitto Bridge # 3	NA	NA	2.4 Hrs	320	140	460
		NA	1 Day (24 Hrs)	340	140	480
		NA	14 Days (336 Hrs)	380	140	520

(b)

Pile Location	Pile Name	Restrike or load testing time	Skov - Denver Method (kips)			
			Skin Friction		Tip Resistance	Total Pile Resistance
			$S(t_0)$ ($t0 = 24$ hrs)	S(t)		
Mo-Pac-Railroad Overpass, West Baton Rouge	TP-1	48 Hrs	301	—	—	—
		7 Days		446	292	738
	TP-3	24 Hrs	234	—	—	—
		9 Day		361	38	399
Bayou Liberty	TP-1	3 Days	153	—	—	—
		7 Days		226	37	263
Calcasieu River	TP-1	1 Hr	656	—	—	—
		20 Hrs		—	—	—
		456 Hrs		1135	285	1420
		432 Hrs (Static)		1126	285	1410
	TP-2	1 Hr	623	—	—	—
		96 Hrs		—	—	—
		408 Hrs		1060	599	1659
		383 Hrs(Static)		1072	599	1649
Bogue Chitto Bridge # 1	NA	2.4 Hrs	350	—	—	—
		1 Day (24 Hrs)		—	—	—
		14 Days (336 Hrs)		578	230	808
Bogue Chitto Bridge # 2	NA	2.4 Hrs	250	—	—	—
		1 Day (24 Hrs)		—	—	—
		14 Days (336 Hrs)		414	160	574
Bogue Chitto Bridge # 3	NA	2.4 Hrs	340	—	—	—
		1 Day (24 Hrs)		—	—	—
		14 Days (336 Hrs)		562	140	702

(c)

Pile Location	Pile Name	Restrike or load testing time	Rate Based Method (kips)			
			Skin Friction		Tip Resistance	Total Pile Resistance
			$S(t_0)$ (t_0 = 24 hrs)	S(t)		
Mo-Pac-Railroad Overpass, West Baton Rouge	TP-1	48 Hrs	30	—	—	—
		7 Days		473	292	765
	TP-3	24 Hrs	234	—	—	—
		9 Day		391	38	429
Bayou Liberty	TP-1	3 Days	153			
		7 Days		239	37	276
Calcasieu River	TP-1	1 Hr	656	—	—	—
		20 Hrs		—	—	—
		456 Hrs		1203	285	1487
		432 Hrs (Static)		1200	285	1484
	TP-2	1 Hr	623	—	—	—
		96 Hrs		—	—	—
		408 Hrs		1135	599	1734
		383 Hrs (Static)		1130	599	1730
Bogue Chitto Bridge # 1		2.4 Hrs	350	—	—	—
		1 Day (24 Hrs)		—	—	—
		14 Days (336 Hrs)		628	230	858
Bogue Chitto Bridge # 2		2.4 Hrs	250	222	160	382
		1 Day (24 Hrs)		—	—	—
		14 Days (336 Hrs)		448	160	608
Bogue Chitto Bridge # 3		2.4 Hrs	340	—	—	—
		1 Day (24 Hrs)		—	—	—
		14 Days (336 Hrs)		610	140	750

CONCLUSIONS

The pile setup data from the LA-1 relocation project and other sites, including the data from the production piles and the nine test piles, have been evaluated. The restrike data from the production piles were specifically treated. The prediction model development was based on the average unit skin friction. Because of insufficient restrike data for some production piles, the average unit skin friction at the reference time had to sometimes be determined by grouping the restrike data from multiple piles in the adjacent area. After the 18-month research work, one can draw the following conclusions:

- The pile setup parameters *A* of the Skov-Denver model, with the reference time of 24 hours, range from 0.5 to 0.7 from different data sources for the PPC piles driven into the typical south Louisiana clayey soils at the LA-1 relocation project. If all the pile setup data at the site are compiled together for the prediction model development, the setup parameter *A* turns out to be 0.57. The second prediction model that was established on the selected piles with long-term restrikes or long-waiting load testing data (data collected at or longer than 200 hours after the EOD) has presented an *A* value of 0.65.
- The established capacity growth rate-based model is a better prediction model for the long term resistances of the piles than the Skov-Denver model, since it offers the capability of predicting the ultimate pile resistances.
- The ultimate shaft capacities of the piles were about 1.85 times the measured shaft capacities at the 24-hour restrike based on the first rate-based model. The ultimate shaft capacities would be 1.99 times the measured shaft capacity at the 24-hour restrike if the second rate-based model is employed, which is established on the selected pile testing data.
- In general, the piles at the LA-1 relocation project reached about 90~95% of the ultimate shaft capacities at two weeks after pile installation based on the established rate-based models.
- Preliminary model verification was done by applying the models for pile capacity predictions. If the rate-based model, which is based on the overall pile testing data, was used, 70 percent of the long-time pile capacity records (≥ 336 hours) were under predicted. If the second one was used, around 50 percent of the long-time records were under predicted.
- A carefully executed pile load test program will yield a better setup prediction model than the massive restrikes on production piles.
- Selection of the reference time does not cause a large statistical variation of *A* values.

- It is hard to evaluate the accuracy and reliability of the predictions. The prediction models were established on a small portion of long-term restrike data or long-waiting pile load testing data. Of all the 115 restrike records of production piles and 9 load testing piles, there are only 24 records that were achieved at or longer than two weeks after the EOD. In order to improve the predictability of the mathematical models, more long-term restrike or long-waiting pile testing data should be obtained.
- Various empirical equations have been established for the relationships between the 24-hour restrike shaft capacity and the CPT log-based shaft capacities, which were calculated from the LCPC method, Schmertmann method, de Ruiter and Beringen method, and average of the three methods. It is found that the relationship between the quad root of the ratio of the measured 24-hour restrike shaft capacity to the calculated shaft capacity and the calculated shaft capacity has presented the best correlation.
- Preliminary implementation work of the LRFD calibration was done to incorporate the pile setup effect for the resistance factors. The resistance factors corresponding to the Skov-Denver and the rate-based prediction models were calculated, respectively. They are very close numerically.
- The research team has followed other researchers by assuming that the pile setup effect only applies to the shaft capacity and that the predicted total capacity is equal to the predicted shaft capacity plus the tip resistance measured at around 24-hour restrike or at the end of driving, or the first available restrike after the end of driving. From all the data analyses, model evaluations, and predictions, it was found that the assumption is appropriate.

RECOMMENDATIONS

Based on the available dynamic and static field-testing data collected by LADOTD, preliminary work has been done in developing mathematical models to predict the pile capacity growth. The achievements have shown that pile capacity considering pile setup is predictable. However, the predictability of the models still needs to be improved with more dynamic and static testing data. It shows that more long-term pile capacity measurements may be able to give a larger predicted pile capacity. Therefore, a reliable and accurate prediction model depends largely on the availability of a large volume of long-term pile restrike data. Achievements of long-term data should be the focus of pile setup research at the next stage.

It is recommended that the developed prediction models be used to consider pile setup by pile foundation engineers in their design and construction work in different ways. In addition to the traditional restrike and load testing, the Skov-Denver and rate-based models are employed to estimate pile capacity after the end of driving as an additional tool. The beneficial use of the predicted pile setup could avoid the unnecessary increase in pile length if the measured pile capacity does not meet the design requirement during dynamic testing for construction quality control. Pile setup predictions need to be constantly validated from field measurements. These models will continue to be modified and improved. Eventually, they will become more robust in pile foundation design and construction and play an important role in engineering practice, like other pile design methods that are being used.

In order to implement the pile setup prediction in engineering practice, a detailed step-by-step implementation manual will help engineers get familiar with the pile setup prediction procedures using the mathematical models. A one- or two-day workshop should be conducted for prediction job training. A window-based software program, similar to the DRIVEN and the PileConeAnalysis etc., should be developed to make the pile setup computations simple and easy.

In addition to collecting long-term pile testing data to improve the developed statistics-based models, research attention needs to be paid to incorporate the mechanism of pile setup in the prediction models. Pore pressure and soil aging must be reflected in the mechanistic prediction models to make predictions more rational, accurate, reliable, and effective.

The recommended research efforts need to focus on the following:

- ***Collection and/or creation of long-term restrike data and/or long-waiting load testing data***

  More dynamic monitoring and static or statnamic load testing data should be collected or created by performing more fields testing, with special attention to the long-term data.

- ***Field study of the pile setup mechanisms***

  Fully-instrumented piles designed and installed at some typical clayey soil sites with long-term restrike and pile load testing should be performed. All the collected data should be carefully analyzed, and shaft capacity and tip resistance should be acquired for any testing event. Excess pore pressure, settlements, lateral displacements of piles, and lateral earth pressure on pile walls should be continuously monitored for a long period of time. Unit skin friction and tip resistance should be back- calculated from the strain gauge measurements on pile reinforcements at different elevations of the test piles.

- ***Laboratory study of the pile setup mechanisms***

  Laboratory research should be conducted to determine the pore pressure dissipation mechanism because of pile installation and friction angle growth on the pile wall because of soil aging. Triaxial compression tests and direct shear tests should be performed on undisturbed and remolded soil samples. Clay sensitivity and thixiotropy should be studied.

- ***Validation, modification, and improvement of the mathematical model***

  As one of the goals, future research should be to develop a mechaniscally-based model incorporating the aging factors previously described. This model should be able to predict the long-term pile setup with greater confidence.

LIST OF ACRONYMS, ABBREVIATIONS, & SYMBOLS

BOR	Beginning of Restrike
CAPWAP	CAse Pile Wave Analysis Program
CIP	Cast-in-Place
COV	The Coefficient of Variation
CPT	Cone Penetration Test
EOD	End of Driving
EOID	End of Initial Driving
FORM	First-Order Reliability Method
LADOTD	Louisiana Department of Transportation and Development
LCPC	The French Central Bridge and Pavement Laboratory Method
LTRC	Louisiana Transportation Research Center
LRFD	Load and Resistance Factor Design
MVFOSM	The Mean-Value-First-Order-Second-Moment
NC	The North Connector
PDA	Pile Driving Analyzer
PI	Principal Investigator
PRC	Project Review Committee
RFP	Request for Proposal
SC	The South Connector
SSR	Sum of Squared Residual

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APPENDIX A

Literature Review and Data Collection

Review of the State of Art on Pile Setup Prediction

Setup is predominately associated with an increase in shaft resistance. The complete mechanisms contributing to set-up are not well understood, but it is likely related to dissipation of excess pore water pressures, subsequent remolding, and reconsolidation of soil, which is displaced and disturbed during pile driving. After excess pore water pressures have dissipated, soil aging may account for additional setup. A number of empirical relationships have been proposed to estimate or predict the setup capacity and have demonstrated reasonable success in a number of studies. Empirical relationships are limited in widespread application by the relationships having been based on combined (shaft and toe) resistance determinations, interdependence of back-calculated or assumed variables, and the complexity of the mechanisms contributing to the setup.

During pile installation, soils around the pile are significantly disturbed and remolded. Excessive pore pressures are generated in saturated clays. The excessive pore pressure will dissipate and pile will regain its capacity, which could be used to explain short-term capacity increase (*[3], [4], [5], [2], [6], [7], [8]*. Cases have been reported where the shaft resistance of piles driven in clayey soils kept increasing over a period of time much longer than the duration of soil reconsolidation. Percentage-wise, the capacity increase of piles driven into soft clays tends to be greater than that of piles driven into stiff clays *[31]*. The long-term capacity increase results from other causes. Examples were presented by Schmertmann with regard to the time-strength changes in different types of soils *[10], [11]*. Mechanical aging caused an increase in the drained friction angle. Karlsrud and Haugen conducted axial tension tests on more than 20 piles in overconsolidated clay and found that pile capacities continued to increase another 22 percent within the next 30 days after the excess pore pressure dissipation for 6 days after the end of driving *[12]*. Komurka et al. illustrated a three-phrase pile setup, as shown in the following figure *[1]*. Kehoe indicated that setup occurs primarily in the shaft shear and found that the capacity of square pre-stressed concrete piles increased an average of 58 percent at one and 200 percent at the other 11 days after the piles were driven in mixed clayey soils *[13]*.

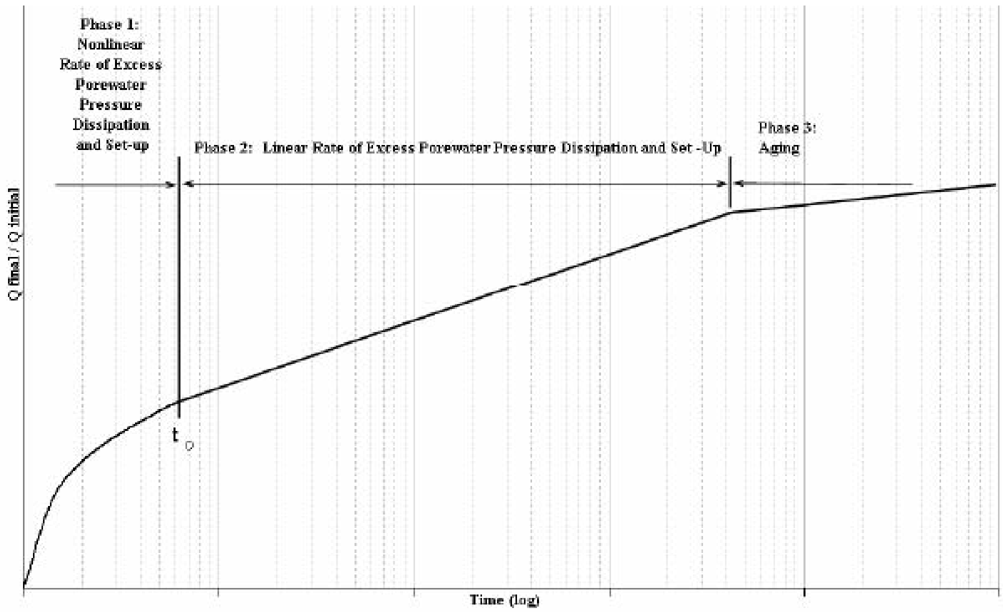


Figure 39
Idealized schematic of setup phases *[1]*

Skov and Denver examined four case histories of tested piles in clay, chalk, and coarse sands and analyzed the data from static loading test and restrikes with dynamic measurements performed at a certain time after the initial driving *[16]*. They found that pile shaft resistance increases with time. After CAPWAP analyses were carried out for different restrike blows for two piles driven in chalk and clay soils, they presented in Figure 40 a database for the distribution of shaft and toe resistance at driving and restriking. After statistically analyzing the database, an equation was developed to quantify the development of setup capacity. After a certain period of elapsed time, the time dependent increase in pile capacity could be considered approximately linear with the logarithm of time, as shown in the following equation:

$$\frac{Q}{Q_0} = 1 + A\,\mathbf{log}_{10}\left(\frac{t}{t_0}\right) \tag{15}$$

where, t is the time after initial driving, t_0 is the time elapsed after initial driving from which the increase in pile capacity bears a linear relationship with logarithmic time scale, and Q_0 is the capacity at time t_0, which depends on the soil type. *A* is a statistics-based coefficient called pile setup parameter.

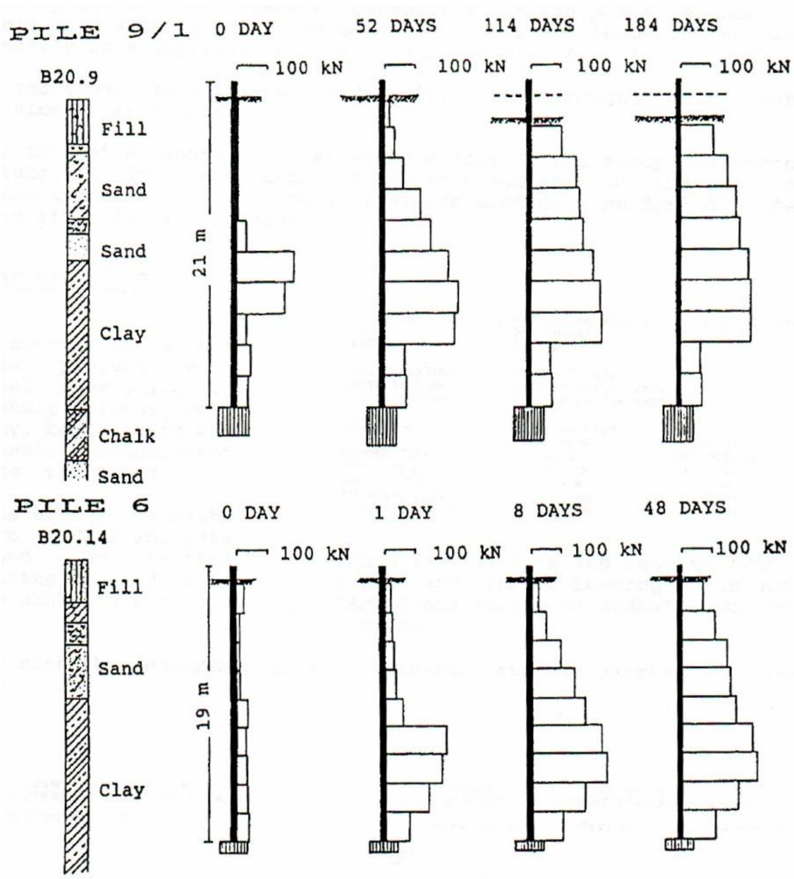


Figure 40
Distribution of shaft and toe resistances from CAPWAP analysis at driving and restriking (Skov and Dever *[19]*)

The empirical equation has been widely used for estimating pile setups by a good number of researchers and engineers *[17]*, *[18]*, *[32]*, *[14]*, *[33]*, *[9]*, *[24]*, *[19]*, *[20]*, *[29]*.

A database containing significant amount of pile testing data in clay is developed by Yang and Liang to analyze the setup effect statistically *[29]*. They incorporated the setup effect into reliability-based load and resistance factor design of driven piles and applied reliability-based techniques to develop separate resistance factors to account for different degrees of uncertainties that are associated with the measured pile setups. In the database, 16 piles are used to investigate the correlation between the measured CAPWAP results and the predicted setup capacities. In the developed framework, the setup effects are accounted for using the statistical parameters with the first-order reliability methods (FORM). Based on the AASHTO LRFD bridge design specifications, the resistance factors for the pile capacity at the initial end of driving (Q_0) are higher than those for Q_{setup} at the given reliability level because the uncertainties for Q_0 are less than those for Q_{setup}, as presented in Figure 41. Figure 42 shows the required resistance with and without considering the setup effects. When the target reliability index (β) is lower than 3.0, the incorporation of setup effect into the design of driven piles can advantageously enhance the prediction of design capacity. The setup effect would be ignored if a target reliability index β is chosen to be 3.0, which corresponds to 0.5, 1.25, and 1.75 for the conservative preset resistance factor Q_0, the dead load factor, and the live load factor, respectively. If the target reliability index β takes 2.33, the resistance factor for Q_{setup} can be conservatively taken as 0.30, corresponding to 0.65, 1.25, and 1.75 for the preset resistance factor Q_0, dead load factor, and live load factor, respectively. The factors of safety in the allowable stress design method are about 3.0 and 5.8 for the recommended load and resistance factors in LRFD at the target reliability index of β = 2.33 and 3.00, respectively.

