Assessment of Current AASHTO LRFD Methods for Static Pile Capacity Analysis in Rhode Island Soils

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July 2013

URITC PROJECT NO. 000149

PREPARED FOR
UNIVERSITY OF RHODE ISLAND
TRANSPORTATION CENTER

1. Report No.	2. Government Accession No.	3. Recipients Catalog No.

4. Title and Subtitle	5. Report Date
Assessment of Current AASHTO LRFD Methods for Static Pile Capacity Analysis in Rhode Island Soils	July 2013
	6. Performing Organization Code N/A
7. Author(s) Aaron S. Bradshaw and Sean Davis	8. Performing Organization Report No. N/A
9. Performing Organization Name and Address Department of Civil and Environmental Engineering University of Rhode Island Bliss Hall Kingston, RI 02881	10. Work Unit No. (TRAIS) N/A 11. Contract or Grant No. URI 000149
12. Sponsoring Agency Name and Address University of Rhode Island Transportation Center 75 Lower College Rd Kingston, RI 02881	13. Type of Report and Period Covered Final 14. Sponsoring Agency Code

15. Supplementary Notes
N/A

16. Abstract

This report presents an assessment of current AASHTO LRFD methods for static pile capacity analysis in Rhode Island soils. Current static capacity methods and associated resistance factors are based on pile load test data in sands and clays. Some regions of Rhode Island including Providence and Narragansett Bay are underlain by very silty soils. Therefore, the use of the AASHTO pile capacity methods is uncertain in these soils, which can have important safety or cost implications. To address this objective static loading test data were compiled from recent bridge projects within the state. The capacity of the test piles were also predicted using the Nordlund method and SPT method as specified by AASHTO. The measured and predicted capacities were compared to assess both the accuracy and the precision of the methods as well as calibrate preliminary resistance factors. The results showed that capacities of high-displacement piles were overpredicted in the majority of the cases. Gross overpredictions were observed at the Jamestown bridge site. Preliminary resistance factors of 0.20 and 0.42 were calibrated for the Nordlund and SPT methods, respectively, for high-displacement piles driven to glacial till in Providence.

17. Key Words	18. Distribution Statement
Driven piles, static capacity methods, resistance factors, LRFD, AASHTO bridge specifications	No restrictions. This document is available to the Public through the URI Transportation Center, Carlotti Administration Building, 75 Lower College, Rd., Kingston, RI 02881

19. Security Classification (of this report)	20. Security Classification (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	34	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized (art. 5/94)

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1. INTRODUCTION

1.1 Background

The current AASHTO bridge design code mandates the use of Load and Resistance Factor Design (LRFD) for design. In the design of a foundation system it is necessary to predict the geotechnical capacity of the pile, in addition to the structural capacity, such that the pile type, pile size, and embedment depth can be selected to support the structural loads. Static capacity calculations are always performed during the design phase, and a load testing program may or may not be implemented during the construction phase depending on the size of the project. Therefore, the design plans and project costs are based largely on static capacity calculations and thus it is important that these are accurate to avoid unsafe designs, uneconomical designs, or costly re-designs.

The philosophy of design in civil engineering practice will continue toward reliability-based design. The reliability based design concept is illustrated in Figure 1. Two histograms are shown in the figure, one representing the distribution of the applied loads and the other representing the distribution of resistance (i.e. strength). The overlap of the two distributions indicates the cases where the loads are higher than the resistance. The overlap is mathematically related to the probability of failure; the greater the overlap the higher the probability of failure. The objective is to design the structural component such that the resistance is higher than the load. The design equations ensure that a minimum level of reliability (defined as one over the probability of failure) is achieved.

Load and Resistance factor Design (LRFD) is one form of reliability-based design. The LRFD design equation is as follows:

$$\sum \phi R_n \geq \sum \gamma Q_n \tag{1}$$

ϕ=resistance factor, R_n=nominal resistance, γ=load factor, and Q_n=nominal load. As shown in Figure 1, the resistance factors and load factors can be thought of as shifting the distributions such that the overlap (i.e. probability of failure) is not higher than the accepted level. Therefore, the resistance factors are generally less than one to reduce the resistance and the load factors are greater than one to increase the loads. The major advantage to the method is that it allows all components of the uncertainty to be isolated and not lumped together into one "global" factor of safety such as is used in the Allowable Stress Design (ASD) method.

Relative to the structural engineering community, the geotechnical engineering community has been slower to adopt LRFD in design practice. However, the AASHTO bridge design specifications (AASHTO 2007) now requires LRFD for geotechnical design. AASHTO (2007) specifies the static capacity methods for driven piles and associated resistance factors and these are listed in Table 1. The static capacity methods in Table 1 are categorized by soil type as "sand", "clay" or "mixed" soils. Sand generally implies that the pile loading occurs under drained conditions, whereas clay is assumed to be loaded under undrained conditions. When silty soils are loaded it is generally assumed that non-plastic silt is drained and therefore treated as "sand" and organic silt is treated as "clay". However, many of the static loading tests from which the static capacity methods are based, where derived from load tests in sands and clays.

This issue is important because many areas of Rhode Island are underlain by silty soils particularly in and around Narragansett Bay including Providence (Baxter et al. 2005). Non-plastic uniform silts in particular were deposited as glacial lake sediments during the last glacial retreat. These soils have been shown to be problematic leading to ground movements during construction (Bradshaw et al. 2007) and softened pile response (Bradshaw et al. 2012). Two of the most recent bridge projects including the Jamestown bridge and the Sakonnet River Bridge are located at sites underlain by silty soils. The accuracy of pile capacity predictions in these soils is uncertain and has been an area of research interest (e.g., Kim et al. 2009).

The current resistance factors shown in Table 1 were calibrated from national databases and previous factors of safety. Note that the resistance factors are specific to the method used, the loading direction, and the type of soil. The intent of the current resistance factors in the code is to provide a sufficiently low value such that the design is safe for analyses performed in a wide range of pile types and soil conditions. However, given the high variability in pile and soil types from region to region the sample tests used to calibrate the resistance factors may not represent the population of all possible soil conditions. For this reason Paikowsky et al. (2004) recommends that calibrated resistance factors be checked against case studies to test their validity and modified as necessary.

As of 2008 12 known DOTs are following LRFD specifications with their own regionally calibrated resistance factors. The regionally calibrated resistance factors reported are equal to or greater than those recommended by AASHTO (AbedelSalam et al. 2010). The Minnesota DOT has successfully implemented LRFD and is moving towards regionally calibrating resistance factors that reflect their design and construction practices as well as the soil profile (Dasenbrock et al. 2009). An investigation into the current design practices in Iowa lead to an ongoing research project aimed at creating calibrated LRFD resistance factors (Roling et al. 2011). Kim (2005), Kebede (2010) also suggest regionally calibrated resistance factors for the North Carolina DOT and Missouri DOT, respectively.

