TRC1204

LRFD Site Specific Variability in Laboratory and Field Measurements and Correlations

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Final Report

2015

RESEARCH

LRFD Site Specific Variability in Laboratory and Field Measurement Correlations

PROJECT OBJECTIVES

The research project included 1) determining the variance in reliability between the current site characterization techniques and state-of-the-art site characterization techniques including full-scale load testing and 2) developing resistance factors as associated with the geotechnical engineering properties of the soil and soil-structure interaction.

SCOPE

The research program consisted of performing nine full-scale load tests on Drilled Shaft foundations within the State of Arkansas. The tests were performed at the testing sites at Siloam Springs (SSATS), Turrell (TATS), and Monticello (MATS). The majority of the research focused on the comparison between predicated and measured axial capacity measurements for drilled shaft foundations.

FINDINGS

The major findings of the study are summarized below.

1. It is recommended to utilize the MODOT and/or the UofA geotechnical investigation method to 1) more accurately determine the soil property values at project sites and 2) more efficiently design drilled shaft foundations (DSF). Specifically, the use of the cone penetration test is recommended as a fast and efficient method to determine soil property values, but should not be utilized in dense to very dense sandy soils.
2. The FB-Deep software program is recommended to design DSF in alluvial and deltaic soils within the state of Arkansas
3. Embedment lengths of production DSF within moderately hard to hard limestone in Northwest Arkansas can be less than 10 feet. An rock embedment length of 4 feet for a 4 foot diameter DSF was determined to be adequate at the SSATS.
4. A Bayesian updating/Monte Carlo design methodology and resistance factor values were determined for the design of total resistance, unit side resistance, and unit side resistance of DSF in alluvial and deltaic soils in the state of Arkansas for the strength/service limit states (5%D, 1%D, and 1.27cm) as included in Race (2015).
5. Resistance factor values for total resistance design of DSF in alluvial and deltaic soils in Arkansas were determined to be greater, on average, (ranging from 0.57 to 0.80) than the national recommended value of 0.58 for the strength limit state.
6. The use of DSF as bridge foundations in the state of Arkansas is 32 percent more cost efficient as compared to the use of driven piles in terms of cost per ton of resistance in rock.
7. Significant cost savings may be obtained by modifying the geotechnical investigation methods. The cost per ton of resistance as obtained using the UofA and AHTD geotechnical investigation methods were $24.11 and $82.70 at the SSATS and $75.47 and $141.57 at the TATS, respectively.
8. It is recommended that every DSF be proof-tested, to at least the design load, until a large database is created to 1) avoid major construction problems and 2) increase the load test database of DSF in the state of Arkansas.

FINAL REPORT: AHTD TRC1204

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Abstract

Geotechnical investigation methods and design software programs were examined to determine the "best" method/program to design drilled shaft foundations (DSF) in the state of Arkansas. The University of Arkansas method, consisting of a California split spoon sampler in cohesionless soils and unconsolidated undrained triaxial compression tests in cohesive soils, is recommended for all geotechnical investigations conducted within the state of Arkansas. The cone penetration test, referred to as the Missouri Department of Transportation (MODOT) method, is also recommended for use in cohesive soils and loose to medium dense cohesionless soils commonly found in Eastern Arkansas. Similarly, it is recommended that the FB-Deep software program be utilized when designing DSF in alluvial and deltaic soils within Arkansas.

Currently, the Arkansas State Highway and Transportation Department (AHTD) does not employ DSF for bridge foundations in mixed soils (clay, sand, and or clay/sand interbedded). Driven piles are the most common type of foundation system used in Arkansas to resist the axial and lateral loads associated with bridge foundations; however, driven piles are not the most efficient foundation type in terms of cost per ton of resistance. By constructing and testing three DSF at three different test sites (rock, alluvial deposits, and deltaic deposits within the state of Arkansas), it was determined that the use of DSF could save up to $262,800 per site, provide additional lateral resistance, and provide designers with additional predicted versus measured information.

Resistance factor values were calibrated for the design of DSF in alluvial and deltaic soil deposits within the state of Arkansas for the total resistance, unit side resistance, and unit end bearing resistance for the strength/service limit states (5%D, 1%D, and 0.5in.). Consequently, cost savings for the design of DSF using total resistance for the strength limit state may be up to $463,800 (29.7 percent of the total project cost) for a hypothetical project that includes 24 DSF instead of piling, depending on: the site, the utilized geotechnical investigation method, and the design software program that was utilized. The recommended geotechnical investigation method and the design software program for DSF constructed within soil deposits within the state of Arkansas are the UofA method and the FB-Deep program, respectively.

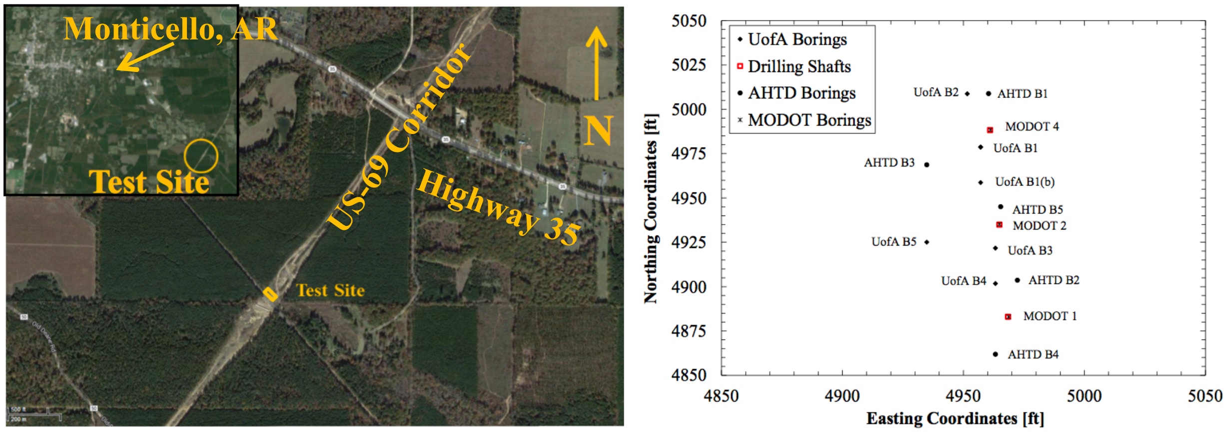
Introduction

Detailed geotechnical investigations and full-scale load testing were performed on DSF at three sites across the state of Arkansas. The purpose for the geotechnical investigations and full-scale load tests, as associated with the TRC-1204 project, was to develop resistance factors. These resistance factors will be utilized by bridge designers, within the AHTD, to enable Load and Resistance Factor Design of deep foundations (DSF) within the state of Arkansas. The methods and materials that were utilized to complete the scope of work for the TRC-1204 project, including: the geotechnical investigation methods, the DSF construction, the resistance factor calibration, and the cost analyses are discussed herein. Moreover, the results that were obtained are also presented and discussed. These results include 1) bias factors that were developed by dividing the measured resistance (as obtained from full-scale bi-directional load cell [BLC] tests and cross-hole sonic logging [CSL] tests) by the predicted resistance (as obtained from computer programs that determined the resistance by utilizing the properties that were obtained from the detailed geotechnical investigation) and 2) resistance factors that were obtained by utilizing the Bayesian updating technique along with Monte Carlo simulations. Construction concerns that were encountered during the TRC-1204 project are presented, recommendations for the implementation of DSF within the state of Arkansas are discussed, and cost savings associated with the utilization of DSF are detailed.

