Final Report

CALIBRATING RESISTANCE FACTORS FOR LOAD AND RESISTANCE FACTOR DESIGN FOR STATNAMIC LOAD TESTING

UF Project No.: 4910450482312

Contract No.: BC-354, RPWO #42

Submitted to:

Florida Department of Transportation
605 Suwannee Street, MS 30
Tallahassee, FL 32399-0450

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March 2003

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
Final Report		

4. Title and Subtitle	5. Report Date
Calibrating Resistance Factor in the Load and Resistance Factor Design of Statnamic Loading Test	March 2003
	6. Performing Organization Code
	8. Performing Organization Report No.
7. Author(s) Michael C. McVay, Ching L. Kuo, and Amy L. Guisinger	4910-4504-823-12
9. Performing Organization Name and Address	10. Work Unit No. (TRAIS)
University of Florida Department of Civil and Coastal Engineering 365 Weil Hall / P.O. Box 116580 Gainesville, FL 32611-6580	11. Contract or Grant No. BC354, RPWO #42
12. Sponsoring Agency Name and Address	13. Type of Report and Period Covered Final Report March 31, 2001 – March 30, 2003
Florida Department of Transportation Research Management Center 605 Suwannee Street, MS 30 Tallahassee, FL 32301-8064	14. Sponsoring Agency Code

15. Supplementary Notes

Prepared in cooperation with the Federal Highway Administration

16. Abstract

Since 1994 Statnamic load testing has been used by the FDOT to estimate drilled shaft static capacities. However, until recently insufficient Statnamic and conventional static load testing data existed together to estimate LRFD resistance factors, ϕ, for piles and shafts in variable soil types.

Using a database of thirty-seven drilled shafts and piles from around the world (contribution from AFT Inc., and Berminghammer Co.), the resistance factors were estimated based on the FOSM (First Order Second Moment, FHWA) approach. Using a Reliability Index, $\beta = 2.5$, LRFD resistance factors of 0.70 and 0.65 were determined for driven piles and drilled shaft embedded in rocks and non-cohesive soils (insufficient number of cases for cohesive soils).

Numerical simulations (LS-DYNA) of Statnamic tests in sands and rock under varying load rates gave very similar back-computed static resistance using the Unloading Point Method (UPM). Changing the magnitude not duration of the Statnamic dynamic load, also gave similar static resistance (UPM) with the exception of very high skin friction cases.

17. Key Words	18. Distribution Statement
LRFD, reliability analysis, driven pile, drilled shaft, Statnamic Load Testing, rate factor, resistance factor	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA, 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	129	

Form DOT F 1700.7 (8-72)

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	ml
gal	gallons	3.785	liters	l
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3

NOTE: Volumes greater than 1000 l shall be shown in m^3.

Symbol	When You Know	Multiply By	To Find	Symbol
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	l
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
psi	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
ml	milliliters	0.034	fluid ounces	fl oz
l	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg	megagrams	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	psi

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised August 1992)

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CHAPTER 1

INTRODUCTION

1.1 General

The utilization of drilled shafts as an alternative to driven piles for Florida Department of Transportation's bridge foundations has seen an increased use due to loads and geological conditions. However, increasing loads/diameter of drilled shafts has resulted in field load testing problems due to the limitations of conventional load test equipment. As an alternative, Berminghammer Foundation Equipment has developed the Statnamic device with a 7500-ton capacity, in the early 1980s. Because of its large loading capacity, quick and easy mobilization, the device has become a viable alternative for load testing drilled shafts.

Unfortunately, there exists no recommended published LRFD resistance factors, ϕ or ASD (Allowable Stress Design) F.S. for the back-computed Statnamic static capacities. Moreover, in 1998 when the FDOT contracted the University of Florida to calibrate LRFD resistance, ϕ, factors for Florida, the Statnamic test was not considered due to insufficient data. As a consequence, many engineers assumed that the resistance factor, ϕ, for Statnamic testing was the same as that of a conventional load test (0.75). However, because of the dynamic nature of the test, the latter may not be true.

Moreover, since the Statnamic device is relatively new to practicing engineers, their theoretical understanding of the test and data interpretation is limited. Therefore, a database needed to be established (measured and predicted capacities), and the LRFD resistance factors, ϕ, and associated ASD F.S. had to be established. Also, as part of this work, a summary of the available literature, as well as modeling (numerical) of the test (rate of loading, soil/rock resistance: inertia, damping and stiffness) was investigated to better understand the assumptions and limitations of the Statnamic device and results.

1.2 Objectives

To fulfill the objectives of this research, the overall project was divided into four major tasks as follow:

Task 1. Literature Review and Evaluation

a) Review and study the available publications on Statnamic load testing.
b) An overview of the Statnamic loading concept and the methodologies (i.e., static capacity) proposed to interpret the Statnamic measurement.

Task 2. Database Collection and Analysis

a) Collect a database including Statnamic, and conventional load tests on driven piles and drilled shafts and related soil conditions.
b) Data was separated by geologic formation and type of foundation.
c) Perform a numerical simulation of rapid loading on a deep foundation.

Task 3. Calibration of Resistance Factor

a) Following the methodology used in the previous FDOT research project "Calibrating Resistance Factors in the Load and Resistance Factor Design for Florida Foundation (McVay et al. 1998)," a calibration of the resistance factor for the Statnamic test was performed using reliability theory.

b) A meeting with FDOT Geotechnical Engineers will be scheduled to present the findings and solicit comments.

Task 4. Report Preparation

a) Finalize the findings of the literature review and evaluation, and recommend a resistance factor to be used for Statnamic load testing.

CHAPTER 2

LITERATURE REVIEW

2.1 The Development of the Statnamic Test

Through a better understanding of load-transfer mechanisms of deep foundations, geotechnical engineers have gained more confidence in the design of high capacity piles/drilled shafts. Given the load limitation of conventional top down static testing, many novel or innovative methods for conducting load tests on high capacity deep foundations have been suggested: Statnamic, Osterberg, etc. This effort focuses on the former, Statnamic testing.

The Statnamic concept was first visualized in Hamilton, Ontario, Canada in 1985, and was first proposed in 1986, then referred to as Inertial Load Testing. In 1988, TNO Building & Construction Research of the Netherlands joined the development. Statnamic load testing was developed as a means of being able to fully mobilize a foundation up to a 600 tons capacity and was expected to be used together with conventional PDA measuring equipment (Bermingham, 1998). In 1988, a small model Statnamic device was built and tested in Hamilton, Ontario. Statnamic was introduced into the Market in 1992, and in 1994 FDOT performed its first Statnamic load test on the Victory Bridge project. The capacity of the Statnamic device dramatically increased from about 0.1 MN of the Model tester in 1988 to a 60 MN device in 1997 due to market demands. In addition to axial load testing, the Statnamic device was adopted/suggested for lateral loading test in 1994.

Currently an ASTM Specification is being proposed to address pile load testing under rapid axial compressive load (Janes et al. 2000), which includes Statnamic loading.

2.2 Fundamental Concepts of Statnamic Testing

The Statnamic apparatus as shown in Figure 2-1 consists of a pressure chamber and reaction mass placed atop the shaft/pile to be tested.

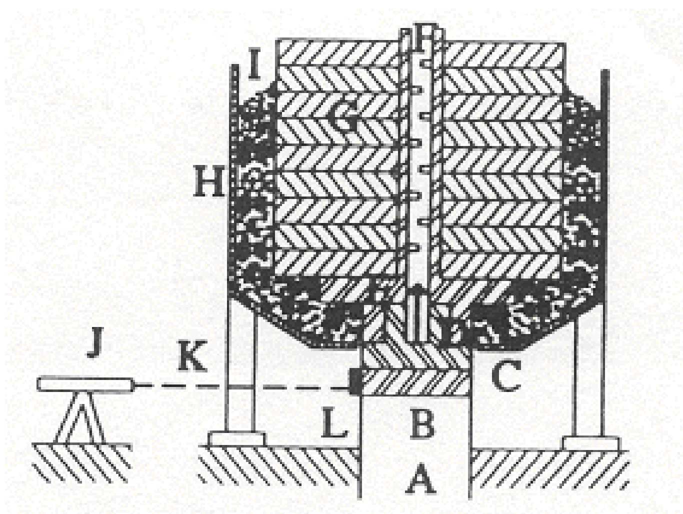


Figure 2-1. Statnamic apparatus

Solid fuel is burned within the pressure chamber. As the pressure increases, an upward force is exerted on a set of reaction masses while an equal and opposite force pushes downward on the pile/drilled shaft. In Fig. 2-2 the fuel has been ignited. The burning of the fuel generates high pressures and the reaction mass is accelerated. At this stage the actual loading of the pile takes place. The reaction force pushes the pile into the ground. The force signal and the displacement signal are record by TNO's Foundation Pile Diagnostic System (FPDS).

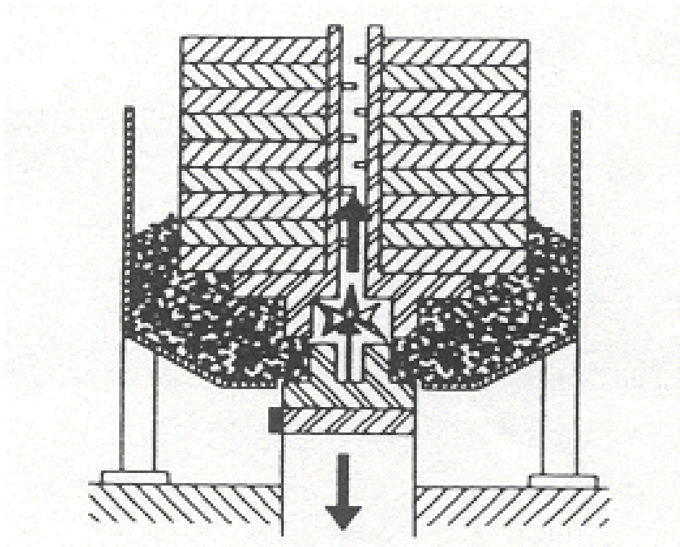


Figure 2-2. Fuel ignition

The upward movement of the reaction mass results in space, which is filled by gravel (Fig. 2-3). Gravity causes the gravel to flow over the pile head as a layer.

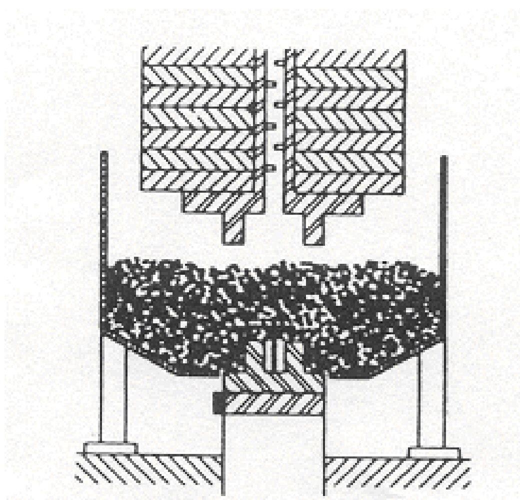


Figure 2-3. Space fills with gravel

When the reaction mass falls back it lands on the gravel, which cushions the force on the pile/shaft (Fig. 2-4).

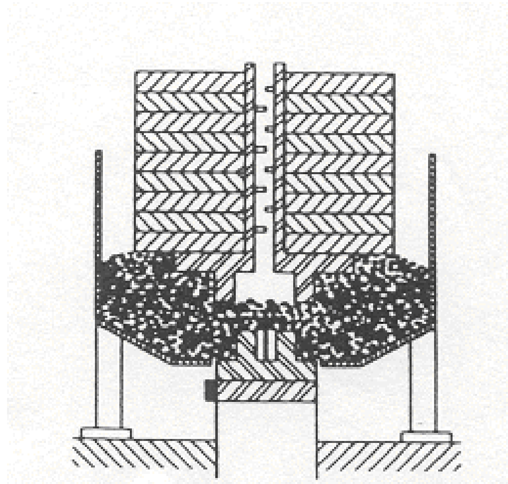


Figure 2-4. Gravel catching mass

In addition to the gravel catching mechanism presented in this description there are two other catching devices commonly used: the hydraulic and mechanical catching mechanisms, which are implemented for tests of 4 MN and less. In the hydraulic apparatus, the reaction weights rest on hydraulic rams located at each corner of the Statnamic device. This device stops the fall of the reaction mass at the apex of its rise. The mechanical catching system is currently limited to underwater test. This device is equipped with a locking device to prevent motion in the downward direction.

The propellant typically accelerates the reaction masses in the neighborhood of 20 gs. The reaction masses weigh approximately 5 to 10% of the desired load, and the loading duration is about 120 ms. Built-in instrumentation including load cell and laser sensor record load and displacement during the entire test. Figure 2-5 shows typical load, time and displacement results of the Statnamic load test.

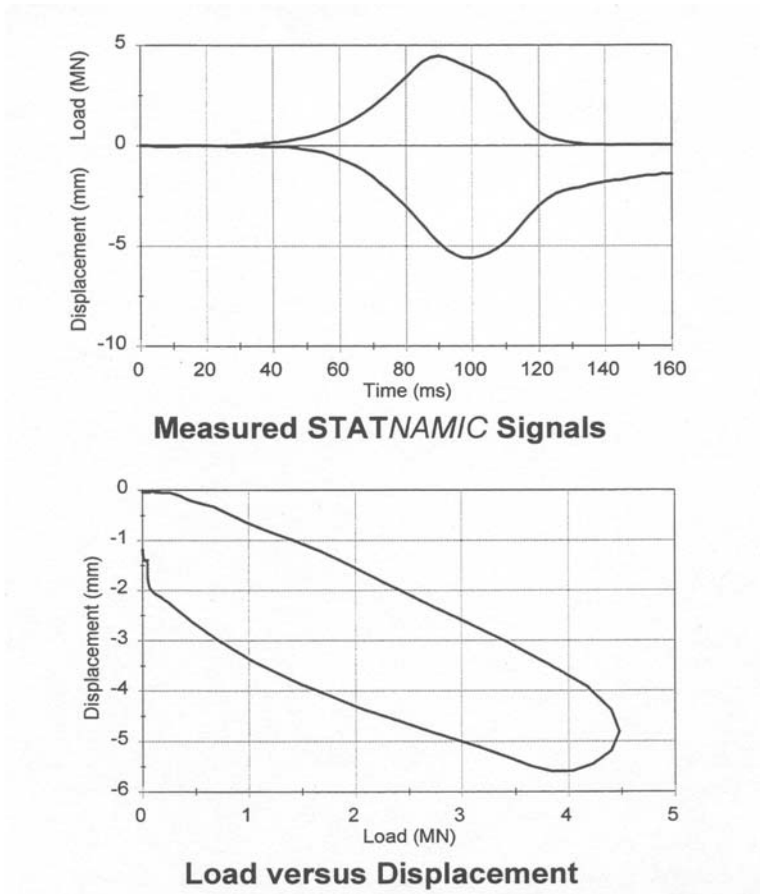


Figure 2-5. Typical results of Statnamic load test

2.3 Interpretation of Statnamic Load Test

Although Statnamic load tests have been performed since 1989, the first published approach for interpreting the Statnamic test was proposed by Middendorp in 1992 and is generally referred to as the Unloading Point Method (UPM). This method is based on the assumption that driven piles/drilled shafts behave as a single degree of freedom (SDOF) or rigid body. The latter (rigid body) assumes that no stress-wave propagation, i.e., strain dependent ($\sigma = E\,\varepsilon$) motion occurs within the pile/shaft. For the latter to be valid, the particle motion at any point on the pile/shaft (top, bottom, middle, etc.) is the same.

For a SDOF event, i.e., Statnamic loading, may be characterized as follows:

$$F_{stn}(t) = F_u(t) + F_v(t) + F_a(t)$$

$$= F_u(t) + C \bullet v(t) + m \cdot a(t) \qquad (2\text{-}1)$$

where

$F_{stn}(t)$ is the Statnamic applied loading

$F_u(t)$ is the static soil resistance

$F_v(t) = C \bullet v(t)$ is the damping force of the soils due to dynamic effects and determined as the product of the damping coefficient (C) and the velocity of the pile (v)

$F_a(t) = m \bullet a(t)$ is the inertia force and is determined from the product of pile acceleration and the mass of the pile.

Rewriting Equation 2-1, for the static pile capacity or static soil resistance $F_u(t)$:

$$F_u(t) = F_{stn}(t) - C \bullet v(t) - m \bullet a(t) \qquad (2\text{-}2)$$

Equation 2-2 can be solved for any time of loading (t) for the pile/shaft's static response (i.e., load-deflection response) if the damping (C) can be established. Note v(t) and a(t) are measured (same for any point on pile/shaft), and m is determined from the weight of the pile/shaft divided by 32.2 ft-sec^2 (English units). Middendrop (1992) identified 5 key areas on a typical Statnamic load-displacement curve (Figure 2-6) as well as recommending a procedure to calculate the damping coefficient and static capacity:

1. In area 1 (Fig. 2-6), the Statnamic reaction mass is placed on the pile/shaft top. The load displacement behavior is fully elastic. The measured load and displacement at the end of area 1 are called F_{stat} and u_{stat} (i.e., no dynamic forces: $F_v = F_a = 0$). The spring stiffness k_1 in this area can be calculated with

$$k_1 = F_{stat} / u_{stat} \qquad (2\text{-}3)$$

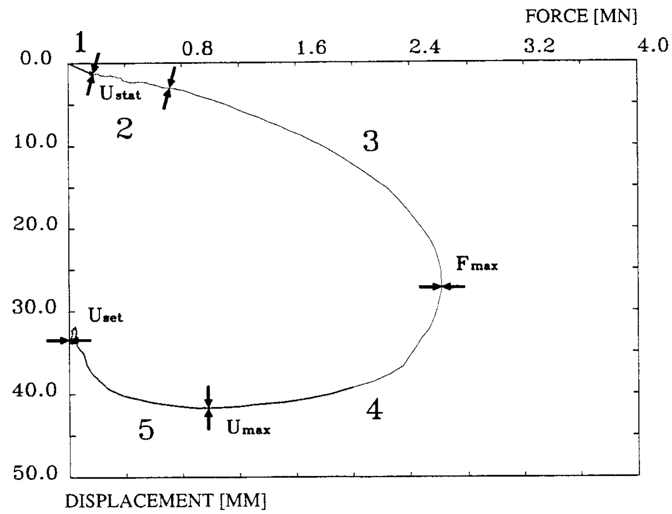


Figure 2-6. Key areas for Statnamic analysis

2. In area 2 (Fig. 2-6) the reaction mass is launched, i.e., Statnamic loading starts. It is argued that the soil resistance is elastic, and inertia and damping forces are acting on the pile/shaft. Middendorp developed the following expression for the damping coefficient for a certain time t_2, corresponding with displacement u_2, velocity v_2, acceleration a_2 and Statnamic load F_{stn2} in area 2 as:

$$C_2 = (F_{stn2} - k_1 \bullet u_2 - m \bullet a_2)/v_2 \tag{2-4}$$

Knowing C_2, the static resistance, F_u can be calculated as

$$F_u(t) = F_{stn}(t) - C_2 \bullet v(t) - m \bullet a(t) \tag{2-5}$$

3. In area 3, the velocity and inertia increase and the maximum Statnamic load is reached. The static soil resistance reaches its ultimate strength and yields at a value F_{uy}.