Major bridge projects performed by RIDOT provide excellent case studies for this purpose. For example, the Sakonnet bridge project in Rhode Island was the first bridge project in the United States to utilize the new AASHTO LRFD code. The bridge is founded on six-foot diameter pipe piles driven into the underlying silty soils. Paikowsky et al. (2010) briefly discusses the validity of resistance factors for piles at the Sakonnet River Bridge project. However, there have been no comprehensive studies of resistance factors for driven piles in Rhode Island soils. Given that future bridge projects will encounter similar soil types, validation of static capacity methods and resistance factors is critical to optimizing designs and avoiding issues during construction.

1.2 Objectives

The objectives of this project were to:

- Assess the accuracy and precision of current AASHTO LRFD recommended static capacity methods for piles driven in Rhode Island soils, and
- Develop region-specific resistance factors for driven piles in Rhode Island soils that could be used for design.

1.3 Scope of Work

Five main tasks were completed to address the project objectives that included the following:

- Compile static loading test data from test piles installed on Rhode Island bridge projects,
- Interpret the ultimate resistance of the test piles from the static loading test data using the methods specified by AASHTO,
- Predict the static capacities of the selected test piles using the static capacity methods specified by AASHTO,
- Compare the predicted and measured test pile capacities to assess the accuracy and precision of the AASHTO static capacity methods,
- Perform preliminary region-specific calibration of resistance factors for use with the AASHTO static capacity methods.

1.4 Structure of Report

This report is structured in three remaining sections. Section 2 describes the methodology, Section 3 discusses the results, and Section 4 presents a brief summary and conclusions.

2. METHODOLOGY

This section describes the methods used in this study including the compilation of static loading test data, interpretation of capacity from static loading test data, prediction of static capacity, and preliminary calibration of resistance factors.

2.1 Compilation of Geotechnical Data

A database of static loading test results was compiled from available geotechnical reports from six major bridge projects completed within Rhode Island. A list of projects, locations, and reports is summarized in Table 2. The project locations are also shown on the map in Figure 2. The following information was documented for each test pile: pile type and dimensions, instrumentation details (if used), pile driving record, measured load-displacement curve, load transfer curves (if available) as measured with telltales or strain gages, and closest boring log.

2.2 Interpretation of Capacity from Static Loading Test Data

Consistent with AASHTO (2007) the ultimate capacities were interpreted for each of the test piles using Davisson's criterion. The offset line is defined by the following equation (Hannigan et al. 2005):

$$s = \frac{PL}{AE} + 4.0 + 0.008B \tag{2}$$

where s=movement of the pile head in mm, P=test load in kN, L=pile length in mm, A=cross sectional area of pile in m^2, E=modulus elasticity of pile in kPa, and B=pile width in mm. The intersection of the offset line with the load-movement curve indicates the ultimate resistance.

2.3 Prediction of Static Pile Capacity

Static capacity was "predicted" for the test piles using the methods specified in the AASHTO (2007). At each test pile location a layered design soil profile was developed based on stratigraphic changes observed in the boring logs and SPT blow counts. Average properties were obtained within each layer and a unit shaft resistance for each layer was calculated and integrated over the shaft area to determine the shaft resistance. The unit toe resistance was also determined and integrated over the toe area to obtain the toe resistance. Consistent with AASHTO procedures the analysis of H-piles used the plugged box area for both shaft and toe resistance calculations.

Nordlund Method

The Nordlund method (Nordlund 1963) is a theoretically-based method calibrated from a load test database of various pile types in “sands”. It is by far the preferred method for estimating pile capacity in cohesionless soils (Hannigan et al. 1998, Paikowsky 2004) and has the highest resistance factor in the AASHTO code. The major advantage is that it can accommodate different pile types including tapered piles. The equation in AASHTO for unit shaft resistance is given by:

$$f_s = K_\delta C_F \sigma'_v \frac{\sin(\delta + \omega)}{\cos(\omega)} \tag{3}$$

where f_s =unit shaft resistance, K_d =coefficient of lateral earth pressure at mid-point of soil layer (chart), C_F =correction factor for K_d when δ does not equal the friction angle of the soil (chart), σ'_v =effective overburden stress at mid-point of soil layer, δ =interface friction angle (chart), and ω =angle of pile taper from vertical. The unit toe resistance is given by:

$$q_t = \alpha_t N'_q \sigma'_v \leq q_l \tag{4}$$

where, q_t =unit toe resistance, a_t =dimensionless coefficient, dependent of pile depth-width relationship (chart), N'_q =bearing capacity factor (chart), σ'_v =effective overburden stress at pile toe, and q_l = limiting unit toe resistance determined from a correlation to effective strength friction angle (ϕ') originally developed from cone penetration test data. The overburden stress in Equation 2 was also limited to 150 kPa.

Consistent with Paikowsky (2004) the correlation originally proposed by Peck, Hanson and Thornburn (1978) was used to determine the effective strength friction angle (in degrees) from the following equation (Kulhawy and Mayne 1990):

$$\phi' \approx 54 - 27.6034\exp(-0.014N) \tag{5}$$

where N =raw (uncorrected) blow counts in blows per foot. It is difficult to interpret high blow counts that might be obtained in dilative silts, gravelly soils, and till. It is anticipated that the friction angle of the till in particular could exceed 40 degrees given the well-graded and very dense nature of these soils. However, consistent with Paikowsky (2004) the average calculated friction angle within a given layer was limited to 36 degrees that would include some level of conservatism that would likely be used in engineering practice. Informal conversations with local geotechnical engineers suggest that friction angle limitations of around 36 degrees are commonly used because they have shown to yield capacity predictions that are more consistent with static loading test results.

SPT Method

The SPT method (Meyerhof 1976) is an empirical method based on a database of driven piles in sand. The equations in AASHTO for unit shaft resistance (in kips/ft^2) for high-displacement and low-displacement piles (e.g., H-piles) respectively are as follows:

$$f_s = \frac{(\overline{N}_1)_{60}}{25} \tag{6}$$

$$f_s = \frac{(\overline{N}_1)_{60}}{50} \tag{7}$$

where $(\overline{N}_1)_{60}$ = average SPT blow count along pile shaft corrected for overburden stress and hammer energy in blows/ft. The unit toe resistance is calculated from the following:

$$q_t = \frac{0.8(\overline{N}_1)_{60} D_b}{D} \leq q_l \tag{8}$$

where $(\overline{N}_1)_{60}$ =average corrected blow counts within a depth of 2D below the pile toe D_b= depth of penetration in the bearing strata, and D= pile width or diameter. q_l is taken as 8 times $(\overline{N}_1)_{60}$ for sands and 6 times $(\overline{N}_1)_{60}$ for non-plastic silts. Given that the original figures developed by Meyerhof were truncated at 60 blows per foot, the $(\overline{N}_1)_{60}$ calculated in each soil layer was limited to 60 blows per foot.