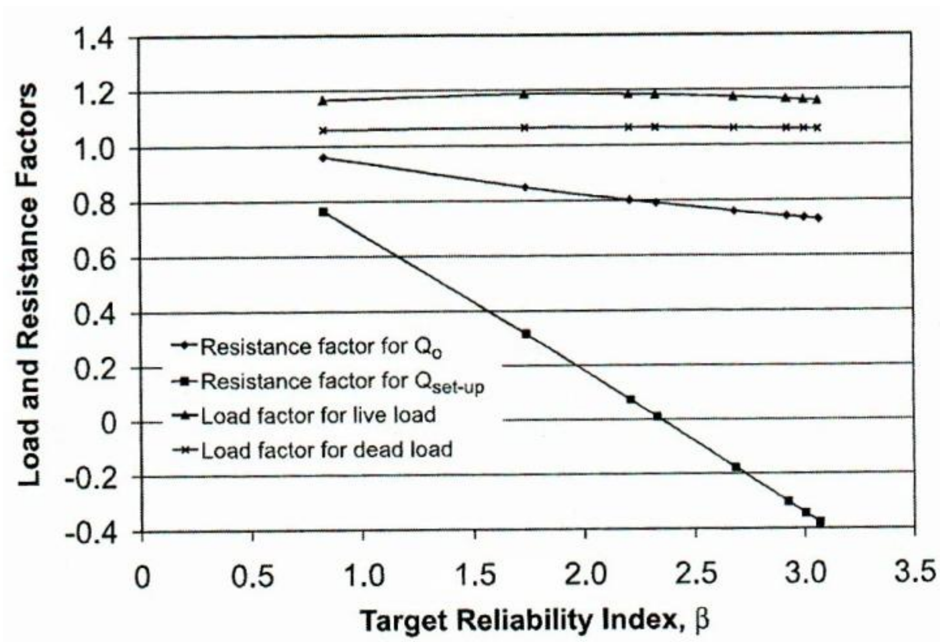


Figure 41
Relationship between the load and resistance factors and target reliability index

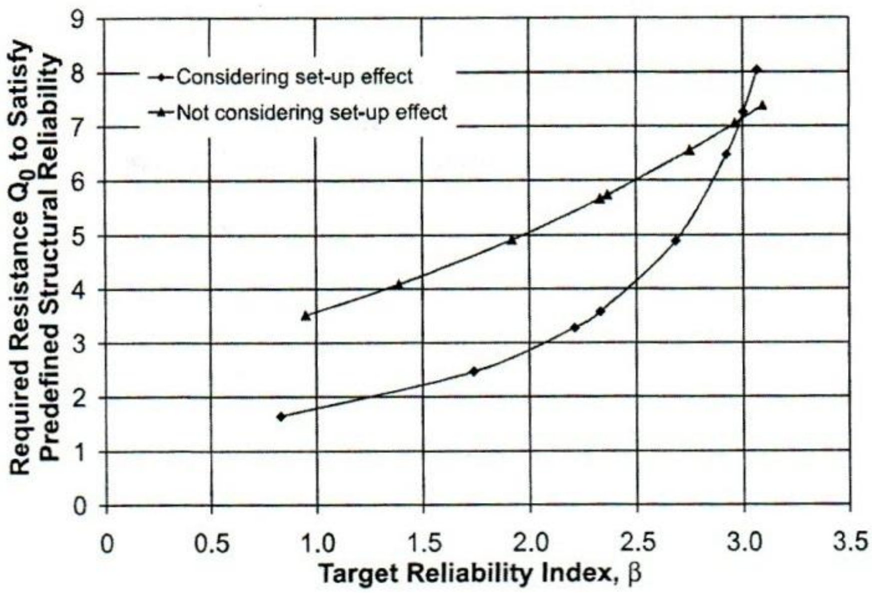


Figure 42
Comparison between the required pile capacity soon after end of driving for the two cases of considering and not considering setup effect *[29]*

Pile Data Collection and Analysis from Various State Highway and Other Agencies Nationwide that Have Geological Conditions Similar to Louisiana

Many field measurements of pile setup have been presented and published. They were given in different database forms from various static and dynamic pile tests conducted by different researchers. In this section, some typical tests for piles driven in clayey soils in various literatures will be selected to present here, and the corresponding background information and observations will be summarized based on the published pile data. The well-documented pile tests are good references and will provide sufficient information in analyzing the testing data of the piles driven in Louisiana soft clayey soils.

Measured Time Effects for the Pile Setup at Different Bridge Construction Sites in Florida. Bullock and his co-workers conducted a test pile program for nearly 5 years in which they well instrumented and installed five 18-in. in diameter, square, prestressed, concrete piles driven into coastal plain soils at four bridge construction sites in northern Florida as given in Figure 43 *[19], [34]*.

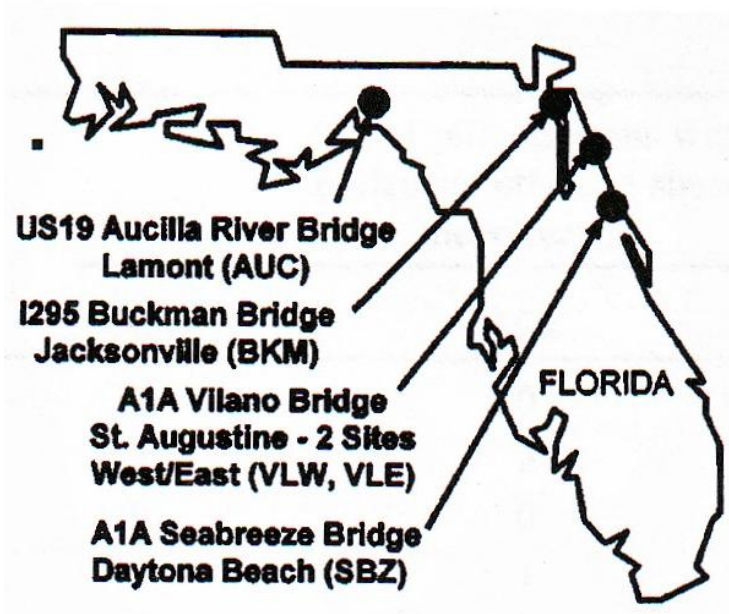


Figure 43
University of Florida side shear test pile sites *[19]*

The primary soils in which test piles were driven include soft to medium, stiff silty clays, and dense fine and medium sands, as described in Table 42. They performed dynamic tests and CAPWAP analyses during initial driving and two restrikes to measure short-term side shear setup. After adjusting the CAPWAP end bearing for each individual pile to be one constant value for all of the dynamic tests, as seen in Table 33, the dynamic and static shear estimates for the roughly continuous log-linear progression were presented in Figure 44. An O-cell was cast into the tip of each pile; strain gauges were at soil boundaries, and total stress cells and pore pressure cells were centered in one pile face between adjacent strain gauge elevations. Each pile test series included from three to six static tests with 15 to 1,727 days total setup time. In the long-term staged tests, shear strains, total horizontal earth pressure, and pore pressure were instrumented at different segments of each individual pile. Eventually, shear force and average shear stress acting on the pile wall were calculated over time in repeated tests in an effort to investigate the time effect on the side shear setup. Bullock and McVay et al. presented the general information and detailed results for the five test piles *[33]*,*[35]*.

They found that all pile segments showed setup with similar average magnitudes in all soils and at all depths. The setups continued long after the dissipation of pore pressures. The soil aging-induced post-dissipation setup proceeded at approximately a constant horizontal effective stress.

The dynamic and static test results confirmed a linear trend of side shear resistance versus the logarithm of time. Figure 44 shows that the semi log-linear side shear setup factors were bounded within the range of *A* = 0.1 to 0.4.

Table 32
Soil and test pile summaries *[19]*

Test site	Primary soil type	Pile length (m)	Pile penetration (m)	Date driven
I295 Buckman Bridge Jacksonville, FL	Dense fine sand	10.05	9.16	03/26/94
Aucilla River Bridge US19 (SR20) Aucilla, FL	Soft to med. stiff Si. clay. & Fn. sand	21.33	19.19	03/30/94
Vilano Bridge East AIA, Vilano Beach, FL	Dense fine sand	11.88	10.68	04/14/94
Vilano Bridge West AIA Vilano Beach, FL	Soft to med. stiff silty clay	19.96	18.40	08/24/94
Seabreeze Bridge SR430 EB Daytona Beach, FL	Med. to fn. sand & silty clay	26.84	25.12	01/26/96

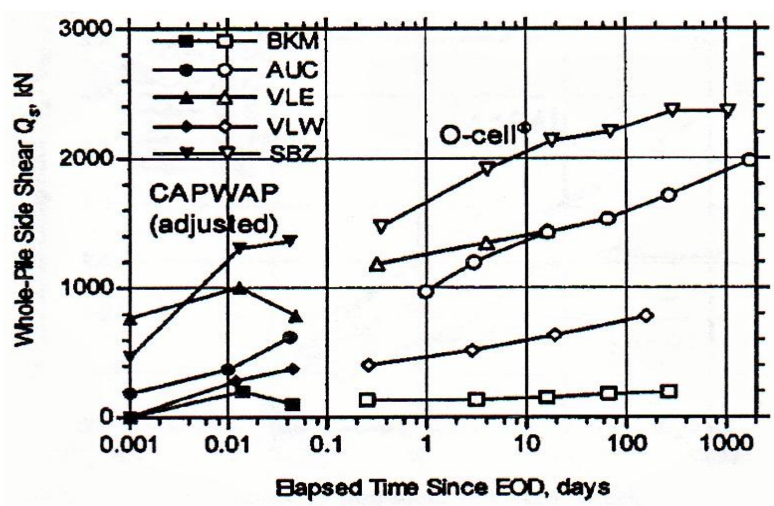


Figure 44
Increase in pile side shear capacity with time *[19]*

Table 33
Pile testing results after CAPWAP analyses *[19]*

Test site	Time after EOD, t (min)	(days)	Type of blow 4	Pile tip elev. (m)	*CAPWAP* capacity Side shear (kN)	End bearing (kN)	Total (kN)	Adj. capacity End bearing (kN)	Side shear (Q_{s_1} kN)
BKM	1	0.001	EOD	−4.62	876	961	1837	1837	0
	20	0.014	BOR	−4.69	873	1164	2037	1837	200
	65	0.045	BOR	−4.74	882	1058	1939	1837	102
AUC	1	0.001	EOD	−2.37	683	705	1388	1200	188
	15	0.010	BOR	−2.41	870	700	1570	1200	370
	60	0.042	BOR	−2.51	1045	778	1824	1200	624
VLE	1	0.001	EOD	−9.53	1274	1399	2673	1900	773
	18	0.013	BOR	−9.54	1450	1454	2905	1900	1005
	69	0.048	BOR	−9.57	1401	1290	2691	1900	791
VLW	1	0.001	EOD	−16.90	286	259	545	540	5
	17	0.012	BOR	−16.92	465	356	821	540	281
	65	0.045	BOR	−17.00	495	422	916	540	376
SBZ	1	0.001	EOD	−22.63	703	766	1470	1000	470
	19	0.013	BOR	−22.76	1379	925	2304	1000	1304
	61	0.042	BOR	−22.84	1775	587	2362	1000	1362

Note: EOD=End of driving ($t \approx 1$ min), BOR=Beginning of restrike; BKM=Buckman Bridge test site; AUC=Aucilla River test site; VLE=Vilano Bridge East test site; VLW=Vilano Bridge West test site; SBZ=Seabreeze Bridge test site.

Bearing Capacity of the Friction Piles Driven in San Francisco Young Bay Mud. A few 6-in. diameter pile piles, 20 to 22 ft. long, were driven about 15 ft. deep into a stratum of soft, saturated clay at a site near the San Francisco-Oakland Bay Bridge on the east site of San Francisco Bay *[15]*. The soil condition is shown in Figure 45, consisting of about 4 ft. of fill, 5 ft. of sandy clay, and at least 30 ft. of organic silty clay known locally as "bay mud." The load tests were conducted at 3 hours, 21 hours, 3 days, 7 days, 14 days, 23 days, and 33 days after the end of driving. Reese and Seed found that the tested piles had quite low supporting capacity when the piles were first driven into the saturated soft clay *[15], [3]*. Then, it was found that there was an increase in bearing capacity of the pile with time. The ultimate load of 6200 lb. at the final test was 5.4 times as large as the ultimate load of 1150 lb. measured in the first test. Eighty-eight percent of the increase was completed 8 days after the end of driving and the remaining 12 percent during the last 25 days, as shown in Figure 46. Seed and Reese attributed the increase in pile capacity to the dissipation of excess pore pressure *[3]*. However, they also noticed that there was an increase in bearing capacity of the pile with no apparent increase in effective pressure, which was "difficult to believe."

During and after driving, pore pressure and total pressure on the pile wall were measured. The pressures during and immediately after driving are given in Table 34, the residual pressures in Table 35, and pressure changes with time are shown in Figure 47. The presented data indicated that the pressures built up rapidly as the pile was driven and then dropped off rapidly when driving was stopped. The total pressure decreased rapidly during the first part of the test and reached equilibrium relatively soon.

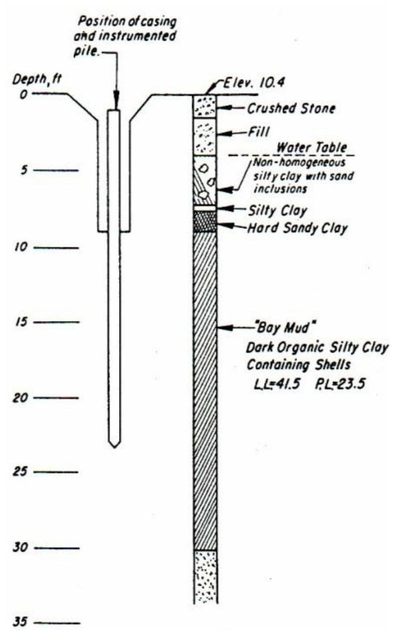


Figure 45
Soil profile of the 'Bay Mud" *[15]*

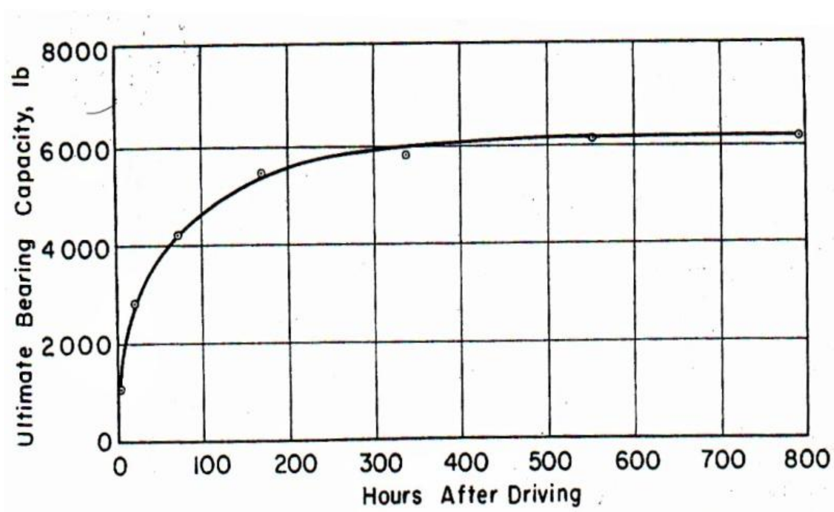


Figure 46
Increase in ultimate bearing capacity *[15]*

Table 34
Pressures during and at the end of driving ***[15]***

Pile Penetration Below Ground Surface, ft	Depth Where Pressure Was Measured, ft	Distance from Pressure Gage to Bottom of Casing, ft	Total Pressure, lb per sq ft	Pore-Water Pressure, lb per sq ft
11.4....	10.4	2.4	1100	...
12......	11	3.0	...	575
13......	12	4.0	2740	860
15......	14	6.0	3290	1620
15......	12	4.0	1870	...
17......	16	8.0	3290	2580
17......	14	6.0	2040	...
19......	18	10.0	3790	...
20......	17	9.0	2900	...
20......	19	11.0	...	3430
21......	20	12.0	4180	...
23......	22	14.0	4390	3810
23......	16	8.0	2310	...
23......	14	6.0	1490	...

Table 35
Residual pressure long after the end of driving

PORE-WATER PRESSURES

Gage	Depth, ft	Measured Pressure, lb per sq ft	Computed Hydrostatic Pressure, lb per sq ft
No. W-1......	22.0	1105	1102
No. W-2......	20.0	1090	976
No. W-3......	18.0	825	850
No. W-5......	14.0	560	598
No. W-6......	12.0	445	472
No. W-7......	10.0	335	346

TOTAL PRESSURES

Gage	Depth, ft	Measured Pressure, lb per sq ft	Measured Pressure Minus Pore-Water Pressure, lb per sq ft	Gross Estimated Effective Overburden Pressure, lb per sq ft	Gross Effective Pressure Ratio
No. T-1.....	22.0	1295	193	1354	0.14
No. T-2.....	20.0	1295	319	1256	0.25
No. T-3.....	18.0	1480	630	1158	0.54
No. T-4.....	16.0	1020	296	1060	0.28
No. T-5....	14.0	830	232	962	0.24