Methods and Materials

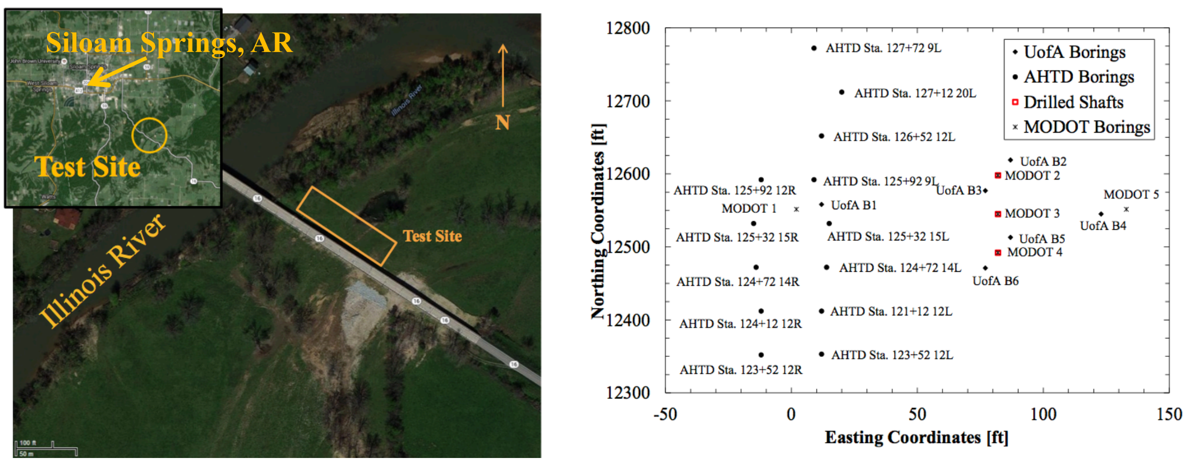
Geotechnical Investigation Method

Within the state of Arkansas, detailed geotechnical investigations were performed at the three test sites. 1) The Monticello Arkansas Test Site (MATS), is located in the southeastern portion of Arkansas and is comprised of deltaic soil deposits (Figure 1a). 2) The Siloam Springs Arkansas Test Site (SSATS) is located in the northwest portion of Arkansas and is comprised of cherty soil underlain by limestone and shale rock (Figure 2a). 3) The Turrell Arkansas Test Site (TATS) is located within the Mississippi Embayment in the northeastern portion of Arkansas, within the New Madrid Seismic Zone, and is comprised of alluvial soil deposits (Figure 3a). The site investigations that were performed at the MATS and TATS included drilling traditional boreholes (10 at the MATS, 19 at the SSATS, and 12 at the TATS) and five attempted cone penetration test (CPT) soundings within a 929m^2 testing area. The borehole layout at the MATS, SSATS, and TATS are presented in Figures 1b, 2b, and 3b, respectively.



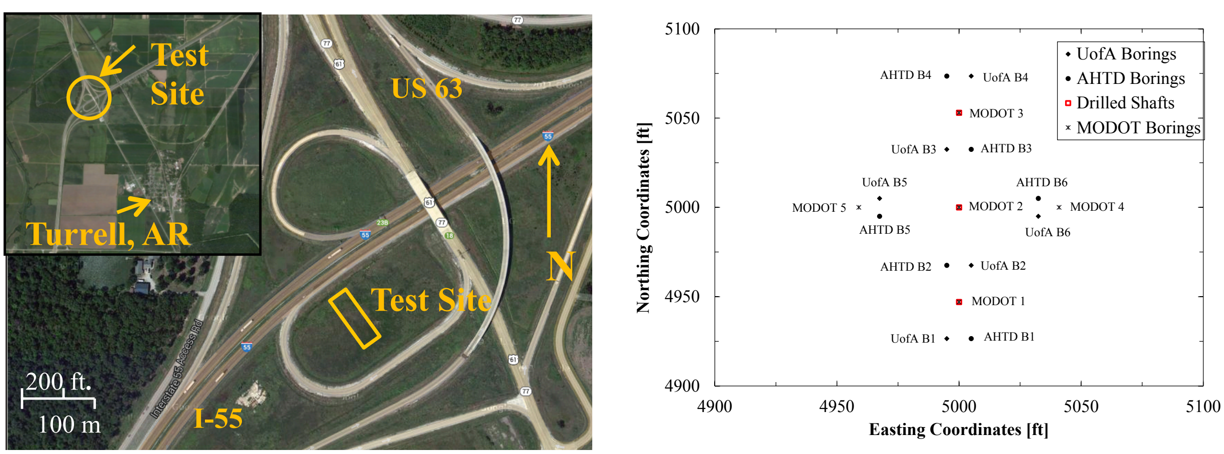
(a) (b)

Figure 1. a) Relative location of the MATS and b) geotechnical investigation borehole and DSF locations at the MATS.



(a) (b)

Figure 2. a) Relative location of the SSATS and b) geotechnical investigation borehole and DSF locations at the SSATS.



(a) (b)

Figure 3. a) Relative location of the TATS and b) geotechnical investigation borehole and DSF locations at the TATS.

The AHTD geotechnical investigation method included the use of SPT (ASTM D1586 2012), that utilized a standard split-spoon sampler (1.18 inch diameter) at five-foot intervals, in all types of soil. In rock, the rock quality designation (RQD) and recovery values were obtained for the AHTD method. The UofA geotechnical investigation method included the use of 1) the SPT that utilized a California sampler (2.4 inch diameter), in cohesionless soils, 2) the Osterberg hydraulic fixed-piston Shelby tube sampler in soft to firm clay, 3) the Pitcher barrel Shelby tube sampling in stiff to hard clay, and 4) unconfined compressive testing of rock samples. For one of the UofA boreholes at each test site, samples were continuously retrieved, but for the other UofA boreholes, samples were taken at every five-foot intervals. The MODOT sampling method included the use of a 100-kN capacity five-channel (tip resistance, sleeve friction, pore pressure, seismic, tilt) cone following ASTM D3441 (2012) testing procedures, in all soils.

The methods for determining the soil properties, based on the soil sampling and testing methods, are presented in Table 1. For example, the blow count values were obtained by following the procedures outlined in ASTM D1586 (2012) for the AHTD and UofA methods, or were calculated from the CPT measurements (MODOT method) by using Equation 1. The total unit weight and undrained shear strength values for cohesive soils and the total unit weight and friction angle values for non-cohesive soils were correlated from Vanikar (1986) for the AHTD method. The undrained shear strength and total unit weight values, as obtained from CPT measurements (for the MODOT method), were calculated using Equations 2 and 3, respectively. The undrained shear strength values (UofA method) were directly obtained from unconsolidated undrained (UU) triaxial compression tests, as performed following the procedures outlined in ASTM D2850 (2012). The total unit weight values for the Uof A method were calculated from mass and volume measurements collected for trimmed sample of extruded soil sections that were obtained from Shelby tubes (clay) or from mass and volume measurements obtained from 15.24cm long soil sections that were recovered from the California split spoon sampler (sand).

Table 1. Soil property determination method for various soil sampling and testing methods.

Soil Property	Soil Sampling Method		
	AHTD	**MODOT**	**UofA**
Corrected Blow Count	Calculated[1]	Calculated[2]	Calculated[3]
Undrained Shear Strength	Correlated[4]	Calculated[5]	Measured[6]
Total Unit Weight	Correlated[4]	Calculated[7]	Measured[8]

[1]Corrected for hammer efficiency
[2]Equation 1 (originally in Robertson and Cabal [2012])
[3]Empirical equation from Race and Coffman (2013)
[4]Vanikar (1986)
[5]Equation 2 (originally in Robertson and Cabal [2012])
[6]UU test
[7]Equation 3 (originally in Robertson and Cabal [2012])
[8]Based on diameter, length, and weight measurements

$$N_{60} = \frac{\left(\frac{q_t}{p_a}\right)}{8.5\left(1-\frac{I_c}{4.6}\right)}$$ (Robertson and Cabal 2012) Equation 1

$$c_u(ksf) = \frac{q_t - \sigma_v}{N_{kt}}$$ (Robertson and Cabal 2012) Equation 2

$$\frac{\gamma_t}{\gamma_w} = 0.27\left[\log\left(R_t\right) + 0.36 \cdot \log\left(\frac{q_t}{p_a}\right)\right] + 1.236$$ (Robertson and Cabal 2012) Equation 3

Within Equations 1 through 3, N_{60} is the energy corrected blow count, q_t is the tip resistance, p_a is the atmospheric pressure, I_c is the soil behavior type index, c_u is the undrained shear strength, σ_v is the vertical overburden pressure, N_{kt} is a cone factor value (14 for this study), γ_t is the total unit weight, γ_w is the unit weight of water, and R_t is the friction ratio.