It is interesting to note that Meyerhof's original method used blow counts corrected for overburden stress only (Hannigan et al. 1998). The additional energy correction would increase the level of conservatism in hammers that have efficiencies of less than 60%. Meyerhof's original method also included an additional equation for q_t when the pile toe was close to a stratigraphic boundary that would consider the contributions of the soil above and below the boundary. Equation 8 assumes that the soil above the bearing stratum has no contribution to the bearing capacity, which is conservative. For the Providence piles D_b was taken as the depth of penetration into the very dense sand or till layer. If the piles did not encounter a distinct bearing stratum as was common at the Narragansett Bay sites, D_b was taken as the entire pile embedment.

2.4 Preliminary Calibration of Resistance Factors

Various methods are available to calibrate resistance factors from load test data (Allen et al. 2005). As a preliminary step the First Order Second Moment (FOSM) method was used to calibrate resistance factors in this study. FOSM provides a closed-form equation to calculate resistance factor assuming that the both the resistance and the loads are log-normally distributed. The method has been used extensively in resistance factor calibration studies from the literature. The method yields resistance factors that are approximately 10% lower than the more robust and accurate FORM method (Paikowsky 2004) and thus is anticipated to be conservative.

If only the dead load and live loads are considered the resistance factor (φ) is calculated from the following equation (Barker et al. 1991):

$$\phi = \frac{\lambda_R (\gamma_D \frac{Q_D}{Q_L} + \gamma_L) \sqrt{\frac{(1 + COV_{Q_D}^2 + COV_{Q_L}^2)}{(1 + COV_R^2)}}}{(\lambda_{Q_D} \frac{Q_D}{Q_L} + \lambda_{Q_L}) \exp\{\beta_T \sqrt{\ln[(1 + COV_R^2)(1 + COV_{Q_L}^2 + COV_{Q_D}^2)]}\}} \quad (9)$$

where, γ_D ,γ_L = dead and live load factors, Q_D/Q_L = dead load to live load ratio, λ_{QD}, λ_{QL} = dead load and live load bias factors, COV_Q = coefficient of variation of the load, λ_R = resistance bias factor, COV_R= coefficient of variation of the resistance, β_T = target reliability index.

3. RESULTS

This section presents the results including the soil conditions, static loading test capacities, a comparison of measured and predicted capacities, and preliminary resistance factors.

3.1 Soil Conditions

The soil conditions in Rhode Island are largely influenced by the last glacial period that occurred approximately 15,000 year ago (Murray 1988; Baxter et al. 2005). The Wisconsinan ice sheet advanced southward, retreated, and then advanced again depositing glacial till and leaving two sets of terminal moraines; one located along what is now the south coastline of Rhode Island and the other offshore in the area of Block Island. Deep bedrock valleys were formed from the scouring and widening of ancient rivers that formed during the glacial advancement. The deep bedrock valleys are generally located in what is now Narragansett Bay.

As the glacier receded water became impounded between the terminal moraines to the south and the ice sheet to the north. Large deposits of outwash soils were deposited in the glacial lake including glaciofluvial deposits that tend to be sandy and glaciolacustrine deposits that are silty. The inorganic silt deposits are non-plastic, very thick in some locations, and may contain seasonal varves. The silt deposits in Providence, for example, are composed of greater than 95% fines. Eventually the terminal moraines ruptured initiating river erosion and formation of the current bay. Sea level rise eventually deposited organic silts in some locations and fill was also placed in many coastal areas from urban development.

The boring information reflects the geologic history of the region. A typical soil profile from one of the Providence sites is shown in Figure 3. As shown in the figure the conditions typically consist of thin layers of fill and organic silt, underlain by thicker layers of sand and silt outwash deposits, underlain by till and bedrock. Bedrock was typically encountered at a depth of approximately 35 m below the ground surface. A typical soil profile from one of the Narragansett Bay sites is shown in Figure 4. The conditions also consist of thick sequences of sand and silt outwash deposits but bedrock is much deeper (>50 m).

3.2 Static Loading Test Capacities

A typical load-movement curve obtained from the static loading test is shown in Figure 5. The pile load test data from all test piles are stored electronically and also presented in Davis (2012).

For this study it was necessary to utilize piles that were loaded to their ultimate resistance defined using Davisson's criterion. There were some pile types (e.g. tapered piles and H-piles) that had only one or two piles in the data set and therefore did not provide enough data for statistical analysis. Therefore, of the 40 test piles that are in the database only 14 of the piles are included in this study as summarized in Table 4. All of the 14 test piles were high-displacement piles consisting of either square precast prestressed concrete (PPC) or pipe piles driven closed-ended.

One concern in interpreting load test data for piles in silty soils is the drainage conditions during load testing. Analysis of pore pressure data during pile driving (Bradshaw et al. 2007) and during load testing (Bradshaw et al. 2012) has shown that excess pore pressures generally dissipate in the silts within about ½ to 1 hour after driving. Therefore, it was assumed in this study that the load test results represent drained loading conditions.

As shown in Table 4 the ultimate resistances at the two Providence sites ranged from 1,334 to 2,580 kN. The ultimate resistances at the one Narragansett Bay site ranged from 738 to 4,626 kN.

3.3 Static Capacity Predictions

The ultimate resistances predicted for the selected test piles using the Nordlund and SPT methods are summarized in Table 5. The predicted capacities using the Nordlund method ranged from 1,361 kN to 7,197 kN for the Providence sites (Civic Center and I-195 Interchange) and 6,815 kN to 42,712 kN for the Jamestown Bridge site in Narragansett Bay. The capacities predicted using the SPT method ranged from 2,358 kN to 3,407 kN for the Providence sites and 5,556 kN to 19,577 kN for the Jamestown bridge site.

3.4 Comparison of Predicted and Measured Capacities

The predicted and measured ultimate resistances of the selected test piles are summarized in Table 5. The accuracy and precision of the predictions were assessed by calculating a bias for each test pile defined as the ratio of the measured capacity to the predicted capacity. A summary of the bias data is presented in Table 6. The mean of the bias data was used to evaluate the accuracy of the model predictions, whereas the coefficient of variation (COV), defined as the ratio of standard deviation to the mean, was used to assess the precision of the predictions.