4. In area 4, the Statnamic load decreases. Because of the inertia of the pile, the displacement is still increasing. At the end of this area, i.e., time t_{umax}, the displacement reaches a maximum value, u_{max}, and the velocity becomes zero. Due to

this zero velocity, the damping force becomes zero and the Statnamic load minus the inertia force equals the static soil resistance at this point.

$$F_u(t_{umax}) = F_{stn}(t_{umax}) - m \bullet a(t_{umax}) \qquad (2\text{-}6)$$

Next, assuming that the pile/shaft's ultimate static resistance, i.e., F_{uy}, remains constant throughout zone 4, i.e., $F_u(t_{umax}) = F_{uy}$, the damping within zone 4 is calculated at time t_{umax} as:

$$C_4 = (F_{stn4} - F_{uy} - m \bullet a_4) / v_4 \qquad (2\text{-}7)$$

Based on average C_4 computed from Equation 2-7, the static resistance, F_u can be calculated in zone 3 and 5 as:

$$F_u(t) = F_{stn}(t) - C_4 \bullet v(t) - m \bullet a(t) \qquad (2\text{-}8)$$

5. In area 5, the pile is unloading and the final settlement of the pile U_{set} is observed at the end of this area.

Figure 2-7 shows the typical derived static capacity from Statnamic data.

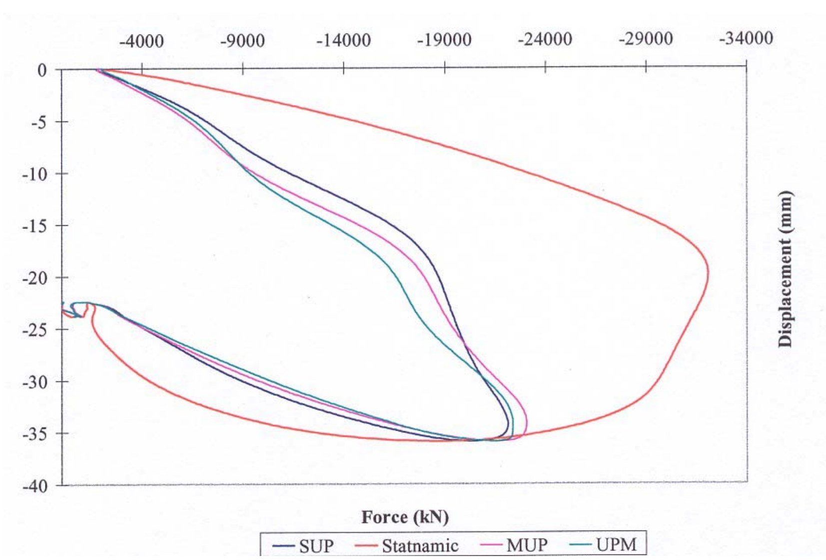


Figure 2-7. Typical derived static capacity from Statnamic data

2.4 Modification of Unloading Point Method (UPM)

Many researchers continued to explore a better methodology to interpret the test results. Following are brief discussions of these different approaches.

2.4.1 Correction for Stress Wave Phenomena

Middendorp and Bielefeld (1995) indicated that UPM might over predict the static capacity by as much as twenty percent (20%), if stress wave phenomena are present, i.e., short Statnamic (STN) load duration relative to the pile length. Middendorp (1995) characterized the significance of the stress wave phenomena through a wave number, $N_w = D/L$, where D ($D = c \bullet T$) is the distance that the stress wave (c) travels during the duration of load (T), and L is pile length. They concluded that for $N_w > 12$, UPM is valid (i.e., single degree of freedom response) while for $N_w < 12$ the stress wave phenomena needs to be accounted for. For example, a concrete pile with a wave speed of 4000 m/s, and a typical STN load duration (0.1 sec), the UPM approach would best predict the static capacity on a pile shorter than 32 m. A flow chart (Figure 2-8) was developed, based on N_w, for the appropriate analysis. In the case of $N_w < 12$, a number of recommendations were proposed. Brief discussions of each recommendation are as follows; the detailed evaluation can be found in the publication by Middendrop and Bielefeld (1995).

Increase of Statnamic loading duration. Increase in Statnamic load duration will lead to higher N_w. This can be done by increasing the reaction mass or vent distance (Bermingham et. al, 1995)

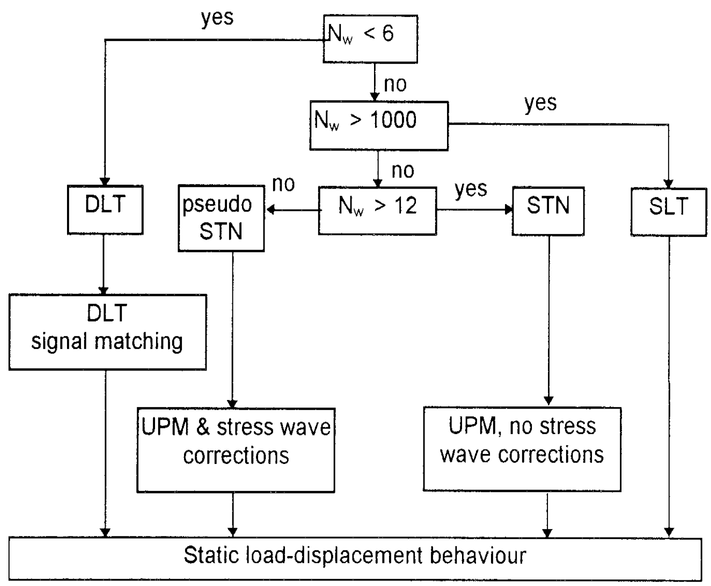


Figure 2-8. Load test as function of N_w

Statnamic signal matching. Signal matching by a stress wave simulation program, such as CAPWAP analysis became a routine method for Dynamic Load Testing. For Statnamic, a similar approach was adopted by Chin (2000), Naggar and Baldinelli (2000), and Foekken et al. (2000).

Stress wave analysis. Some piles, such as offshore piles penetrate partly into the soil. The wave number Nw can be increased by stress wave analysis, and an alternative Statnamic load-displacement diagram can be derived from a pile with the length below the ground and pile head at the ground surface.

Additional instrumentation. The standard instrumentation of STN consists of a load cell and a displacement transducer to measure the load and displacement of the pile head. Additional strain and acceleration transducers can be placed near the pile toe and along the shaft. These signals yield additional information to characterize the influence of stress wave phenomena.

Stress wave capacity factor. By performing Statnamic (STN) load testing and conventional static load testing, a stress wave capacity factor defined as the ratio of Statnamic Capacity and Static Capacity, is determined. This value can be used directly after Statnamic testing to compensate for stress wave phenomena.

2.4.2 Modified Unloading Point Method (MUP)

In this method, since the top and bottom (toe) of the pile/shaft is instrumented (accelerometers and strain gauges), Lewis (1999) proposed using an average of these values. It should be noted that variations in results from the top and bottom (acceleration, etc.) of the pile/shaft are indications that a single degree of freedom approach (UPM) is questionable (i.e., stress wave propagation is realized). Generally, using an average value (MUP) results in lower static capacities as shown in Figure 2-7 above.

2.4.3 Segmental Unloading Point Method (SUP)

Although the MUP may provide better interpretation of the test data, it still considers the response as a single degree of freedom instead of multiple degrees of freedom with stress wave propagation. The Segmental Unloading Point Method (SUP) was developed to overcome this problem (Lewis, 1999). By installing several levels of strain gages along the pile/shaft in addition to the transducers at the pile's top and toe, the pile is divided into segments separating each of the gage locations. The acceleration, velocity, displacement and force are determined for each segment, with each segment considered a rigid body (i.e., single degree of freedom). Using the standard UPM approach, the static capacity of each individual segment is determined, and the total estimated static capacity is obtained by summing the capacity of the individual segments.

2.4.4 Alternative Methods

Although many researchers have performed stress wave analysis (Seidel, 1996; Nishimura et al. 1998) and/or Finite Element analysis (Matsumoto, 1998; Horikoshi et al. 1998) on Statnamic testing, they do not provide step-by-step repeatable procedures for estimating the static capacity. Consequently, the majority of Statnamic reduction is done either by the Unloading Point Method (UPM) or Segmental Unloading Point Method (SUP).

2.5 Rate Effects on Load Testing

The loading duration of the Statnamic test is relatively long compared to other impact loading (pile driving) as shown in the Table 2-1 where Statnamic load testing is classified as Kinetic Testing. However, the pile-soil system is still considered subject to dynamic load effects (inertia, and damping). For instance, Ishida et al. (1998) performed axial load testing with different loading rates on piles embedded in sandy soil. His results indicate that both skin friction and unit end bearing resistance tended to rise as the loading speed increases. Bea and Audibert (1979) found that as the rate of loading on a pile founded in clay increased above the rate of a static load test, its ultimate capacity also increased. Wave and earthquake simulation, have shown pile ultimate axial capacity of 50 % to 70% over static values. The rate of loading in a Statnamic test is even higher with typical peak accelerations in excess of 20 g. The corresponding increase in axial pile capacity is large but as yet difficult to quantify for the purpose of predicting the static pile capacity from Statnamic data (Hyde and Anderson, 1998).

Table 2-1. Characteristics of Load Test Methods

	Integrity Testing	High Strain Dynamic Testing	Kinetic Testing	Static Testing
Mass of hammer (kg)	0.5-5	20000-10000	2000-5000	N/A
Pile Peak Strain	2-10	500-1000	1000	1000
Pile Peak Velocity (μ str)	10-40	2000-4000	500	10-3
Peak Force (kN)	2.2	2000-10000	2000-10000	2000-10000
Force Duration (ms)	0.5-2	5-20	50-2000	107
Pile Acceleration (g)	50	500	0.5-1	10-14
Pile Displacement (mm)	0.01	10-50	50	>20
Relative Wave Length	0.1	1.0	10	106

2.6 Case Histories

Since Statnamic was first introduced into the market in 1992, it is estimated that approximately 300 tests per year are performed worldwide (Bermingham, 1998). Although, a very limited number of Statnamic load test are performed with static load test for comparison. Justason et al. (1998) reported good agreement between conventional load tests and UPM or SUP results from Statnamic testing in sandy soil and rocks. However, Hajduk et al. (1998) reported poor agreement in clay, and Ng & Justason (1998) showed a need for further research in shale. Consequently, there is a great need to collect a database of Statnamic vs. measured conventional test results and estimate the appropriate LRFD resistance, ϕ, factors. The latter is a focus of this research.

2.7 General Review of the Interpretation of Statnamic Data

As given by the force equilibrium (Eq. 2-1) equation, Statnamic Loading is a dynamic event, which generates inertia, damping and static forces. In order to evaluate the static resistance, a number of assumptions and back calculations have to be performed. For instance, the major assumptions of the Unloading Point Method are that 1) pile or a segment of pile will behave as a rigid body and 2) the acceleration has no impact on the soil–pile interface behavior. In addition, the damping coefficient shown in Equation 2-2 is not a physical property of soil and cannot be measured in the laboratory or through in-situ testing.

Janes et al. (1994) noted that significant over prediction of pile capacity in a number of cases where static yield/failure was achieved. Also, in a number of cases where failure was not achieved, the computed static capacity exceeded the peak Statnamic load response because the pile/shaft's acceleration at the time of maximum displacement was negative (i.e., inertia forces would add instead of subtract to static resistance). Finally, there is confusion as to the definition of static capacity especially when excessive deformations are not recorded in the test.

Seidel (1996) performed a review of the analysis of Statnamic pile tests and concluded, "… although the test method has potential, the analytical method, which has been promulgated, suffers from some fundamental flaws, and cannot therefore be considered as reliable." It is considered that an alternative analysis based on rigorous physical modeling is necessary before the test method can be used with confidence. However, at the current stage, it is believed that with an appropriate resistance factor or safety factor used in design, the Statnamic test is a good tool for practical engineering.

2.8 Advantage and Disadvantage of Statnamic Load Testing

Statnamic testing appears to offer a number of advantages over other test types, including:

1. the test is quick and mobilization is quick and economical;
2. high loading capacity is available;
3. the loading is accurately centered and can be applied to both single pile and pile group;
4. the loading can be applied both vertically and laterally;
5. the test is quasi-static and does not develop potentially damaging compressive and tensile stress in the test pile/shaft; and
6. the load is measured using a calibrated load cell and does not rely on the pile/shaft material and cross section properties.

Inevitably, there are also some potential shortcomings, including:

1. certain assumptions need to be made in the interpretation of the test, especially in relation to the unloading of the pile;
2. currently, it cannot be used in uplift;
3. it cannot provide information on time-dependent settlement or movement; and
4. if insufficient dynamic force is applied, the pile/shaft failure capacity cannot be ascertained.

CHAPTER 3

DATABASE COLLECTION AND EVALUATION

3.1 Data Collection

Prior to this research, the FDOT database included thirteen-drilled shafts, which were subject to Statnamic loading as shown in Table 3-1. This data was from four major bridge projects in Florida. Due to the design loads of the shafts, Osterberg testing (hydraulic jack at tip) was undertaken instead of conventional load tests to compare with Statnamic testing. From the thirteen tests, only the Statnamic load test from Hillsborough Bridge had settlements that satisfied FDOT failure criterion.

Table 3-1. Summary of UF Drilled Shaft Data

Site Name	Pile Name	No. of Cycles	Test Date	Soil Type	Pile Size	Statnamic Load vs. Displacement Curve	Derived Static Load vs. Displacement Curve	Failure*	Other Test**
17th Street Bridge	LTS01	3	04/29/1998	sand/limestone	d=48” L=120’	Top	Yes	No	OST
	LTS02	2	05/12/1998	sand/limestone	d=48” L=142’	Top	Yes	No	OST
	LTS03	2	06/22/1998	sand/limestone	d=48” L=100’	Top	Yes	No	OST
Gandy Bridge	26-2 26-1	1	12/17/1994 11/28/1994	sand/limestone	d=48” L=33’	Top	Yes	No	OST
	52-4 52-3	1	12/13/1994 11/21/1994	sand/limestone	d=48” L=56’	Top	Yes	No	OST
	91-4 91-3	1	12/08/1994 11/11/1994	sand/limestone	d=48” L=71’	Top	Yes	No	OST
Hillsborough Bridge	4-14	2	06/26/1996	sand/limestone	d=48” L=71’	Top	Yes	Yes	OST
Victory Bridge	19-1 19-2 TH5	1	01/03/1995 12/06/1994 11/23/1994	sand/limestone	d=48” L=45’ d=48” L=48’ d=48” L=43’	Top	Yes	No	OST

*Failure refers to plunging during Statnamic Load Test
**OST – Osterberg Test

In the case of driven piles, the FDOT database (Table 3-2) had fifteen piles with both Statnamic data and conventional top down load tests. Seven of the test piles were in Florida and the rest were from Taiwan and Japan. Ten of the fifteen piles had sufficient settlements to obtain the FDOT failure capacity.