Design Software Program/Equations

FB-Deep and SHAFT are two commercially available programs that are used to predict the axial capacity and the load-movement response of DSF. The methods listed in the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance

Factor Design (LRFD) Bridge Design Specifications (AASHTO 2007) and in the Federal Highway Administration (FHWA) report FHWA-NHI-10-016 (Brown et al. 2010) are utilized within the FB-Deep and SHAFT programs. Specifically, FB-Deep (2012) is a software program developed by the Bridge Software Institute at the University of Florida while the SHAFT (2012) program was commercially released in 1987, by ENSOFT, Inc., under the direction of Dr. Lymon C. Reese. Soil parameters such as soil type, blow count (N), total unit weight (γ_T), and undrained shear strength (c_u), as obtained or correlated from SPT or CPT data, may be utilized within the FB-Deep program to predict the static axial capacity and load-movement response. Likewise, the amount of axial movement, quantity of load, and the distribution of load along the DSF are predicted using FB-Deep and SHAFT. Additionally, LRFD reduction factors for side friction and tip resistance in each soil layer may be specified in the programs for each geostrata layer. For completeness, the design steps and methodology that were utilized to perform the aforementioned analyses for the Arkansas sites, by using FB-Deep and SHAFT, are further described in detail in Bey (2014).

DSF Construction

At each test site, three DSF were constructed based on the designed length presented in Table 2. The DSF at the SSATS were drilled by Aldrich Construction and poured by GCC Midcontinent Concrete Company in July 2013, and were tested by GEI Consultants, Inc. (CSL) and Loadtest, Inc. (BLC) in September 2013. The DSF at the TATS were drilled by McKinney Drilling Company and poured by Razorback Concrete from October to December 2013, and were tested by GEI Consultants, Inc. (CSL) and Loadtest, Inc. (BLC) in January 2014. The DSF at the MATS were drilled by McKinney Drilling Company and poured by Select Concrete Company in September to October 2014, and were tested by GEI Consultants, Inc. (CSL) and

Loadtest, Inc. (BLC) in October 2014. Further discussion of the construction of the DSF for the SSATS and the TATS is presented in Bey (2014) and for the MATS is presented in Race (2015).

Table 2. Designed and as-constructed values for the DSF at the test sites.

Test Site	DSF Designation	Design Capacity (Ton)	Design Parameters Diameter (ft)	Design Parameters Length (ft)	As-Constructed Parameters Diameter (ft)	As-Constructed Parameters Length (ft)
MATS	North 4ft		4	91.5	4.3	91.5
	Center 6ft	1303.7	6	72	6.2	72
	South 4ft		4	91.5	4.5[a]	91.5
SSATS	East 4ft		4	26	4	23
	Center 6ft	1112.5	6	26	6	21
	West 4ft		4	26	4	26
TATS	North 4ft		4	86.5	4.1	87
	Center 6ft	986.6	6	62	6.1	62
	South 4ft		4	86.5	4.2[b]	86.5

[a]Diameter value estimated from the concrete pour volumes
[b]Partial collapse in the side wall from 20 to 30 feet below the ground surface (no sonicaliper® after collapse)

Resistance Factor Calibration

The calibration of resistance factor values for the alluvial and deltaic deposits within the state of Arkansas was performed using the data from the aforementioned BLC tests that were completed on the DSF at the MATS and the TATS. A load test database was created for the total resistance from the BLC test data from the MATS and the TATS (Table 3). The predicted resistance of the DSF was calculated at movement values of five percent of the diameter of the respective DSF. The measured resistance values were interpolated to a movement value of five percent of the diameter by using the equivalent top-down load-movement curve. Furthermore, the mean and variance of the bias factor values were calculated for each combination of geotechnical investigation method and design software program.

Table 3. Summary of DSF load test database for DSF constructed in Arkansas (strength limit state for total resistance).

Location	Dia. (m)	Length (m)	Software Program	Geotechnical Investigation Method	Measured Resistance [MN] +	Predicted Resistance [MN] *	Bias Factor
MATS	1.33	27.89	SHAFT	AHTD	13.7	13.3	1.03
MATS	1.33	27.89	SHAFT	MODOT	13.7	16.2	0.85
MATS	1.33	27.89	SHAFT	UofA	13.7	14.3	0.96
MATS	1.33	27.89	FB-Deep	AHTD	13.7	13.9	0.99
MATS	1.33	27.89	FB-Deep	MODOT	13.7	14.9	0.92
MATS	1.33	27.89	FB-Deep	UofA	13.7	13.1	1.04
MATS	1.89	21.95	SHAFT	AHTD	17.7	14.9	1.19
MATS	1.89	21.95	SHAFT	MODOT	17.7	18.9	0.94
MATS	1.89	21.95	SHAFT	UofA	17.7	15.7	1.13
MATS	1.89	21.95	FB-Deep	AHTD	17.7	14.9	1.19
MATS	1.89	21.95	FB-Deep	MODOT	17.7	17.9	0.99
MATS	1.89	21.95	FB-Deep	UofA	17.7	15.0	1.19
MATS	1.37	27.89	SHAFT	AHTD	13.6	13.7	1.00
MATS	1.37	27.89	SHAFT	MODOT	13.6	16.4	0.83
MATS	1.37	27.89	SHAFT	UofA	13.6	14.4	0.95
MATS	1.37	27.89	FB-Deep	AHTD	13.6	14.3	0.96
MATS	1.37	27.89	FB-Deep	MODOT	13.6	15.3	0.89
MATS	1.37	27.89	FB-Deep	UofA	13.6	13.5	1.01
TATS	1.22	26.21	SHAFT	AHTD	11.0	6.9	1.60
TATS	1.22	26.21	SHAFT	MODOT	11.0	8.1	1.35
TATS	1.22	26.21	SHAFT	UofA	11.0	9.2	1.20
TATS	1.22	26.21	FB-Deep	AHTD	11.0	5.9	1.87
TATS	1.22	26.21	FB-Deep	MODOT	11.0	8.6	1.28
TATS	1.22	26.21	FB-Deep	UofA	11.0	8.6	1.28
TATS	1.83	18.89	SHAFT	AHTD	11.8	7.1	1.66
TATS	1.83	18.89	SHAFT	MODOT	11.8	9.4	1.26
TATS	1.83	18.89	SHAFT	UofA	11.8	9.2	1.27
TATS	1.83	18.89	FB-Deep	AHTD	11.8	6.3	1.86
TATS	1.83	18.89	FB-Deep	MODOT	11.8	8.5	1.39
TATS	1.83	18.89	FB-Deep	UofA	11.8	8.9	1.32
TATS	1.22	26.52	SHAFT	AHTD	8.7	7.3	1.21
TATS	1.22	26.52	SHAFT	MODOT	8.7	8.6	1.02
TATS	1.22	26.52	SHAFT	UofA	8.7	9.6	0.91
TATS	1.22	26.52	FB-Deep	AHTD	8.7	5.9	1.48
TATS	1.22	26.52	FB-Deep	MODOT	8.7	8.6	1.02
TATS	1.22	26.52	FB-Deep	UofA	8.7	8.7	1.01

+Interpolated to 5%D Displacement

*Predicted at 5%D Displacement

To calibrate the resistance factor values, the amounts of uncertainty within the software programs and within the geotechnical investigation methods were accounted for by using a bias factor for the resistance (λ_S); as presented in Equation 4 within Figure 4, the λ_S value was defined as the ratio between the measured resistance value (R_m) and the predicted resistance value (R_p). The Bayesian updating method was utilized to determine the values of an updated mean and updated variance in relation to prior distributions. Specifically, Equations 5 and 6, as presented within Figure 4, were utilized to determine the posterior mean and posterior variance values for the alluvial and deltaic deposits in Arkansas.