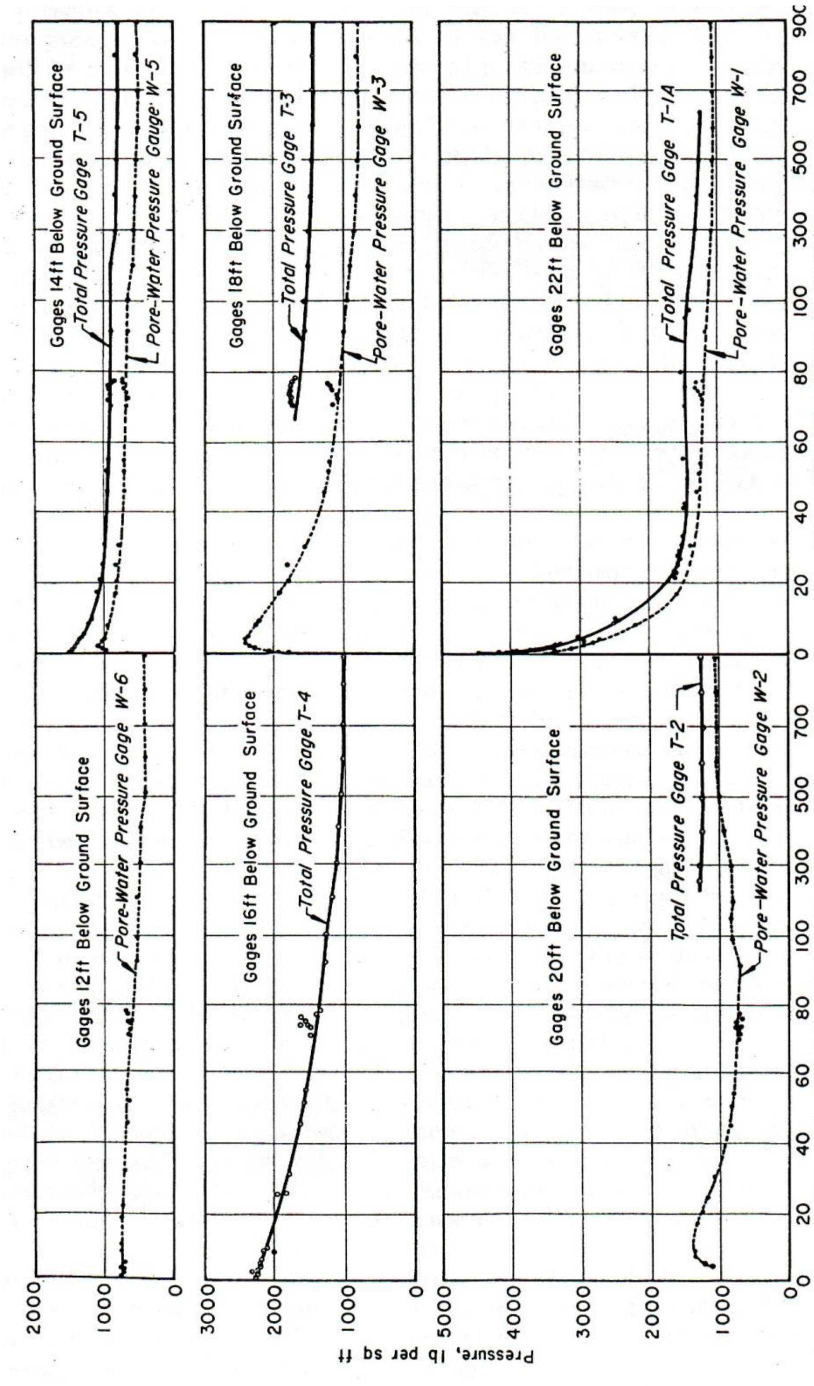


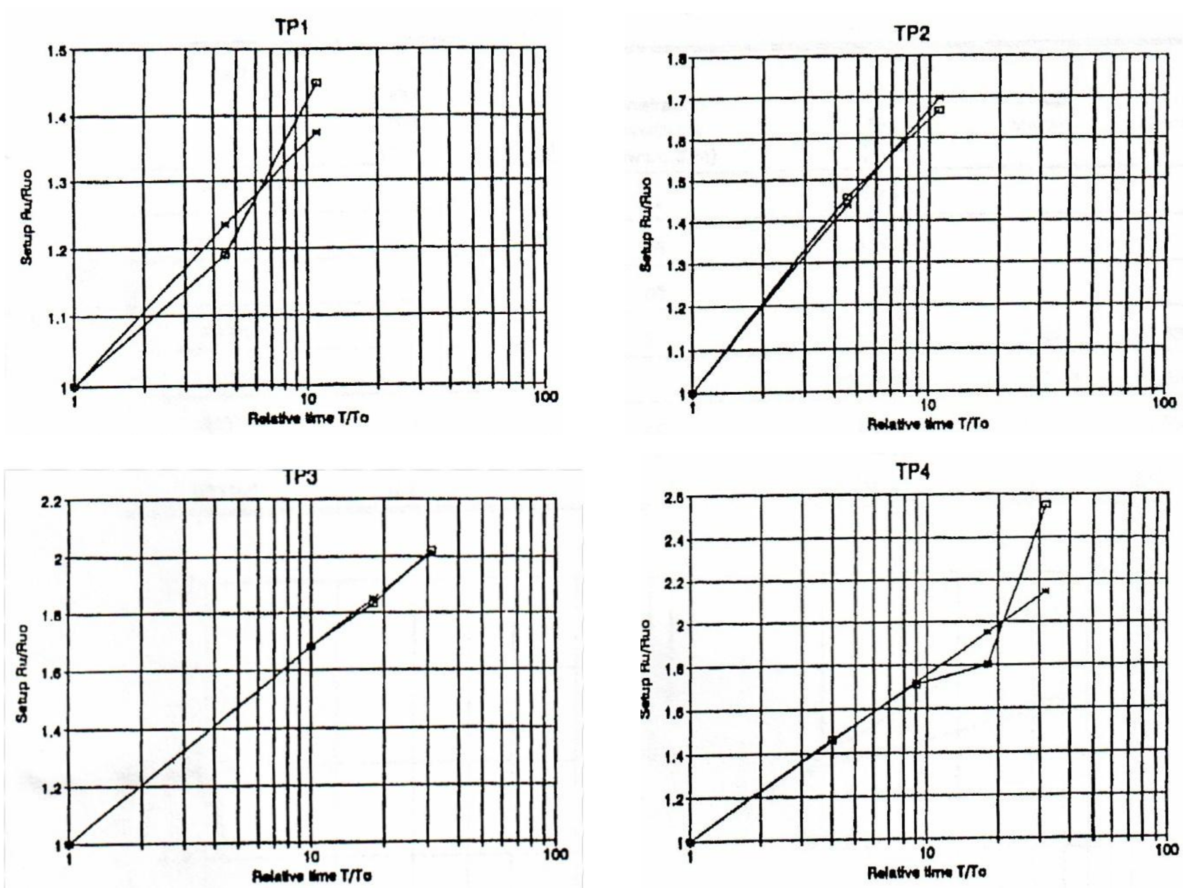
Figure 47
Pressure measurements at pile wall with time

Setup Effect in Cohesive Soils. Svinkin et al. and Svinkin and Skov confirmed the time dependent soil setup formulas developed by Skov and Denver after studying seven prestressed concrete piles that were tested for a bridge approach *[16], [17], [24]*. The piles were driven into the soils consisting of approximately 25.6 m of mainly gray clays followed by a bearing layer of silty sand, with a penetration depth of about 24.4 m for each pile. Three to four dynamic tests and/or static loading tests were performed for each pile installation. Pile descriptions, the elapsed time after the end of initial driving, penetration resistance, and the time dependent ultimate capacity of tested piles are summarized in Table 36.

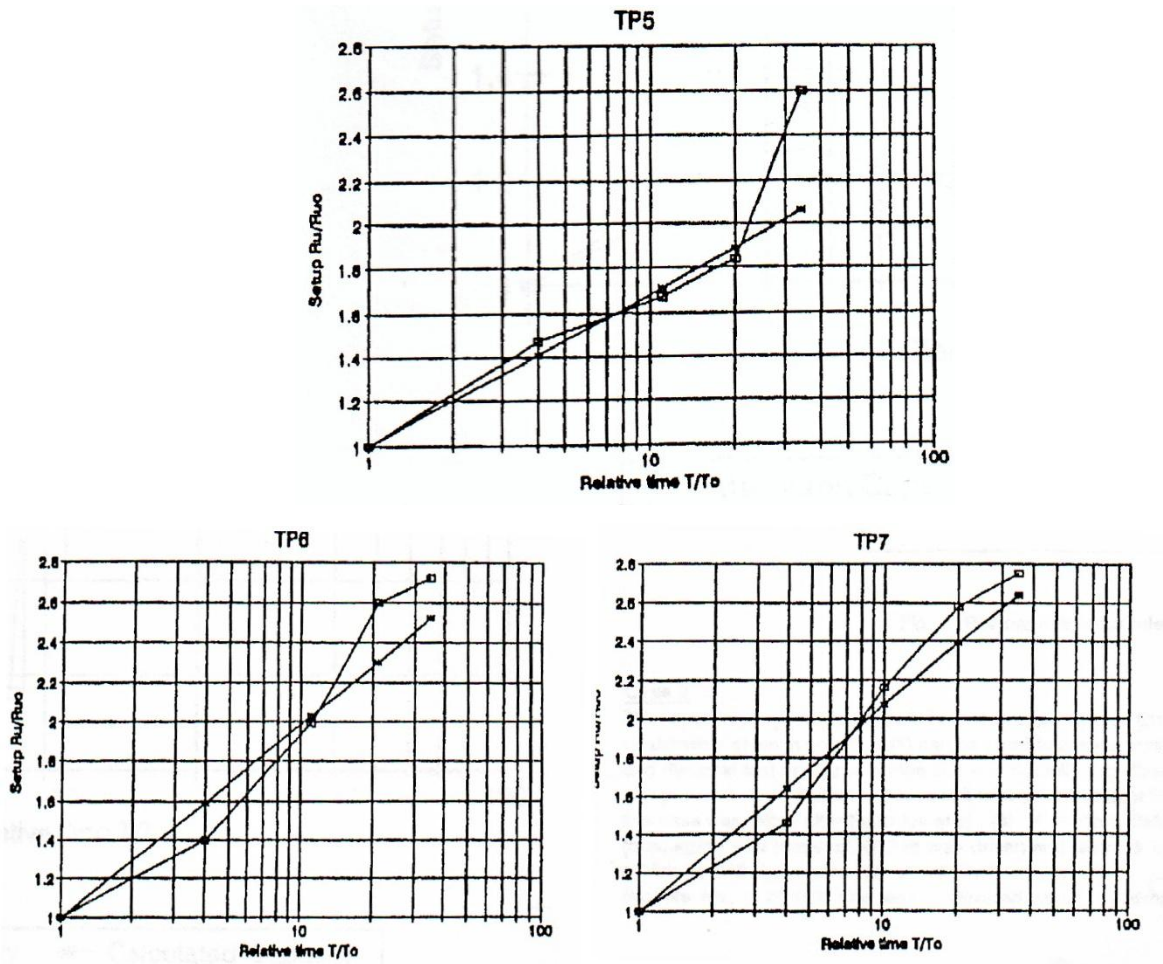
Table 36
Static and dynamic pile test data for the pre-stressed concrete piles in clay

Pile No.	Pile Description	Test	Time after EOID (days)	Penetration Resistance (blows/0.3 m)	Factor A	Ru (kN)	Setup Measd	Setup Calcd
TP1	1372 x 127 mm Cylinder	EOID	-	38	0.36	752	-	-
		RSTR-1	2	>240		2451	1	1
		RSTR-2	9	>240		2927	1.19	1.23
		RSTR-3	22	>240		3545	1.45	1.37
TP2	1372 x 127 mm Cylinder	EOID	-	48	0.67	712	-	-
		SLT-1	2	-		1913	1	1
		SLT-2	9	-		2789	1.46	1.44
		SLT-3	22	-		3189	1.67	1.70
TP3	610 x 610 mm (305 mm D. void)	EOID	-	10	0.68	267	-	-
		RSTR-1	1	21		912	1	1
		RSTR-2	10	72		1530	1.68	1.69
		RSTR-3	18	144		1672	1.83	1.85
		SLT	31	-		1841	2.02	2.01
TP4	762 x 762 mm (475 mm D. void)	EOID	-	14	0.76	200	-	-
		RSTR-1	1	23		890	1	1
		RSTR-2	4	60		1299	1.46	1.46
		RSTR-3	9	>240		1517	1.70	1.70
		RSTR-4	18	168		1601	1.80	1.95
		SLT	32	-		2273	2.55	2.14
TP5	762 x 762 mm (475 mm D. void)	EOID	-	23	0.69	262	-	-
		RSTR-1	1	59		952	1	1
		RSTR-2	4	96		1401	1.47	1.41
		RSTR-3	11	91		1588	1.67	1.71
		RSTR-4	20	>240		1748	1.84	1.89
		SLT	34	-		2473	2.60	2.06
TP6	914 x 127 mm Cylinder	EOID	-	15	0.99	400	-	-
		RSTR-1	1	34		885	1	1
		RSTR-2	4	64		1241	1.40	1.60
		RSTR-3	11	162		1766	2.00	2.02
		RSTR-4	21	113		2300	2.60	2.30
		SLT	35	-		2406	2.72	2.52
TP7	914 x 127 mm Cylinder (spliced)	EOID	-	32	1.07	454	-	-
		RSTR-1	1	32		876	1	1
		RSTR-2	4	102		1285	1.47	1.63
		RSTR-3	10	168		1890	2.16	2.08
		RSTR-4	20	186		2260	2.58	2.40
		SLT	35	-		2406	2.75	2.64

From observation, Svinkin et al. found that pile capacity sharply increased at 1 or 2 days after the end of driving and that the capacity-time relationship is close to linear when data are plotted on a logarithmic time scale for all seven piles *[17]*. The static loading tests and dynamic testing exhibit similar trends of pile capacity increase with time. As samples, the measured capacity and calculated capacity versus time are plotted in Figure 48 (a) and (b). The correlations confirm a good agreement between the tested capacity and calculated results. The pile setup factors ranges from 0.36 to 1.07 for all seven piles.



(a)



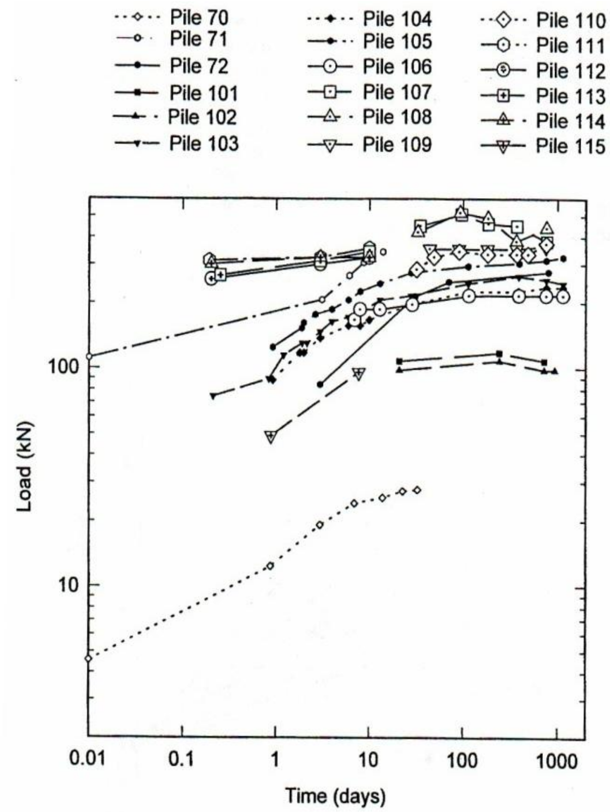
(b)

Figure 48
Measured and calculated capacities for the seven test piles *[17]*

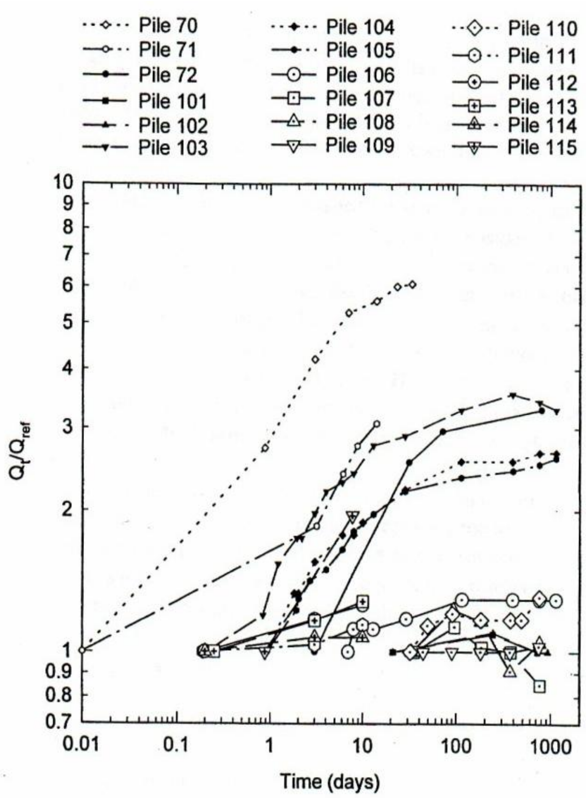
Time Dependent Increase in Axial Capacity of Driven Piling. Long et al. developed a database from various pile tests in published literature and presented some observations based on the collected pile data to quantify effects of time on the axial capacity of driven piles *[9]*. The database contains both static and dynamic load tests that were sorted into three groups based on the three primary subsurface profiles: clays, sands, and mixed soils as shown in Table 37.

The graph of axial pile capacity versus time in Figure 49 is shown for piles driven in clays. The axial pile capacity for piles driven in clay displays an increase with time. The time dependent increase in clay varies considerably. In some cases, the increases are up to six times the initial bearing capacity at the end of driving. The largest increase in axial capacity develops in the first

20 to 30 days after driving, which is probably due to dissipation of excess pore pressures. For times greater than 20 to 30 days, the pile capacity continues to increase for about half of the piles. The capacity remains constant with time for rest of the piles. Time effects on pile capacity level out around 100 days after driving. Pile load testing is generally not feasible after 100 days after driving. However, the graphs provide evidence that piles continue to increase their loading-carrying capacity with time after 100 days.



(a)



(b)

Figure 49
Axial capacity (a) and normalized capacity
(b) with time for the tested piles driven in clay *[9]*

Table 37
A database for the load pile tests for the time dependent pile capacity *[9]*

Pile #	Paper Ref #	Paper pile#	Pile type	Soil type		Density	Strength (kPa)
1-8	*(16)*	T5-10, J5-4,LT2-172 T5-107, 10B-4, TP-4, TP-7, TP-9	mono 3 gauge	sand	fine/med	30-50%	
9	*(16)*	TP-11	mono 5 gauge	sand	fine/med	30-50%	
10	*(16)*	L-18-2	timber	sand	fine/med	30-50%	
11	*(16)*	PP3	pipe	sand	fine/med	30-50%	
12	*(16)*	P6	mono 3g hollow	sand	fine/med	30-50%	
13	*(16)*	TP5	mono 5g hollow	sand	fine/med	30-50%	
14	*(16)*	TP8	mono 3g hollow	sand	fine/med	30-50%	
15	*(16)*	TP4	mono 5g concrete	sand	fine/med	30-50%	
16	*(16)*	TP10	mono 5g concrete	sand	fine/med	30-50%	
17	*(16)*	TP7	mono concrete	sand	fine/med	30-50%	
18	*(16)*	TP9	mono concrete	sand	fine/med	30-50%	
19-23	*(19)*	CT1, CT2, CT3, CT4, CT5	prestressed concrete	sand	silty	dense	
36	*(24)*	pile 2	sq prestress concrete	sand		dense	
50-54	*(13)*	1, 4, 5, 2, 3	prestress conc.pipe	sand	fine		
62	*(12)*	case II	steel pipe-	sand	various		
63	*(12)*	case 4	precast concrete	sand	silt		
73	*(17)*	H-/concr		sand		med dense	
70	*(5)*		close-end pipe	clay	silty		15-35
71	*(6)*	pile	open-end pipe	clay	soft		
72	*(9)*		wood	clay			
101	*(8)*	E	timber	clay			16
102	*(8)*	F	timber	clay			16
103	*(8)*	1	reinf concrete	clay			16
104	*(8)*	2	reinf concrete	clay			16
105	*(8)*	3	reinf concrete	clay			16
106	*(8)*	4	NP30 steel grdr	clay			16
107	*(8)*	26	timber. box	clay			16
108	*(8)*	27	timber, box	clay			16
109	*(8)*	28	reinf concrete	clay			16
110	*(8)*	29	reinf concrete	clay			16
112-114	*(8)*	1, 2, 2	capped pie-pile	clay			100
115	*(8)*	4.5	monotube	clay			23
24	*(25)*	A-2	thin wall pipe/concrete	mixed			
25	*(25)*	A-4	thin wall pipe/concrete	mixed			
26	*(25)*	B-2	12 HP63	mixed			
27	*(25)*	B-4	12 HP63	mixed			
29-32	*(25)*	F-1, G-1, H-1, I-1,	heavy wall pipe/concrete	mixed			
33	*(25)*	B-3	12 HP63	mixed			
34	*(25)*	E-4	heavy wall pipe/concrete	mixed			
35	*(21)*	none	HP 360x40x176	mixed			
37-42	*(26)*	TP5, TP6, TP7, TP11, TP30, TP31	circl prestr concrete	mixed	soft clay/dense sand		
55-57	*(27)*	PC1, PC2, PC3	prestress concrete	mixed			
58	*(28)*	SP1	steel pipe -closed end	mixed			
59-61	*(12)*	P9/1, P5, P6	precast concrete	mixed			
64-67	*(28)*	TP-1, TP-2, TP-3, TP-4	pipe-close end	mixed			
68	*(29)*	HP	HP 14x73	mixed			
69	*(29)*	Pipe	close-end pipe	mixed			

Characterization of Pile Capacity with Time in the Cooper Marl, Located in Charleston, South Carolina. Camp III and Parmar studied the time dependent setup for the piles driven into the Cooper Marl soils in the coastal area of Charleston, South Carolina *[14]*. The subsurface profiles consist of stiff, cohesive calcareous marine deposit that is generally more than 30 m thick with principal material properties given in Table 38. Numerous piles driven in the soils have been statically or dynamically tested at various times after installation. It has long been recognized that the driven piles have experienced tremendous setup. The established pile setup database was employed to back-calculate the setup factors for the empirical linear relationship between pile capacity and logarithmic time.