Table 3-2. Summary of UF Driven Pile Data

<table>
<tr><th>Site Name</th><th>Pile Name</th><th>Test Date</th><th>Soil Type</th><th>Pile Size</th><th>Statnamic Load vs. Displacement Curve</th><th>Derived Static Load vs. Displacement Curve</th><th>Failure*</th><th>Type of Test**</th></tr>
<tr><td rowspan="2">Taiwan High Speed Rail</td><td>Site No. 3</td><td>11/30/2000</td><td>sand/silt/clay</td><td></td><td>Top</td><td>Yes</td><td></td><td>STD</td></tr>
<tr><td>TP-4</td><td>02/16/2001</td><td>sand/silt/clay</td><td></td><td>Top</td><td>Yes</td><td></td><td>STD</td></tr>
<tr><td rowspan="4">St. George Island Bridge</td><td>LT-1</td><td>10/12/2000</td><td>sand/limestone</td><td>d=54”
t=8”
L=80’</td><td>Top</td><td>Yes</td><td>Yes</td><td>STD</td></tr>
<tr><td>LT-2</td><td>10/13/2000
10/28/2000</td><td>sand/limestone</td><td>d=54”
t=8”
L=80’</td><td>Top</td><td>Yes</td><td>No</td><td>STD</td></tr>
<tr><td>LT-3</td><td>10/27/2000</td><td>sand/limestone</td><td>d=54”
t=8”
L=80’</td><td>Top</td><td>Yes</td><td>No</td><td>STD</td></tr>
<tr><td>LT-5</td><td>10/26/2000</td><td>sand/limestone</td><td>d=54”
t=8”
L=80’</td><td>Top</td><td>Yes</td><td>No</td><td>STD</td></tr>
<tr><td>Japan</td><td>T1</td><td>1994/1995</td><td>clay/sand/silt</td><td>od=300mm
id=180mm
L=7m</td><td>Top</td><td>Yes</td><td></td><td>STD</td></tr>
<tr><td>Japan</td><td>T2</td><td>1994/1995</td><td>clay/sand/silt</td><td>od=300mm
id=180mm
L=7m</td><td>Top</td><td>Yes</td><td></td><td>STD</td></tr>
<tr><td>Japan</td><td>T3</td><td>1994/1995</td><td>clay/sand/silt</td><td>od=300mm
id=180mm
L=7m</td><td>Top</td><td>Yes</td><td></td><td>STD</td></tr>
<tr><td>Japan</td><td>T4</td><td>1994/1995</td><td>clay/sand/silt</td><td>od=300mm
id=180mm
L=7m</td><td>Top</td><td>Yes</td><td></td><td>STD</td></tr>
<tr><td>Japan</td><td>T5</td><td>1994/1995</td><td>clay/sand/silt</td><td>od=300mm
id=180mm
L=7m</td><td>Top</td><td>Yes</td><td></td><td>STD</td></tr>
<tr><td>Japan</td><td>T6</td><td>1994/1995</td><td>clay/sand/silt</td><td>d=400mm
L=13m</td><td>Top</td><td>Yes</td><td></td><td>STD</td></tr>
<tr><td rowspan="3">Bayou Chico Bridge Replacement</td><td>Pier 5</td><td>1999</td><td>sand/silt</td><td>s=24”
L=54’</td><td>Top</td><td>Yes</td><td>Yes</td><td>STD</td></tr>
<tr><td>Pier 10</td><td>1999</td><td>clay/sand/silt</td><td>s=24”
L=87’</td><td>Top</td><td>Yes</td><td>Yes</td><td>STD</td></tr>
<tr><td>Pier 15</td><td>1999</td><td>clay/sand/silt</td><td>s=24”
L=44’</td><td>Top</td><td>Yes</td><td>Yes</td><td>STD</td></tr>
</table>

*Failure refers to plunging during Statnamic Load Test
**STD – Static Load Test

To perform the LRFD calibration of Statnamic testing, it is necessary to have Statnamic and conventional load testing results available simultaneously on the same project site. Therefore, AFT and Berminghammer were requested to provide Statnamic and conventional static load test data, which would be used to assess the LRFD resistance, ϕ, factors (shafts, piles, etc.). In an effort to obtain more Statnamic versus conventional load test data, the FHWA database was also queried. A total of 61 cases, in which 27 were drilled shafts and 34 were driven piles, were collected as shown in Table 3-3.

Table 3-3. Summary of Statnamic Testing Data

Sheet	File	Pile Type	Soil Type	Same/Near	Order*	Comp. Method**	Static Method***	Country Code	Site ID	Pile ID	SLT Capacity (kN)	SLD w/ RF Capacity (kN)	SLD Capacity (kN)	◊ w/RF	◊
1	Florida LS1	DS	ROCK	NEAR	0	DAV	1	USA	FLLS	TP-1/2	6200	6225	6480	0.995984	0.95679
2	Florida LS2	DS	ROCK	NEAR	0	PE	1	USA	FLLS	TP-3/4	5600	4750	4950	1.178947	1.131313
3	JFK P10a	Pipe	ROCK	SAME	2	DAV	2	USA	JFK	OMSF P-10	N/F	N/F	N/F	N/F	N/F
4	NNO	Pipe	ROCK	SAME	2	DAV	3	JPN	NNO	T1	4380	4670	5087	0.937901	0.861018
5	STGLT1	DP	ROCK	SAME	2	DAV	2	USA	STG	LT-1	9400	N/F	N/F	N/F	N/F
6	TFC-241	DS	ROCK	SAME	2	DAV	1	TWN	TFC	241	N/F	N/F	N/F	N/F	N/F
7	TFC-532	DS	ROCK	SAME	1	DAV	1	TWN	TFC	532	N/F	N/F	N/F	N/F	N/F
8	Ohito	DS	SAND	SAME	2	DAV	3	JPN	OHITO	TEST	23680	N/F	N/F	N/F	N/F
9	BQE	DS	SAND	NEAR	0	DAV	1	USA	BQE	SA 1&8	N/F	N/F	N/F	N/F	N/F
10	BCPier10	DP	SAND	SAME	2	DAV	2	USA	BC	Pier10	3380	4410	5000	0.76644	0.676
11	BCPier15	DP	SAND	SAME	2	DAV	2	USA	BC	Pier 15	3820	3000	3322	1.273333	1.14991
12	BCPier5	DP	SAND	SAME	2	DAV	2	USA	BC	Pier 5	3500	3570	3957	0.980392	0.884508
13	STGLT5	DP	SAND	SAME	2	DAV	2	USA	STG	LT-5	12270	N/F	N/F	N/F	N/F
14	ShonanT6	PIPE	SAND	SAME	1	DAV	2	JPN	SHONAN	T6	1100	950	1042	1.157895	1.055662
15	ShonanT5	OTHER	SAND	SAME	2	DAV	2,3	JPN	SHONAN	T5	446	460	489	0.969565	0.912065
16	ShonanT2	OTHER	SAND	SAME	1	DAV	3,2	JPN	SHONAN	T2	533	N/F	N/F	N/F	N/F
17	ShonanT1	OTHER	SAND	SAME	2	DAV	3,2	JPN	SHONAN	T1	434	N/F	N/F	N/F	N/F
18	ashaft10	DS	SILT	SAME	2	DAV	2	USA	Aub	10	1420	1530	2191	0.928105	0.648106
19	ashaft8	DS	SILT	SAME	2	PE	2	USA	Aub	8	1700	1680	2450	1.011905	0.693878
20	ashaft7	DS	SILT	SAME	2	PE	2	USA	Aub	7	2230	2430	3530	0.917695	0.631728
21	ashaft5	DS	SILT	SAME	2	PE	2	USA	Aub	5	2800	2230	2890	1.255605	0.968858
22	ashaft3	DS	SILT	SAME	2	DAV	2	USA	Aub	3	1013	1200	1730	0.844167	0.585549
23	ashaft2	DS	SILT	SAME	2	PE	2	USA	Aub	2	2230	2030	2890	1.098522	0.771626
24	ashaft1	DS	SILT	SAME	2	PE	2	USA	Aub	1	2400	2050	2970	1.170732	0.808081
25	NIA TP 12a	Pipe	SILT	NEAR	0	DAV	2	USA	NIA	TP-1&2a	1230	1285	1790	0.957198	0.687151
26	NIA TP 12b	Pipe	SILT	NEAR	0	DAV	2	USA	NIA	TP-1&2b	1300	950	1380	1.368421	0.942029
27	NIA TP 13a	Pipe	SILT	NEAR	0	DAV	2	USA	NIA	TP-1&3a	1210	1225	1404	0.987755	0.861823
28	NIA TP 13b	Pipe	SILT	NEAR	0	DAV	2	USA	NIA	TP-1&3b	1300	1136	1750	1.144366	0.742857
29	NIA TP 910a	Pipe	SILT	NEAR	0	DAV	2	USA	NIA	TP-9&10a	1810	1900	N/F	0.952632	N/F
30	NIA TP 910b	Pipe	SILT	NEAR	0	DAV	2	USA	NIA	TP-9&10b	2380	1890	3850	1.259259	0.618182
31	Contraband T114	DP	CLAY	SAME	2	DAV	1	USA	LC	T-114.5	1830	2015	3070	0.908189	0.596091
32	Contraband X123	DP	CLAY	SAME	1	DAV	1	USA	LC	X-123	2470	2600	N/F	0.95	N/F
33	NIA TP 56a	Pipe	CLAY	NEAR	0	DAV	2	USA	NIA	TP-5&6a	1668	1937	N/F	0.861125	N/F
34	NIA TP 56b	Pipe	CLAY	NEAR	0	DAV	2	USA	NIA	TP-5&6b	2190	2070	2600	1.057971	0.842308
35	Amherst 2	DS	CLAY	SAME		DAV	4	USA		2	1214	796	1244	1.525126	0.975884
36	Amherst 4	DS	CLAY	SAME	0	DAV	1	USA	0	4	965.00	1071	1617	0.901027	0.596784
37	S9002T2	DS	ROCK	SAME	2	DAV	QML	CAN	KIN	P1	4550	3340	3500	1.362275	1.3
38	S9003T1	DS	ROCK	SAME	2	DAV	QML	CAN	KIN	P4	N/F	N/F	N/F	N/F	N/F
39	S9004T1	AC	SAND	NEAR	0	DAV	STD	CAN	DFC	P30	1310	1330	1350	0.984962	0.97037
40	S9004T2	AC	SAND	NEAR	0	DAV	STD	CAN	DFC	P44	1340	N/F	N/F	N/F	N/F
41	S9006T1	Pipe	ROCK	NEAR	0	DAV	STD	CAN	QP	P209	1560	1480	1800	1.054054	0.866667
42	S9007T3	Pipe	SILT	SAME	2	DAV	STD	CAN	BOU	P4	N/F	797	1540	N/F	N/F
43	S9010T1	DP	SILT	SAME	2	DAV	STD	USA	COL	P4	2470	1815	2360	1.360882	1.04661
44	S9010T2	DP	SILT	SAME	2	DAV	STD	USA	COL	P7	2880	N/F	N/F	N/F	N/F
45	S9101T2	DP	SILT	SAME	2	DAV	STD	USA	I-2	P2	8600	N/F	N/F	N/F	N/F
46	S9101T3	DP	SILT	SAME	2	DAV	STD	USA	I-2	P30	5300	N/F	N/F	N/F	N/F
47	S9102T2	Pipe	CLAY	NEAR	0	DAV	STD	CAN	LAM	P586	1040	1140	2550	0.912281	0.407843
48	S9105T1	Pipe	ROCK	SAME	2	DAV	STD	CAN	CPW	PT1	2200	2390	2550	0.920502	0.862745
49	S9206T1	DS	SAND	SAME	1	DAV	STD	GER	FRA	P6	N/F	N/F	N/F	N/F	N/F
50	S9206T2	DS	SAND	SAME	1	DAV	STD	GER	FRA	P586	N/F	N/F	N/F	N/F	N/F
51	S9207T1	T	ROCK	SAME	2	DAV	STD	USA	ONO	PT1	N/F	N/F	N/F	N/F	N/F
52	S9209T1	DS	SAND	SAME	2	DAV	STD	USA	I-4	PT1	7130	5790	6370	1.231434	1.119309
53	S9209T2	DS	SAND	SAME	2	DAV	STD	USA	I-4	PT1	7170	N/F	N/F	N/F	N/F
54	S9303T1	DS	SAND	SAME	2	DAV	STD	USA	CUP	P4	N/F	N/F	N/F	N/F	N/F
55	S9303T2	DS	SAND	SAME	2	DAV	STD	USA	CUP	P2	N/F	N/F	N/F	N/F	N/F
56	S9305T1	DS	SAND	SAME	2	DAV	STD	USA	I-4	PT1	N/F	N/F	N/F	N/F	N/F
57	S9306T2	Pipe	CLAY	NEAR	0	DAV	STD	USA	CAL	P49	1360	580	892	2.344828	1.524664
58	S9307T1	AC	SAND	NEAR	0	DAV	STD	USA	21_	P2	N/F	N/F	N/F	N/F	N/F
59	S9310T4	H	ROCK	NEAR	0	DAV	STD	USA	COU	P124	N/F	N/F	N/F	N/F	N/F
60	YKN	DP	SAND	SAME	2	DAV	CYC	JPN	YKN	No. 5	2770	2210	2700	1.253394	1.025926
61	HASAKI	Pipe	SAND	SAME		DAV		JPN	HASAKI	No. 6	1890	1300	1490	1.453846	1.268456

*Order: 0 – Side by Side (test different piles/shafts); 1 – Statnamic, then Static Test on same pile/shaft; 2 – Static then Statnamic Test on same pile/shaft

** Comp Method: Dav-Davisson; PE – Projected Envelope Failure Estimates

***Static Method: 1 –FHWA Quick; 2 – ASTM Static; 3 – Cyclic; 4 – CRP; 5 – Other

3.2 Data Evaluation

Although a large amount of Statnamic load tests have been performed worldwide over the past ten years, a very limited number also included conventional static load tests for calibration and comparison purposes. Also, many of the Statnamic tests were proof load tests which the loading did not reach the failure state. Therefore, only 37 of the 61 data achieved FDOT/Davisson failure criteria for both the Statnamic and the conventional static load test as shown in Table 3-4.

Table 3-4. Summary of Statnamic Testing Data Used in Calibration

Sheet	File	Pile Type	Soil Type	Same/Near	Order*	Comp. Method*	Static Method***	Country Code	Site ID	Pile ID	SLT Capacity (kN)	SLD w/ RF Capacity (kN)	SLD Capacity (kN)	λ w/RF	λ
1	Florida LS1	DS	ROCK	Near	0	DAV	1	USA	FLLS	TP-1/2	6200	6225	6480	0.995984	0.95679
2	Florida LS2	DS	ROCK	Near	0	PE	1	USA	FLLS	TP-3/4	5600	4750	4950	1.178947	1.131313
4	NNO	Pipe	ROCK	Same	2	DAV	3	JPN	NNO	T1	4380	4670	5087	0.937901	0.861018
10	BCPier10	DP	SAND	SAME	2	DAV	2	USA	BC	Pier10	3380	4410	5000	0.76644	0.676
11	BCPier15	DP	SAND	SAME	2	DAV	2	USA	BC	Pier 15	3820	3000	3322	1.273333	1.14991
12	BCPier5	DP	SAND	SAME	2	DAV	2	USA	BC	Pier 5	3500	3570	3957	0.980392	0.884508
14	ShonanT6	PIPE	SAND	SAME	1	DAV	2	JPN	SHONAN	T6	1100	950	1042	1.157895	1.055662
15	ShonanT5	OTHER	SAND	SAME	2	DAV	2,3	JPN	SHONAN	T5	446	460	489	0.969565	0.912065
18	ashaft10	DS	SILT	SAME	2	DAV	2	USA	Aub	10	1420	1530	2191	0.928105	0.648106
19	ashaft8	DS	SILT	SAME	2	PE	2	USA	Aub	8	1700	1680	2450	1.011905	0.693878
20	ashaft7	DS	SILT	SAME	2	PE	2	USA	Aub	7	2230	2430	3530	0.917695	0.631728
21	ashaft5	DS	SILT	SAME	2	PE	2	USA	Aub	5	2800	2230	2890	1.255605	0.968858
22	ashaft3	DS	SILT	SAME	2	DAV	2	USA	Aub	3	1013	1200	1730	0.844167	0.585549
23	ashaft2	DS	SILT	SAME	2	PE	2	USA	Aub	2	2230	2030	2890	1.098522	0.771626
24	ashaft1	DS	SILT	SAME	2	PE	2	USA	Aub	1	2400	2050	2970	1.170732	0.808081
25	NIA TP 12a	Pipe	SILT	Near	0	DAV	2	USA	NIA	TP-1&2a	1230	1285	1790	0.957198	0.687151
26	NIA TP 12b	Pipe	SILT	Near	0	DAV	2	USA	NIA	TP-1&2b	1300	950	1380	1.368421	0.942029
27	NIA TP 13a	Pipe	SILT	Near	0	DAV	2	USA	NIA	TP-1&3a	1210	1225	1404	0.987755	0.861823
28	NIA TP 13b	Pipe	SILT	Near	0	DAV	2	USA	NIA	TP-1&3b	1300	1136	1750	1.144366	0.742857
29	NIA TP 910a	Pipe	SILT	Near	0	DAV	2	USA	NIA	TP-9&10a	1810	1900	N/F	0.952632	
30	NIA TP 910b	Pipe	SILT	Near	0	DAV	2	USA	NIA	TP-9&10b	2380	1890	3850	1.259259	0.618182
31	Contraband T114	DP	CLAY	SAME	2	DAV	1	USA	LC	T-114.5	1830	2015	3070	0.908189	0.596091
32	Contraband X123	DP	CLAY	SAME	1	DAV	1	USA	LC	X-123	2470	2600	N/F	0.95	
33	NIA TP 56a	Pipe	CLAY	Near	0	DAV	2	USA	NIA	TP-5&6a	1668	1937	N/F	0.861125	
34	NIA TP 56b	Pipe	CLAY	Near	0	DAV	2	USA	NIA	TP-5&6b	2190	2070	2600	1.057971	0.842308
35	Amherst 2	DS	CLAY	SAME		DAV	4	USA		2	1214	796	1244	1.525126	0.975884
36	Amherst 4	DS	CLAY	SAME	0	DAV	1	USA	0	4	965.00	1071	1617	0.901027	0.596784
37	S9002T2	DS	ROCK	SAME	2	DAV	QML	CAN	KIN	P1	4550	3340	3500	1.362275	1.3
39	S9004T1	AC	SAND	NEAR	0	DAV	STD	CAN	DFC	P30	1310	1330	1350	0.984962	0.97037
41	S9006T1	Pipe	ROCK	NEAR	0	DAV	STD	CAN	QP	P209	1560	1480	1800	1.054054	0.866667
43	S9010T1	DP	SILT	SAME	2	DAV	STD	USA	COL	P4	2470	1815	2360	1.360882	1.04661
47	S9102T2	Pipe	CLAY	NEAR	0	DAV	STD	CAN	LAM	P586	1040	1140	2550	0.912281	0.407843
48	S9105T1	Pipe	ROCK	SAME	2	DAV	STD	CAN	CPW	PT1	2200	2390	2550	0.920502	0.862745
52	S9209T1	DS	SAND	SAME	2	DAV	STD	USA	I-4	PT1	7130	5790	6370	1.231434	1.119309
57	S9306T2	Pipe	CLAY	NEAR	0	DAV	STD	USA	CAL	P49	1360	580	892	2.344828	1.524664
60	YKN	DP	SAND	SAME	2	DAV	CYC	JPN	YKN	No. 5	2770	2210	2700	1.253394	1.025926
61	HASAKI	Pipe	SAND			DAV		JPN	HASAKI	No. 6	1890	1300	1490	1.453846	1.268456

*Order: 0 – Side by Side; 1 – Statnamic, then Static; 2 – Static then Statnamic Test
**Comp Method: Dav-Davisson; PE – Projected Envelope Failure Estimates
***Static Method: 1 –FHWA Quick; 2 – ASTM Static; 3 – Cyclic; 4 - CRP; 5 - Other

The characteristics of the data shown in Table 3-4 are summarized as follows.