As shown in Table 6 the majority of the calculated bias was less than 1.0 indicating an overprediction of capacity. It is interesting to note that the capacities were still overpredicted despite limiting the friction angle to 36 degrees and SPT blow counts to 60 bpf. The bias for the piles from the two Providence sites (Civic Center and I-195 Interchange) ranged from 0.27 to 0.98 for the Nordlund method and 0.50 to 1.04 for the SPT method. The bias values from the Jamestown bridge site were much lower than the Providence sites ranging from 0.11 to 0.17 for Nordlund and 0.12 to 0.29 for the SPT method. The very low bias values at the Jamestown site is consistent with the gross overpredictions in pile capacity that were made during the test pile program that led to significant design changes during construction (Richardson 2011).

The statistics of the bias were calculated for three of the sites to get an overall measure of the accuracy and precision of the static capacity methods. The results, summarized in Table 7, show that at the Providence sites the predictive accuracy was highest for the SPT method with a mean bias of 0.77 and 0.78. This was surprising considering the high uncertainty in SPT measurements particularly in silty soils that may undergo undrained or partially drained response during penetration. The accuracy of the Nordlund method was lower with a mean bias of 0.65 and 0.58.

The accuracy of the predictions at the Jamestown site were significantly lower than the Providence sites for both static capacity methods with a mean bias of 0.13 and 0.22. This suggests that current static capacity methods do not accurately predict capacity in silty outwash soils, particularly for high-displacement piles. This could explain why low-displacement piles (e.g. open-ended pipe piles) and a novel "donut pile" concept (Fronda et al. 2008) were used on the most recent Sakonnet River Bridge project. The cause of the gross overprediction of capacity in high-displacement piles in silty outwash soils is uncertain at this time but some mechanisms have been proposed including lateral stress arching and friction fatigue (Richardson 2011).

The SPT method also had a slightly higher precision than the Nordlund method. COVs of less than 0.2 typically suggest low variability (high precision) in the data. The COV ranged from 0.28 to 0.44 at the Providence sites for both methods and was 0.22 and 0.33 for the Jamestown bridge site. The lower variability at the Jamestown site was likely due to the fact that the data is from one site having consistent soil conditions.

3.5 Resistance Factor Calibration Results

The analysis in the previous section showed gross overpredictions of capacity for high-displacement piles in silty outwash soils. Given the unlikely case that high-displacement piles would be used in silty outwash soils on future projects, resistance factors were not calculated for these pile conditions. Preliminary calibration of resistance factors was performed only for high-displacement piles bearing on very dense sand or till in Providence.

All load parameters were selected from Strength Case I from the AASHTO specification with the following parameters: γ_D=1.25, λ_{QD}=1.05, COV_{QD}=0.1, γ_L=1.75, λ_{QL}=1.15, COV_{QL}=0.2. The dead to live load ratio typically ranges from 2 to 2.5. Sensitivity analysis within this range showed negligible affect on the calculated resistance factor and thus an intermediate value of 2.25 was used. The target reliability index (β_T) is related to the probability of failure and can range from 2.5 to 3.5 depending on the level of pile redundancy (Moses and Verma 1987). Consistent with AASHTO (2007) this study utilized values of 2.33 for redundant piles (i.e. pile groups) and 3.0 for non-redundant piles. These reliability indices correspond to probability of failures of 1% and 0.1%, respectively.

The resistance parameters λ_R and COV_R needed in Equation 9 were determined from the bias data in Table 6. In reliability analyses the probability of failure is controlled by the overlap of the tails of the load and resistance distributions. For the resistance, the lower tail region is most important as the upper tail region has no effect on the accuracy of the calibration (Allen et al. 2005). To ensure that the lognormal distributions represent the lower tail region, the standard normal variable (z) is plotted as a function of the bias as shown in Figures 6 and 7. As shown in the figures the data overall did not show distinct normal or lognormal trends. The data were reviewed and no justification was warranted for the removal of any of the data points. Consistent with Allen et al. (2005) the mean and COV of the bias were modified to visually fit a lognormal distribution to the tail region of the data (Figures 6 and 7). The fits were selected to be conservative (i.e. the curves were placed at or slightly to the left of the measured data). Note, however, there were only two data points available to fit the curve for the Nordlund method.

The fitted statistics were then used to calculate the resistance factors for two levels of reliability (i.e. redundant and non-redundant piles) using Equation 6.

Table 8 compares the calibrated resistance factors with the current AASHTO factors for both the Nordlund and SPT methods. The calibrated resistance factors for the Nordlund method are 0.20 for redundant piles (i.e. pile groups) and 0.14 for non-redundant piles (i.e. single piles). The resistance factors for the SPT method are 0.42 and 0.34. The calibrated resistance factors for the Nordlund method are significantly less than the current AASHTO resistance factor of 0.45. This is important considering that Nordlund is the preferred method for sands and yet the results suggest that it would be unconservative for the design of high-displacement piles bearing in dense sands/till in Providence. The resistance factor for the SPT method is higher than the current factor of 0.30 in AASHTO suggesting that more economical designs could be achieved with the calibrated resistance factor.

4. SUMMARY AND CONCLUSIONS

The objective of this project was to assess current AASHTO LRFD methods for static pile capacity analysis in Rhode Island soils. To accomplish this objective, load testing data were compiled from six major bridge projects in the state and the ultimate resistance of the test piles was interpreted using Davisson's criterion. Static capacity predictions were made for the test piles using both the Nordlund method and SPT method as specified in the AASHTO bridge specifications. Bias data were calculated to assess the accuracy and precision of the methods and to calibrate region-specific resistance factors using the First Order Second Moment (FOSM) method.

The results showed overprediction of capacity in the majority of the test piles. Capacities were grossly overpredicted at the Jamestown bridge site that was consistent with significant design changes that were made during construction. For piles driven to till at the Providence sites the SPT method had both a higher accuracy and precision as compared to the Nordlund method, which was surprising considering the uncertainty in the SPT particularly in silty soils that may exhibit drained or partially drained conditions during penetration. Based on the analysis of 10 high-displacement piles driven to till in Providence the calibrated resistance factors for the Nordlund method were 0.20 for redundant piles (i.e. pile groups) and 0.14 for non-redundant piles (i.e. single piles). The resistance factors for the SPT method were 0.42 and 0.34, respectively. The calibrated resistance factors for the Nordlund method were lower than the current AASHTO resistance factor of 0.45 but higher than the current factor of 0.30 for the SPT method.

Future research should include expanding the static loading test database in Rhode Island, possibly by gathering data from local consulting firms. The database is a work in progress and is critical for improving reliability-based design methods. It is also essential that if pile load tests are performed in the future, that they be tested to failure not just to the design load so that the true ultimate resistance can be determined. Also, more rigorous methods such as the Monte Carlo method should be employed to provide more accurate and likely less conservative calculations of resistance factors.