Table 38
Summary of Cooper Marl soil properties *[14]*

% Wat.	LL	PI	% Fines	Unit Weight (kN/m^3)	OCR	S_u, (kPa)	Effective Shear Strength Parameters, (kPa)
30 to 60	50 to 100	20 to 70	55 to 85	16.5 to 18.1	3 to 7	96 to 239	$\phi' = 33° \text{ to } 40°$ $c' = 19 \text{ to } 38$

Note: 1 kN/m^3 = 6.37 pcf, 1kPa = 0.021 ksf

The assembled database consisted of piles either statically tested or dynamically tested with a pile driving analyzer on two or more occasions. It includes 12-in., 14-in., 18-in., and 24-in. square prestressed concrete piles and 12-in. and 14-in. H-piles, representing 14 sites and 114 testing events. The typical setup magnitudes measured in the setup factor are illustrated in Figure 50. The setup factor shown on the y-axis is as the ratio of the pile capacity at the beginning of restrike (BOR) to the capacity at the end of driving capacity. Samples of mobilized capacity versus the log of time are plotted since the end of driving in Figures 51 and 52.

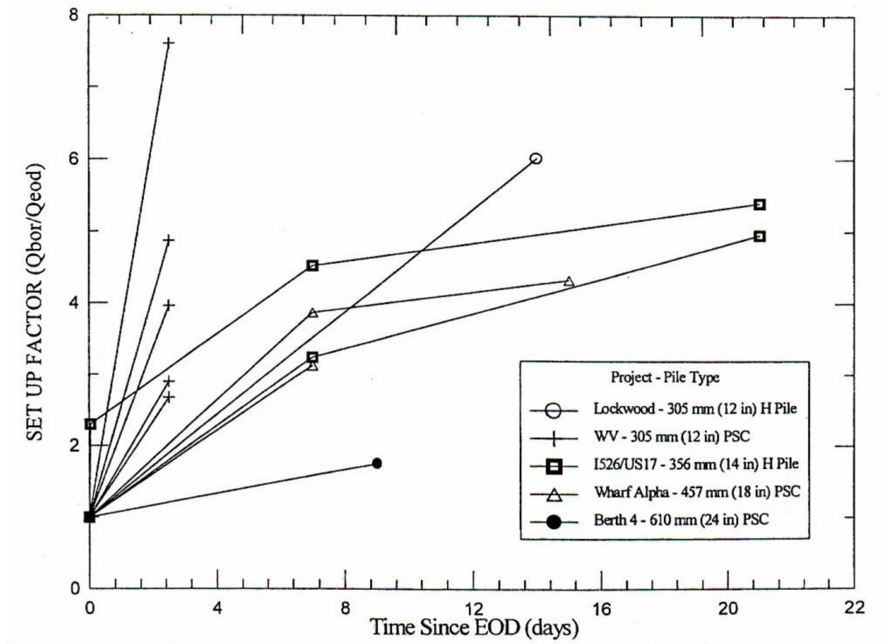


Figure 50
The pile setup factor versus time *[14]*

They contain data from both prestressed concrete piles and H-piles. The capacity and elapsed time in Figures 51(a) and 52(a) were normalized in accordance with the following equation and plotted in Figures 51(b) and 52(b), respectively. As illustrated in the figures, the rate of capacity gain with time as represented by the slope of the lines seems to be fairly similar for each of the four pile sizes. Equation for the capacity versus the elapsed time is re-written as:

$$\frac{Q}{Q_0} = A\,\mathbf{log}\left(\frac{t}{t_0}\right) + 1 \tag{16}$$

where, t represents time since the end of driving, Q is the pile capacity at time t, and t_0 is the time after installation at which point the capacity gain becomes linear on a log(t) plot. Q_0 is the pile capacity at time t_0, and *A* is the setup parameter that is the function of soil type and equal to the slope of the linear portion of the normalized capacity gain versus log(t) plot. If an assumed t_0 value of 2 days is used, the *A* values are back-calculated with relatively small scatters. Consequently, it is concluded that equation (16) that was proposed by Skov and Denver is feasible to predict the long-term pile capacity based on the results of a relatively short-term pile capacity.

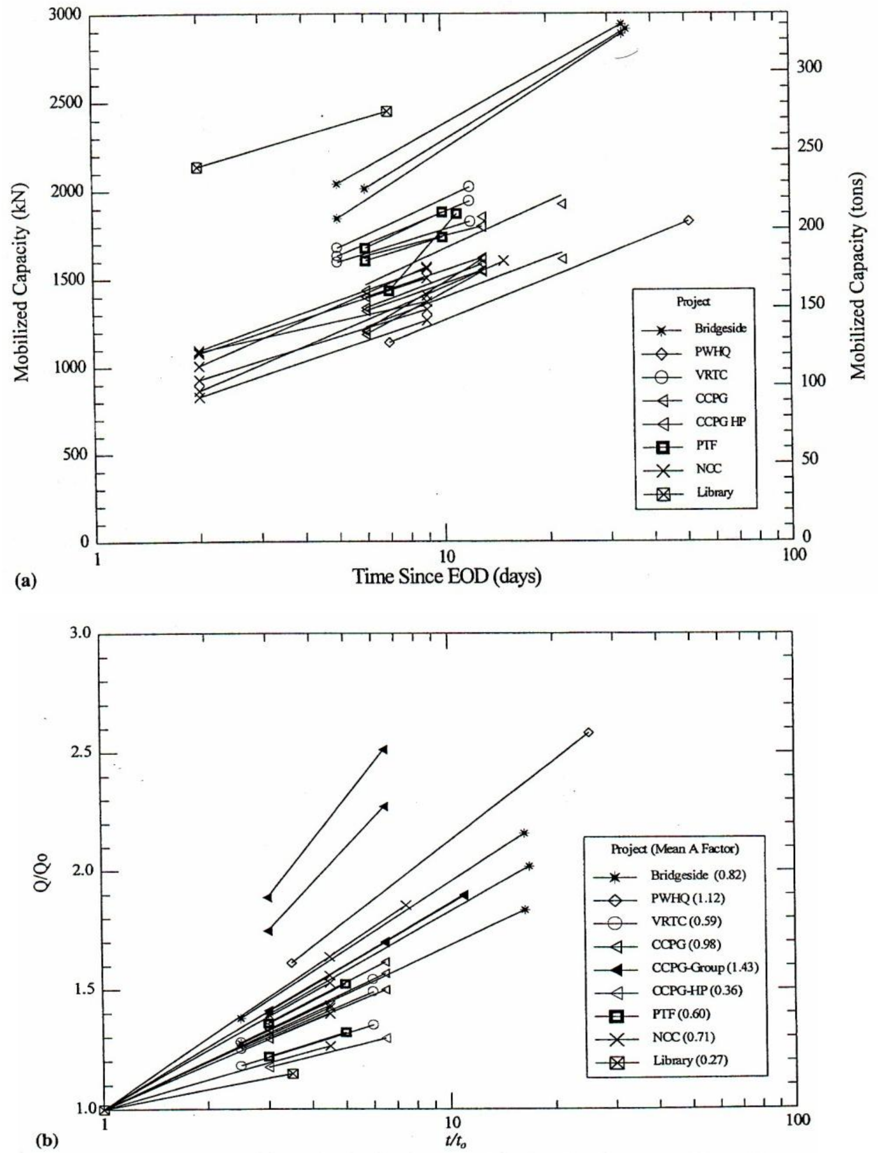


Figure 51
(a) Capacity versus time (b) normalized capacity versus time for 12-in. piles *[14]*

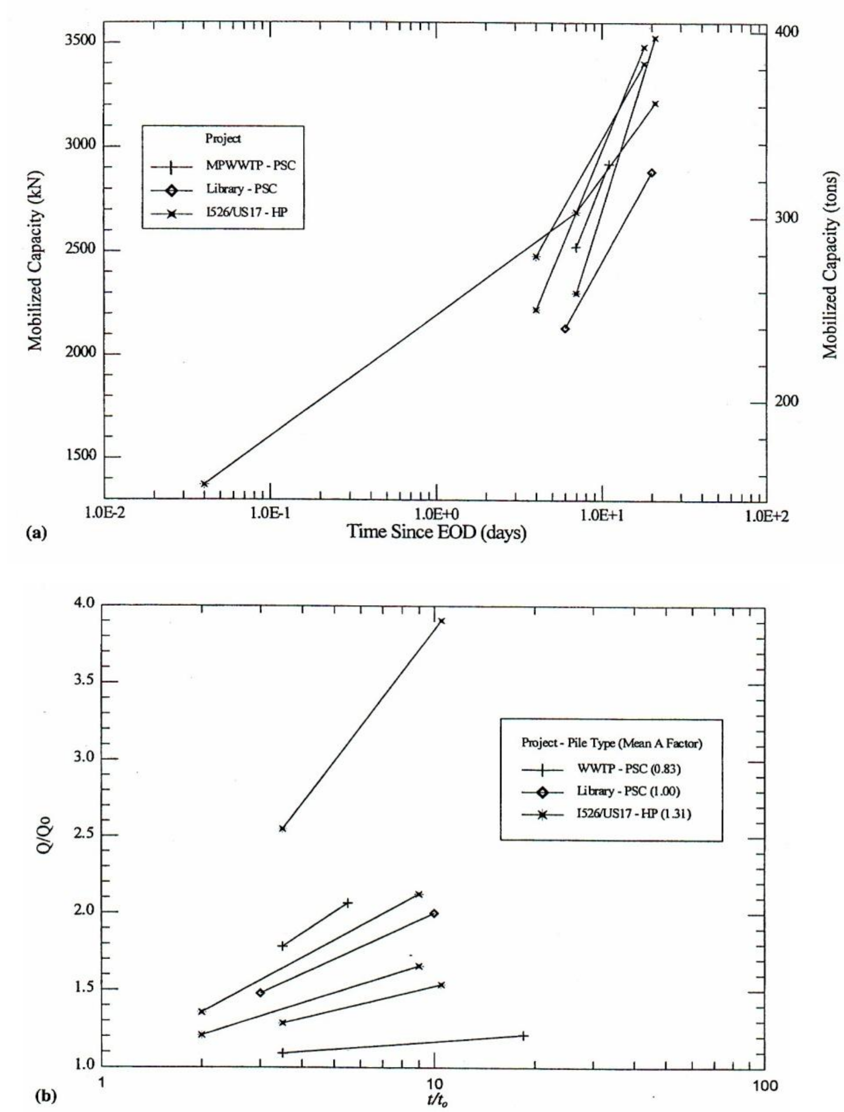


Figure 52
(a) Capacity versus time (b) normalized capacity versus normalized time for 14-in. piles *[14]*

Measured Pile Setup During Load Testing and Production Piling. Attwooll and his co-researchers investigated nine sets of full-scale load tests that were performed at the I-15 Corridor Reconstruction Project site through downtown Salt Lake City *[32]*. The surficial sediments encountered along the project alignment consist mainly of lacustrine clays and silts with minor fine sand lenses or of recent stream alluvial deposits of sand, silt, and clay. The piles and pile driving are summarized in Table 39. Static compression, dynamic monitoring of pile installation, and restrikes using high-strain testing and analysis methods consistently provided data indicating large capacity gain with time (setup) regardless of the subsurface conditions. The setup data were presented in Table 40. In Figure 53, the unit friction derived from signal matching of EOI (end of installation) results represent the very low values caused by pile installation disturbance. The pile capacity before the beginning of restrike (BOR) results in the same figure, which were obtained 93 days after installation and shows a remarkable increase in shaft resistance that grows appreciably with depth.

Table 39
Test pile information *[32]*

Location	Diameter	Length	End of Installation		Pile Type
			Blowcount	Approximate Driving Energy	
	m	m	Blows/300 mm	kN m	
600N	0.324	35.4	10	27	Friction, in clay
400S	0.324	27.4	4	70	Friction, in clay
600S	0.610	32.9	22	70	Friction, in clay
1700S	0.324	23.2	20	40	Friction in clay/endbearing
Roper Yd.	0.406	24.7	58	70	Friction in clay/endbearing
Tomahawk	0.324	26.5	50	30	Friction in clay/endbearing
3300S	0.324	26.5	50/200 mm	30	Friction in clay/endbearing
5300S	0.406	20.1	53	60	Friction in clay/endbearing
7200S	0.324	10.1	68	70	Endbearing/friction in sand

Table 40
Pile load test results and pile setup *[32]*

Location	Dynamic Capacity at End of Installation kN	Load Tested Days after Driving	Axial Test Load Measured kN	Pile Setup (Column d - Column b) kN	Setup Ratio (Col d/ Col b)	Shaft Area sq m	Setup Unit Friction (Col e/Col g) kPa
a	b	c	d	e	f	g	h
600N	534	38	2313	1779	4.3	36.0	49.4
400S	690	40	2669	1979	3.9	27.9	70.9
600S	623	43	3648	3025	5.9	63.1	47.9
1700S	801	42	2113	1312	2.6	23.6	55.6
Roper Yd.	2114	42	4093	1979	1.9	31.5	62.8
Tomahawk	1448	45	2892	1444	2.0	26.9	53.7
3300S	1713	21	3337	1624	1.9	26.9	60.4
5300S	2581	25	4449	1868	1.7	25.6	73.0
7200S	2670	11	3203	533	1.2	10.2	52.3

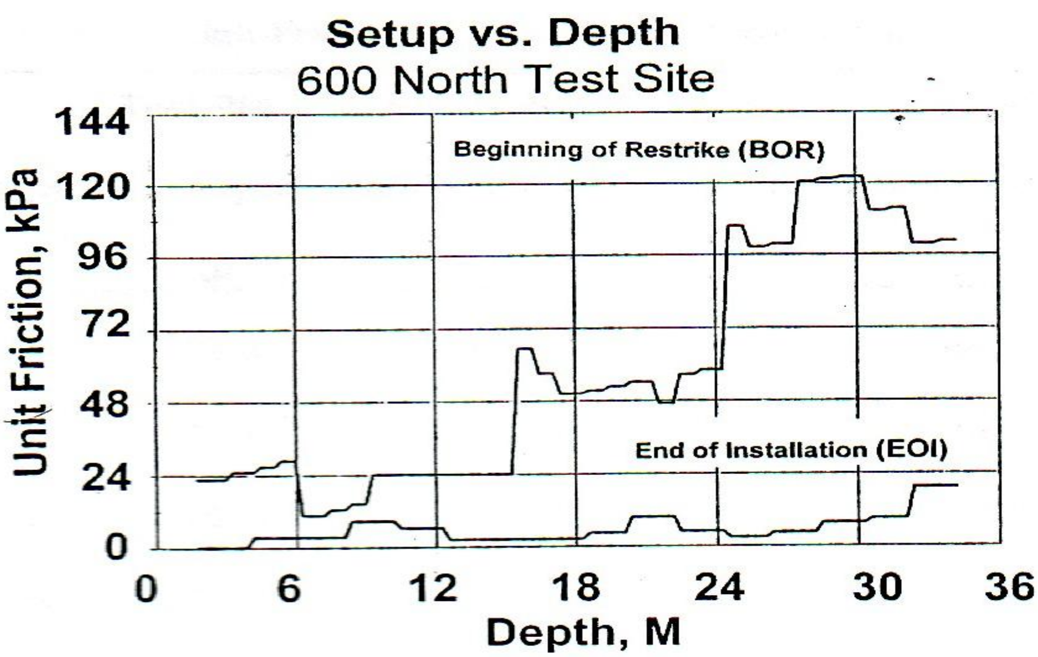


Figure 53
Unit friction capacities of EOI and BOR versus time *[32]*

Attwooll et al. applied various methods to the load test data to correlate the measured pile capacity to the EOI data from the dynamic test results in an attempt to estimate the ultimate pile capacity to be made during production pile driving based on the EOI dynamic test data *[32]*. The resulting increases in terms of average unit shaft friction are shown in Figure 54. As indicated, the small scatter in the average unit friction gains was experienced regardless of the pile diameter, penetration depth, or location within the range of the parameters tested. The setup unit shaft friction was evaluated using the approach, which best predicted the measured pile capacity when added to the EOI capacity estimates, as shown in Table 41. The unit friction predicted the measured pile capacity from the load tests within a range of about ± 17 percent for all piles and within about ± 10 percent for the more common combination friction and tip resistance. The calculated results are plotted and presented in Figure 54 and Table 41, respectively.