1. Data includes 14-drilled shafts, 15 prestressed concrete piles and 15 steel pipe piles.
2. Twenty-seven (27) cases were performed in the United States, 5 from the state of Florida; 5 cases from Japan; and 5 cases from Canada.
3. According to the soil conditions of the test sites, 6 cases were found primarily in rock, 9 cases in sand, 14 cases in silt, and 8 cases in clay.
4. Approximately 62% of the cases had both the Statnamic test and conventional static load test performed on the same pile/shaft.
5. The length of the steel pipe piles ranged from 36 ft to 126 ft in length and 13 inches to 31 inches in diameter.
6. The size of the prestressed concrete piles ranged from 23 ft to 177 ft in length and 16 inches to 36 inches in diameter.
7. The drilled shafts ranged in length from 4.5 ft to 60 ft and 28 inches to 39 inches in diameter.
8. All of the derived static curves were computed using the Unloading Point Method (UPM) except for one test, which was performed prior to 1992, where the method of analysis was unknown. In addition, a rate factor (RF) for specific soils, η, proposed by Dr. Gray Mullins of the University of South Florida (2002), was applied to the results of UPM. The rate factors for different soil types are given in Table 3-5.

Table 3-5. Rate Factors

Soil	Rate Factor, η
Sands	0.91
Silts	0.69
Clays	0.65
Rocks	0.96

Source: Mullins, 2002

As identified by Mullins (2002), these rate factors (η) were developed to unify the average λ values obtained for four general soil types.

Detailed descriptions of the load test results including load-displacement curves are attached in the Appendix. Statistics and reliability analyses were performed on the database and are presented in the following chapters.

CHAPTER 4

CALIBRATION OF RESISTANCE FACTOR

4.1 Methodology of Calibration

The basic design criteria, regardless of the prediction method, assume that the resistance is greater than the applied loads with an acceptable level of safety. Design procedures developed by engineers to provide acceptable margins of safety include the following: (1) Allowable Stress Design (ASD) using a single global factor of safety; (2) Limit State Design (LSD) using partial factors of safety; and (3) reliability-based Load and Resistance Factor Design (LRFD).

Allowable Stress Design (ASD) has been the traditional design basis in civil engineering since it was first introduced in the early 1800's. Up until today, in most design codes, the foundation design has been based on ASD. In ASD, a global factor of safety is applied to the resistance such that the estimated stresses (or loads) do not exceed the reduced resistances. The relationship can be expressed as:

$$\frac{R_n}{FS} \geq Q_D + Q_L + Q_E \tag{4-1}$$

Where R_n is the nominal resistance, Q_D and Q_L are nominal values of dead and live load, Q_E is the environmental load such as wind, earthquake, etc., and FS is the factor of safety. Although this concept is simple and useful, the risk or level of safety associated with a value of FS depends on its definition and application, and a computed value of FS greater than one does not necessarily ensure safety (Smith 1981, 1985).

Limit State Design (LSD) has received increasing attention in geotechnical and structural engineering literature over the last 20 years. In LSD, two limit states are considered: (1) Ultimate Limit States (ULS) and (2) Serviceability Limit States (SLS). ULS pertains to structural safety and applies separate partial factors of safety on loads and strengths. SLS represents conditions that affect function and/or service requirements. An advantage of LSD is that it provides a clearer methodology for the separation of ULS and SLS. LSD is utilized to satisfy the following criteria:

ULS: Factored resistance ≥ Factored load effects

SLS: Deformation ≤ Tolerable deformation to remain serviceable

However, in the United States, geotechnical engineers in practice have not adopted this concept.

Load and Resistance Factor Design (LRFD) utilizes the concept of partial factors and can be viewed as an extension of the LSD. However, as the factors in LSD were determined by experience and judgement, the factors for LRFD may be determined using probability and reliability theory. The LRFD criterion is expressed in the following general form:

$$\varphi R_n \geq \sum \gamma_i Q_i \tag{4-2}$$

Where φR_n is the factored resistance.

φ is the resistance factor that considers the uncertainties in resistance

R_n is the nominal resistance estimated from engineering analyses

$\sum \gamma_i Q_i$ is the factored loads

γ_i is a load factor that considers the uncertainties in a component of load effects

Q_i is a component of load.

The resistance factor, φ, is similar in concept to the global factor of safety, and the factored resistance approach is similar to conventional ASD and may be viewed as a logical extension of ASD. The LRFD method has several advantages over the conventional ASD method including:

1. accounting for the uncertainties and variability in both loads and resistances;
2. providing more uniform levels of safety for various types of structures and materials; and
3. providing similar design concepts and procedures for various superstructures and substructures.

The primary objective of this research was to determine the resistance factor used in the LRFD method for the application of Statnamic load testing on various geotechnical engineering foundation designs for FDOT Projects. This will provide a smooth transition for geotechnical engineers from the ASD method to the LRFD method. In general, two procedures are used to calibrate the resistance factor: 1) fitting with ASD and 2) using reliability. However, without established experience of the safety factor used in ASD, the calibration of fitting with ASD cannot be performed. Therefore, only the reliability evaluation will be performed and the procedure is explained in the following section.

4.2 Calibration Using Reliability Theory

The reliability theory provides a valuable tool to compute the risk level or probability of failure in existing or new design codes. Based on reliability theory, the calibration of resistance factors corresponding to a given set of load factors consists of the following steps. For a detailed discussion about reliability theory and its applications refer to Barker et al. (1991) and Tobias and Trindade (1996).

(1) Estimate the level of safety or inherent reliability in current design methods.

In the reliability model, loads and resistance are considered to be random variables that can be described by probability density functions or frequency distributions. As long as the resistance, R, is greater than the load effects, Q, there exists a margin of safety for the limit states under consideration. The probability of failure or the realization of a limit state, P_f, can be expressed as

$$P_f = P(R < Q) = P\left[(R - Q) < 0\right] \tag{4-3}$$

If R and Q are assumed to be a lognormal distribution, Equation 4-3 can be rewritten as

$$P_f = P\left[\ln\left(R/Q\right) < 0\right] = 1 - F_u \left[\frac{\ln\left[\left(\overline{R}/\overline{Q}\right)\sqrt{\left(1+V_R^2\right)/\left(1+V_Q^2\right)}\right]}{\sqrt{\ln\left[\left(1+V_R^2\right)\left(1+V_Q^2\right)\right]}}\right] \tag{4-4}$$

where $\overline{R}$ and $\overline{Q}$ are the mean values, V_R and V_Q are the coefficients of variation (standard deviation divided by mean value) of R and Q, and $F_u()$ is the standard normal distribution function. Instead of specifying a probability of failure, a common approach is to express reliability in terms of a safety or reliability index, β, as follows.

$$\beta = \frac{\ln\left[\left(\overline{R}/\overline{Q}\right)\sqrt{\left(1+V_R^2\right)/\left(1+V_Q^2\right)}\right]}{\sqrt{\ln\left[\left(1+V_R^2\right)\left(1+V_Q^2\right)\right]}} \tag{4-5}$$

If bias factors, λ, defined as the ratio of the mean value to the nominal value, are introduced and the loads consist of only dead and live loads, Equation 4-5 can be rewritten as:

$$\beta = \frac{\ln\left[\frac{\lambda_R F_S \left(Q_D/Q_L + 1\right)}{\lambda_{QD} Q_D/Q_L + \lambda_{QL}} \sqrt{\frac{1+V_{QD}^2+V_{QL}^2}{1+V_R^2}}\right]}{\sqrt{\ln\left[\left(1+V_R^2\right)\left(1+V_{QD}^2+V_{QL}^2\right)\right]}} \tag{4-6}$$

where Q_D and Q_L are the nominal values of the dead and live loads. F_S is the factor of safety. λ_R, λ_{QD}, and λ_{QL} are the bias factors of the resistance, dead load and live load, respectively. V_R, V_{QD} and V_{QL} are the coefficients of variation of the resistance, dead load and live load, respectively.

Table 4-1 shows the relationship between reliability index, β (for $2 < \beta < 6$) and probability of failure, P_f as suggested by Rosenblueth and Esteva (1972):

$$P_f = 460e^{(-4.3\beta)} \tag{4-7}$$

Table 4-1. Relationship Between Probability of Failure and Reliability Index for Lognormal Distribution (from Barker et al. 1991)

Reliability Index, β	Probability of Failure, P_f		Probability of Failure, P_f	Reliability Index, β
2.5	0.99×10^{-2}		1.0×10^{-1}	1.96
3.0	1.15×10^{-3}		1.0×10^{-2}	2.50
3.5	1.34×10^{-4}		1.0×10^{-3}	3.03
4.0	1.56×10^{-5}		1.0×10^{-4}	3.57
4.5	1.82×10^{-6}		1.0×10^{-5}	4.10
5.0	2.12×10^{-7}		1.0×10^{-6}	4.64
5.5	2.46×10^{-8}		1.0×10^{-7}	5.17

Based on Equations 4-6 and 4-7 and the coefficients of variation of R and Q determined from probability analyses, the reliability index and probability of failure of the corresponding factor of safety, F_S can be calculated.

(2) Observe the Variation of the Reliability Indices.

As shown in Equation 4-6, the reliability index is not only affected by the uncertainty of the load effects and soil resistances, but also by the ratio of dead load to live load. In order to evaluate the effect of the ratio of dead to live load, which depends on the type of structure, a range of ratios from one (1) to nine (9) will be used in the calibration process.

(3) Select a target reliability index based on the level of safety or probability of failure used in the current design method.

Following Step (1), in which the reliability index was calculated based on the level of safety of the current design method, the target reliability index will be used for calibration of the resistance factor. This will ensure that designs utilizing LRFD method will not significantly deviate from the current ASD method.

(4) Calculate resistance factors consistent with the selected target reliability index.

For a given reliability index, β_T, and considering only dead plus live loads, the resistance factor can be expressed as

$$\phi = \frac{\lambda_R \left(\gamma_D Q_D / Q_L\right) \sqrt{\left(1+V_{QD}^2+V_{QL}^2\right) / \left(1+V_R^2\right)}}{\left(\lambda_{QD} / Q_D / Q_L + \lambda_{QL}\right) \exp\left[\beta_T \sqrt{\ln\left(\left(1+V_{QD}^2+V_{QL}^2\right)\left(1+V_R^2\right)\right)}\right]} \tag{4-8}$$

The resistance factor calculated from Equation 4-8 (from First Order Second Moment, Barker et al. 1991) may be used to validate or verify the new design approach by comparing designs based on calibrated resistance factors with designs obtained from conventional ASD. Recalibrating and modifying the resistance factors may also be required.

4.3 Statistic and Reliability Analysis

To better understand the behavior of the Statnamic load testing under different soil conditions and foundation types, analyses were performed on seven (7) scenarios as shown in Table 4-2. Analyses were also performed on the data before and after the rate factor was applied.

Table 4-2. Summary of Statistical Analysis

Case	With Clay						Without Clay					
	With RF			Without RF			With RF			Without RF		
	λ_R	σ_R	V_R	λ_R	σ_R	V_R	λ_R	σ_R	V_R	λ_R	σ_R	V_R
All data	1.11	0.28	0.25	0.88	0.24	0.27	1.10	0.18	0.16	0.89	0.20	0.22
Rock	-	-	-	-	-	-	1.07	0.17	0.16	1.00	0.18	0.18
Sand and silt	-	-	-	-	-	-	1.10	0.18	0.16	0.87	0.19	0.22
Clay	1.18	0.52	0.44	0.82	0.40	0.49	-	-	-	-	-	-
Drilled shaft	1.10	0.20	0.18	0.87	0.23	0.26	1.08	0.16	0.15	0.88	0.23	0.26
Driven pile	1.12	0.32	0.29	0.89	0.25	0.28	1.10	0.21	0.19	0.89	0.18	0.20

Note: Rate factor for sands = 0.91 Rate factor for clays = 0.65
Rate factor for silts = 0.69 Rate factor for rocks = 0.96

Where:

γ_D	1.25
γ_L	1.75
λ_{QD}	1.08
λ_{QL}	1.15
COV_{QD}	0.128
COV_{QL}	0.18
Q_D / Q_L	2

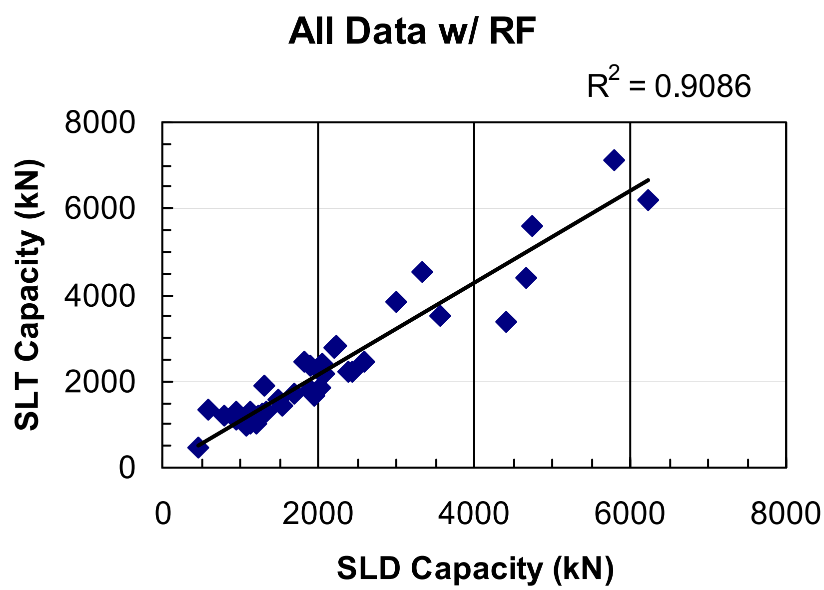
γ = load factors D = dead load
λ = bias factors L = live load
COV = coefficient of variation

Figures 4-1 to 4-8 show comparisons of the static load capacities and the corresponding Statnamic derived static capacities. Based on the statistical analyses, the bias factor and coefficients of variation for the ratio of static capacity to Statnamic derived static capacity are shown in Table 4-2. In general, the bias factor (mean) of the ratio of measured static capacity to derived Statnamic static capacity for all cases without applied Rate Factors ranged from 0.82 to 1.00. The latter indicates that the Statnamic derived static capacity tends to slightly over predict the static capacity. However, if rate factors (Table 3-5) are applied, the bias factors ranged from 1.07 to 1.18. The coefficients of variation do not have significant differences with or without rate factors and ranged from 0.16 to 0.32, except in the case of clays. The coefficients of variation for static capacities from the Statnamic tests in clays were 0.40 and 0.52 with and without the rate factor, respectively.

Because the safety factor for the Statnamic load test in ASD is unknown, the target reliability corresponding to current practices in ASD cannot be established.

However, according to previous research in calibrating resistance factors in the load and resistance factor design for Florida foundations (McVay et al. 1998), the target reliability, β_T, used for driven piles was 2.5, while 3.5 was used for drilled shafts. Considering the characteristics of the Statnamic Load Test compared to the predictions using SPT97 for driven piles, and the FHWA methodology for drilled shafts, it is recommended that a target reliability of 2.5 and 3.0 should be used for driven piles and drilled shafts, respectively. Using Equation 4-7, the calculated resistance factors for different cases with and without rate factor are shown in Table 4-3.

It was found that the overall resistance values were significantly affected by including the clay cases. Consequently, it was decided that the clay cases should be separated from the overall calibration. By doing so, the resistance factors, as shown in Table 4-4, of 0.70 and 0.65 may be used for the Statnamic load test on driven piles and drilled shafts in rock as well as noncohesive soils, respectively. If there is a significant thickness of clay in the soil profile, the resistance factors should be reduced to 0.60 for both driven piles and drilled shafts. If the driven piles and drilled shafts are embedded primarily in clay, a resistance factor for Statnamic Testing is not recommended due to insufficient data in the database, as well as the rate effects in the clays. In the case of the latter, it is well known that clays have inherent slow excessive pore-water pressure dissipation characteristics. Consequently, any excessive pore-water pressure generation in a dynamic event (i.e., Statnamic Test) would dissipate well after the test, making estimation (i.e., Unloading Point Method) of Static Capacity during the Statnamic Event difficult. Consequently, the resistance factors provided here for clays should be used with extreme care and should be used with other methods for verification purpose.