Prior distribution parameters from Paikowsky (2004) or Abu-Farsakh et al. (2010) were utilized within the Bayesian updating procedure. The national database from Paikowksy (2004) included 44 DSF that were designed by using the design procedures discussed in Brown et al. (2010) and geotechnical investigation data from SPT, CPT, and undrained shear strength tests in cohesionless soils, cohesive soils, and mixed cohesionless and cohesive soils. Specifically, the national database, from Paikowsky (2004), was utilized as a prior distribution because the national data encompassed a variety of 1) soil types, 2) geotechnical investigation methods, and 3) design procedures from sites across the United States of America. The regional database from Abu-Farsakh et al. (2010) was based on 22 DSF that were designed by utilizing the SHAFT program and then tested with a BLC. The soil, in which each DSF was constructed, consisted primarily of interbedded cohesionless and cohesive soils; the soil was sampled utilizing blow count values from SPT in cohesionless soils and undrained shear strength values in cohesive soils. The distribution parameters that were proposed in Abu-Farsakh et al. (2010) were utilized in this study because the soil types within the Abu-Farsakh et al. (2010) database were within close proximity to the Arkansas sites and included deposits to the soil that were comparable to the deposits at the test sites in Arkansas.

The limit state equation that was utilized to calibrate the resistance factor values for DSF is presented as Equation 7 within Figure 4. The limit state [g(x)] is utilized to determine the resistance factor (ϕ) by incorporating the live load value (Q_{LL}), the dead load value (Q_{DL}), the average bias factor for 1) the dead load (λ_{DL}), 2) the live load (λ_{LL}), and 3) the resistance (λ_R), and the load factors associated with the dead load and with the live load (γ_{DL} and γ_{LL}, respectively). The bias factors associated with various load types, load factors, and coefficient of variation values (COV) were recommended in AASHTO (2007) to calibrate resistance factors (Table 4). For completeness, the steps utilized during the implementation of the Bayesian updating and the Monte Carlo simulation techniques are presented in Figure 4. For each geotechnical investigation method and each design software program, a resistance factor was determined for a reliability index (β) of 3.0.

Table 4. Loading factors as recommended from AASHTO (2007).

Load Type	**Load Factor (γ)**	**Mean Bias Factor (λ)**	**Standard Deviation (σ)**	**COV** $\left(\frac{\sigma}{\lambda}\right)$
Dead Load	1.25	1.08	0.14	0.13
Live Load	1.75	1.15	0.21	0.18

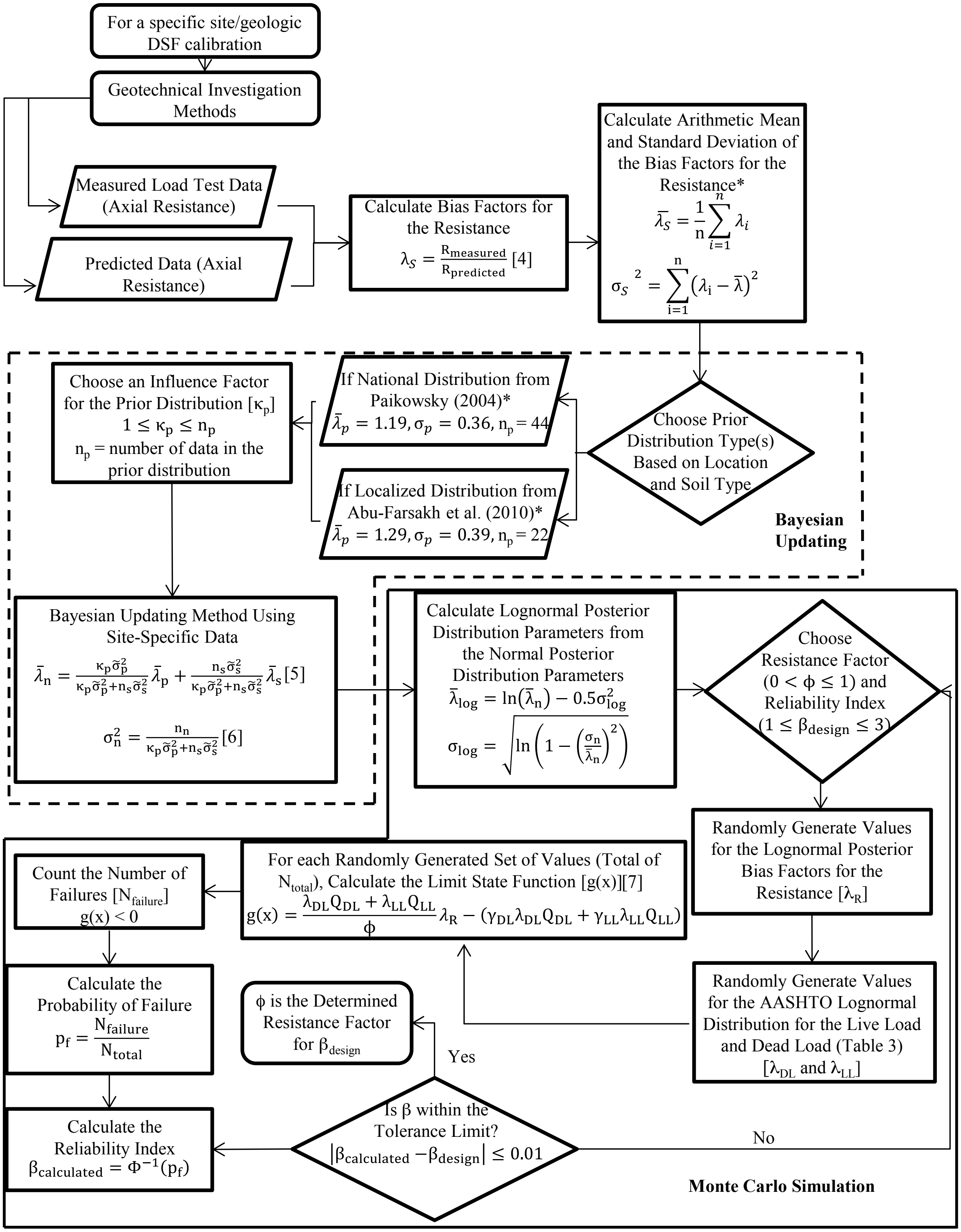


Figure 4. Flowchart of the Bayesian updating method utilized in conjunction with the Monte Carlo simulation method (modified from Race 2015).

Cost Analysis

Two analyses were performed to evaluate the cost efficiency of the geotechnical investigation methods and the calibrated resistance factors. The unit cost per ton of resistance when using the AHTD and UofA geotechnical investigation methods in rock (at the SSATS) and in alluvial soil deposits (at the TATS) was determined to evaluate the cost implications on various types of infrastructure. Furthermore, the cost of five provided/hypothetical loading conditions was investigated to determine the total project cost from the determined unit cost of the DSF (Table 5). Similarly, potential cost savings for 22 or 24 DSF designed and constructed for a hypothetical project at the SSATS and TATS sites, respectively, were calculated based on the calibrated resistance factor values. The required lengths for four-foot diameter DSF were calculated by using the various software programs in conjunction with the data obtained from the various geotechnical investigation methods, and the corresponding resistance factor value. The costs associated with the DSF projects were then compared to determine the possible cost savings that can be obtained by using different geotechnical investigation methods or by performing a resistance factor calibration study.

Table 5. Provided and hypothetical loading conditions for the cost analysis of the DSF at the SSATS and the TATS.