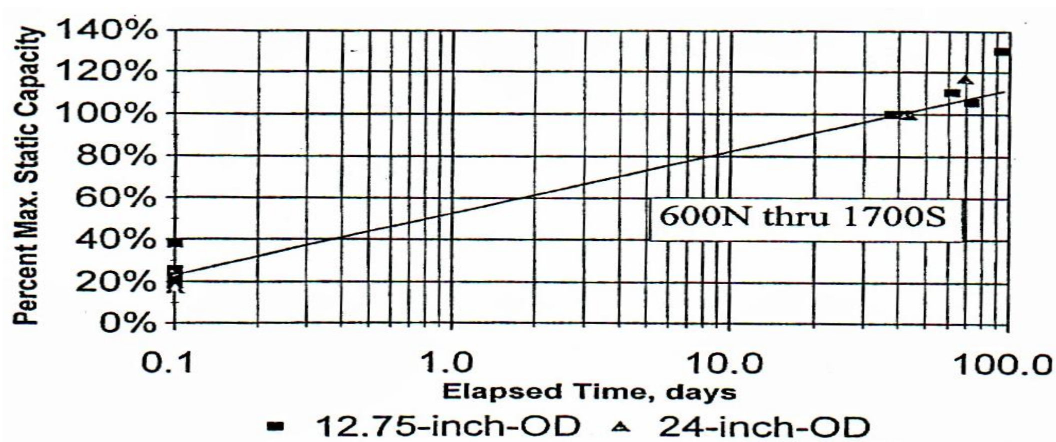


Figure 54
The predicted pile capacity versus the elapsed time *[32]*

Table 41
Pile capacity prediction by unit setup method *[32]*

Location	Dynamic Capacity at End of Installation (EOI) kN	Shaft Area sq m	Assumed Unit Setup kPa	Predicted Capacity kN	Measured Axial Test Load kN	Predicted/ Actual
600N	534	36.0	57.5	2604	2313	1.13
400S	690	27.9	57.5	2294	2669	0.86
600S	623	63.1	57.5	4251	3648	1.17
1700S	801	23.6	57.5	2158	2113	1.02
Roper Yd.	2114	31.5	57.5	3925	4093	0.96
Tomahawk	1448	26.9	57.5	2995	2892	1.04
3300S	1713	26.9	57.5	3260	3337	0.98
5300S	2581	25.6	57.5	4053	4449	0.91
7200S	2670	10.2	57.5	3257	3203	1.02
					Average:	1.01
					Range:	86%-117%
						91%-104%*

* Based on combination friction/endbearing piles only

APPENDIX B

Restrike and Static and Statnamic Load Testing Data

Restrike Data of the Production Piles at the Construction Segments of South Connector, Mainline, and Ramp-N1

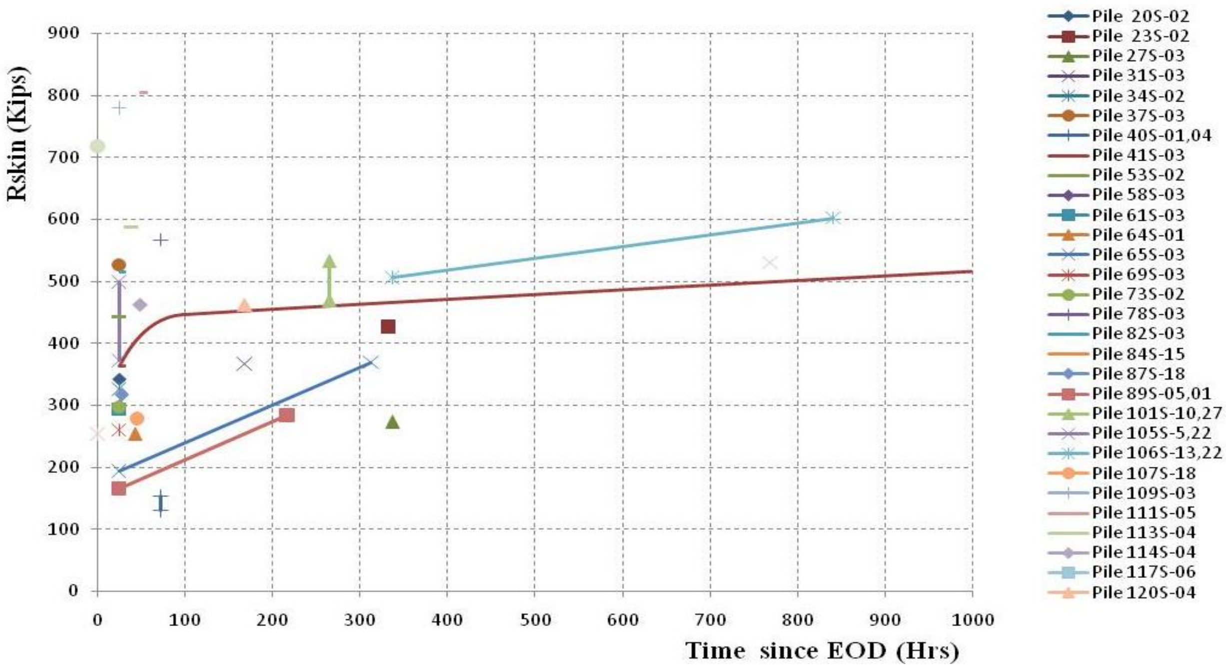


Figure 55
Shaft capacity change with time from the restrikes at the mainline

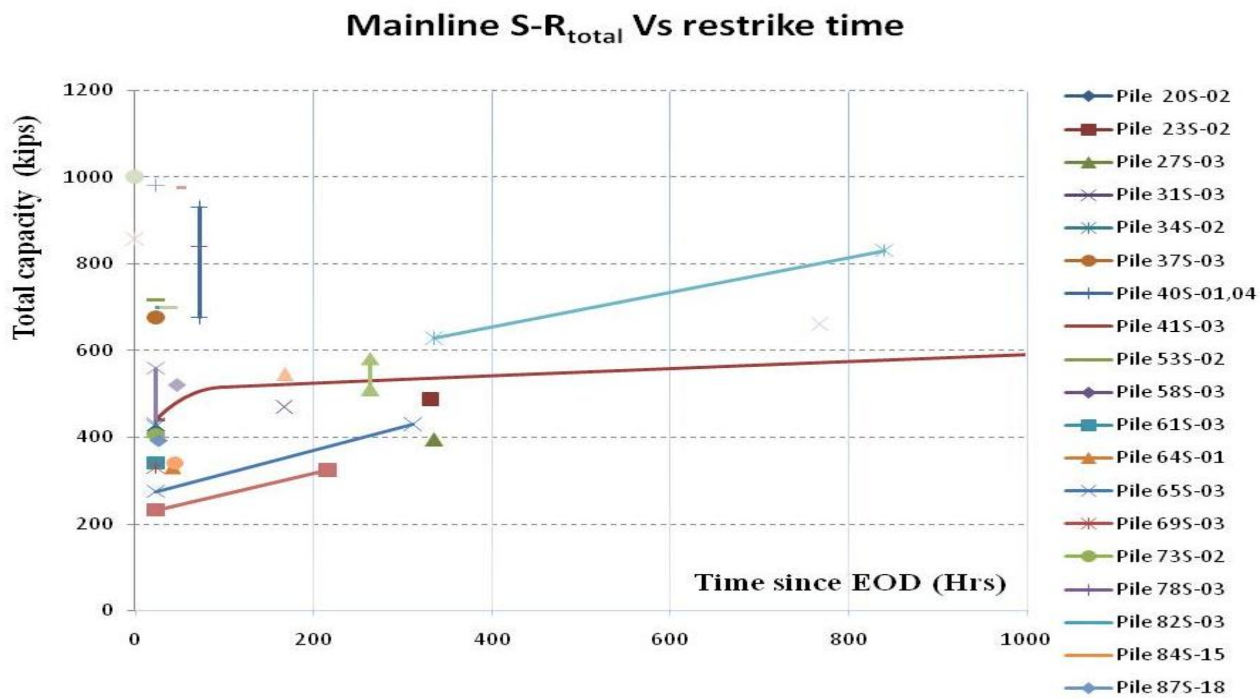


Figure 56
Total capacity change with time from the restrikes at the mainline

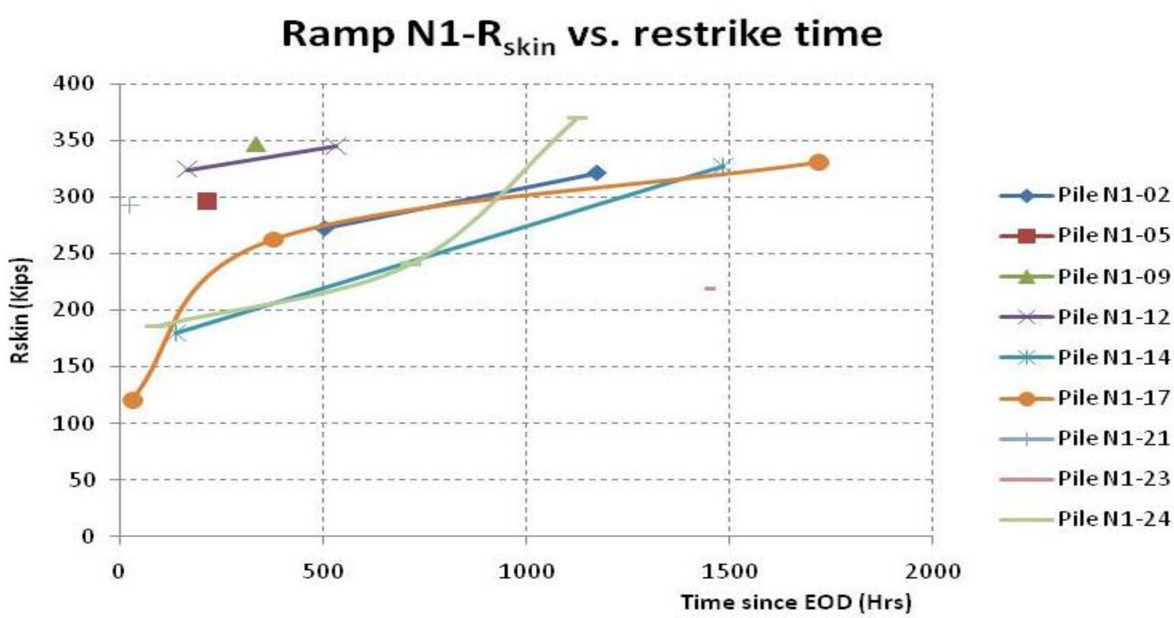


Figure 57
Shaft capacity change with time from the restrikes at ramp N1

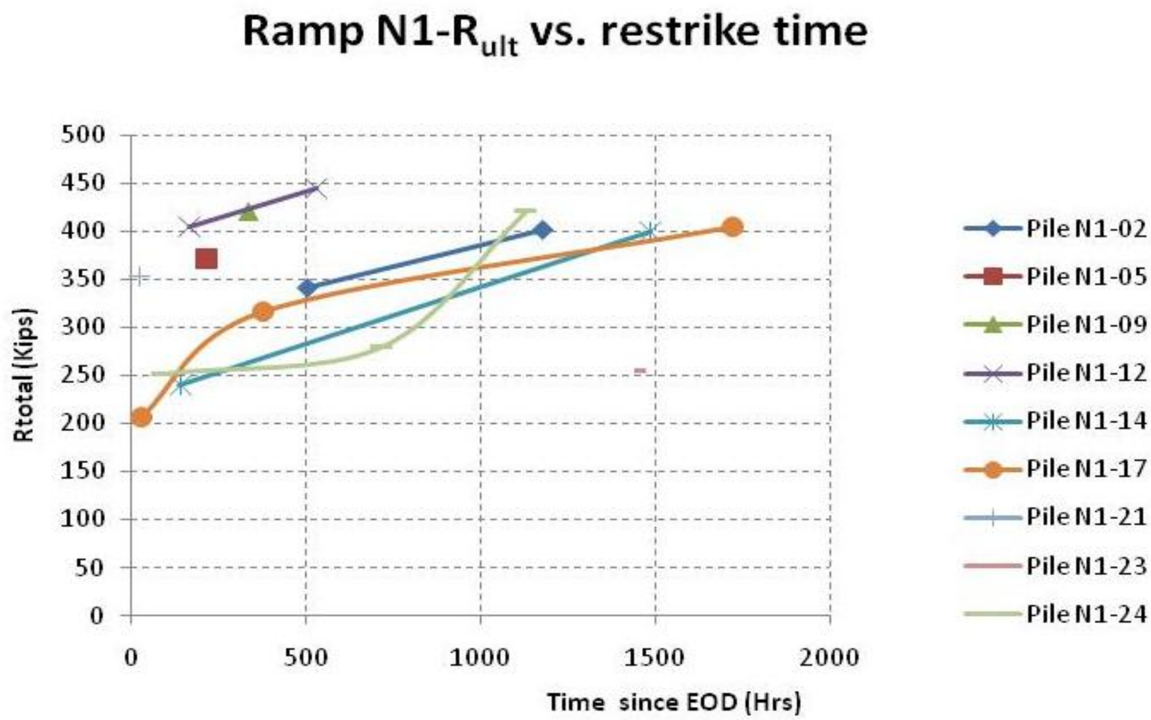


Figure 58
Total capacity change with time from the restrikes at ramp N1

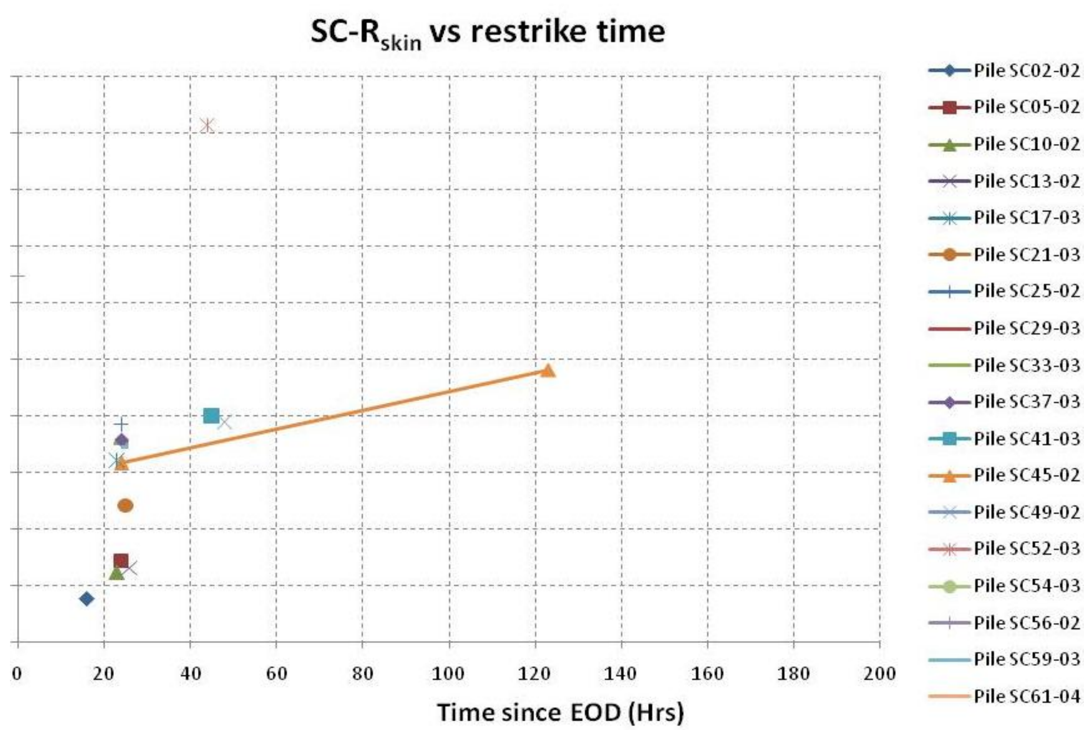


Figure 59
Shaft capacity change with time from the restrikes at South Connector

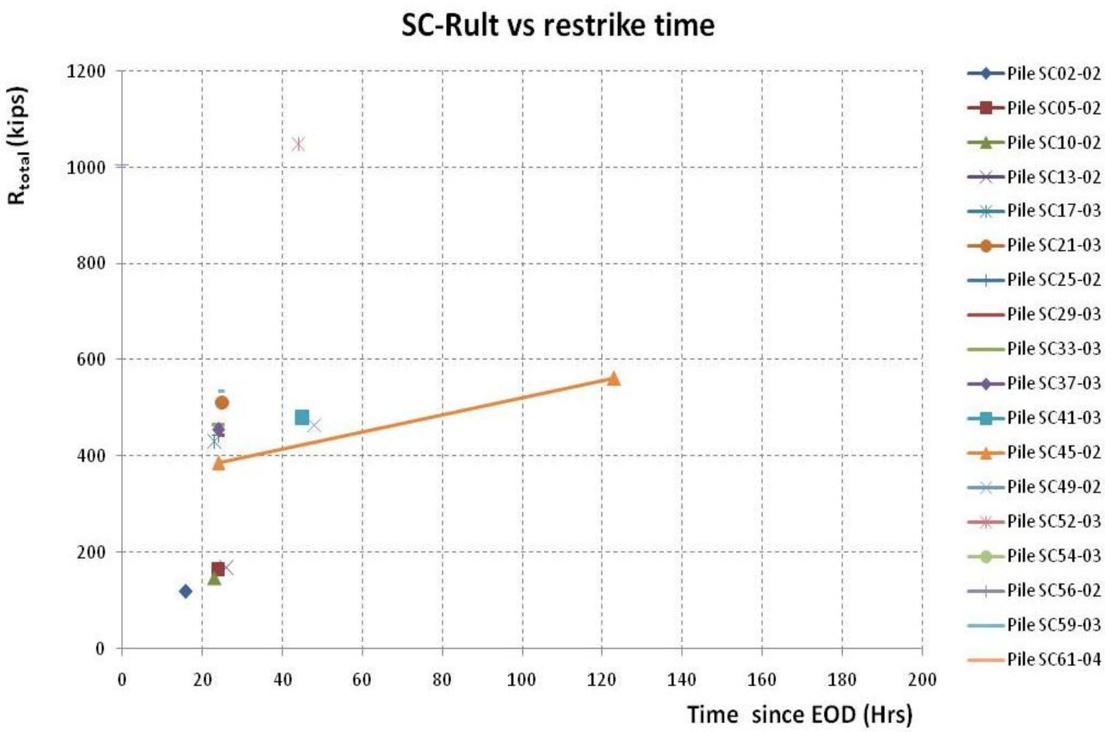


Figure 60
Total capacity change with time from the restrikes at South Connector

Table 42
Pile type, capacity, soil information, and other information of the production piles at the mainline

Pile	Pile Type	Restrike Date	Time (Hrs)	Penetration Length (ft)	Soil Type	R_{skin} (kips)	R_{tip} (kips)	R_{tot} (kips)
20S-02	24" SQ. PPC Solid	1/4/2007	24	81.01	Major clay with silt	343	72	415
23S-02	24" SQ. PPC Solid	1/24/2007	332	83.15	Major clay with silt	427	60	487
27S-03	24" SQ. PPC Solid	2/6/2007	336	83.15	Major clay with silt	273	122	395
31S-03	24" SQ. PPC Solid	2/6/2007	168	90.41	Major clay with sand	367	103	470
34S-02	24" SQ. PPC Solid	2/6/2007	24	89.84	Major clay with sand	326	99	425
37S-03	30" SQ. PPC Solid	3/20/2007	24	144.84	Major clay with silt	526	150	676
40S-01	30" SQ. PPC Solid	7/23/2007	72	72.74	Major clay with sand	130	800	930
40S-04	30" SQ. PPC Solid	7/23/2007	72	72.74	Major clay with sand	153	523	676
41S-03	30" SQ. PPC Solid	3/30/2007	24	75.57	Major clay with sand	364	76	440
41S-03	30" SQ. PPC Solid	4/2/2007	96	75.57	Major clay with sand	446	68	514
41S-03	30" SQ. PPC Solid	10/26/2007	5040	75.57	Major clay with sand	800	100	900
45S-03	30" SQ. PPC Solid	8/21/2007	eod	161.58	Major clay with silt	254	604	858
47S-03	30" SQ. PPC Solid	9/6/2007	eod	157.58	Major clay with silt	717	283	1000
49S-03	30" SQ. PPC Solid	Skipped	Skipped	158.91	Major clay with silt	NA	NA	NA
53S-02	30" SQ. PPC Solid	10/3/2007	24	163.54	Major clay with silt	443	274	717
58S-03	24" SQ. PPC Solid	12/07/07	24	121.29	Major clay with silt	298	42	340
61S-03	24" SQ. PPC Solid	12/19/07	24	120.82	Major clay with silt	294	46	340
64S-01	24" SQ. PPC Solid	01/07/08	42	121.39	Major clay with silt	254	76	330
65S-03	24" SQ. PPC Solid	01/09/08	24	117.53	Major clay with sand	193	82	275

(continued)