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Table 1. Static capacity methods and associated resistance factors for driven piles (AASHTO 2007).

Condition/Resistance Determination Method		Resistance Factor
Nominal Resistance of Single Pile in Axial Compression—Static Analysis Methods, φ_{stat}	Skin Friction and End Bearing: Clay and Mixed Soils	
	α-method (*Tomlinson, 1987*; *Skempton, 1951*)	0.35
	β-method (*Esrig & Kirby, 1979*; *Skempton, 1951*)	0.25
	λ-method (*Vijayvergiya & Focht, 1972*; *Skempton, 1951*)	0.40
	Skin Friction and End Bearing: Sand	
	Nordlund/Thurman Method (*Hannigan et al., 2005*)	0.45
	SPT-method (Meyerhof)	0.30
	CPT-method (Schmertmann)	0.50
	End bearing in rock (*Canadian Geotech. Society, 1985*)	0.45
Block Failure, φ_{b1}	Clay	0.60
Uplift Resistance of Single Piles, φ_{up}	Nordlund Method	0.35
	α-method	0.25
	β-method	0.20
	λ-method	0.30
	SPT-method	0.25
	CPT-method	0.40
	Load test	0.60
Group Uplift Resistance, φ_{ug}	Sand and clay	0.50
Horizontal Geotechnical Resistance of Single Pile or Pile Group	All soils and rock	1.0
Structural Limit State	Steel piles	See the provisions of Article 6.5.4.2
	Concrete piles	See the provisions of Article 5.5.4.2.1
	Timber piles	See the provisions of Article 8.5.2.2 and 8.5.2.3
Pile Drivability Analysis, φ_{da}	Steel piles	See the provisions of Article 6.5.4.2
	Concrete piles	See the provisions of Article 5.5.4.2.1
	Timber piles	See the provisions of Article 8.5.2.2
	In all three Articles identified above, use φ identified as "resistance during pile driving"	

Table 2. Summary of projects and test piles compiled in the static load test database.

Site	Location	Pile Types	Number of Piles	Reference
Capital Center	Providence	Square PPC	3	Maguire (1983)
Civic Center Interchange (I)	Providence	Square PPC, H	5	Maguire (1986a)
Civic Center Interchange (II)	Providence	CP	5	Maguire (1986b)
Providence Place Mall	Providence	Square PPC, H	4	Maguire (1998)
I-195 Interchange	Providence	Square PPC, H, CP	8	GTR (2004)
Jamestown Bridge	Narragansett Bay	Square PPC, H, Composite	8	Sverdrup & Parcel (1982)
Sakonnet River Bridge	Narragansett Bay	H, OP	7	Haley & Aldrich (2008)

Notes: PPC = Precast Prestressed Concrete, CP = Closed-ended Pipe, OP = Open-ended Pipe, H=Steel H-pile

Table 3. Correlation used to determine soil unit weights in this study.

SPT-N (blows/0.3 m)	γ_{sat} (kN/m^3)
Sands	
0-2	15.7
3-4	15.7
4-10	16.5
10-20	17.3
20-30	18.1
30-40	18.9
>40	19.6
Clays	
0-2	16.5
2-4	17.3
4-8	18.1
8-15	18.9
15-30	19.6
>30	19.6

Table 4. Summary of test piles utilized in this study and their ultimate resistances interpreted using Davisson's criterion.

Project	Pile	Type	Length (m)	Bearing Soil	Ultimate Resistance (kN)
Civic Center Interchange	B-2 (pier 1)	14" PPC	23.2	Till	2,580
	A-4	14" PPC	32.9	Till	2,162
	B-1	13.38" CP	32.3	Till	2,491
	A-3	13.38" CP	37.2	Very Dense Outwash	2,313
	B-6	13.38" CP	28.0	Till	2,002
	A-5	9.63" CP	29.0	Till	1,334
I-195 Interchange	PPC#2 (Area1)	14" PPC	28.0	Till	2,135
	PP#2 (Area1)	14" CP	26.5	Till	2,224
	PPC#2 (Area2)	14" PPC	34.8	Till	1,957
	PP#1 (Area2)	14" CP	35.4	Till	2,491
Jamestown Bridge	TTP-1	24" PPC	31.1	Outwash	1,601
	TTP-4	24" PPC	54.3	Till	4,626
	WATP-1	20" PPC	23.8	Outwash	738
	WATP-2	20" PPC	35.4	Outwash	2,135

Table 5. Summary of static capacity predictions for test piles. The ultimate load interpreted from the static loading tests are also shown for reference.

Location	Pile	Ultimate Resistance (kN)		
		Static Loading Test	Nordlund	SPT
Civic Center Interchange	B-2 (pier 1)	2,580	4,075	3,318
	A-4	2,162	6,352	2,358
	B-1	2,491	3,514	2,393
	A-3	2,313	4,288	2,664
	B-6	2,002	2,958	3,995
	A-5	1,334	1,361	2,478
I-195 Interchange	PPC#2 (Area1)	2,135	4,128	3,407
	PP#2 (Area1)	2,224	2,522	2,402
	PPC#2 (Area2)	1,957	7,197	3,407
	PP#1 (Area2)	2,491	3,754	2,527
Jamestown Bridge	TTP-1	1,601	11,307	5,556
	TTP-4	4,626	42,712	19,577
	WATP-1	738	6,815	6,205
	WATP-2	2,135	12,811	9,648

Table 6. Summary of bias data.

Location	Pile	Nordlund	SPT
Civic Center Interchange	B-2 (pier 1)	0.63	0.78
	A-4	0.34	0.92
	B-1	0.71	1.04
	A-3	0.54	0.87
	B-6	0.68	0.50
	A-5	0.98	0.54
I-195 Interchange	PPC#2 (Area1)	0.52	0.63
	PP#2 (Area1)	0.88	0.93
	PPC#2 (Area2)	0.27	0.57
	PP#1 (Area2)	0.66	0.99
Jamestown Bridge	TTP-1	0.14	0.29
	TTP-4	0.11	0.24
	WATP-1	0.11	0.12
	WATP-2	0.17	0.22

Table 7. Summary of bias statistics by site.

Location	Statistical Parameter	Nordlund	SPT
Civic Center Interchange	Mean	0.65	0.77
	COV	0.33	0.28
I-195 Interchange	Mean	0.58	0.78
	COV	0.44	0.27
Jamestown Bridge	Mean	0.13	0.22
	COV	0.22	0.33

Table 8. Comparison of resistance factors for piles bearing in till in Providence.