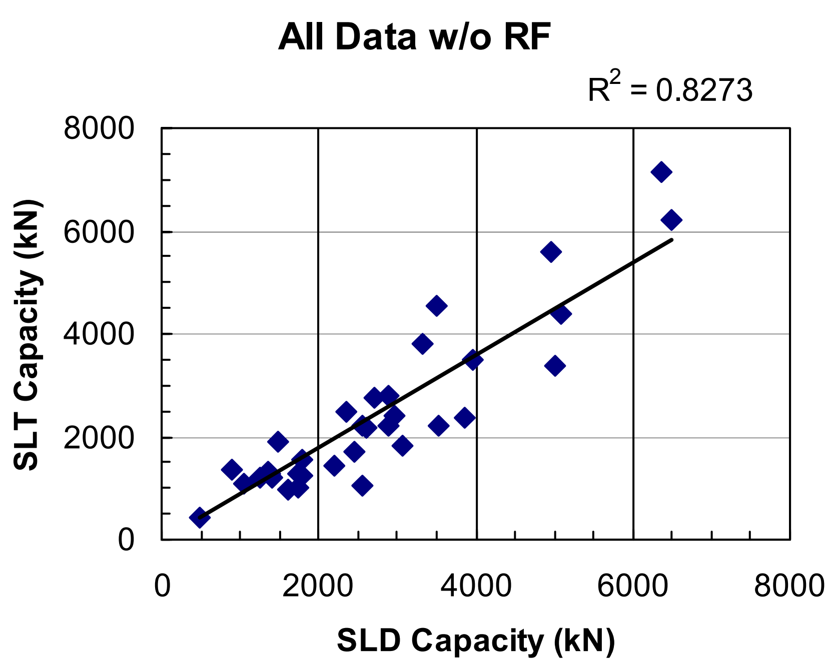
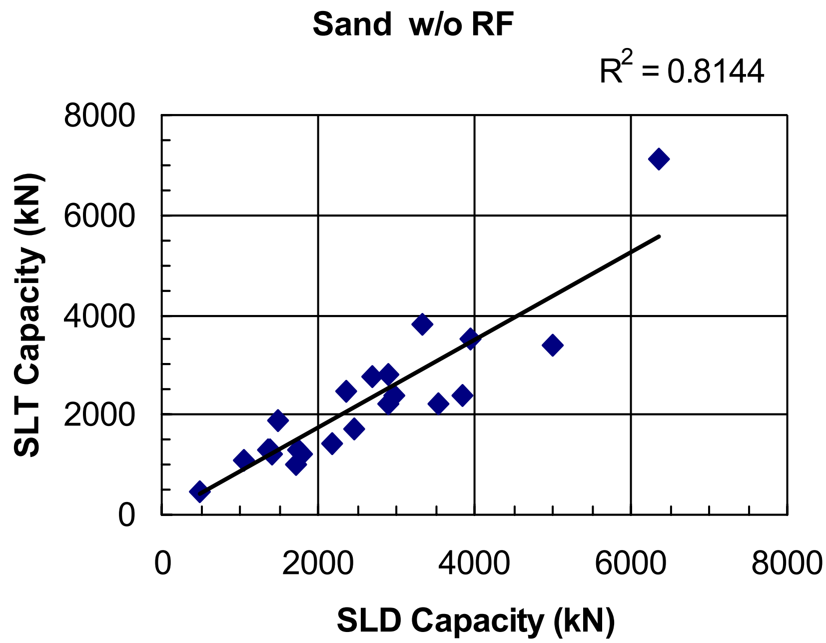


Figure 4-1. Static load test results (SLT) vs. Statnamic capacity (SLD) for all data



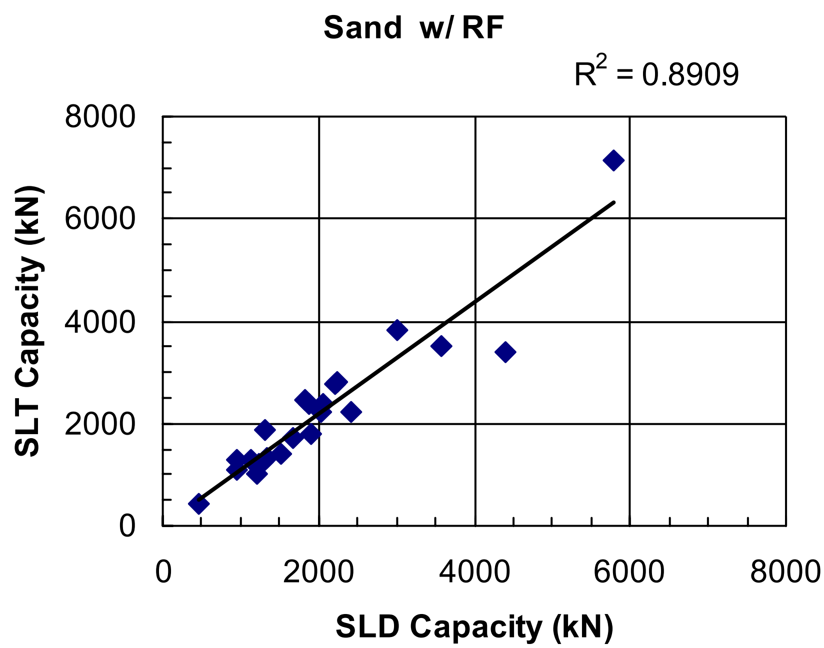
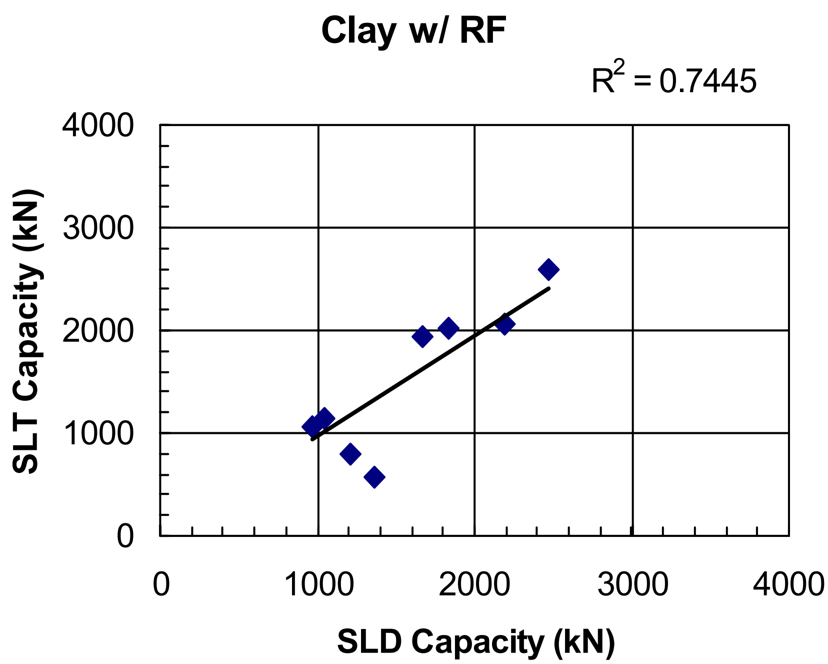


Figure 4-2. Static load test results (SLT) vs. Statnamic capacity (SLD) for sands



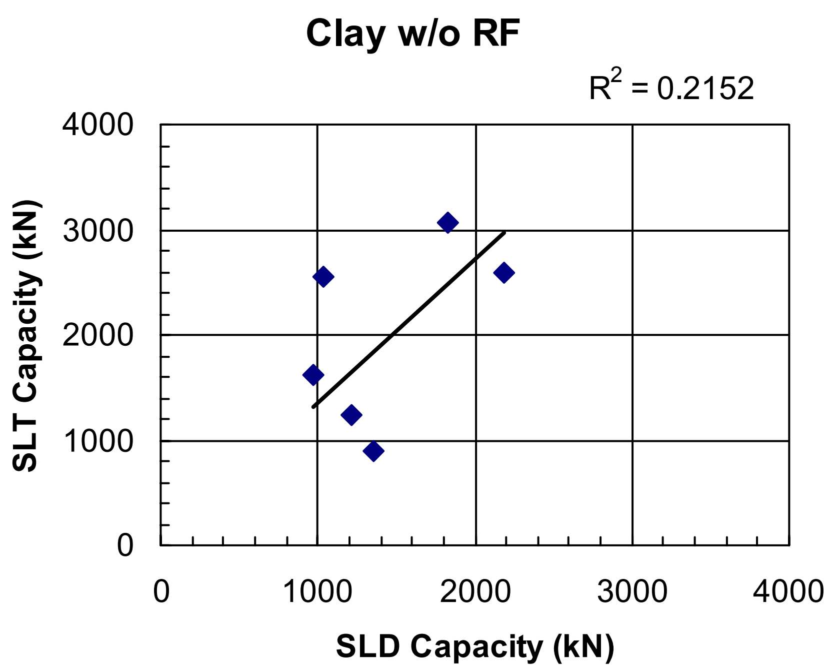
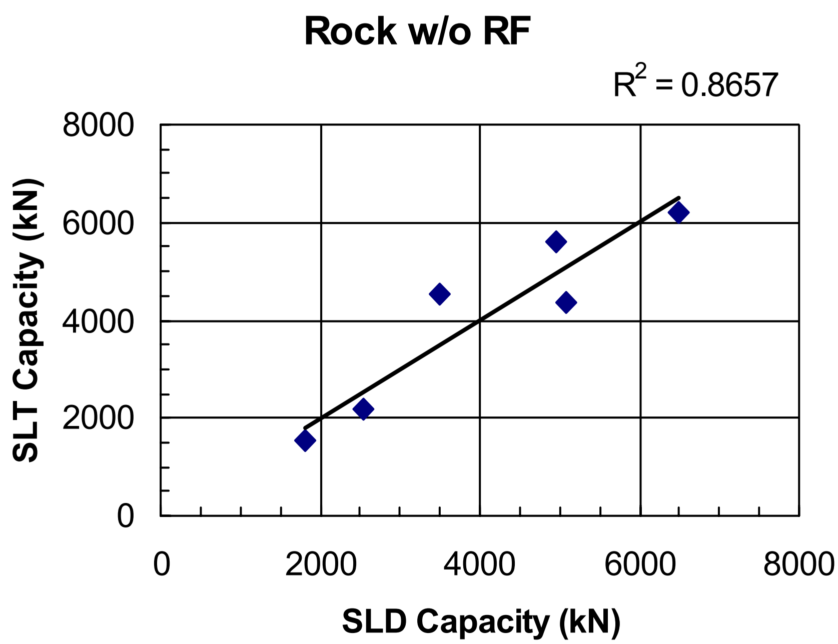


Figure 4-3. Static load test results (SLT) vs. Statnamic capacity (SLD) for clays



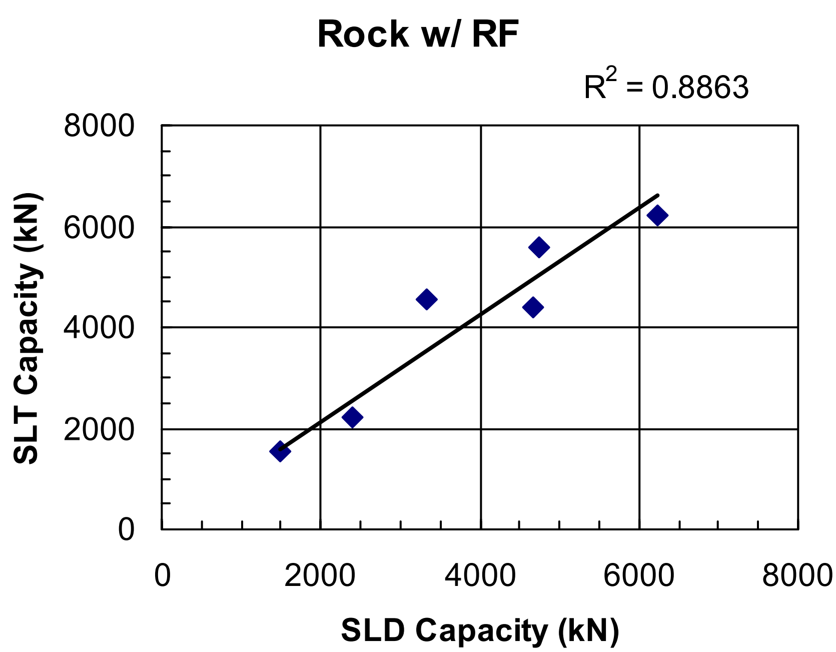
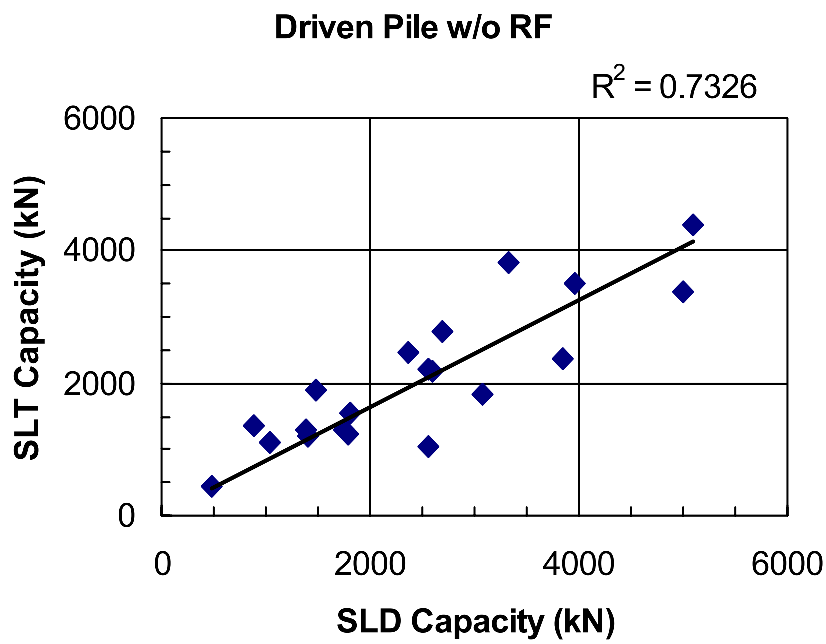


Figure 4-4. Static load test results (SLT) vs. Statnamic capacity (SLD) for rocks



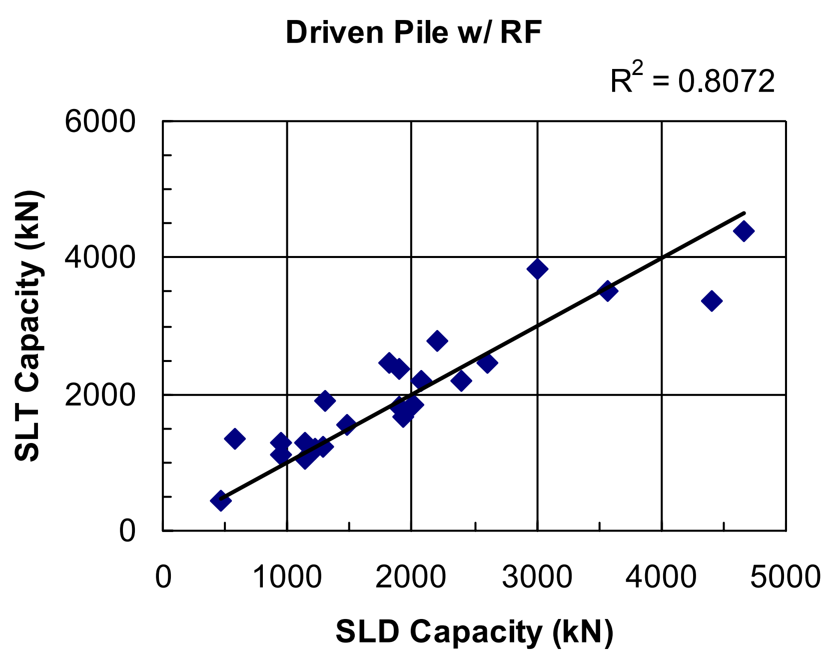
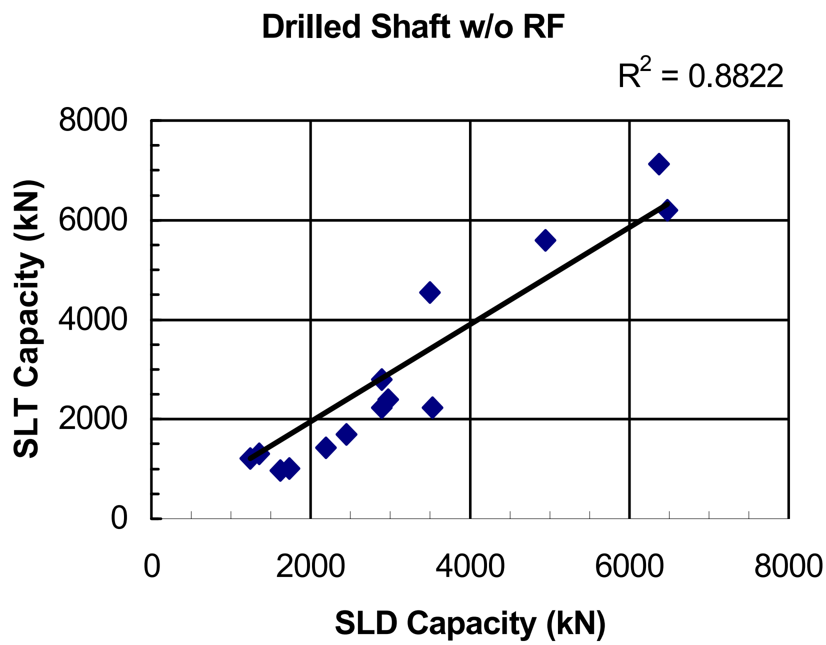


Figure 4-5. Static load test results (SLT) vs. Statnamic capacity (SLD) for driven piles with all data