65S-03	24" SQ. PPC Solid	01/21/08	312	117.53	Major clay with sand	369	60	430
69S-03	24" SQ. PPC Solid	01/18/08	24	101.53	Major clay with sand	260	70	330
73S-02	24" SQ. PPC Solid	02/07/08	24	105.63	Major clay with sand	297	109	406
78S-03	30" SQ. PPC Solid	10/15/2007	72	153.08	Major clay with silt	567	272	839
82S-02	30" SQ. PPC Solid	11/13/2007	24	154.8	Major clay with silt	515	185	700
84S-15	24" SQ. PPC Solid	03/07/08	24	98	Major clay with sand	165	67	232
84S-15	24" SQ. PPC Solid	03/15/08	216	98	Major clay with sand	284	40	324
87S-18	24" SQ. PPC Solid	03/28/08	27	103	Major clay with sand	317	76	393
89S-05	24" SQ. PPC Solid	04/21/08	90	143	Major clay with silt	347	169	515
89S-21	24" SQ. PPC Solid	04/16/08	24	143	Major clay with silt	374	51	425
101S-10	24" SQ. PPC Solid	04/15/08	264	146	Major clay with sand	532	49	582
101S-27	24" SQ. PPC Solid	04/15/08	264	146	Major clay with sand	469	41	510
105S-05	24" SQ. PPC Solid	03/20/08	24	149	Major clay with sand	499	60	559
105S-22	24" SQ. PPC Solid	03/20/08	24	149	Major clay with sand	372	57	430
106S-13	24" SQ. PPC Solid	02/28/08	840	149	Major clay with sand	603	228	831
106S-22	24" SQ. PPC Solid	02/28/08	336	149	Major clay with sand	506	122	628
107S-18	24" SQ. PPC Solid	01/14/08	45	119	Major clay with sand	278	62	340
109S-03	30" SQ. PPC Solid	02/11/08	24	140.95	Major clay with silt	780	200	980
111S-05	30" SQ. PPC Solid	02/06/08	48	160.89	Major clay with silt	805	172	977
113S-04	30" SQ. PPC Solid	01/17/08	38	178.85	Major clay with silt	588	112	700

(continued)

114S-04	30" SQ. PPC Solid	01/14/08	48	178.82	Major clay with silt	462	58	520
117S-06	24" SQ. PPC Solid	04/16/08	3168	88.52	Major clay with silt	299	76	375
120S-04	24" SQ. PPC Solid	11/1/2007	168	137.03	Major clay with silt	462	84	546
123S-03	24" SQ. PPC Solid	11/16/07	768	138.48	Major clay with silt	530	131	661

Table 43
Pile type, capacity, soil information, and other information of the production piles at South Connector

Pile	Pile Type	Restrike Date	Time (hrs)	Penetration Length (ft)	Soil Type	R_{skin} (kips)	R_{tip} (kips)	R_{tot} (kips)
SC02-02	16" SQ. PPC Solid	7/28/2006	16	77.66	Major clay with sand	76	43	119
SC05-02	16" SQ. PPC Solid	8/3/2006	24	82.46	Major clay with sand	143	22	165
SC10-02	16" SQ. PPC Solid	8/10/2006	23	81.58	Major clay with sand	123	25	148
SC13-02	16" SQ. PPC Solid	8/14/2006	26	80.74	Major clay with sand	130	38	168
SC17-03	24" SQ. PPC Solid	9/12/2006	23	138.04	Major clay with sand	321	110	431
SC21-03	24" SQ. PPC Solid	9/20/2006	25	152.44	Major clay with sand	241	270	511
SC25-02	24" SQ. PPC Solid	11/2/2006	24	138.39	Major clay with sand	385	59	444
SC29-03	24" SQ. PPC Solid	11/9/2006	24	137.41	Major clay with sand	355	89	444
SC33-03	24" SQ. PPC Solid	11/16/2006	24	137.41	Major clay with sand	351	116	467
SC37-03	24" SQ. PPC Solid	11/30/2006	24	138.41	Major clay with sand	358	98	456
SC41-03	24" SQ. PPC Solid	12/8/2006	45	138.41	Major clay with sand	399	81	480
SC45-02	24" SQ. PPC Solid	12/14/2006	24	138.27	Major clay with sand	317	69	386
SC45-02	24" SQ. PPC Solid	12/18/2006	123	138.27	Major clay with sand	481	80	562

(continued)

SC49-02	24" SQ. PPC Solid	12/21/2006	48	135.77	Major clay with sand	389	76	464
SC52-03	30" SQ. PPC Piles	4/11/2007	44	167.64	Major clay with sand	914	134	1048
SC54-03	30" SQ. PPC Piles	5/17/2007	648	164.88	Major clay with sand	739	211	950
SC56-02	30" SQ. PPC Piles	No Restrike	0	164.4	Major clay with sand	646	360	1006
SC59-03	30" SQ. PPC Piles	6/27/2007	24	128.44	Major clay with sand	345	190	535
SC61-04	30" SQ. PPC Piles	7/17/2007	246	117.81	Major clay with sand	405	149	554

Table 44
Pile type, capacity, soil information, and other information of the production piles at ramp N1

Pile	Pile Type	Restrike Date	Time (Hrs)	Penetration Length (ft)	Soil Type	R_{skin} (kips)	R_{tip} (kips)	R_{tot} (kips)
N1-24-02	24" SQ. PPC	7/24/2007	88	118.1	Major Clay with silt	186	66	252
N1-24-02	24" SQ. PPC	8/23/2007	717	118.1	Major Clay with silt	241	39	280
N1-24-03	24" SQ. PPC	9/6/2007	1128	118.1	Major Clay with silt	370	50	420
N1-23-06	24" SQ. PPC	01/17/08	1440	78.35	Major Clay with silt	219	36	255
N1-21-03	24" SQ. PPC	8/8/2007	24	118.24	Major Clay with silt	293	60	353
N1-17-02	24" SQ. PPC	8/23/2007	30	118.24	Major Clay with silt	120	87	207
N1-17-03	24" SQ. PPC	9/6/2007	377	118.24	Major Clay with silt	262	54	316
N1-17-02	24" SQ. PPC	11/1/2007	1721	118.24	Major Clay with silt	331	74	405
N1-14-02	24" SQ. PPC	9/6/2007	140	118.1	Major Clay with silt	180	60	240
N1-14-02	24" SQ. PPC	11/1/2007	1484	118.1	Major Clay with silt	327	72	399
N1-12-02	24" SQ. PPC	9/17/2007	166	118.03	Major Clay with silt	324	81	405

(continued)

N1-12-02	24" SQ. PPC	10/3/2007	532	118.03	Major Clay with silt	345	100	445
N1-09-03	24" SQ. PPC	10/3/2007	336	117.29	Major Clay with silt	347	74	420
N1-05-03	24" SQ. PPC	10/11/2007	215	116.82	Major Clay with silt	296	75	372
N1-02-03	24" SQ. PPC	11/1/2007	504	117.94	Major Clay with silt	272	69	341
N1-02-03	24" SQ. PPC	11/29/07	1176	117.94	Major Clay with silt	321	80	401

Pile Load Testing Data at the LA-1 Relocation Project

The following pile load testing data were achieved from the "Report on Pile Load Test Program," LA-1 Improvements, Federal Project No. HP-NH-T021(002), State Project No. 700-29-0112, prepared by Wilbur Smith Associates for LADOTD.

Table 45
30-in. PPC pile - T3

Event	Date	Time	t (hours)	Ru (kips)	Rs (kips)	Rt (kips)
End of Driving	6/4/2004	7:22 PM	0.0	880	333	548
Restrike 2 hrs	6/4/2004	9:21 PM	2.0	914	334	580
Restrike 24 hrs	6/5/2004	7:00 PM	23.6	1065	414	650
Restrike 72 hrs	6/7/2004	4:35 PM	69.2	1187	537	649
Restrike 7 days	6/11/2004	1:44 PM	162.4	1297	655	641
Load Test	6/17/2004		312.0	1650	1129	521

Table 46
30-in. pipe pile - T3

Event	Date	Time	t (hours)	Ru (kips)	Rs (kips)	Rt (kips)
End of Driving	6/1/2004	4:03 PM	0.0	215	163	52
Restrike 2 hrs	6/1/2004	6:19 PM	2.3	485	427	58
Restrike 4 hrs	6/1/2004	8:09 PM	4.1	715	634	81
Restrike 24 hrs	6/2/2004	4:10 PM	24.1	834	733	101
Restrike 48 hrs	6/3/2004	4:55 PM	48.9	885	777	108
Restrike 72 hrs	6/4/2004	8:20 PM	76.3	907	798	110
Restrike 7 days	6/8/2004	8:35 PM	172.5	958	842	115
Load Test	6/16/2004		360.0	1597	1163	434

Table 47
54-in. cylinder pile - T3

Event	Date	Time	t (hours)	Ru (kips)	Rs (kips)	Rt (kips)
End of Driving	6/6/2004	2:15 PM	0.0	378	287	91
Restrike 2 hrs	6/6/2004	4:15 PM	2.0	696	596	99
Restrike 4 hrs	6/6/2004	6:06 PM	3.9	798	690	108
Restrike 24 hrs	6/7/2004	2:56 PM	24.7	1027	886	141
Restrike 48 hrs	6/8/2004	10:29AM	44.2	1112	971	141
Restrike 72 hrs	6/9/2004	2:37 PM	72.4	1169	1026	143
Restrike5 days	6/11/2004	11:39AM	117.4	1247	1104	143
Restrike 12 days	6/18/2004	1:55 PM	287.7	1337	1193	144
Load Test	6/22/2004		384.0	1395	1295	100

Table 48
24-in. 160 ft. long PPC pile - T4

Event	Date	Time	t (hours)	Ru (kips)	Rs (kips)	Rt (kips)
End of Drivinq	8/11/2004	3:45 PM	0.0			
Restrike 2 hrs	8/11/2004	5.42 PM	2.0	389	302	87
Restrike 4 hrs	8/11/2004	7:23 PM	3.6	475	381	94
Restrike 6 hrs	8/11/2004	9:32 PM	5.8	517	412	105
Restrike 24 hrs	8/12/2004	12.21PM	20.6	625	518	107
Restrike4 8 hrs	8/13/2004	12:39PM	44.9	820	666	154
Restrike7 2 hrs	8/14/2004	12:16PM	68.5	832	677	155
Restrike 96 hrs	8/15/2004	8:56 AM	89.2	880	724	156
Load T-est	8/17/2004		144.0	861	776	85

Table 49
24-in. 210 ft. long PPC pile - T4

Event	Date	Time	t (hours)	Ru (kips)	Rs (kips)	Rt (kips)
End of Driving	8/11/2004	12:57PM	0.0	730	561	174
Restrike 2 hrs	8/11/2004	4:04 PM	3.1	845	651	194
Restrike 4 hrs	8/11/2004	5:20 PM	4.4	865	656	209
Restrike 6 hrs	8/11/2044	7:35 PM	6.6	898	655	243
Restrike 8 hrs	8/11/2004	9:21 PM	8.4	923	672	251
Restrike 24 hrs	8/12/2004	12:38PM	23.7	920	667	253
Restrike 48 hrs	8/13/2004	1:08 PM	48.2	1027	746	281
Restrike 72 hrs	8/14/2004	1:12 PM	72.3	1197	910	286
Restrike 96 hrs	8/15/2004	9:53 AM	92.9	1197	899	298
Load Test	8/17/2004		144.0	1656	1310	346

Table 50
24-in. 145 ft. long PPC pile - T5

Event	Date	Time	t {hours}	Ru (kips)	Rs (kips)	Rt (kips)
End of Drivinq	7/27/2004	3:43 PM	0.0			
Restrike 3 hrs	7/27/2004	6:18 PM	2.6	341	137	204
Restrike 4 hrs	7/27/2004	7:53 PM	4.2	443	170	273
Restrike 24 hrs	7/28/2004	1:23 PM	21.7	558	272	287
Restrike 48 hrs	7/29/2004	2:20 PM	46.6	600	300	299
Restrike 72 hrs	7/30/2004	1:40 PM	70.0	654	327	326
Restrike 96 hrs	7/31/2004	10:21AM	90.6	641	314	327
Load Test	8/2/2004		144.0	739	696	43

Table 51
24-in. 170 ft. long PPC pile - T5

Event	Date	Time	t (hours)	Ru (kips)	Rs (kips)	Rt (kips)
End of Driving	7/27/2004	1:25 PM	0.0			
Restrike 3 hrs	7/27/2044	4:35 PM	3.2	415	225	191
Restrike 5 hrs	7/27/2004	6:41 PM	5.3	426	227	199
Restrike 7 hrs	7/27/2004	8:54 PM	7.5	469	264	205
Restrike 24 hrs	7/28/2004	1:02 PM	23.6	566	361	205
Restrike 48 hrs	7/29/2004	1:33 PM	48.1	561	356	245
Restrike 72 hrs	7/30/2004	1:22 PM	72.0	748	518	230
Restrike 96 hrs	7/31/2004	9:36 AM	92.2	818	598	220
Load Test	8/2/2004		144.0	769	680	89

Skin Friction Distributions on the Walls of the Selected Piles at Different Restrike Time