Method	Calibrated (Redundant piles)	Calibrated (Non-redundant piles)	AASHTO
Nordlund	0.20	0.14	0.45
SPT	0.42	0.34	0.30

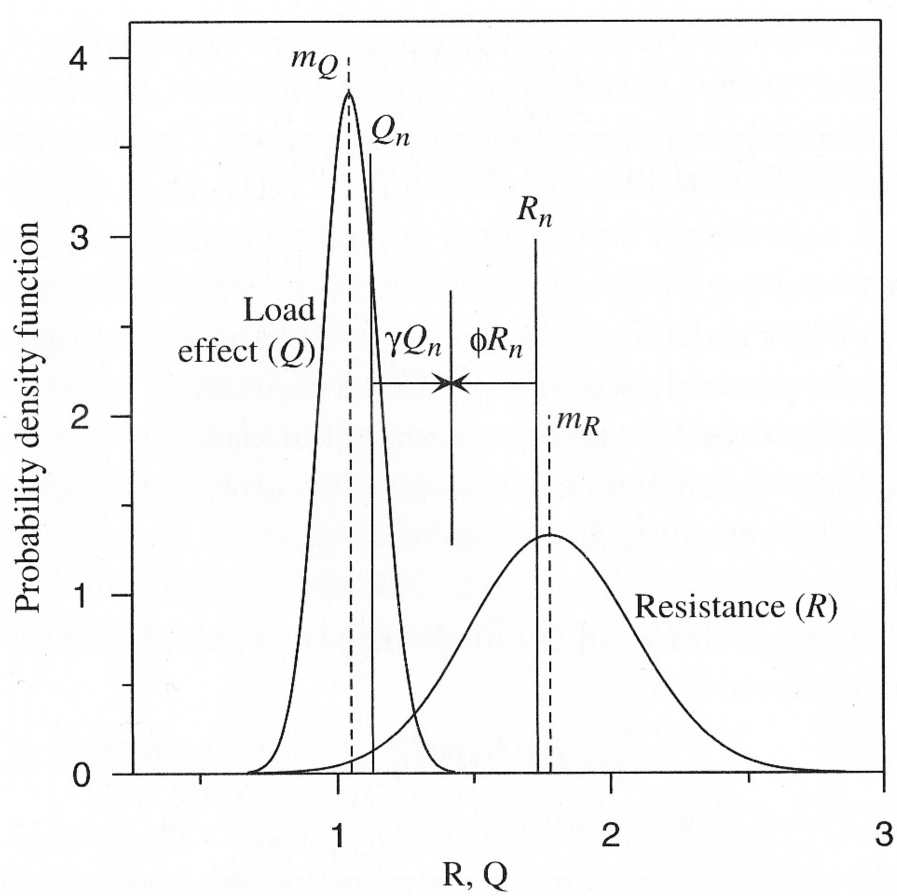

Figure 1. Histogram illustrating the reliability design concept (Paikowsky et al. 2010).

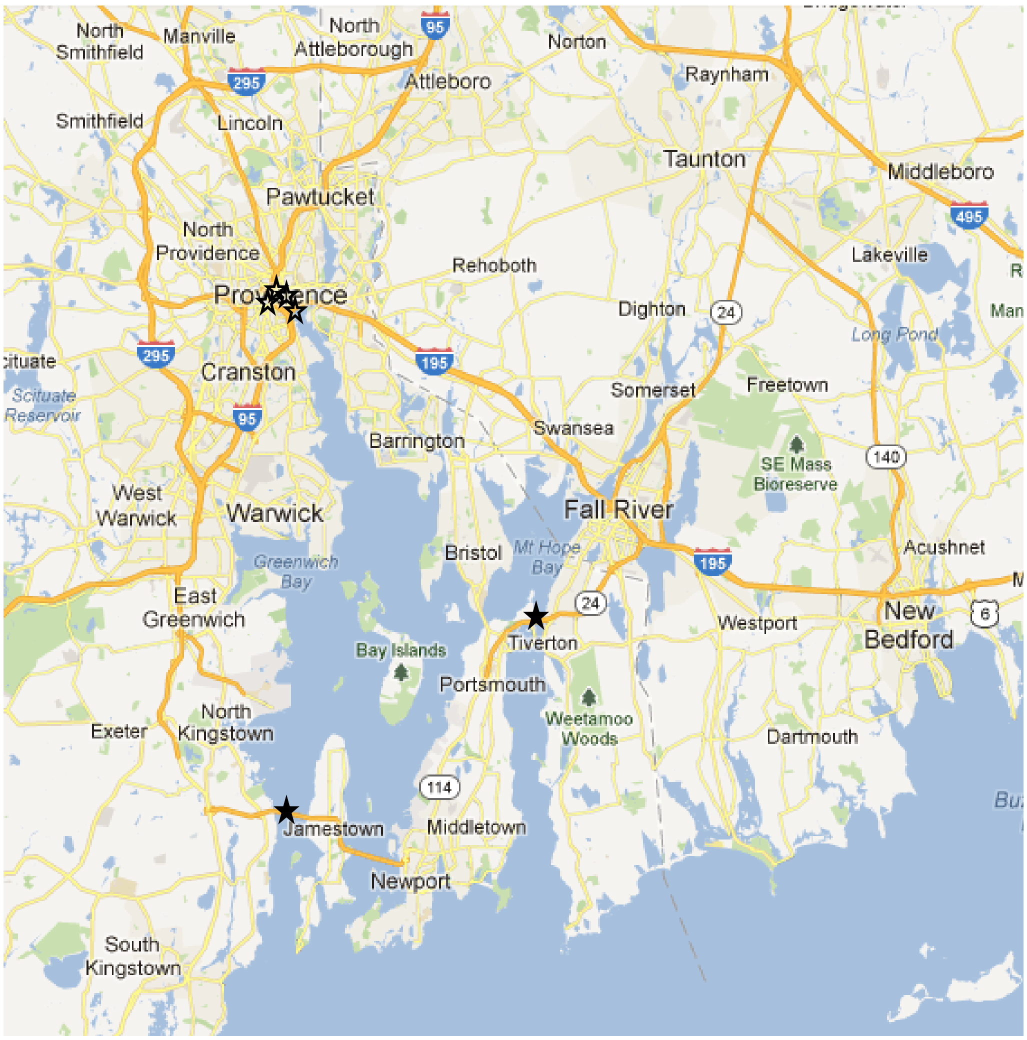

Figure 2. Locations of the project sites utilized in this study (basemap obtained from Google Earth).