Load Condition	Description	Number of Shafts	Max. Axial Load (Ton)
AHTD – Provided SSATS	Single-lane Bridge Superstructure	22	445
AHTD – Provided TATS	Principal Arterial Bridge for On-Ramp Approach	24	395
Hypothetical 1	Heavy Building with Concentrated Loads	50	1700
Hypothetical 2	Large Structure with Less Concentrated Loads	150	850
Hypothetical 3	Medium Structure with Moderate Loads	40	500

Results

Geotechnical Investigation Method

The mean values (and uncertainty) of the corrected blow count (N_{60}), γ_T, and c_u were obtained from the sites and different sampling methods (AHTD, MODOT, and UofA). As obtained from the TATS, an empirical factor (0.55) was utilized in this study to convert raw blow count values that were obtained from the California split spoon sampler to corresponding raw blow count factors for the standard split spoon sampler (Figure 5). At the TATS, the blow count values were similar in the cohesionless soils from the standard split spoon sampler (AHTD method), the CPT equations (MODOT method), and the empirically corrected blow count values from the California split spoon sampler (UofA method), as presented in Figure 6. At both the MATS and the TATS, the AHTD obtained total unit weight values for the cohesive soils, as correlated from blow count values by using the AHTD method, were greater than the values using the MODOT and UofA methods. Conversely, the total unit weight values in cohesionless soils were greater using the UofA method than the MODOT and AHTD methods. In general, the undrained shear strength values of the cohesive soils determined from the AHTD method were less than the values determined from the UofA and MODOT methods. Particularly, the UofA and MODOT obtained undrained shear strength values in high plasticity cohesive soil, as determined using the UofA and MODOT methods, were almost two times the values that were obtained by using the AHTD method.

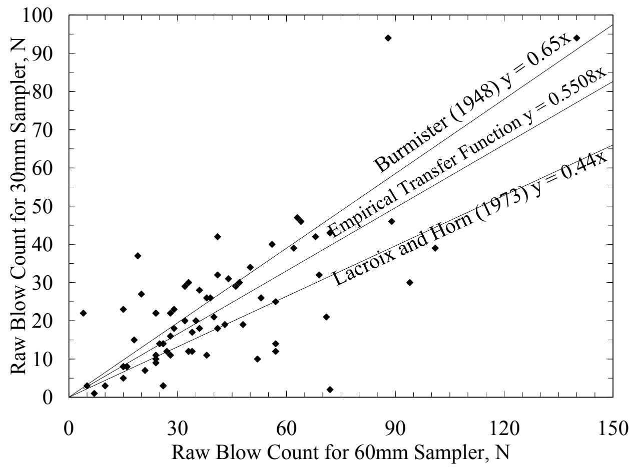


Figure 5. Empirical correlation for blow count values using a standard split spoon sampler and a California split spoon sampler (Race and Coffman 2013).

Design Software Programs

The axial capacity values , as a function of depth, for a four-foot and a six-foot diameter DSF were determined using three geotechnical investigation methods and two software programs for all three of the test sites (Figure 7). Similarly, the predicted load-movement responses were determined from the three geotechnical investigation methods and the two software programs for the as-constructed DSF at the three test sites (Figure 8). In general, the predicted load-movement responses, as obtained from the FB-Deep program were closer to the measured load-movement responses within the mixed cohesive and cohesionless soils at the MATS and the TATS.

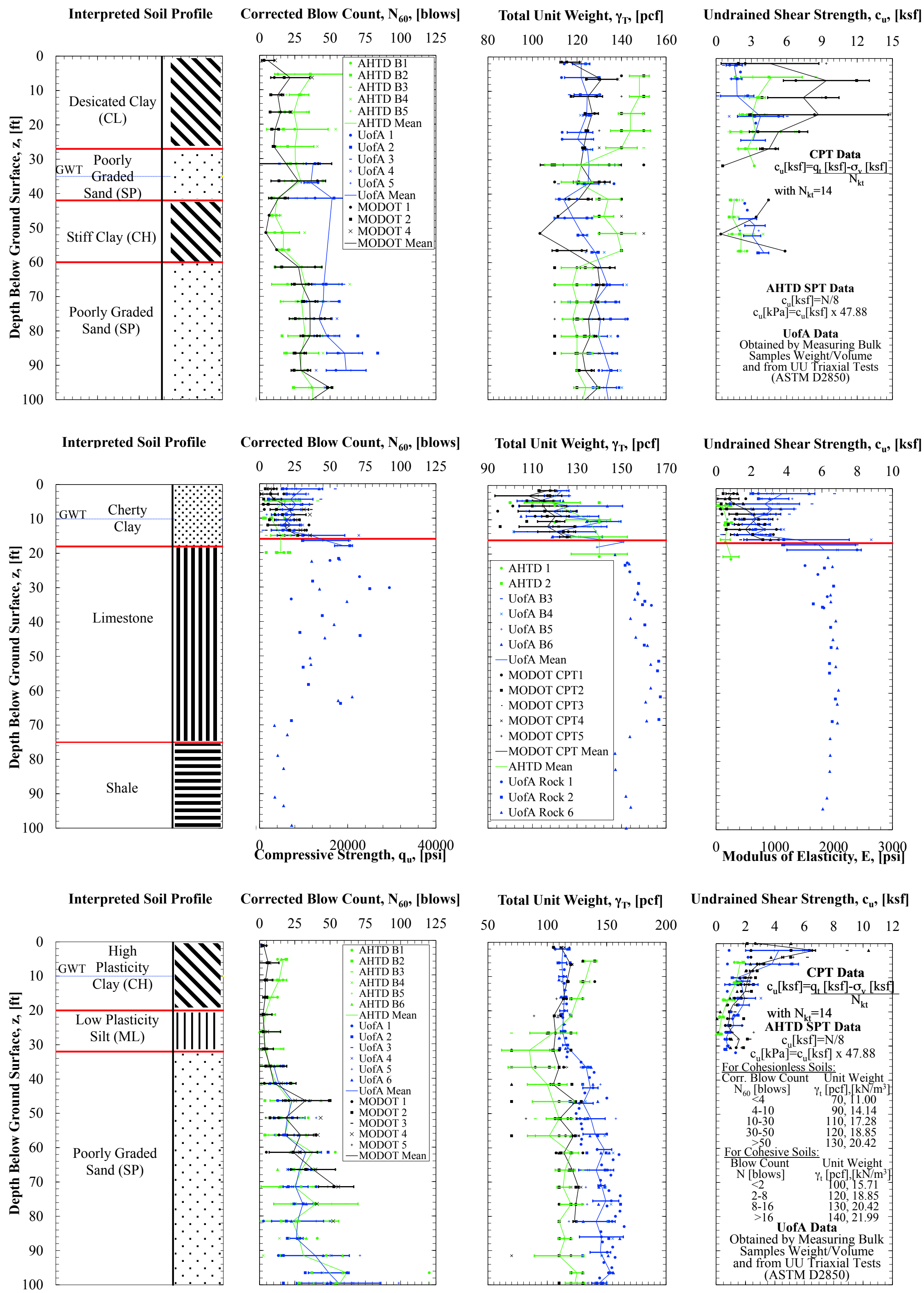


Figure 6. Soil property values from the a) MATS, b) SSATS, and c) TATS.