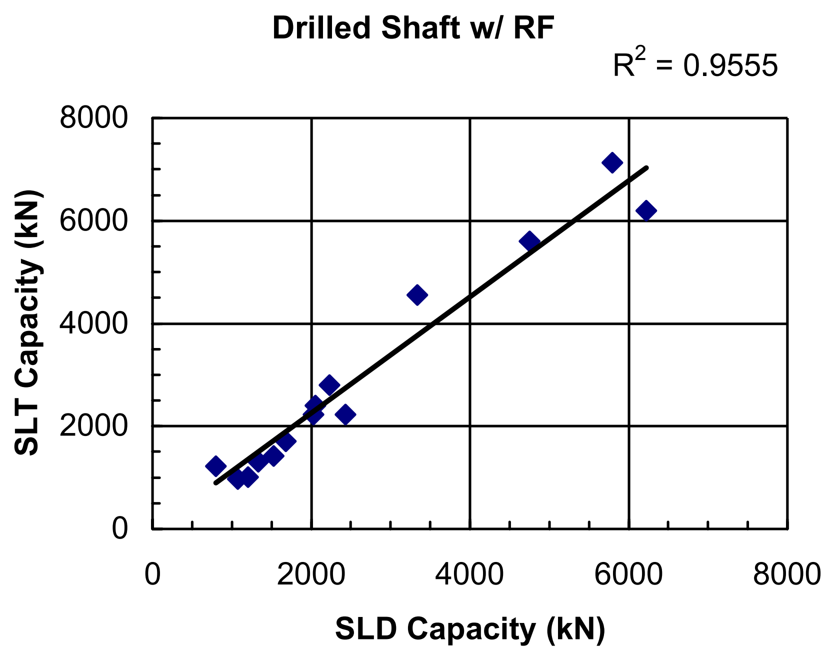
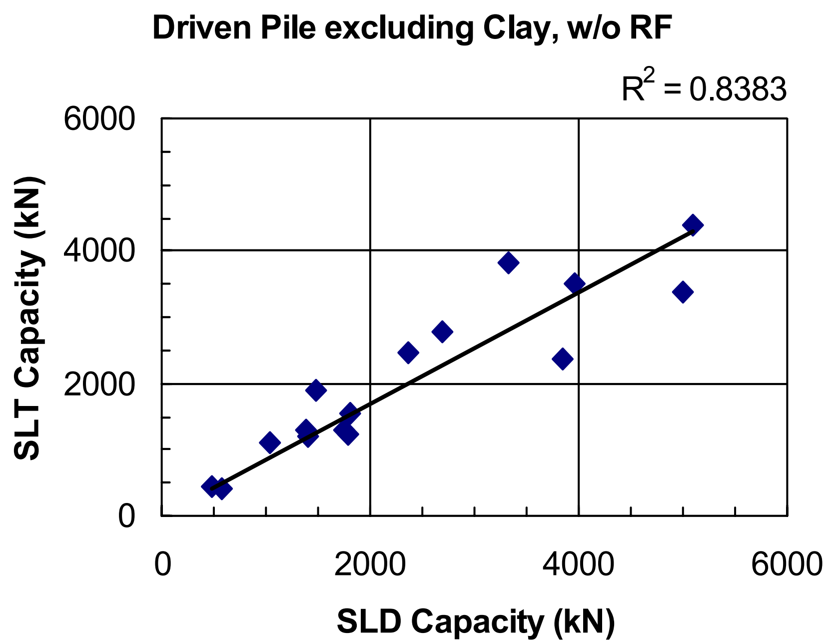


Figure 4-6. Static load test results (SLT) vs. Statnamic capacity (SLD) for drilled shaft with all data



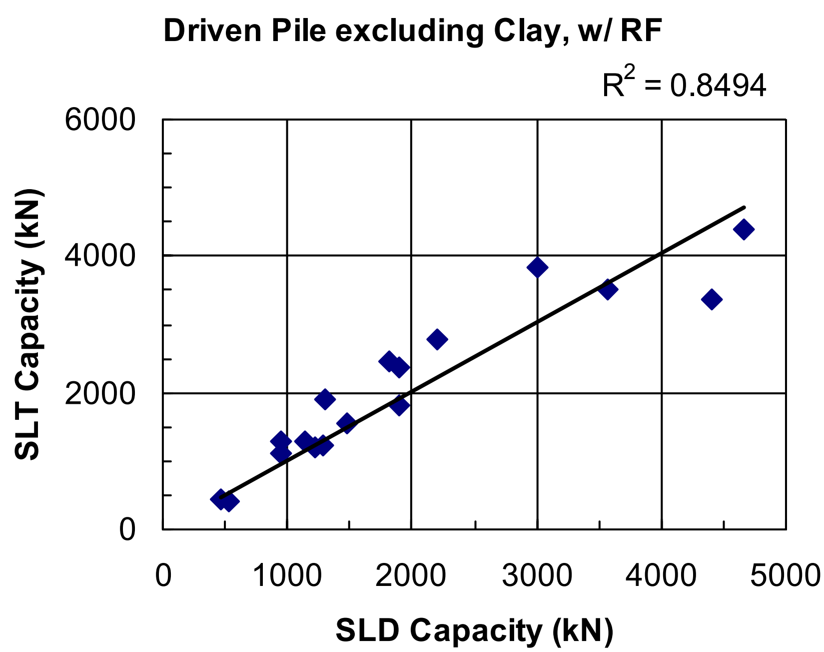
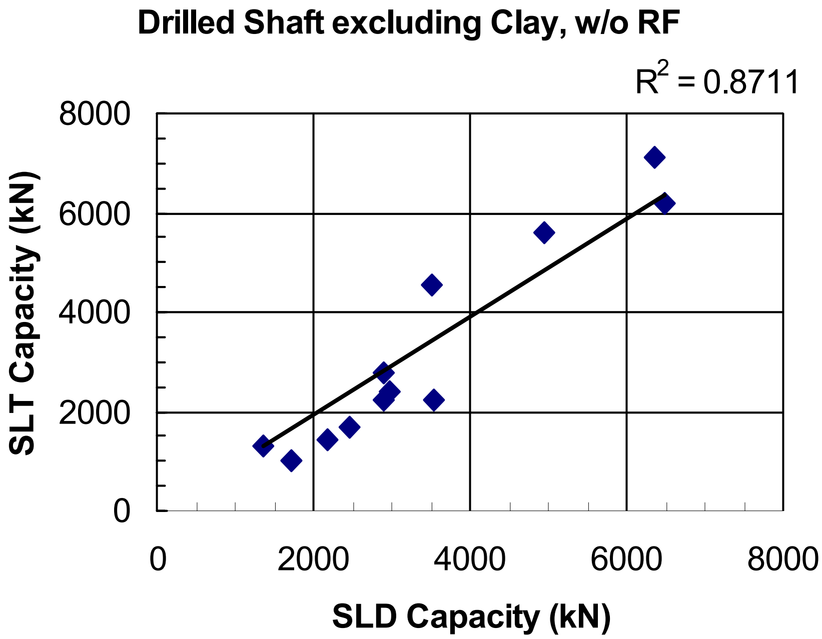


Figure 4-7. Static load test results (SLT) vs. Statnamic capacity (SLD) for driven piles excluding clays



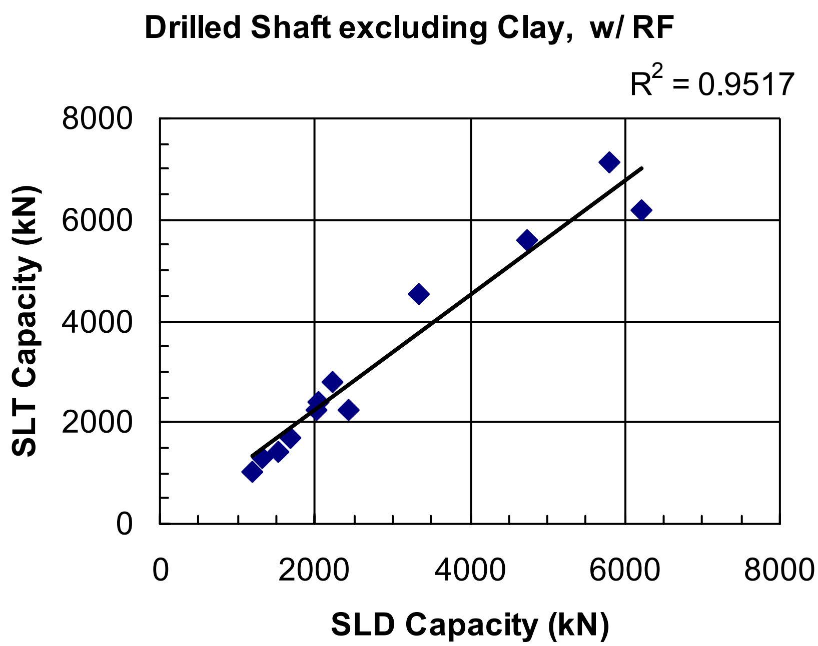


Figure 4-8. Static load test results (SLT) vs. Statnamic capacity (SLD) for drilled shaft excluding clays

Table 4-3. Summary of LRFD Calibration

Case	Resistance Factor (ϕ) w/ β=2.5				Resistance Factor (ϕ) w/ β=3.0			
	With Clay		Without Clay		With Clay		Without Clay	
	w/ RF	w/o RF	w/ RF	w/o RF	w/ RF	w/o RF	w/ RF	w/o RF
All data	0.62	0.47	0.72	0.52	0.52	0.40	0.63	0.45
Rock	-	-	0.72	0.52	-	-	0.63	0.44
Sand and silt	-	-	0.71	0.64	-	-	0.62	0.56
Clay	0.43	0.27	-	-	0.34	0.21	-	-
Drilled shaft	0.70	0.47	0.73	0.48	0.61	0.38	0.64	0.41
Driven pile	0.58	0.47	0.69	0.55	0.49	0.40	0.60	0.47

Table 4-4. Resistance Factors of Statnamic Load Testing

	Rock and Noncohesive Soils	Clays	Sand-Clay-Rock Mixed Layers
Driven pile (β = 2.5)	0.70	0.45	0.60
Driven pile (β = 3.0)	0.65	0.35	0.60

CHAPTER 5

NUMERICAL SIMULATION OF THE STATNAMIC TEST

Finite element modeling has seen tremendous growth in both design and analysis of civil infrastructures. Software such as FB-PIER is used regularly for design of bridge foundations, subject to lateral and axial static loads for both linear and nonlinear material behavior. More recently (past ten years), dynamic simulations, such as vehicle collision, barge impact, etc., are being performed with nonlinear finite element codes. For example, new roadway barrier designs are typically analyzed with the finite element code LS-DYNA. Recently, such codes have been used to model dynamic soil-structure events. An example of the latter is the Takenaka Research Group use of LS-DYNA to simulate Statnamic load testing (Yamashita et al. 2000).

One benefit of finite element modeling is its ability to perform parametric evaluations. For instance, the influence of both rate of loading, or magnitude, as well as pile/shaft geometry may be investigated individually or combined. The former is the focus of this study, since both magnitude and duration are typical variables in sizing a Statnamic test.

The study employs the LS-DYNA finite element program to perform a 3D simulation of a drilled shaft under Statnamic loading conditions. LS-DYNA is a general-purpose two or three dimensional finite element code developed for dynamic simulations of missile penetration, vehicle crashes, etc., involving significant plastic deformation, or penetration. LS-DYNA also has a number of contact-impact algorithms for penetration including the capability of remeshing.

5.1 Finite Element Model

In the study, a drilled shaft 12.5 m in length (11.5-m embedded depth) and 0.90 m in diameter was modeled in a uniform soil (silty sand) deposit. The mesh used for the analysis, Figure 5-1, was composed of 5211 nodes and 248 elements. For accuracy, a denser mesh was employed near the interface (see Fig. 5-1). The soil was characterized with LS-DYNA's elasto-plastic geologic cap model, as described in its manual. The soil parameters were characterized with an angle of internal friction of 32^0, and cohesion of 200 psf. At the boundary of the soil and drilled shaft, an interface element was employed. The latter assumes that the shaft will slide relative to the soil, if the shear exceeds its Coulombic value ($\delta = 30^0$).

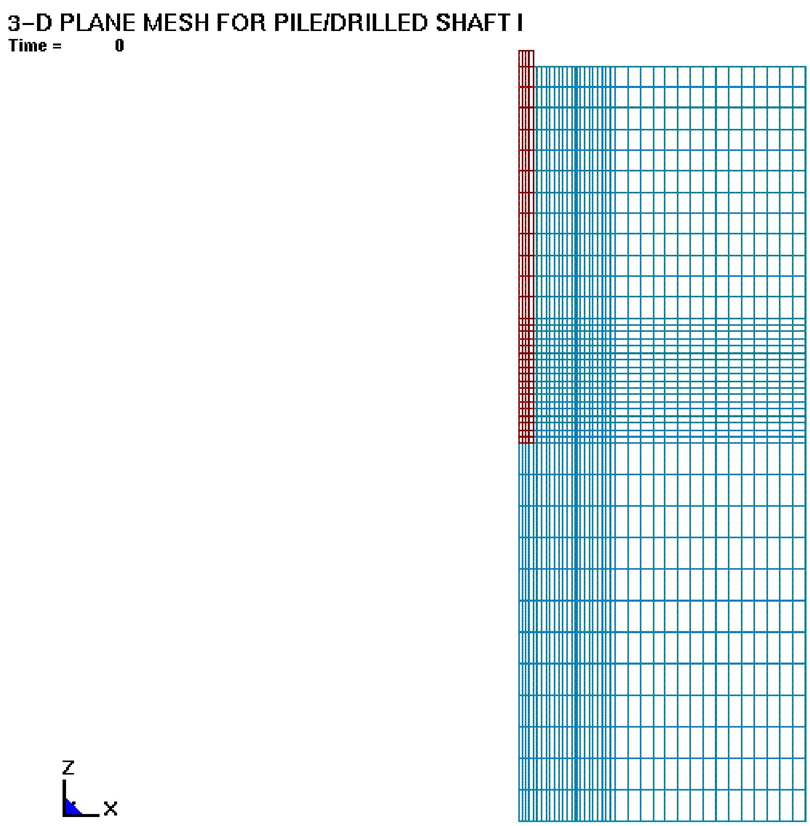


Figure 5-1. Finite element mesh

5.2 Static Load Test Simulation

A total of nine (9) Statnamic load tests and one static load test simulation were performed. All simulations were based on prior reported Statnamic and static load case histories. The static load-time history is shown in Figure 5-2. Note, the maximum load occurs at four seconds, which is much quicker than a typical static load test. However, as identified in section 2.4.1, a stress wave number greater than 1000 is considered static. For this study, i.e., a concrete drilled shaft with a length of 12.5m any load duration greater than 3.125 seconds would be considered static. The latter time (4 secs, Fig. 5-2) was selected due to the significant CPU time required (over 400 hrs) by LS-DYNA for the static analysis. As identified earlier, LS-DYNA employs an explicit time integration, which greatly increases the processing time required when running a non-dynamic simulation. LS-DYNA does offer an implicit solver; unfortunately it does not support the elasto-plastic geologic cap model use to characterize the soil.

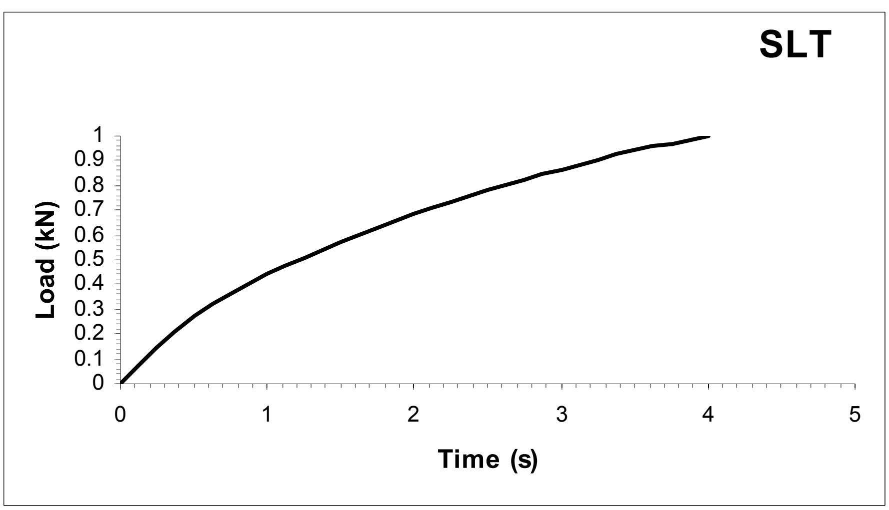


Figure 5-2. Static load history

Shown in Figure 5-3 is the static load-displacement response predicted by LS-DYNA for the static load-time plot, Fig. 5-2. Evident from Figure 5-3, the drilled shaft has not reached plunging failure. Figure 5-3 will be subsequently used to compare UPM predicted static responses from LS-DYNA simulated dynamic response to different loading rates and magnitudes.

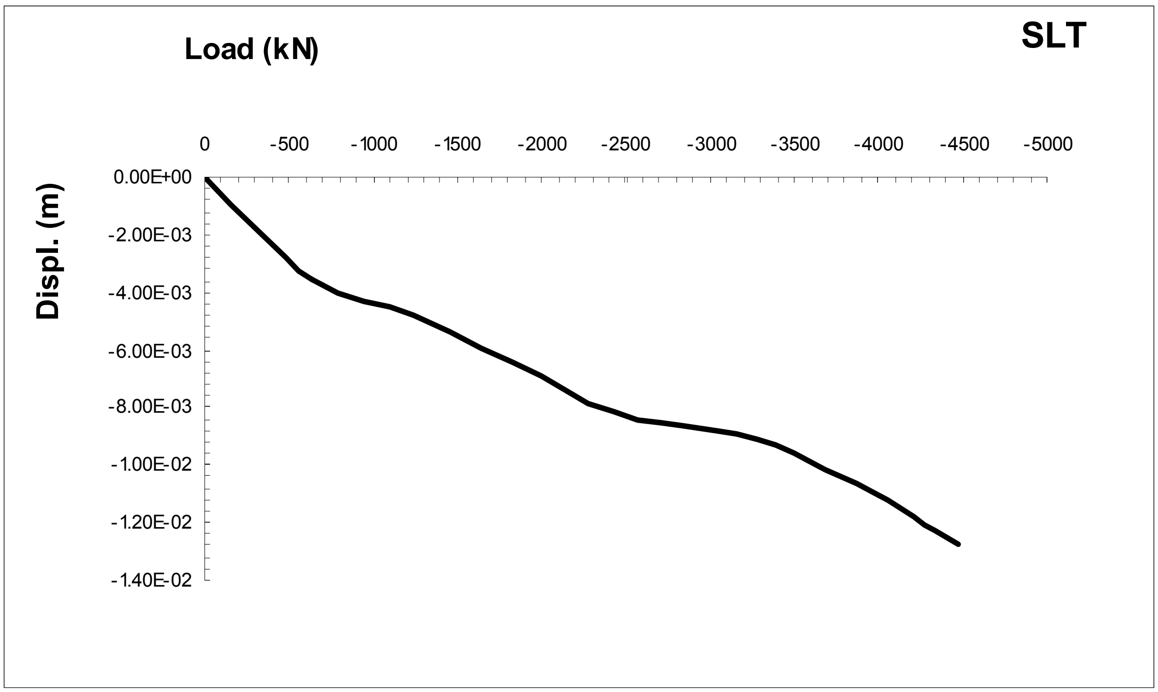


Figure 5-3. Static load-displacement response

5.3 Influence of Load Magnitude

Shown in Figure 5-4 is the normalized Statnamic load (i.e., MN/MN) vs. time curve that was applied to the top of the drilled shaft (Fig. 5-1) for the LS-DYNA simulation. For this portion of the study, only the magnitude of the load was changed, but not the load duration (x axis of Fig. 5-4). The peak load magnitude (Y axis, Fig. 5-5) was increased in individual runs from 1, to 4, 8, 12.5, and 25 MN, respectively.   For each run, the soil conditions, pile dimensions, and mesh (Fig. 5-1) were kept the same.