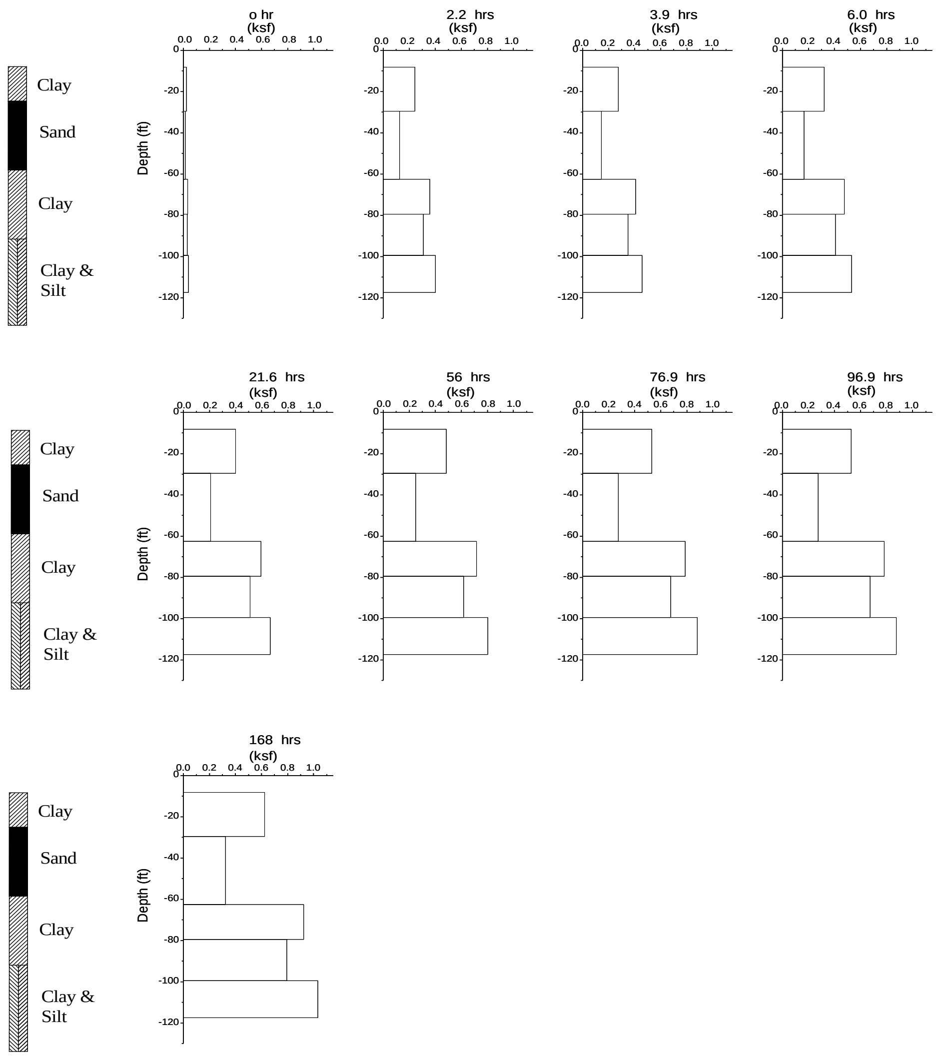


Figure 61
Unit skin friction distribution: 16-in. PPC pile – T2

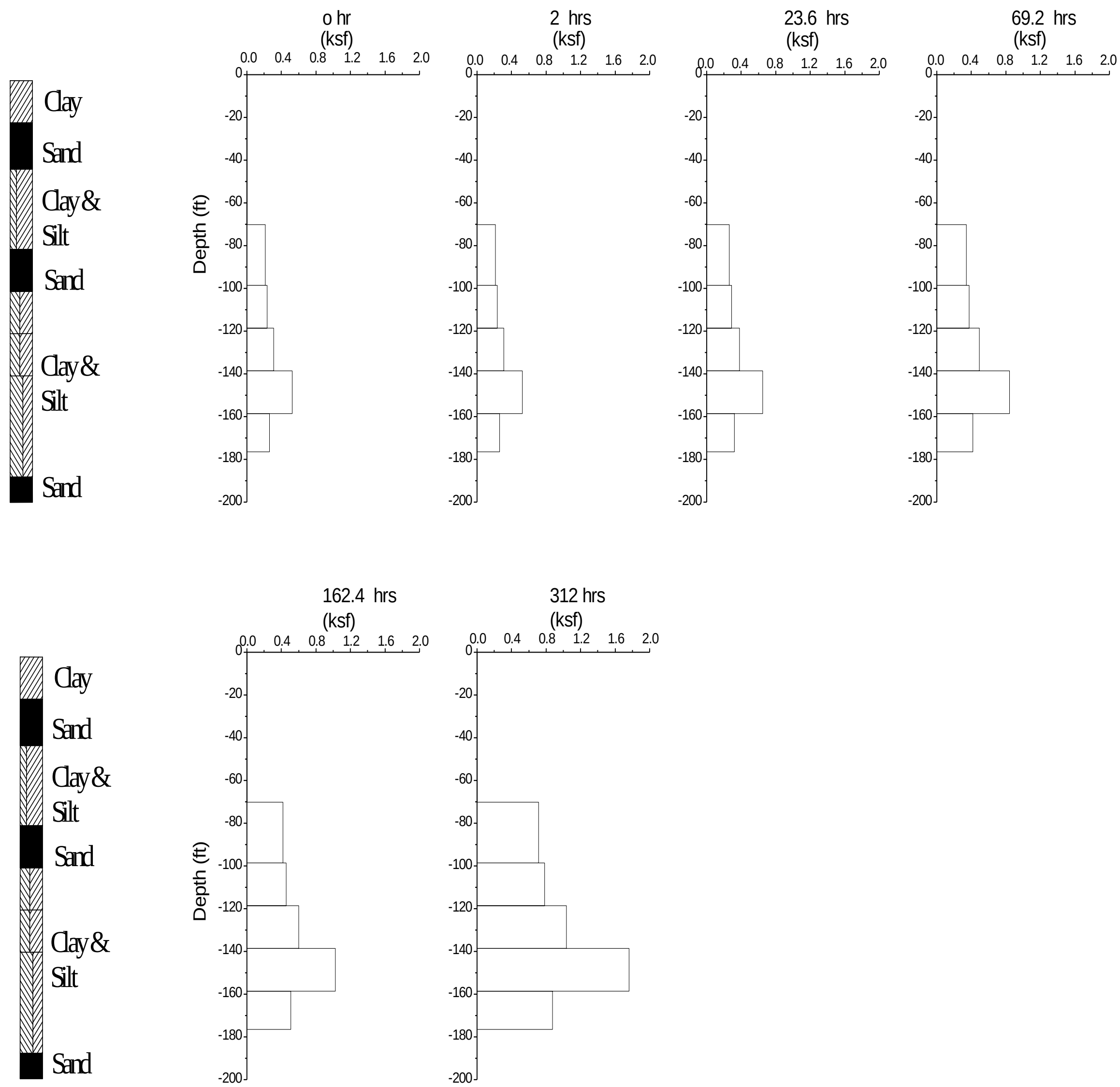


Figure 62
Unit skin friction distribution: 30-in. PPC pile – T3

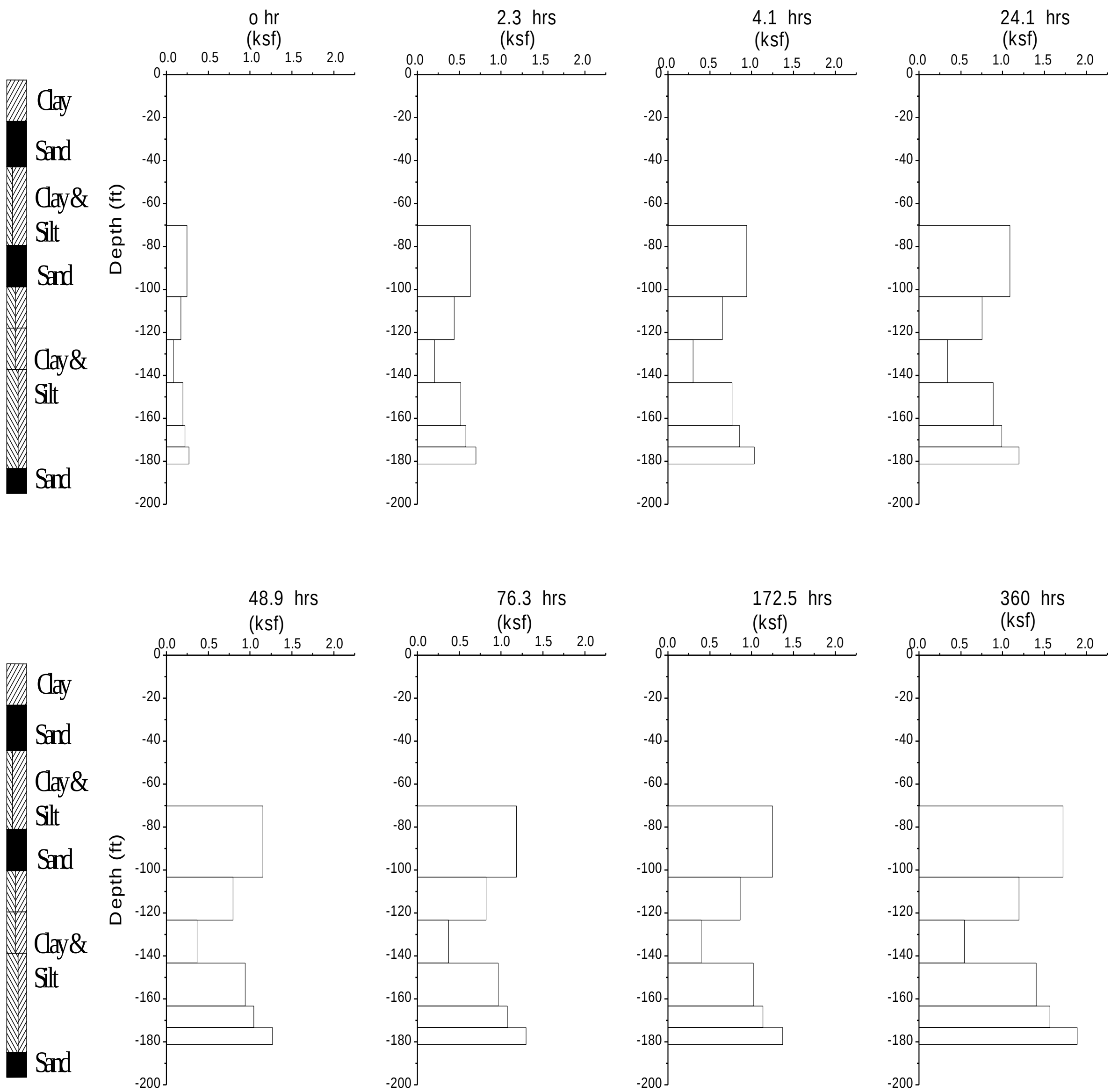


Figure 63
Unit skin friction distribution: 30-in. pipe pile – T3

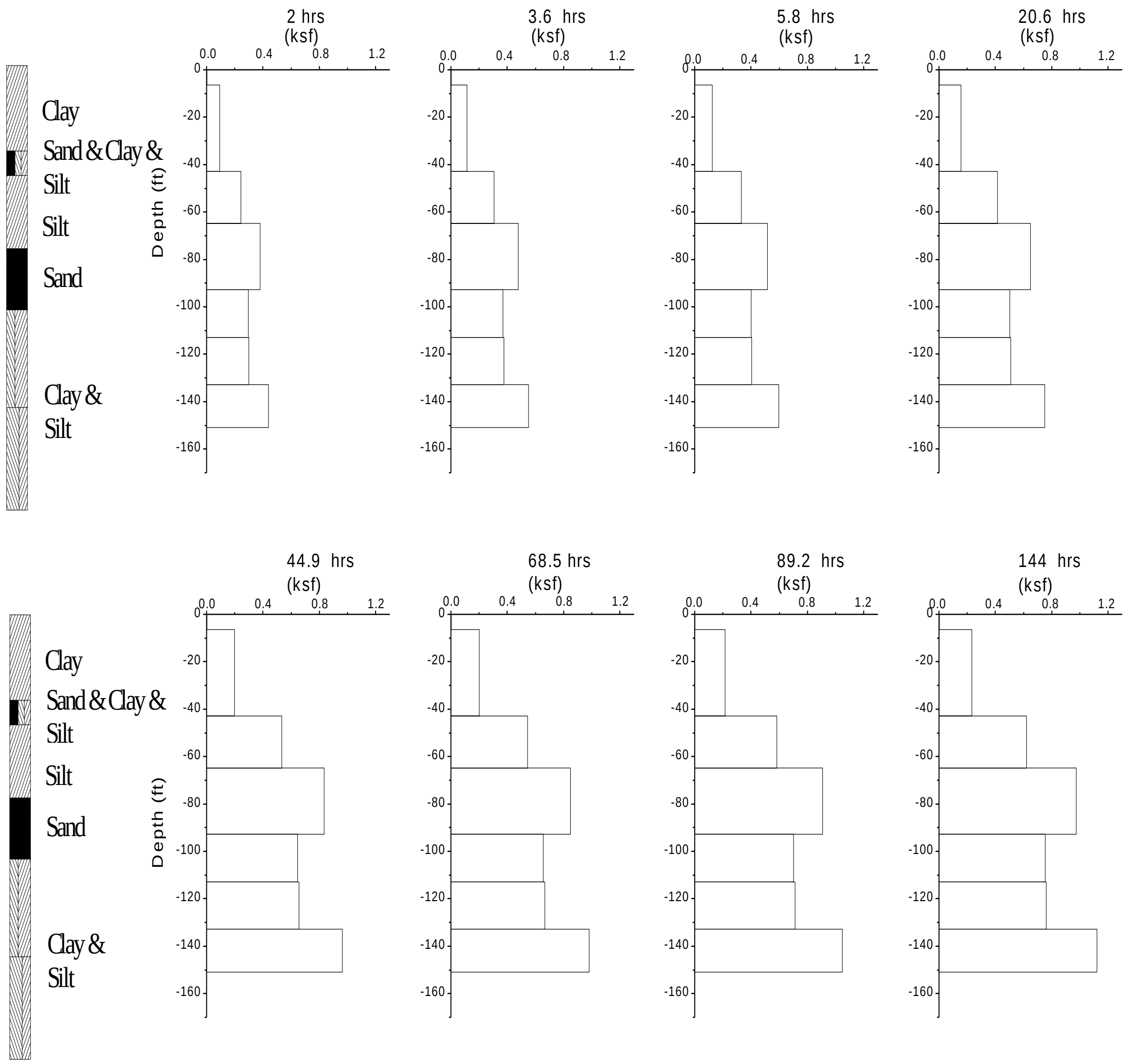


Figure 64
Unit skin friction distribution: 24-in. 160-ft. long PPC pile – T4

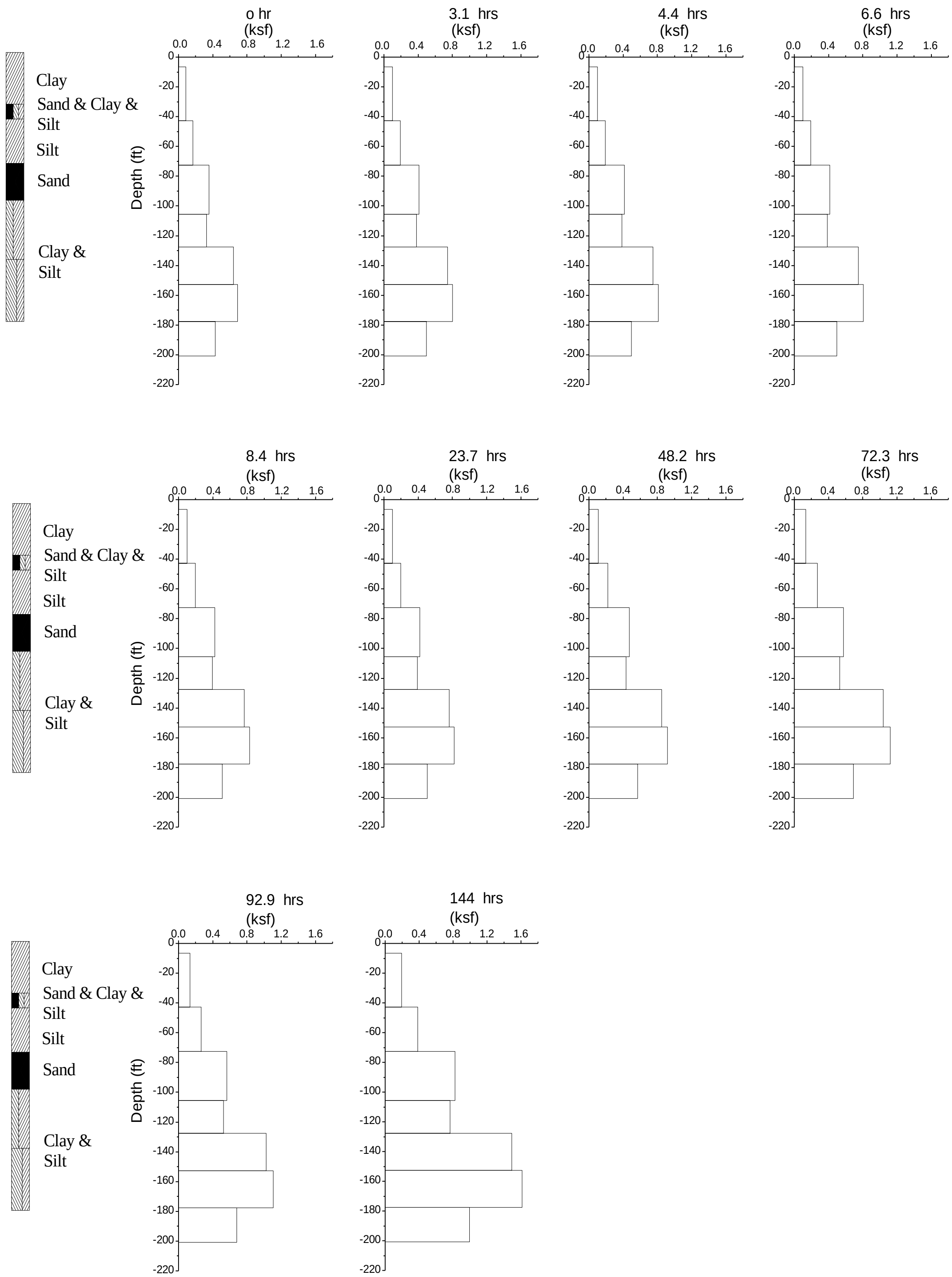


Figure 65
Unit skin friction distribution: 24-in. 210-ft. long PPC pile – T4

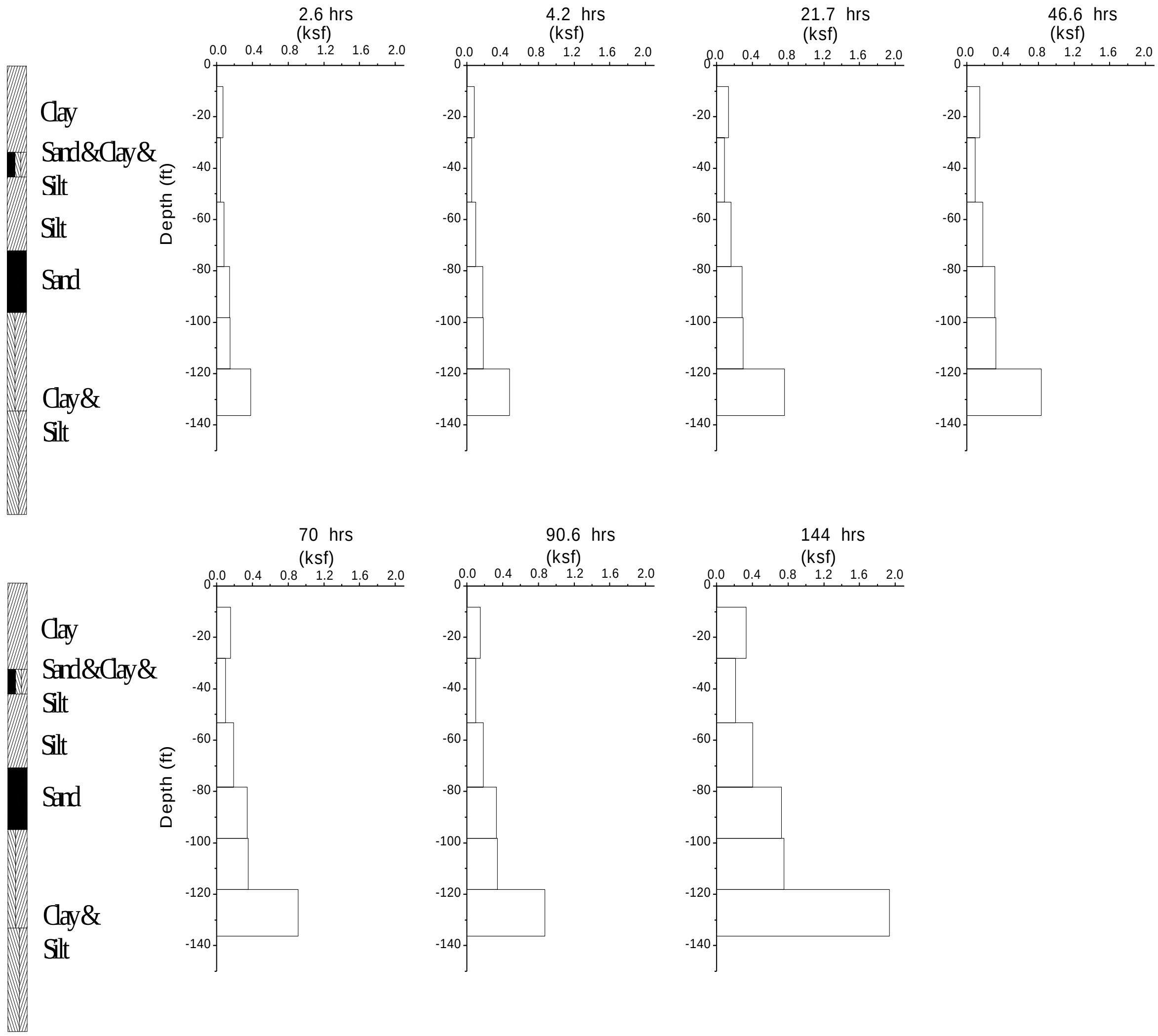


Figure 66
Unit skin friction distribution: 24-in. 145-ft. long PPC pile – T5

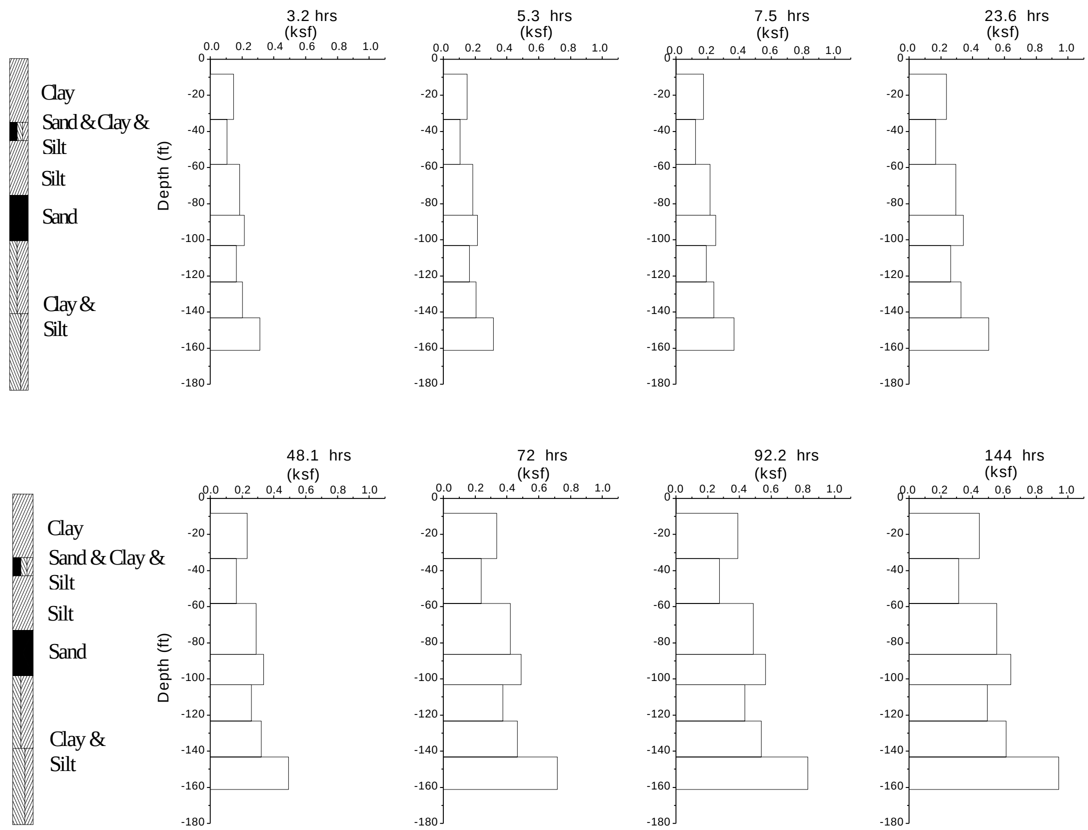


Figure 67
Unit skin friction distribution: 24-in. 170-ft. long PPC pile – T5

APPENDIX C

Setup Parameter *A* at Different Reference Times

Table 52
Setup factor *A* values for the shaft capacity for the cylinder pile at site T2

Restrike time (hrs)	Setup factor *A* values corresponding to different reference time (hrs)					
	t0 = 1.8	t0 = 5.1	t0 = 23.2	t0 =46.4	t0 = 70.3	t0 = 92.8
1.8	—					
5.1	0.62	—				
23.2	0.51	0.34	—			
46.4	0.45	0.29	0.13	—		
70.3	0.48	0.34	0.27	0.47	—	
92.8	0.47	0.32	0.24	0.33	0.12	—
168 (load test)	0.70	0.57	0.61	0.83	0.92	1.28