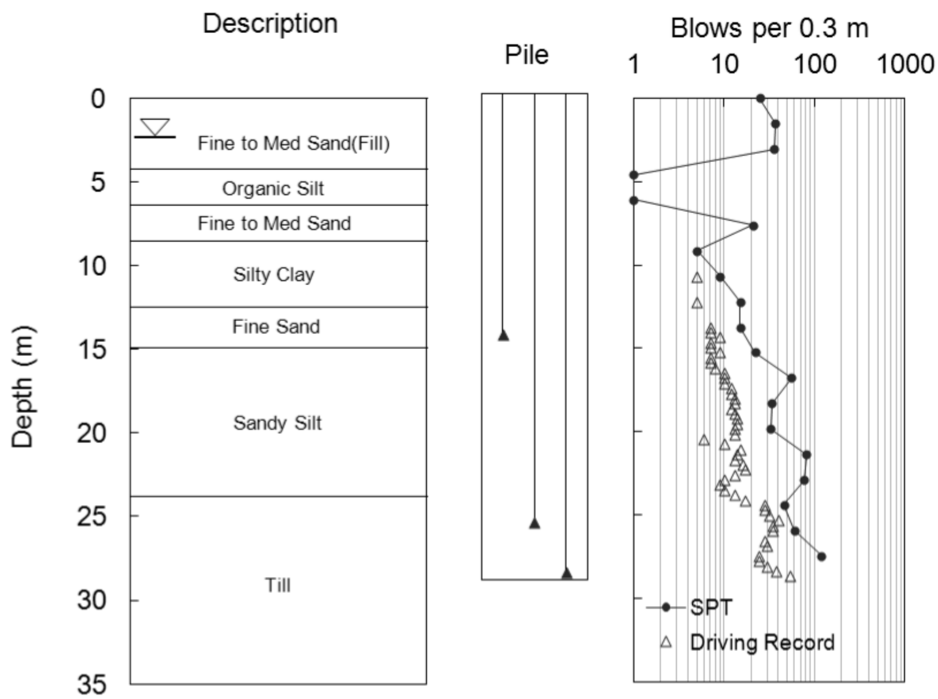

Figure 3. Typical subsurface profile representing the Providence sites. Test pile A3 (Civic Center Interchange) is shown. Triangles on the pile indicate telltale locations (Davis 2012).

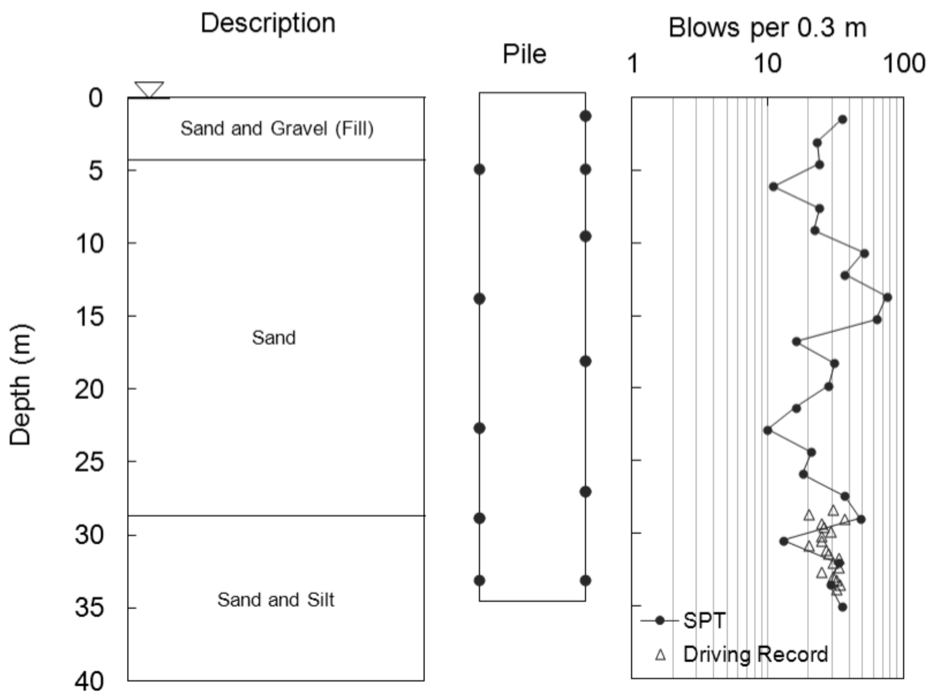

Figure 4. Typical subsurface profile representing Narragansett Bay sites. Test pile HA-HP (Sakonnet River Bridge) is shown. Solid dots on the pile indicate locations of functional strain gages (Davis 2012).

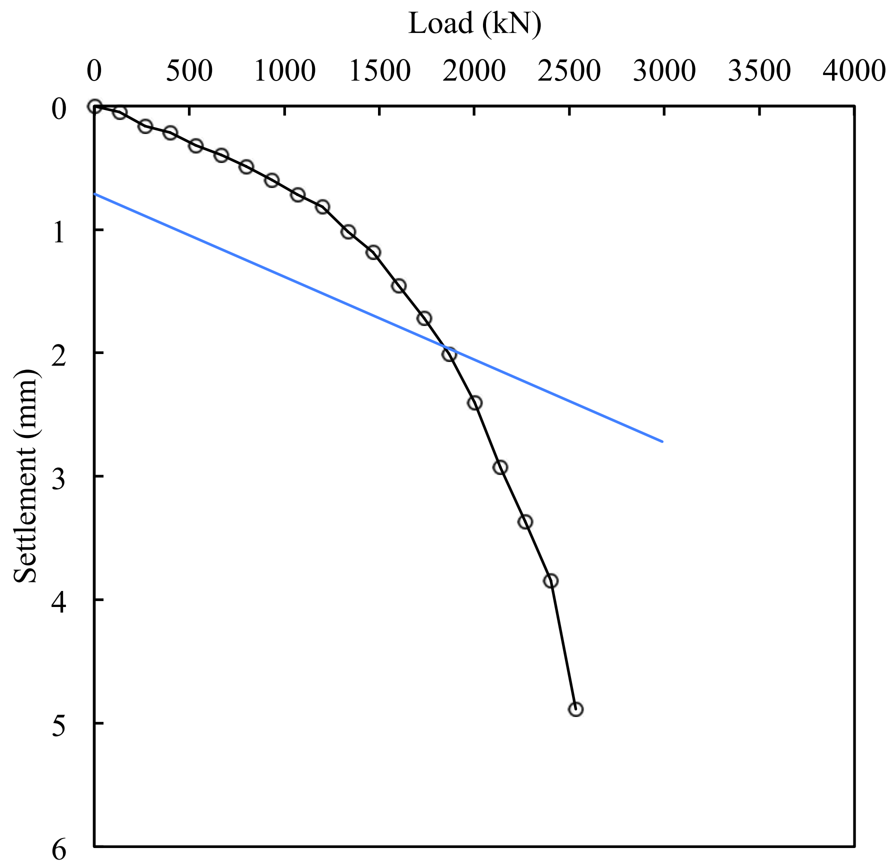

Figure 5. Typical load-movement curve (HA-HP Sakonnet River Bridge). Davisson's offset line is also shown.