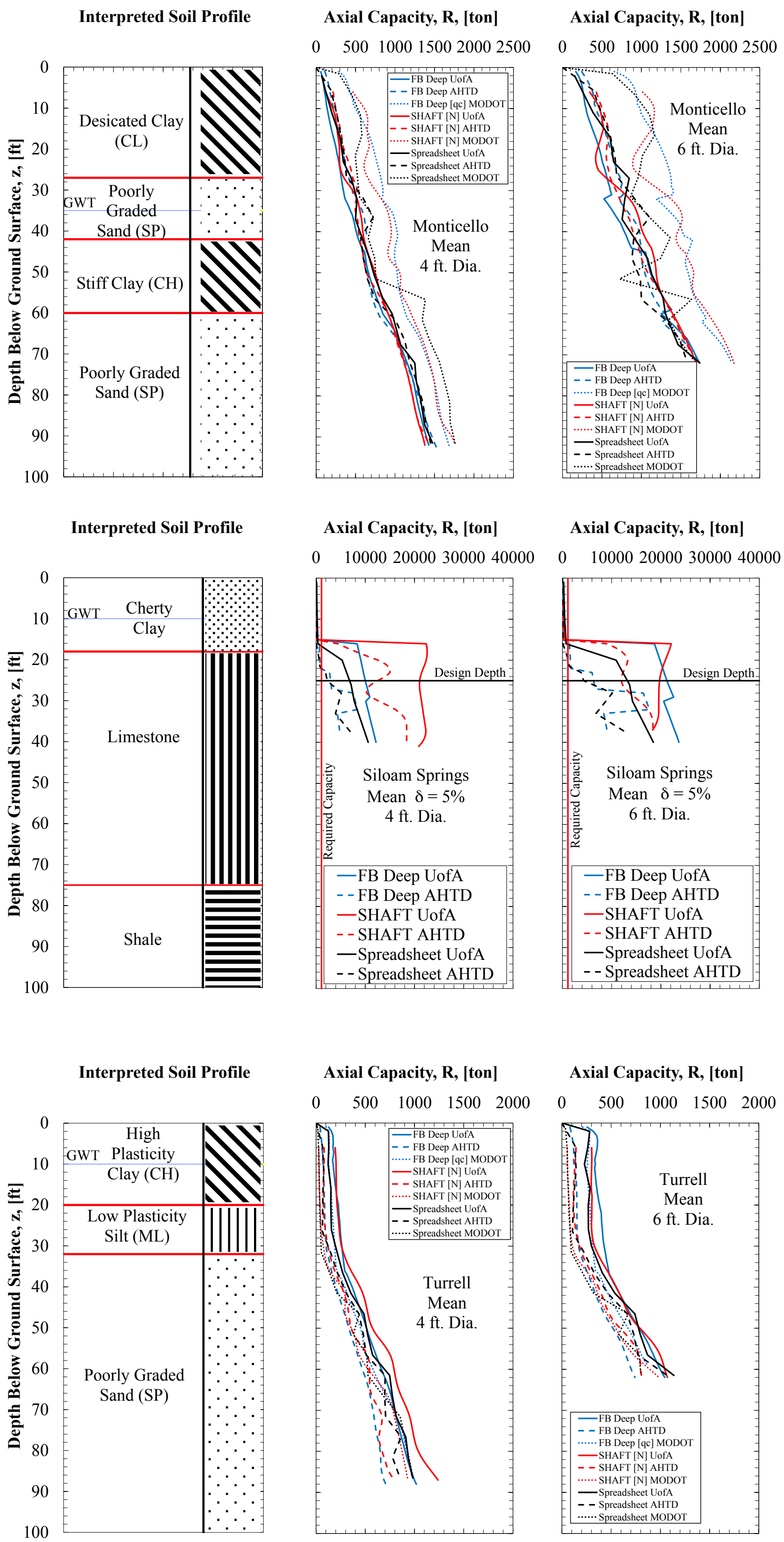


Figure 7. Predicted axial capacity with depth for a four-foot and six-foot diameter DSF (left to right) at the a) MATS, b) SSATS, and c) TATS.

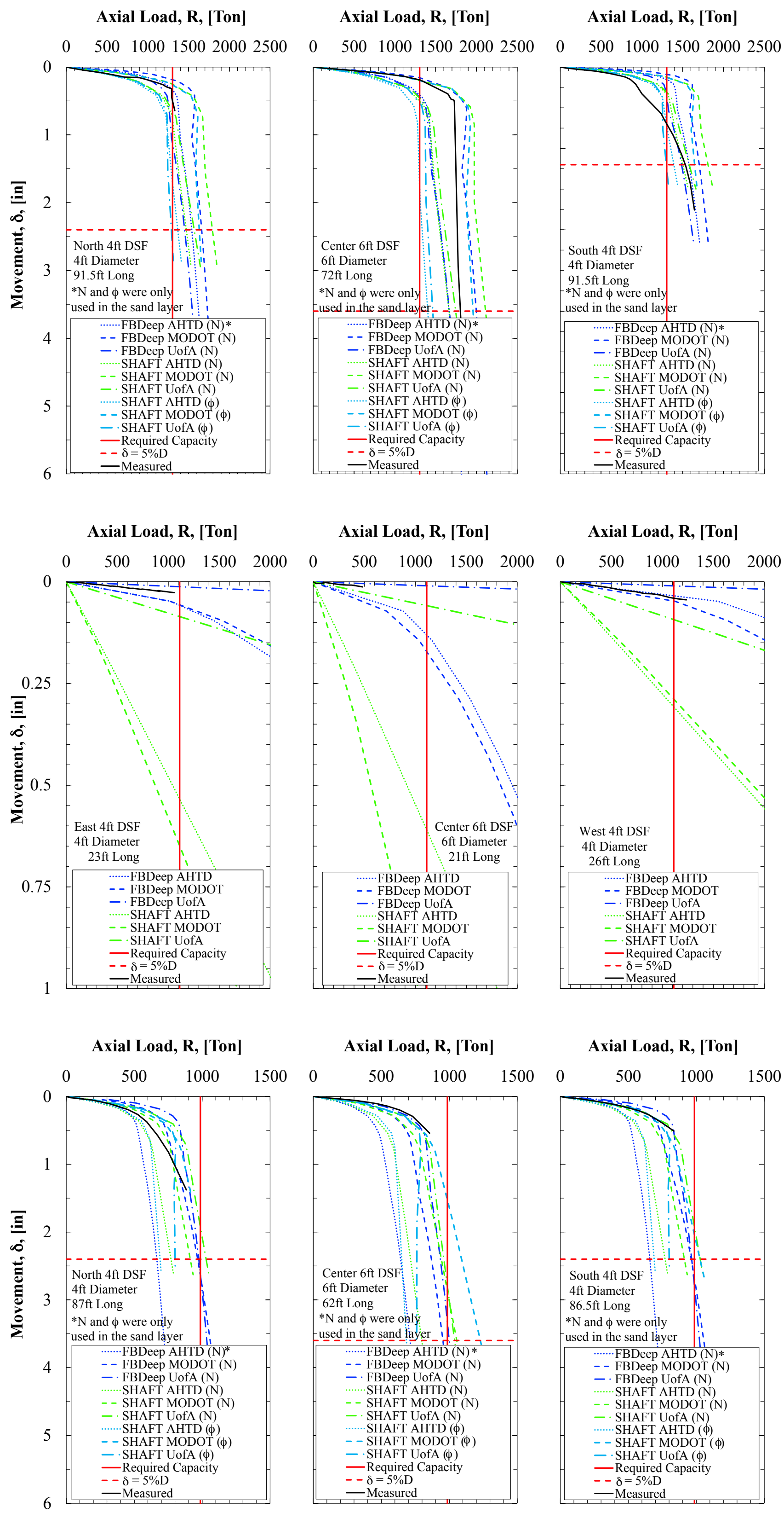


Figure 8. Predicted and measured load-movement response for the constructed DSF at the a) MATS, b) SSATS, and c) TATS.

Construction Concerns

At the SSATS, the four main construction/testing concerns included: 1) poor concrete placement below the load cell for the West four-foot diameter DSF, 2) bad telltale placement (welded to the top steel plate instead of the bottom steel place) within the Center six-foot diameter DSF and West four-foot diameter DSF, 3) shorter than designed rock socket lengths for the Center six-foot and East four-foot diameter DSF, and 4) an extended period of time from the start of the excavation to when the concrete was poured. The construction concerns about the DSF at the SSATS were primarily related to obtaining high-quality of load tests; however, the amount of time between excavation and concrete pouring was also an important consideration particularly regarding the time dependent nature of the rock texture. Based on the construction and testing of DSF at the SSATS, it is recommended that: 1) each piece of instrumentation be verified for correct installation, 2) the rock socket be deep enough to balance the side shear resistance and the end bearing resistance when performing a BLC test, 3) a large enough BLC be utilized, 4) concrete should be poured into the excavation within one day of completing the bottom one diameter of the rock socket excavation, and 5) proper concrete placement should be verified.

At the TATS, a collapse of the sidewall occurred within the North four-foot diameter DSF. Specifically, the collapse occurred within the silt layer (approximately 20 to 30 feet below the ground surface) due to rapid drawdown conditions. The combination of the increased water level to perform the Sonicaliper test®, the high permeability of the sand layer below the silt layer, and the low viscocity of the polymer slurry resulted in the collapse of the silt layer (collapsed volume ranged from 135ft^3 to 703ft^3). Due to the collapse, 2.5 times the amount of movement was observed within the equivalent top-down load-movement curve, at the required load, for the DSF constructed within the collapsed excavation (North DSF) than for the DSF constructed within the uncollapsed excavation (South DSF). Based on the observed results, it is recommended that a DSF that is constructed within a collapsed excavation be either overdrilled (larger diameter and deeper depth by 0.125 diameters) to obtain the required capacity or be proof tested to a movement of 1) one inch or to 2) 1.5 times the required axial capacity.