Table 53
Setup factor *A* values for the 30-in. PPC pile at site T3

Restrike time (hrs)	Setup *A* values with different reference time (hrs)			
	t0=2.0	t0=23.6	t0=69.2	t0=162.4
2.0	—			
23.6	0.22	—		
69.2	0.39	0.64	—	
162.4	0.50	0.69	0.59	—
312 (load test)	1.09	1.54	1.69	2.55

Table 54
Setup factor *A* values for the 54-in. cylinder pile at site T3

Reference time (hrs)	Setup factor *A* values corresponding to different reference time (hrs)						
	t0 = 2.0	t0 = 3.9	t0 =24.7	t0 =44.2	t0 = 72.4	t0 =117.4	t0=287.7
2.0	—						
3.9	0.54	—					
24.7	0.45	0.35	—				
44.2	0.47	0.39	0.38	—			
72.4	0.46	0.38	0.34	0.26	—		
117.4	0.48	0.41	0.36	0.32	0.36	—	
287.7	0.46	0.39	0.32	0.28	0.27	0.21	—
384.0 (load test)	0.51	0.44	0.39	0.36	0.36	0.34	0.68

Table 55
Setup factor *A* values for the 24-in. PPC pile (160 ft. long) at site T4

Reference time (hrs)	Setup factor *A* values corresponding to different reference time (hrs)						
	t0 = 2.0	t0 = 3.6	t0 =5.8	t0 =20.6	t0 = 44.9	t0 =68.5	t0=89.2
2.0	—						
3.6	1.02	—					
5.8	0.79	0.39	—				
20.6	0.71	0.47	0.47	—			
44.9	0.89	0.68	0.69	0.84	—		
68.5	0.81	0.61	0.60	0.59	0.09	—	
89.2	0.85	0.65	0.64	0.62	0.29	0.61	—
144.0 (load test)	0.85	0.65	0.63	0.59	0.33	0.45	0.35

Table 56
Setup factor *A* values for the 24-in. PPC pile (210 ft. long) at site T4

Reference time (hrs)	Setup factor *A* values corresponding to different reference time (hrs)							
	t0=3.1	t0 = 4.4	t0 =6.6	t0 =8.4	t0 = 23.7	t0 =48.2	t0=72.3	t0=92.9
3.1	—							
4.4	0.05	—						
6.6	0.02	-0.00	—					
8.4	0.07	0.09	0.25	—				
23.7	0.03	0.02	0.03	-0.02	—			
48.2	0.12	0.13	0.16	0.15	0.38	—		
72.3	0.29	0.32	0.37	0.38	0.75	1.25	—	
92.9	0.26	0.28	0.32	0.32	0.59	0.72	-0.11	—
144.0 (load test)	0.61	0.66	0.75	0.77	1.23	1.59	1.47	2.40

Table 57
Setup factor *A* values for the 24-in. PPC pile (145 ft. long) at site T5

Reference time (hrs)	Setup factor *A* values corresponding to different reference time (hrs)					
	t0 = 2.6	t0 = 4.2	t0 =21.7	t0 =46.6	t0 = 70.0	t0 =90.6
2.6	—					
4.2	1.16	—				
21.7	1.07	0.84	—			
46.6	0.95	0.73	0.31	—		
70.0	0.97	0.76	0.40	0.51	—	
90.6	0.84	0.64	0.25	0.16	-0.35	—
144.0 (load test)	2.34	2.02	1.90	2.69	3.60	6.05

Table 58
Setup factor *A* values for the 24-in. PPC pile (170 ft. long) at site T5

Reference time (hrs)	Setup factor *A* values corresponding to different reference time (hrs)						
	t0 = 3.2	t0 =5.3	t0 =7.5	t0 =23.6	t0 = 48.1	t0 =72.0	t0 =92.2
3.2	—						
5.3	0.04	—					
7.5	0.47	1.08	—				
23.6	0.70	0.91	0.74	—			
48.1	0.49	0.59	0.43	-0.04	—		
72.0	0.96	1.13	0.98	0.90	2.50	—	
92.2	1.14	1.32	1.16	1.11	2.41	1.44	—
144.0 (load test)	1.22	1.39	1.23	1.13	1.91	1.04	0.71

APPENDIX D

Examples of Pile Capacity Predictions by the Skov-Denver Model and the Rate-Based Model

Skov-Denver Method

$$S(t) = S(t_0)(A \log\left(\frac{t}{t_0}\right) + 1)$$

where,

S(t) = skin friction at time t (hrs);

$S(t_0)$= measured skin friction at reference time (t_0=24 hrs);

t = time elapsed since the end of initial driving;

t_0 = reference time, i.e., 24 hrs; and

A= 0.57 pile set-up parameter, usually 0.5 – 0.7 for Louisiana clayey soils.

Rate-Based Method

$$S(t) = \frac{1.846S(t_0)}{1 + 0.846e^{-0.261(\frac{t}{t_0}-1)}}$$

where,

S(t) = skin friction at time t (hrs),

$S(t_0)$ = measured skin friction at time (t_0=24 hrs),

t = time elapsed since the end of initial driving, and

t_0 = reference time, i.e., 24 hrs.

Predicted Total Pile Capacity

Q(t) = S(t) (predicted skin friction) + $T(t_0)$ (measured tip resistance at the reference time)

Example One

Given:

Pile name: SC45-02, located at South Connector of LA-1 relocation site with following details

Pile type: 24 in. sq. PPC solid

Plan pile length: 155 ft.; penetration length: 138.27 ft.

Soil details: major part clay with sand

Measured pile capacity at 24-hour restrike from CAPWAP analysis: skin friction: 317 kips, tip resistance: 69 kips, total capacity: 386 kips

Measured pile capacity at 123-hour restrike from CAPWAP analysis: skin friction: 481 kips, tip resistance: 80 kips, total capacity: 561 kips

$S(t_0)$= 317 kips $T(t_0)$ = 69 kips t_0 = 24 hours

Required: Predict total capacity of the pile at t = 123 hours

Skov-Denver Method

$$S(t) = S(0)(0.570\log\left(\frac{t}{t_0}\right) + 1)$$

$$S(123\text{ hrs}) = 317(0.570\log\left(\frac{123}{24}\right) + 1)$$

$$S(123\text{ hrs}) = 317 * 1.404$$

S(123 hrs) = 445 kips

Total capacity: Q(123) = 445 +69 = 514 kips

Rate-Based Method

$$S(t) = \frac{1.846S(t_0)}{1+0.846e^{-0.261(\frac{t}{t_0}-1)}}$$

$$S(123\text{ hrs}) = \frac{1.846*317}{1+0.846e^{-0.261(\frac{123}{24}-1)}}$$

$$S(123\text{ hrs}) = \frac{585.18}{1.288}$$

S(123 hrs)= 454 kips

Total capacity: Q(123) = 454 + 69 = 523 kips

Example Two

Given:

Pile name: 41S-03, pile located at mainline with following details

Pile type: 30 in. sq. PPC solid

Plan pile length: 100 ft.; penetration length: 75.57 ft.

Soil details: major part clay with sand

Measured pile capacity at 24-hour restrike from CAPWAP analysis: skin friction: 364 kips, Tip resistance: 76 kips, total capacity: 440 kips

Measured pile capacity at 96-hour restrike from CAPWAP analysis: skin friction: 446 kips, Tip resistance: 68 kips, total capacity: 514 kips

Measured pile capacity at 5040-hour restrike from CAPWAP analysis: skin friction: 800 kips, tip resistance: 100 kips, total capacity: 900 kips

$S(t_0)$= 364 kips $T(t_0)$ = 76 kips t_0 = 24 hours

Required: Predict total capacity of the pile at t = 96 hours and 5040 hours, respectively.

Skov-Denver Method

t = 96 hours

$$S(t) = S(t_0)(0.570 \log\left(\frac{t}{t_0}\right) + 1)$$

$$S(96\ \text{hrs}) = 364(0.570 \log\left(\frac{96}{24}\right) + 1)$$

$$S(96\ \text{hrs}) = 364 * 1.343$$

$$S(96\ \text{hrs}) = 488\ \text{kips}$$

Total capacity: Q(96) = 488 + 76 = 564 kips

t = 5040 hours

$$S(t) = S(t_0)(0.570 \log\left(\frac{t}{t_0}\right) + 1)$$

$$S(96\text{ hrs}) = 364(0.570\log\left(\frac{5040}{24}\right) + 1)$$

$$S(96\text{ hrs}) = 364 * 2.324$$

$$S(96\text{ hrs}) = 846\text{ kips}$$

Total capacity: Q(96 hrs) = 846 + 76 = 922 kips

Rate-Based Method:

t = 96 hours

$$S(t) = \frac{1.846S(t_0)}{1+0.846e^{-0.261(\frac{t}{t_0}-1)}}$$

$$S(96\text{ hrs}) = \frac{1.846*364}{1+0.846e^{-0.261(\frac{96}{24}-1)}}$$

$$S(96\text{ hrs}) = \frac{671.94}{1.38}$$

S(96 hrs)= 485 kips

Total capacity: Q(96 hrs) = 485 + 76 = 561 kips

t = 5040 hours

$$S(t) = \frac{1.846S(t_0)}{1+0.846e^{-0.261(\frac{t}{t_0}-1)}}$$

$$S(5040\text{ hrs}) = \frac{1.846*364}{1+0.846e^{-0.261(\frac{5040}{24}-1)}}$$

$$S(5040\text{ hrs}) = \frac{671.94}{1.00}$$

S(5040 hrs)= 672 kips

Total capacity: Q(5040 hrs) = 672 + 76 = 748 kips

Table 59
Measured and predicted pile capacities

Pile Name	Restrike time (hrs)	Measured total capacity (kips)	Skov- Denver Method (kips)	Rate - Based Method (kips)
SC45-02	123	561	514	523
41S-03	96	514	564	561
	5040	900	922	748

APPENDIX E

Pile Setup Survey Summary

As a part of the research project, a pile setup survey was conducted by sending a pile setup questionnaire to all the states in the United States and provinces in Canada. It turned out that a total of 36 states/provinces returned their responses. Most of the responded states/provinces think that pile setup is an important factor, and some have considered pile setup effect to some extents in their pile foundation design. However, no states/provinces have considered pile setup effect beyond two weeks after the end of driving. They have not thoroughly taken into account pile setup effect mainly because currently there is not a well-developed mathematical model available for setup prediction. All the completed surveys have been summarized and presented in the following two tables.

Table 60
Responses of the states/provinces for the pile setup survey

State/Province	Pile Set-up Considered	Importance of Set-up	Static Load Test Time	Current Pile Design Practice	Comments
Alaska	Yes	Not usually considered	Not usually performed	EOD capacity	Dynamic analysis on selected projects
Alberta	Yes	Not usually considered	EOD for rock, and up to 7 days for clay	No pile setup considered	See Note 4 in Table 61
Arkansas	No	Important, but not usually considered	Not usually performed	Dynamic Analysis	Based on the dynamic formula
British Columbia	Yes	Yes	Usually 7 days for clay, one day for sand	Based on borehole logs or CPT logs	—
Connecticut	Yes	Yes	Not specified, at anticipated pile freeze areas perform dynamic monitoring, instead of static load testing	Varies depending up on site/geology, usually between 1 day and 1 week.	A recent load testing report attached, and the data report could be provided if a ftp site is available.

(continued)

Florida	Yes	Yes	No specific time	End of drive	—
Georgia	Yes	Important and usually considered	3-7 days , depending on time to drive reaction piles and construct load frame	One day after installation	—
Hawaii	No	No	NA	NA	—
Idaho	Yes	Yes	Varies	Varies	—
Illinois	Yes	Important and usually considered	14 days	Capacity one day after installation	—
Iowa	Yes	Yes	After 40 hours	After 40 hours	—
Kansas	Yes	Yes	Rarely done	One day after pile installation	—
Kentucky	Yes	Yes	5 days	One day after pile installation	Design is based on “after setup,” but currently the minimum setup time in field is 1day.
Louisiana	Yes	Yes	14 days after installation	14 days after installation	—
Maryland	Yes	Important in some regions with the state	Not usually performed, only for large structures	Capacity at 3-day re-strike	—
Massachusetts	Not often considered	Important in some soils	Static load test rarely performed. It is usually done >=3 days if it is used (State specification)	Dynamic analysis, 1-2 day re-strike	99% testing dynamic. Up to two week set-up if strength gain expected or needed.
Michigan	Yes	Yes, but only in limited areas of the state	Not common to perform static pile load testing	At 1 or 3 days after pile installation	At 1 or 3 days after pile installation. A pile re-strike is completed if piles drive well past estimated tip elevation.
Mississippi	Yes	Yes	7 days after pile installation	7 days after pile installation	—

(continued)

Missouri	No	Yes	No static load testing performed – dynamic load testing done on some projects at 3-7 days after installation	EOD capacity using the AASHTO Gates Formula or PDA results from dynamic testing	—
New Hampshire	Yes	Yes	7 days	Based on long term capacity, e.g., typically > one month	—
New Jersey	Yes	Yes	7 to 14 days are anticipated for setting up load test apparatus	PDA/CAPWAP dynamic load tests are required for all test piles. Re-strikes are performed to determine the set-up capacity if initial required driving resistance is not achieved.	—
New Mexico	Yes	Yes	7 days after pile installation	48 hours re-strike	—
New York	Yes	Usually considered in design	Rarely performed	Capacity at one day after installation, depending on soil type.	If static load test performed, it is usually done 7 days after EOD.
North Carolina	Yes	Yes	Dynamic load testing is preferred	Use set up to verify bearing of pile that does not obtain capacity during initial driving.	—
Oregon	Yes	Important in some soils	Not usually performed	EOD capacity for cohesion less soils, and 24 hour re-strike for cohesive soils.	See Note 3 in Table 61

(continued)

Pennsylvania	No	Important in some soils	Rarely performed, and dynamic monitoring is routinely used for friction piles	End of driving strength for rock, 3~7 days for soils that potentially provides setup	See Note 2 in Table 61
Saskatchewan	No	Not Important	Not usually performed	Usually EOD capacity, until unexpected capacity occurs, then wait for one day after EOD	—
South Carolina	Yes	Yes	Seven days after pile installation	Varies from 1 hr to 14 days after EOD	—
South Dakota	No	Important, but not usually considered	Not usually performed	EOD capacity, determined during driving operation	No recent static load testing data available.
Tennessee	No	Yes	3 days after installation	3 days after pile installation	Initial design is based on butt bearing and friction estimates, but capacity is based on the 3-days test pile results from static load tests.
Texas	Yes	Usually considered in design	7 days	Based on lab/in-situ soil strength	See Note 1 in Table 61
Utah	Yes	Yes	State performs PDA with wave matching 1 to 2 days after EOID	1 day after pile installation.	—
Vermont	Yes	Important and usually considered	Static load test at 48 hours after EOD	Capacity 48 hours after installation	—
West Virginia	No	No	No static load test, except for bigger project	Only rely on end bearing	—
Wisconsin	Yes	Yes	Generally not perform any static pile load tests	Based on capacity at the end-of-drive	See Note 5 in Table 61
Wyoming	No	No	No static or dynamic tests done	N/A	—

Table 61
Additional survey information

Note 1	Design of piling is based on the strength of soil measured either in-situ or in laboratory testing. We are concerned with pile setup mainly because loss of strength during driving leads to the indication that piling are not capable of carrying their required design loads. Understanding pile setup is necessary to interpret the results of pile driving data.
Note 2	PennDOT practice is typically to base pile capacity on the end of driving as piles are generally driven to rock. However, a waiting period and restrike may be specified due to the nature of the soils or rock or location of the water table or if difficulty is encountered in achieving pile capacity (at the time of driving) and the nature of the soil is such that there may be some set-up. Waiting periods generally range from 3 to 7 days, and a restrike is performed.
Note 3	Oregon will accept piles based on both end-of-initial driving (EOID) and beginning of restrike (BOR) criteria. EOID criteria is generally used with granular (cohesionless) soils where significant soil setup is not anticipated. If we are driving into cohesive soils where we are going to rely on pile setup, we will wait a minimum of 24 hours before restrike. We restrike as minimum of one in ten piles and at least one pile per bent (we don't restrike all piles). This 24-hour time period may be extended depending on the drainage properties (permeability) of the soil and the judgment of the engineer as to how soon they regain strength after driving.
Note 4	Designs follow the *Canadian Foundation Engineering Manual*, we don't rely on pile setup for design capacity since the ability to predict pile setup accurately is generally not reliable. If we do achieve some pile setup, it is a bonus capacity which may come into play as design loadings increase (larger trucks, heavier axle loads, etc.)
Note 5	▪ WisDOT used pile setup on a limited number of projects. If it appears that CIP (cast-in-place) piles may run very long, WisDOT may call for driven length and complete retaps. This occurs on 1-2 projects per year. ▪ Pile setup is extensively used on the Marquette Interchange mega project in Milwauke. ▪ A recent research study has been completed, in an attempt to arrive at a method to estimate pile setup in design phase. Results were not very conclusive and recommendation was to generally assume a 20% increase in pile capacity. ▪ Static load tests were only used in the Marquette Interchange project recently. Tests usually ranged from 5-25 days after EOD.

Questionnaire on Setup (or Freeze) of Driven Piles

Name: _________________________ Title: ______________________

State: ________ Date/Time: ____________

Dear Sir or Madam:

The Louisiana Transportation Research Center is conducting research on the subject of pile set-up, which is the pile capacity increase with time after installation. The purpose of this research is to integrate pile setup into pile foundation design for the state of Louisiana. Please take a few minutes of your time answering the following questions regarding how pile setup has been incorporated into pile foundation design in your state and to what extent. A summary of the collected data will be available for all participants. Your help would be greatly appreciated. If you have any questions, please contact Jay Wang.

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Construction Engineering Technology
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Ruston, LA 71272
Tel: (318) 257-2934
Email: xwang@latech.edu

1. Has pile setup been taken into account to any extent in pile foundation design in your state?
   - ☐ Yes
   - ☐ No

2. Do you think pile setup is important and may contribute significant long-term pile capacity in your state's geology?
   - ☐ Yes
   - ☐ No

If you marked "No" to both questions 1 and 2, please skip to question 6.

3. When do you usually perform static load testing after test pile installation?
   - ☐ One day after installation
   - ☐ Three days after installation
   - ☐ Seven days after installation
   - ☐ Fourteen days after installation
   - ☐ Other, please specify_________________________

4. The current pile design practice at your state is based on pile capacity
   - ☐ At one day after pile installation
   - ☐ At three days after pile installation
   - ☐ At seven days after pile installation

- ☐ At fourteen days after pile installation
- ☐ At one month after pile installation
- ☐ Other, please specify___________________________

5. May we contact you or somebody else for more detailed information?
   - ☐ Yes
   - ☐ No

If your answer is "Yes," please provide contact information as follows.

6. In order to enhance our pile setup research, is it possible to share pile load test data collected in your state (static and/or statnamic test data from test piles, restrike data from production piles, etc.)?
   - ☐ Yes
   - ☐ No

If your answer is "Yes," please provide contact information as follows.

7. Would you like to receive a final survey summary?
   - ☐ Yes
   - ☐ No

Thank you very much for your time, attention, and cooperation!

Information for further contact

☐ For more detailed survey ☐ For pile testing data

Name: ____

Address 1: ____

Address 2: ____

Address 3: ____

City: ____

State: ____

Zip: ____

Email: ____

Phone: ____