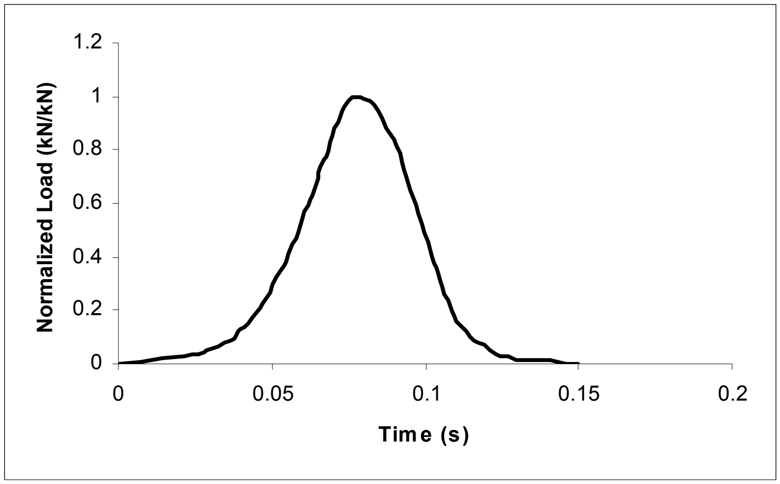


Figure 5-4. Static load vs. time curve

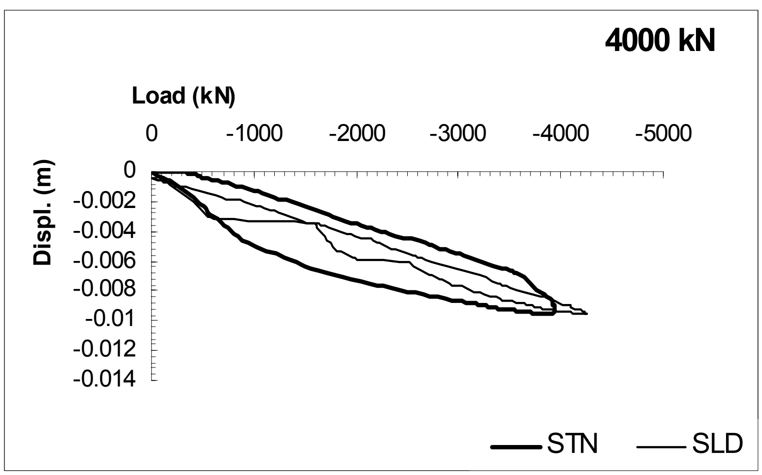


Figure 5-5. 4MN Statnamic load test

Shown in Figure 5-5 is a typical LS-DYNA predicted dynamic response (load vs. displacement, STN) for the 4 MN peak load (Fig. 5-4). The latter is equivalent to the raw measured Statnamic dynamic data, i.e., accelerometer, and the dynamic load cell measurement vs. time at the top of the drilled shaft. Also, shown in Fig. 5-5 is the back-computed static load vs. displacement curve. The derived static load-displacement curve (SLD) was computed using the University of South Florida's SAW program applied to the LS-DYNA dynamic response (Fig. 5-5). The SAW program employs Middendorp's Unloading Point Method, to estimate the shaft's static response. Note the static response is almost linear and agrees closely with LS-DYNA's predicted static load-displacement response (Fig. 5-3).

Presented in Figure 5-6 are the dynamic (STN) load vs. displacement predictions from LS-DYNA for 4, 8, 12.5, and 25 MN, peak loads (Fig. 5-4). As expected with increasing peak load over the same load duration, larger dynamic forces and resistance (inertia, damping, etc.) are developed (i.e., higher loads for same displacements).

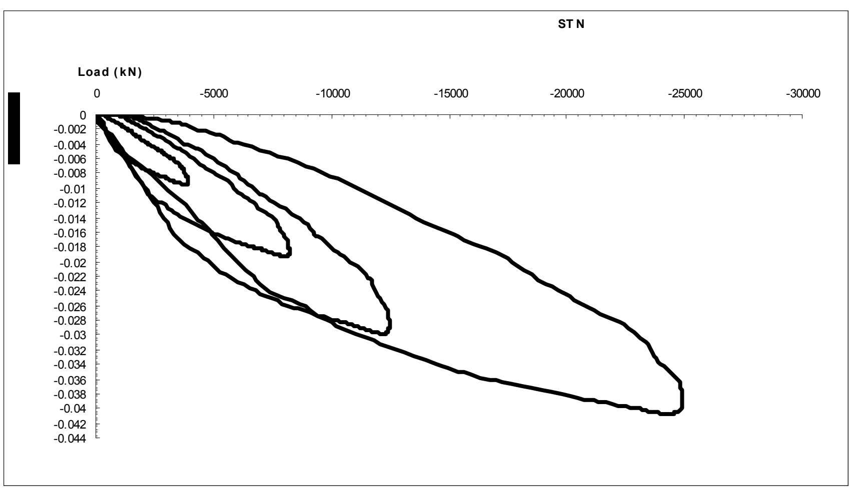


Figure 5-6. Statnamic load displacement curves of increasing maximum load

Shown in Figure 5-7 is the computed static resistance obtained from University of South Florida's SAW program. The static predicted response (SLD) for the 4, 8, 12.5 MN peak dynamic loads give similar results, and they agree closely with the LS-DYNA static prediction (Fig. 5-3). However, the derived static load response for the 25 MN load is much higher throughout the duration of the event. The latter would suggest that either the back-computed damping or inertia forces are too low. In the case of extremely high skin friction, soil would translate downward with the shaft and should be considered as part of the inertia mass of the system. If the latter were to be neglected, higher static resistance would be computed (SAW) as shown here. Since, the shaft is only 0.9 m (35.4 in.) in diameter and is sustaining a 25 MN (2,810 tons) load without failure, it is believed that abnormally high soil unit skin friction was occuring, and the latter test may not be representative of the field scenarios.

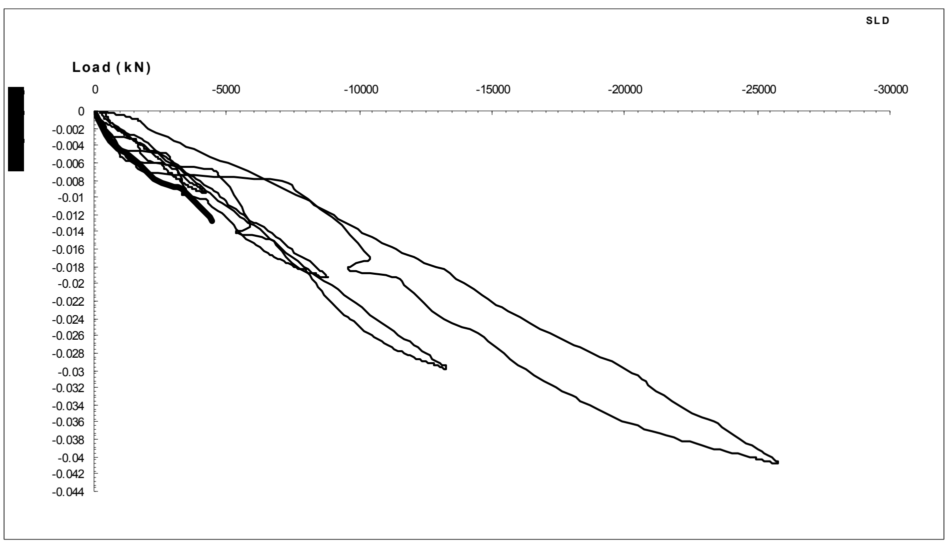


Figure 5-7. Derived static and static load displacement curves
of increasing maximum load

5.4 Influence of Load Duration

One of the advantages of Statnamic load testing besides mobilization speed/cost is the limited time to complete the test (less than a second) versus the conventional static load test (more than a day). Consequently, the influence of load duration on the predicted static response is of great interest. Shown in Figure 5-8 is load vs. duration curves available in the published literature. Generally, (Figure 5-8) load durations of 80, 90, 110, 120, and 240 milliseconds have been employed. Accordingly, LS-DYNA simulations of varying load durations (Fig. 5-8: 80, 90, 110, 120, 240 millisecond) for a 4MN maximum load were performed.

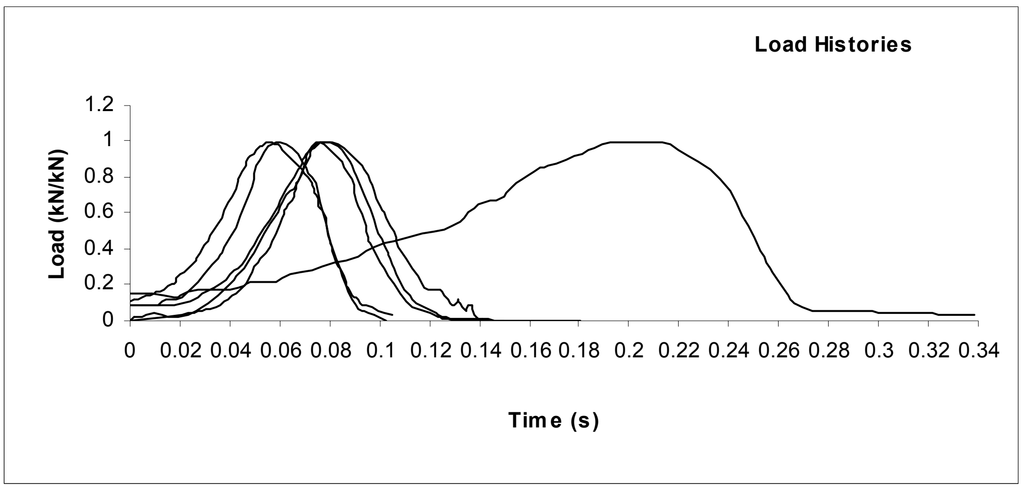


Figure 5-8. Load vs. time for varying load durations

LS-DYNA's predicted Statnamic load-displacement curves (STN) for the various load durations (Fig. 5-8) are given in Figure 5-9. Also shown in Fig. 5-9 is the derived static load vs. displacement curves (SLD) computed from the SAW program. As expected the longer the load duration, the closer the static (SLD) curves are to the dynamic curves (STN). That is the inertia, and damping forces decrease with load duration for same magnitude of loading. A comparison of the dynamic load vs. displacement curves (STN) for all simulations are given in Figure 5-10.

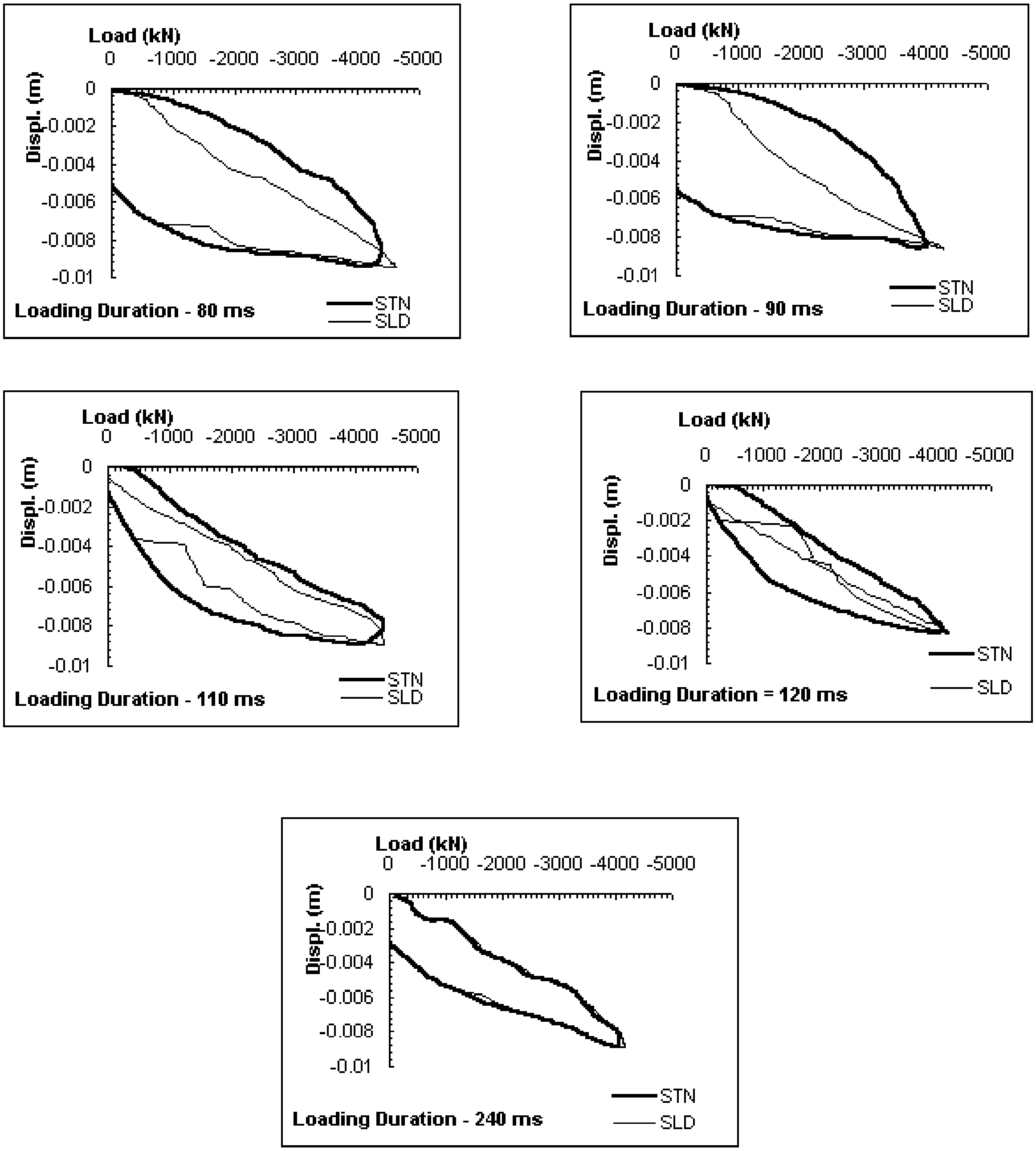


Figure 5-9. Individual derived static and static load-displacement curves for various load durations

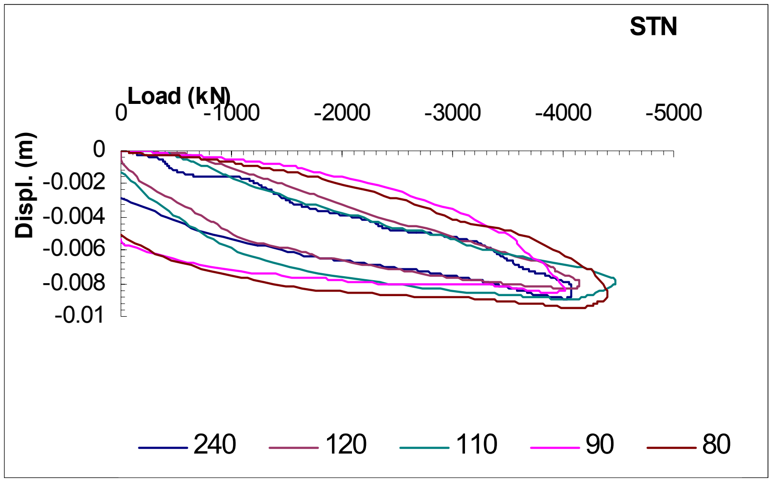


Figure 5-10. Statnamic load-displacement curves of
various load durations

Presented in Figure 5-11 are all of the derived static load-displacement (SLD) curves for the five different load duration (80-, 90-, 110-, 120-, and 240-millisecond) simulations (Fig. 5-8). Evident is that the SAW predicted static responses are all quite similar and agree with LS-DYNA's simulated static load-displacement response of the drilled shaft. It should be noted that a typical Statnamic load duration is approximately 100 milliseconds, which is well within the bounds of this study.

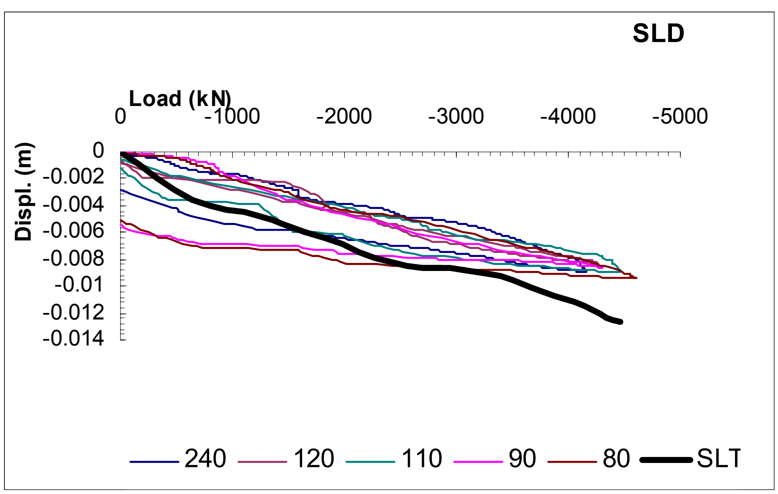


Figure 5-11. Measured and predicted static load-displacement curves

5.5 Conclusions of the Numerical Study

A numerical study of a 0.9-m diameter by 12.5-m long drilled shaft, embedded in silty-sand soil deposit was undertaken with LS-DYNA. Investigated were both the dynamic load magnitude and duration on the predicted static response of the shaft.