The construction problems at the MATS included: an open excavation from two to eight days, significant loss of polymer slurry, high slump concrete, equipment/operator malfunction, poor clean out of the bottom of the excavation, possible collapse at the bottom of the excavation, and premature setup of the concrete during placement. These problems occurred due to poor weather conditions, poor concrete mixing/timing, a substandard equipment operator, and high permeability cohesionless soils being located 80 feet below the ground surface. The influence from the various construction problems included: larger upward and downward movement of the BLC, larger top-down load-movement response, lower load transfer along the length of the DSF, lower unit side shear resistance values, and lower unit end bearing resistance values. It is recommended that the construction methods be considered in the design of DSF to unerstand the effects on the predicted versus measured load-movement response and unit side shear resistance. Similarly, it is recommended that DSF constructed in high permeability cohesionless soils be drilled and poured within a single day to prevent possible excavation collapse or slurry loss from within in the DSF excavation.

Although construction concerns were noted for all three sites, all of the constructed DSF reached the design capacity for the failure limits state. The ability to obtain the design capacity when constructed improperly should provide encouragement to designers of DSF. As discussed within the next section, not only were the design capacities met but the resistance factors that were calculated based on the load tests were higher than the resistance factors that are recommended by the AASHTO standards, even though the construction concerns were observed.

Calibrated Resistance Factor Values

Resistance factor values were calculated using the Bayesian updating and the Monte Carlo simulation technique. The resistance factor values are presented in Table 6 for the geologic specific study, as well as site-specific studies. In general, the resistance factor values for the geologic-specific soil deposits (alluvial and deltaic) within the state of Arkansas were increased. The resistance factor values that were obtained by utilizing the SHAFT software program were typically lower than the values that were obtained by utilizing the FB-Deep software program when the MODOT and UofA data were employed, but greater when the AHTD data was employed. Therefore, because the deltaic and alluvial soil deposit calibration was calculated from six full-scale load tests that were performed on DSF in the state of Arkansas, it is recommended that a larger database of DSF should be utilized to increase the accuracy of the values of the determined resistance factor. However, as discussed in Race and Coffman (2015a, 2015b), construction problems at the MATS and the TATS that affected the axial resistance likely resulted in lower values for the mean bias factor, higher values for the standard deviation of the bias factor, and conservative (lower) calibrated resistance factor values. Therefore, the values that are presented in Table 6 can be immediately implemented by design engineers at AHTD, and these values will continue to be refined with each additional full-scale load test that is completed within the state of Arkanas.

Table 6. Resistance factors for total resistance design determined utilizing the Bayesian updating method for the MATS, the TATS, and the state of Arkansas.

Site	Software Program	Geotechnical Investigation Method	Resistance Factor Values Calculated Utilizing the Bayesian Updating Method*	
			Paikowsky (2004) Prior Distribution	Abu-Farsakh et al. (2010) Prior Distribution
MATS	SHAFT	AHTD	0.754	0.815
		MODOT	0.796	0.796
		UofA	0.885	0.940
	FB-Deep	AHTD	0.695	0.748
		MODOT	0.891	0.908
		UofA	0.861	0.930
TATS	SHAFT	AHTD	0.644	0.693
		MODOT	0.612	0.609
		UofA	0.595	0.572
	FB-Deep	AHTD	0.662	0.745
		MODOT	0.607	0.599
		UofA	0.620	0.630
Non-Rock Arkansas (MATS & TATS)	SHAFT	AHTD	0.616	0.625
		MODOT	0.590	0.570
		UofA	0.705	0.750
	FB-Deep	AHTD	0.585	0.570
		MODOT	0.612	0.603
		UofA	0.740	0.805

*Reliability Index (β) of 3.0

Geologic-specific calibrated resistance factors were calculated for DSF constructed in mixed soil (clay and sand) within Arkansas for the total resistance, unit side resistance, and unit end bearing resistance. A summary of the resulting geologic specific calibrated resistance factors for the Strength I limit state (5%D) are presented in Table 7. The resistance factor values for the total resistance were generally higher than the recommended national resistance factor values (0.58 for a site with low spatial variability). Conversely, the resistance factor values for the unit side and unit end bearing resistance were lower than the national values (but similar in magnitude to the recommended resistance factor values that were obtained from the Louisiana/Mississippi loadtest database that was reported within Abu-Farsakh et al., 2010). The highest efficiency of the calibrated resistance factor values was obtained when designing the DSF using the FB-Deep program and the UofA geotechnical investigation method.

From this Arkansas specific (deltaic and alluvial soil deposit) resistance factor calibration study, it is recommended that more full-scale data be collected to more accurately calibrate resistance factors for 1) the various soil deposits across the state of Arkansas and for 2) the different DSF construction methods. While small quantities of tests may be utilized for site-specific resistance factor calibration, the calculated values for the resistance factor may be higher than the "true" resistance factor for the state. However, it is recommended to utilize the site-specific resistance factor values particularly at sites with low variability. The values for the resistance factors that were calculated for the state of Arkansas were higher than the resistance factors calculated for the TATS due to the collapsed excavation leading to poor dataset at the TATS.

Due to the large DSF resistance in moderately hard to hard limestone, few full-scale load test results were publically available; therefore, it is recommended that any full-scale DSF in moderately hard to hard limestone be added to a national database. Subsequently, resistance factors for DSF in moderately hard to hard limestone may be calibrated once a larger amount of data is available (at least 10 load tests). Resistance factors and design considerations of DSF in moderately hard to hard limestone should then be considered once a national or regional database is established. However, it was determined that the AHTD standard of designing DSF to a minimum embedment depth of 10 feet within rock was not required. All of the shafts at the SSATS, even the shafts with less than one diameter embedment length met the required capacity.

Table 7. Summary of the alluvial and deltaic geologic-specific calibrated resistance factor values for the strength limit state for a reliability index (β) of 3.0.

Design Property	**Soil Type**	**Design Method**	**Geotechnical Investigation Method**	**Resistance Factor (Efficiency, ϕ/λ)** Prior Distribution Source Paikowsky (2004)	Abu-Farsakh et al. (2010)
Total Resistance	Mixed (Clay and Sand)	SHAFT	AHTD	0.616 (0.576)	0.625 (0.584)
			MODOT	0.590 (0.676)	0.570 (0.653)
			UofA	0.705 (0.697)	0.750 (0.741)
		FB-Deep	AHTD	0.585 (0.560)	0.570 (0.546)
			MODOT	0.612 (0.656)	0.603 (0.646)
			UofA	0.740 (0.685)	**0.805 (0.745)**
Unit Side Resistance	Clay	SHAFT	AHTD	0.206 (0.146)	0.134 (0.095)
			MODOT	0.195 (0.106)	0.127 (0.069)
			UofA	**0.214 (0.248)**	0.140 (0.162)
		FB-Deep	AHTD	0.218 (0.109)	0.145 (0.072)
			MODOT	0.204 (0.128)	0.125 (0.079)
			UofA	0.210 (0.159)	0.132 (0.100)
	Sand	SHAFT	AHTD	0.380 (0.182)	0.364 (0.175)
			MODOT	0.361 (0.188)	0.337 (0.175)
			UofA	0.333 (0.212)	0.289 (0.184)
		FB-Deep	AHTD	0.280 (0.167)	0.233 (0.139)
			MODOT	0.305 (0.200)	0.254 (0.166)
			UofA	**0.294 (0.234)**	0.238 (0.189)
Unit End Bearing Resistance	Sand	SHAFT	AHTD	N/A	0.496 (0.118)
			MODOT		0.137 (0.036)
			UofA		0.250 (0.077)
		FB-Deep	AHTD		0.280 (0.448)
			MODOT		0.182 (0.360)
			UofA		**0.287 (0.472)**

Note: the resistance factors with the highest efficiency for the various resistance mechanism are bolded.