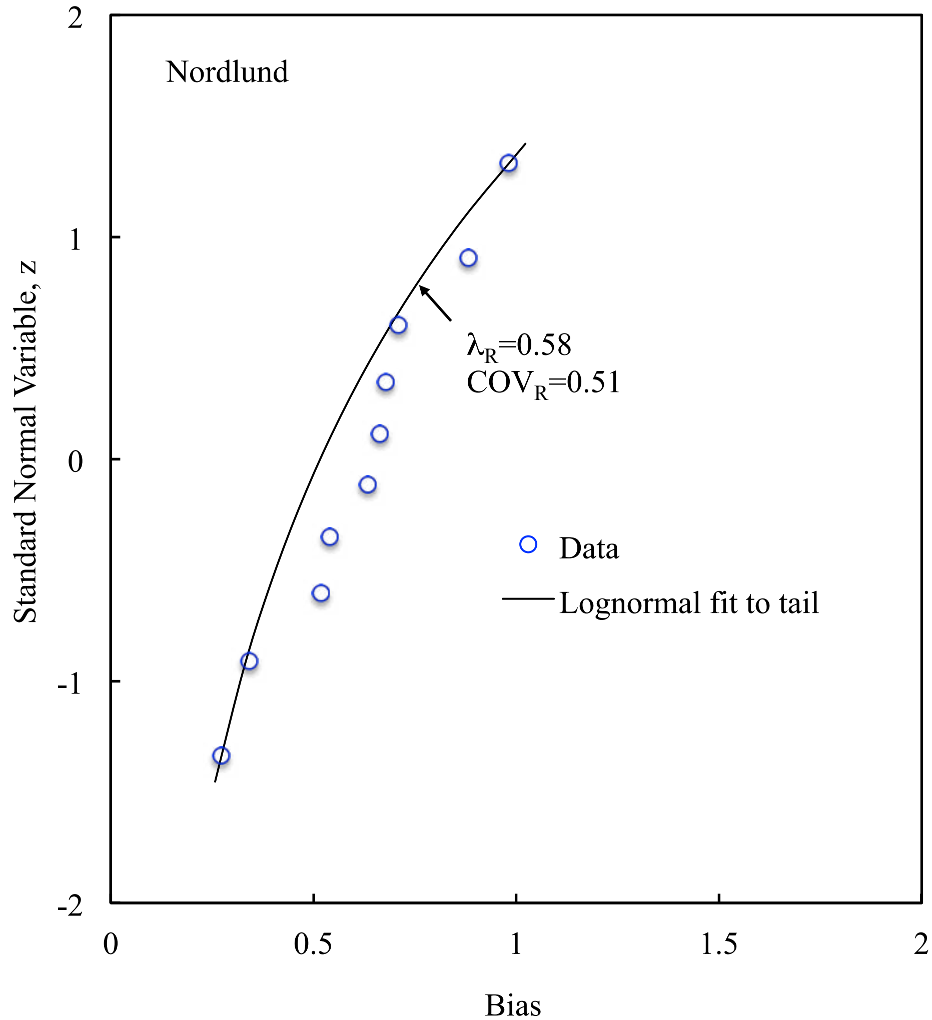

Figure 6. Standard normal variable plotted as a function of bias for the Nordlund method. The lognormal distribution shown was conservatively fit to the lower tail of the bias data.

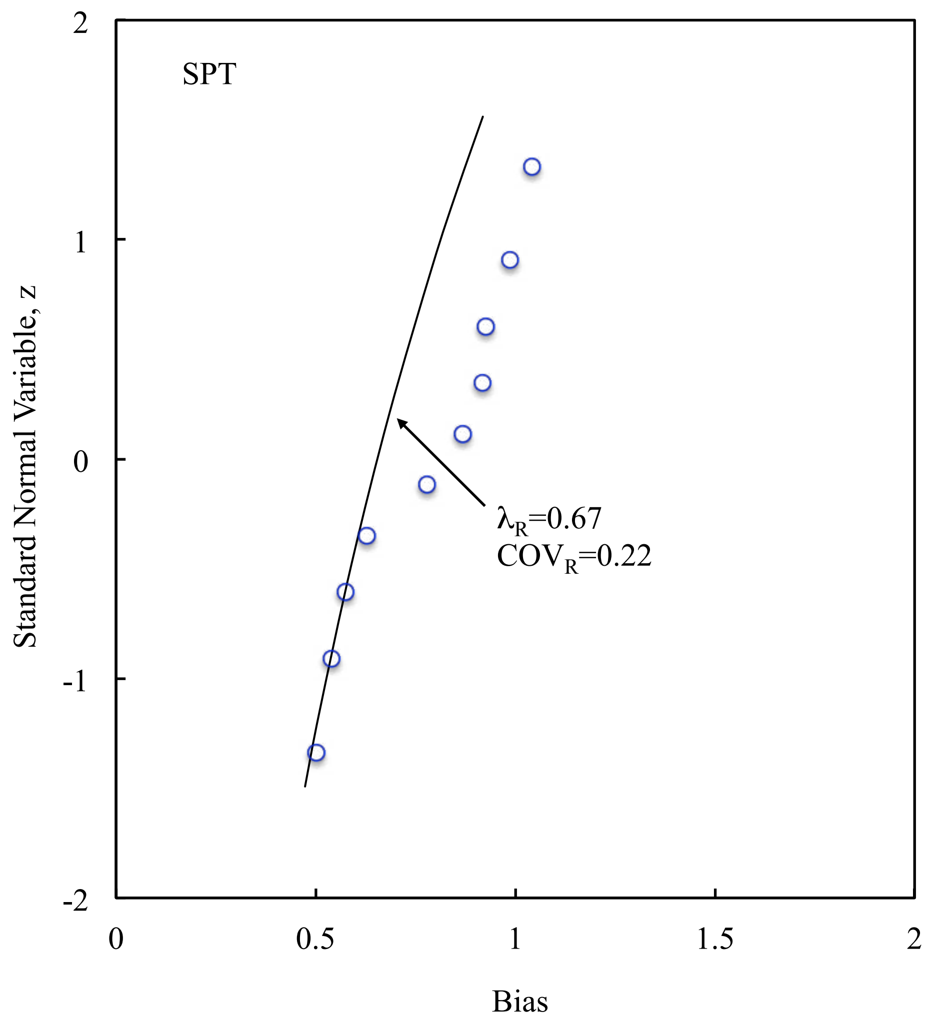

Figure 7. Standard normal variable plotted as a function of bias for the SPT method. The lognormal distribution shown was conservatively fit to the lower tail of the bias data.