Cost Analysis

Unit cost of the DSF based on the AHTD, MODOT and UofA geotechnical investigation methods was determined to be $82.70 and $24.11 at the SSATS (no MODOT) and $141.57, $75.47, and $75.47 at the TATS, respectively, The unit cost estimates were based on the geotechnical investigation estimates (from AHTD), CPT estimates (from MODOT for TATS only), the material take-off, construction estimates (from Aldridge Construction/McKinney Drilling Company, respectively), and load testing estimates (from Loadtest, Inc. for the UofA method only). The cost implications of the DSF at the SSATS is presented in Table 8. The UofA measured method averaged a savings of 220 percent and 28 percent compared to the AHTD and UofA design methods (as further discussed in Bey 2014). For the provided foundation type at the TATS, the total cost of the foundation was determined for DSF using the MODOT and UofA geotechnical investigation methods and for driven piles using the AHTD method (Table 9). The cost of DSF at the TATS using the UofA measured method were 8.8 percent more expensive than driven piles using the AHTD method; however, this was attributed to the unexpected labor costs associated with the extended construction at the TATS. Moreover, it is recommended to utilize DSF in potential seismically sites to benefit from the additional lateral loading capacity of the DSF, which was not investigated for this project.

Table 8. Summary of the cost implications of AHTD and UofA geotechnical investigation methods on various types of infrastructure at the SSATS (modified from Bey 2014).

Load Condition	**Project Cost (in $)**		
	AHTD	**UofA Designed**	**UofA Measured**
AHTD – Provided SSATS	819,789	815,778	556,983
Hypothetical 1	7,039,419	2,817,869	2,130,618
Hypothetical 2	10,554,036	4,223,715	3,155,339
Hypothetical 3	1,664,124	667,750	563,397

Table 9. Summary of the cost implications of the AHTD, MODOT, and UofA geotechnical investigation methods on the infrastructure at the TATS (modified from Bey 2014).

Design Method		**Shaft**	**Design Capacity (Ton)**	**Diameter (ft)**	**Length (ft)**	**Project Cost ($)**
Measured	UofA	S4	818	4	86.5	1,565,577
	UofA	N4	1065	4	87	1,889,668
	UofA	C6	1050	6	62	2,635,067
	MODOT	S4	818	4	86.5	1,570,710
Designed	AHTD	N/A	N/A	1.5	85	1,428,079
	UofA			4	86.5	1,490,577

A cost analysis was performed to compare the benefits of performing a site-specific or geologic specific resistance factor calibration study for the state of Arkansas. For all but one of the combined geotechnical investigation/software program/prior distribution methods (FB-Deep with AHTD data using the Paikowsky 2004 prior distribution), the site-specific resistance factors for the MATS were utilized to save money when compared to utilizing the AASHTO (2007) recommended resistance factors (Table 10). The largest cost savings of $463,800 US dollars (29.7 percent of the total) was obtained by utilizing the site-specific calibrated resistance factors for the MATS as obtained from the SHAFT program, the UofA data, and the prior distribution from Abu-Farsakh et al. (2010). Similarly, the largest cost savings/highest efficiency from the geologic-specific calibration study was obtained by using the FB-Deep program and the UofA geotechnical investigation method as previously presented in Table 8.

Cost savings can be achieved by implementing the UofA geotechnical investigation method for the design of DSF and by calibrating resistance factors based on site-specific/geologic-specific studies. Particularly at sites with three full-scale load tests and minimal problems associated with the measured resistance, site-specific resistance factor values may be utilized to significantly reduce project costs. However, due to the limited number of tests at the TATS and MATS, cost savings was not achieved at some of the test sites due to one poor quality test at each site causing the mean value of the bias factor to decrease and the standard deviation of the bias factor to increase. This decrease in the mean values and increase in the standard deviation values led to a lower and more conservative calibrated resistance factor value. Even though the resistance factor was lower, the additional data that was obtained from the full-scale load tests are invaluable to designers as these data can provide designers with insight into how the foundations will perform if there is a problem during construction

Table 10. Design lengths of a 1.2m diameter DSF by utilizing site-specific resistance factors (prior distribution from Paikowsky 2004) and the subsequent cost for a large project of 1.2m diameter DSF (24 total).

Site	Software Program	Geotechnical Investigation Method	Design Length (m)		Project Cost (USD)	
			Original	Calibrated	Original*	Calibrated+
MATS	SHAFT	AHTD	82	63	1,377,600	1,283,400
		MODOT	62	35	1,041,600	813,000
		UofA	82	55	1,377,600	1,149,000
	FB-Deep	AHTD	77	67	1,293,600	1,350,600
		MODOT	63	34	1,058,400	796,200
		UofA	77	60	1,293,600	1,233,000
TATS	SHAFT	AHTD	91	88	1,528,800	1,703,400
		MODOT	88	82	1,478,400	1,602,600
		UofA	82	81	1,377,600	1,585,800
	FB-Deep	AHTD	101	94	1,696,800	1,804,200
		MODOT	82	77	1,377,600	1,518,600
		UofA	83	78	1,394,400	1,535,400

*Cost included construction equipment, man hours, and materials

+Cost included construction equipment, man hours, materials, and three full-scale load tests

Recommendations

The utilization of the UofA or MODOT geotechnical investigation methods is recommended to increase the efficiency of 1) determining the soil properties and 2) designing DSF in the state of Arkansas. Although the most efficient geotechnical investigation method is the UofA method, the MODOT method is also efficient (for the total resistance design of DSF) and the rapid deployment and data collection of the MODOT method is valuable for time sensitive projects. Similarly, the FB-Deep program is, on average, more efficient than the SHAFT program for the design of DSF in soil deposits within the state of Arkansas.

Until a large database (>30 full-scale load tests) exists for the various soil deposits within the state of Arkansas, it is recommended that every DSF be proof tested, up to the design load, to ensure that the required axial capacity of the DSF can be met (for a specific contractor). Furthermore, the results from the proof tests on DSF may be added to the load test database to more accurately calibrate resistance factors for the geologic-specific areas (alluvial or deltaic deposits and rock) within the state of Arkansas. Finally, the future utilization of DSF within the state of Arkansas is recommended because this foundation technology increases the reliability of the foundation system while reducing the cost.

Conclusions

The most efficient combination of geotechnical investigation method and design software program was examined to determine the “best” combination to DSF within the state of Arkansas. It is recommended that the UofA geotechnical investigation method be utilized in all soil types in combination with the FB-Deep software program to design DSF in alluvial and deltaic soil deposits in Arkansas. Similarly, it is recommended to utilize the UofA geotechnical investigation method rather than the AHTD method in rock because of the significant cost savings that can be obtained (as high as $262,800 for the project that will be completed at the SSATS) by performing unconfined compressive testing at rock sites. Conversely, it was determined that for sites with seismic potential there was benefit in the additional lateral support provided by DSF compared with driven piles to counterbalance the slight crease in cost by using DSF in alluvial soils at the TATS. However, because the full-scale load tests lead to the slight increase in cost by using DSF at TATS, additional knowledge about the foundation performance is gained from the load teats and that knowledge is considered to offset the cost increase.

For the strength limit state (5%D), calibrated resistance factor values were determined for the total resistance, unit side resistance, and unit end bearing resistance design of DSF in the state of Arkansas. From the site-specific and geologic-specific calibration studies that were performed, the calibrated resistance factor values ranged from 0.57 to 0.93 depending upon the geotechnical investigation method and software program that were utilized. Resulting cost savings from site-specific resistance factor calibration study was up to $463,800 for the design of

DSF using total resistance for the strength limit state (at the MATS using SHAFT and UofA methods). The recommended geotechnical investigation method and the design software program for the efficient design of DSF in soil within the state of Arkansas are the UofA method and the FB-Deep program, respectively. However, due to the rapid rate of data collection associated with the MODOT method, the use of the data collected from the MODOT method within the FB-Deep software program is also advisable.

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Additional Publications from this Project

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