Studies of varying the dynamic load magnitude: 1, to 4, 8, 12.5, and 25 MN, but the same time duration of loading revealed:

- Dynamic forces (damping, inertia) increased with load magnitude due to higher particle velocities;
- Back-computed static load-displacement response of the shaft from the Unloading Point Method (UPM) gave reasonable predictions compared with static simulations (LS-DYNA) up to 12.5 MN of loading;
- UPM back-computed static resistance was higher than the static simulations in the case of high unit skin friction (i.e., rock: 7.5 tsf or 725 kPa from 25 MN loading) suggesting more inertia mass (i.e., not just the shaft) or possibly damping may be needed in the UPM analysis.

Simulations of varying the load duration time (80, 90, 110, 120, and 240 milliseconds) with a maximum dynamic load of 4MN revealed:

- Dynamic forces (damping, inertia) increased as load duration decreased due to higher particle velocities;
- For high load durations (240 milliseconds) there was little (negligible) difference between the dynamic and the static response of the shaft;

- UPM back-computed static loads vs. displacements from the shaft's dynamic response with varying load durations were all quite similar.

It should be noted that the latter dynamic analysis does not consider pore water fluid (not available in LS-DYNA) nor viscous soil behavior. Clays and/or saturated soils would be subject to one or both such phenomenon.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

Deep foundations in navigable waterways have seen a gradual change of using many smaller pile elements to larger piles or drilled shaft foundations due to higher loads (lateral) and cost. Conventional static load testing method for validation of design and construction is generally limited (cost) to 9 MN (1000 ton) of vertical load. As an alternative, Berminghammer Foundation Equipment developed the Statnamic device with a 66MN (7500-ton) capacity, in the early 1980s. Because of its cost, capacity, and ease of mobilization, the device has become a viable alternative for load testing piles/shafts. Although a large number of Statnamic Load Tests have been performed worldwide over the past 10 years, limited numbers have been conducted in combination with conventional static load tests. The latter is important for both comparison and the development of the LRFD resistance factors.

LRFD, adopted by the Florida Department of Transportation, provides a probabilistic risk based determination of resistance factors, ϕ. Using First Order Second Moment Approach, identified by Barker et al. (2000), the LRFD resistance factors, ϕ, may be determined from a database of measured conventional static capacities (FDOT 455 specification) versus back-computed static capacities from Statnamic Tests.

A total of thirty-seven (37) pairs of Statnamic and conventional static load tests on piles and drilled shafts were collected for this study. Of the thirty-seven tests, twenty-nine were in predominately cohesionless soils and eight were in cohesive (clays) soils.

For the twenty-nine (29) cases in cohesionless soils, seventeen (17) were for driven piles and twelve (12) were for drilled shafts. The population of the database is small for statistical analysis purposes. Unfortunately, the latter is all the data available (2002) throughout the world (Manufacturer, USA Statnamic load test distributor contributed to the database).

Due to the size of the database, it was decided to separate the LRFD resistance factor determination into: 1) driven piles in rock and noncohesive soils; 2) drilled shafts in rock and noncohesive soils; and 3) driven piles or drilled shafts in sands-clays-rocks mixed layers. Based on the database and probabilistic approach, the recommended resistance factors for the Statnamic load test for deep foundations are as follows:

1. Resistance factor (ϕ) for driven piles in rocks and noncohesive soils = 0.70
2. Resistance factor (ϕ) for drilled shafts in rocks and noncohesive soils = 0.65
3. Resistance factor (ϕ) for driven piles or drilled shaft in sands-clays-rocks mixed layers = 0.60
4. If drilled shafts or driven piles embedded primarily in clays, the Statnamic load test is not recommended unless the conventional static load test is also performed for calibration purposes.

Note that these resistance factors were calibrated using the rate factors identified by Mullins (2002) [Table 3-5]. It should be noted that the rate factors [Table 3-5] for noncohesive soils and rock are above 0.9, which has a significantly less impact as the rate factor (0.65) for cohesive soils.

Numerical analysis of the Statnamic test applied to a drilled shaft founded in silty sand (no water) under varying load durations (80 to 240 milliseconds) revealed little if any difference in the predicted static capacity when back computed from the unloading point method. Varying the magnitude of dynamic load (1 to 25 MN) under the same duration of loading, showed little if any difference in the back-computed static capacity up to and including 12.5 MN. However in the case of 25 MN and high unit skin friction (i.e., rock: 7.5 tsf or 725 kPa), the UPM back-computed static resistance was higher. It was concluded that either more inertia mass (i.e., not just the shaft, but adjacent soil-rock material) or possibly damping was needed in the UPM analysis.

It is recommended that the FDOT continue to collect both Statnamic and conventional static load tests data especially in cohesive soils to increase the database and refine the LRFD resistance factor, ϕ assessment. It is also suggested that further numerical analysis which include rate dependent soil and rock models, as well as pore pressure representation for saturated poorly draining soils (clays, clayey silts) be undertaken, given the proposed Rate Factor (0.65) for cohesive soils.

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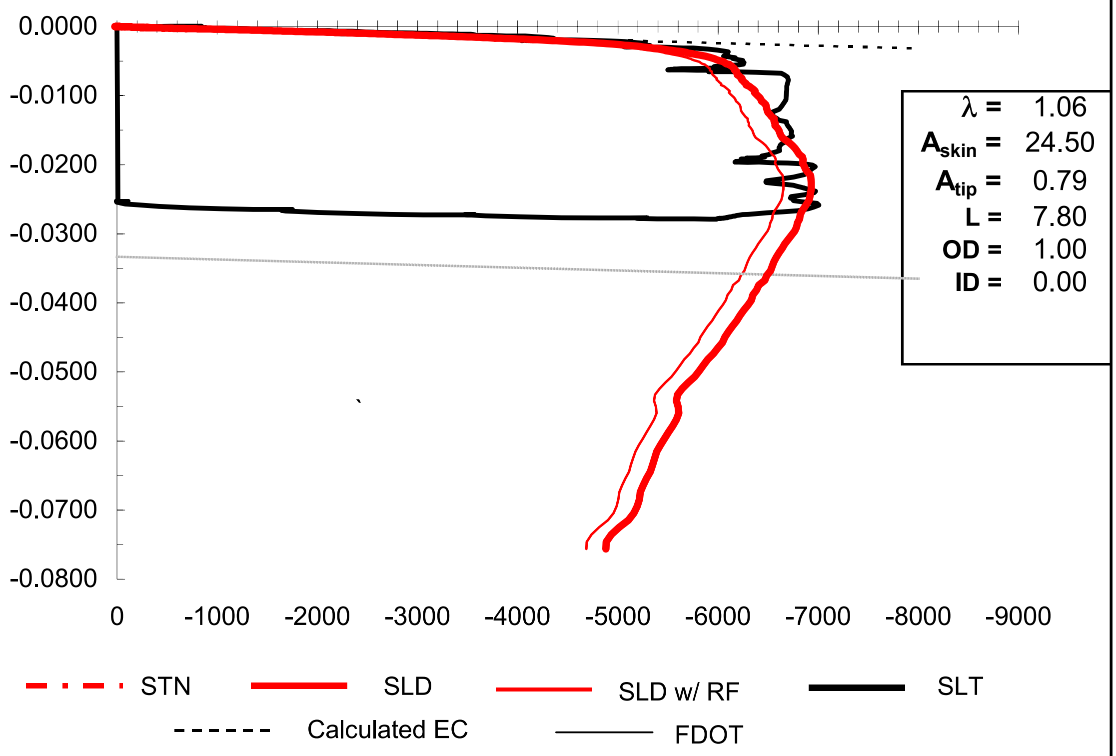
APPENDIX

LOAD-DISPLACEMENT CURVES OF STATNAMIC LOAD TESTING

STATNAMIC TEST SUMMARY SHEET

1	1	2	0	3	1	-	USA	FLLS	TP-1/2

Job Name: Florida LS1
Job Location: Florida



General Soil Profile - Boring B-8

0 to 1.5 m - Silty Sand (SP-SM) Avg N=6
1.5 to 2.5 m - Clayey Sand (SP-SC) Avg N=11
2.5 to 4.5 m - Weak Limestone Avg N=13
2.5 to 30.0 m - Hard Limestone Avg N>50

** Pile Tip @ 7.8 m**
Water table below pile tip

Additional Project Information

Static tests were via anchored reaction beam using QUICK method.

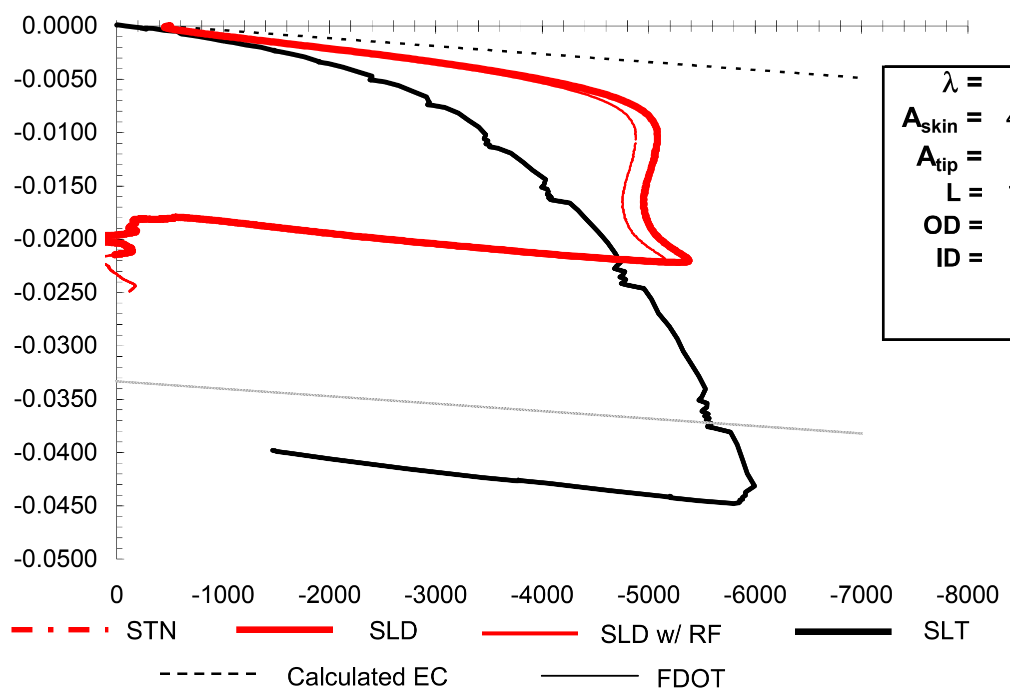
Only first 3 inches of displacement shown for STN test.

STN data unavailable

STATNAMIC TEST SUMMARY SHEET

1	1	2	0	3	1	-	USA	FLLS	TP-3/4

Job Name: Florida LS2
Job Location: Florida



λ =	0.92
A_{skin} =	43.04
A_{tip} =	0.79
L =	13.70
OD =	1.00
ID =	0.00

General Soil Profile - Boring B-8

0 to 2.4 m - Silty Sand (SP-SM) Avg N=3
2.4 to 4.9 m - Sandy Clay (CL) Avg N=7
4.9 to 8.2 m - Soft Clay N=7 to W.O.H.
8.2 to 23.0 m - Weak Limestone Avg N=23

** Pile Tip @ 13.7 m**
Water table 9.5 m

Additional Project Information

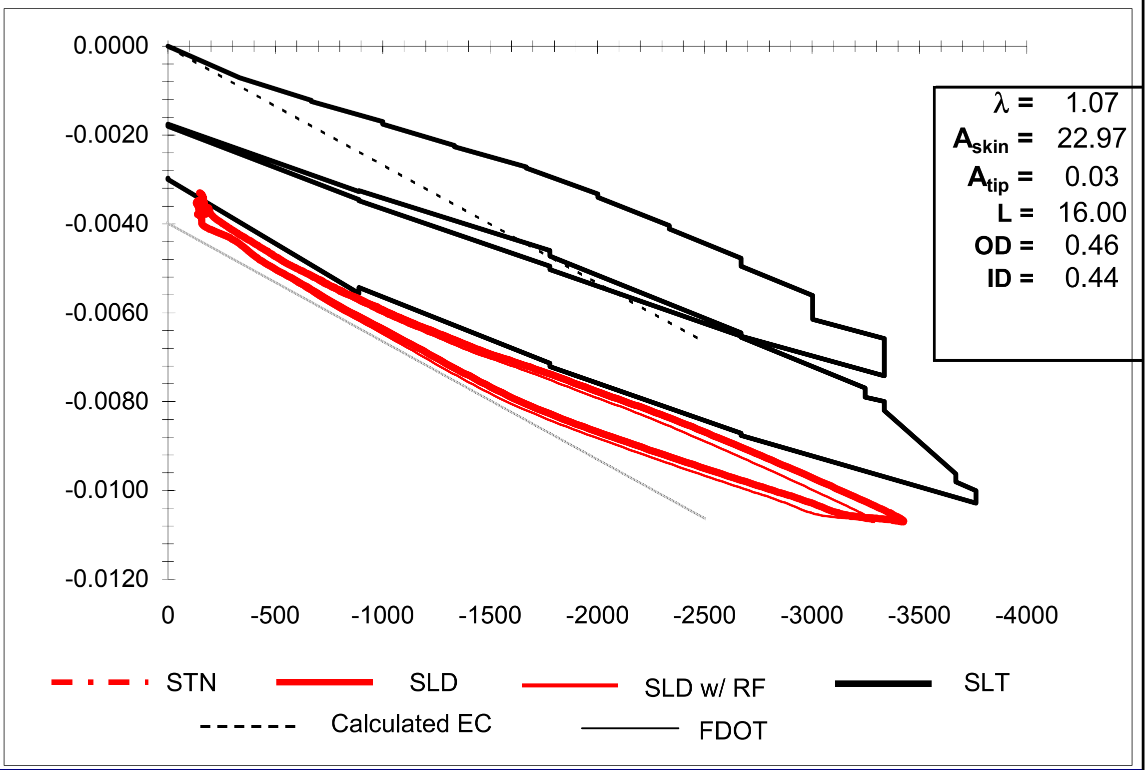
Static tests were via anchored reaction beam using QUICK method.

STN curve not available.

STATNAMIC TEST SUMMARY SHEET

4	1	1	2	3	2	-	USA	JFK	MSF P-1

Job Name: JFK Airport Light Rail System
Job Location: Queens, New York



General Soil Profile - Boring 3-945

0-4 m - fill (sand,gravel,cinders)
4 to 4.2 m - Organic Clay and Peat N=W.O.R.
4.2 to 35 m - Brown medium fine Sand w/ trace Silt Avg N=30

Additional Project Information

Non-uniform polygon (12 sided) steel pile with 18 inch top dia and 8 inch bottom diameter. Known as Tapertube Pile (patent pending). Driven then filled with concrete. Strain data from STN shows less than 10% EB. Static tests were via kentledge with 1 hour holds and 24 hour hold at max load.

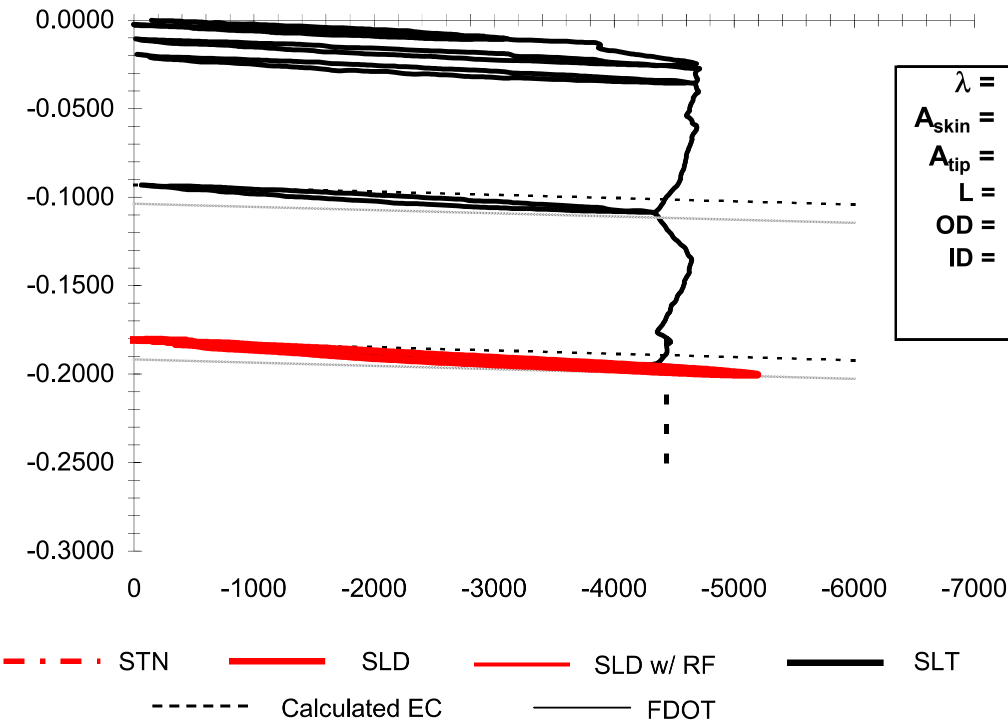
Note: Skin area above wrong due to non-uniform pile.

STN curve not available.

STATNAMIC TEST SUMMARY SHEET

4	1	1	2	4	3	-	JPN	NNO	T1

Job Name: Noto Test Site
Job Location: Noto Peninsula, Japan



λ =	0.89
A_{skin} =	27.65
A_{tip} =	0.03
L =	11.00
OD =	0.80
ID =	0.78

General Soil Profile

1m of surficial clay, founded in 10 m of soft rock with N=15

Additional Project Information

Pile was driven through a soft rock known as Wakura Diatomaceous Mudstone.

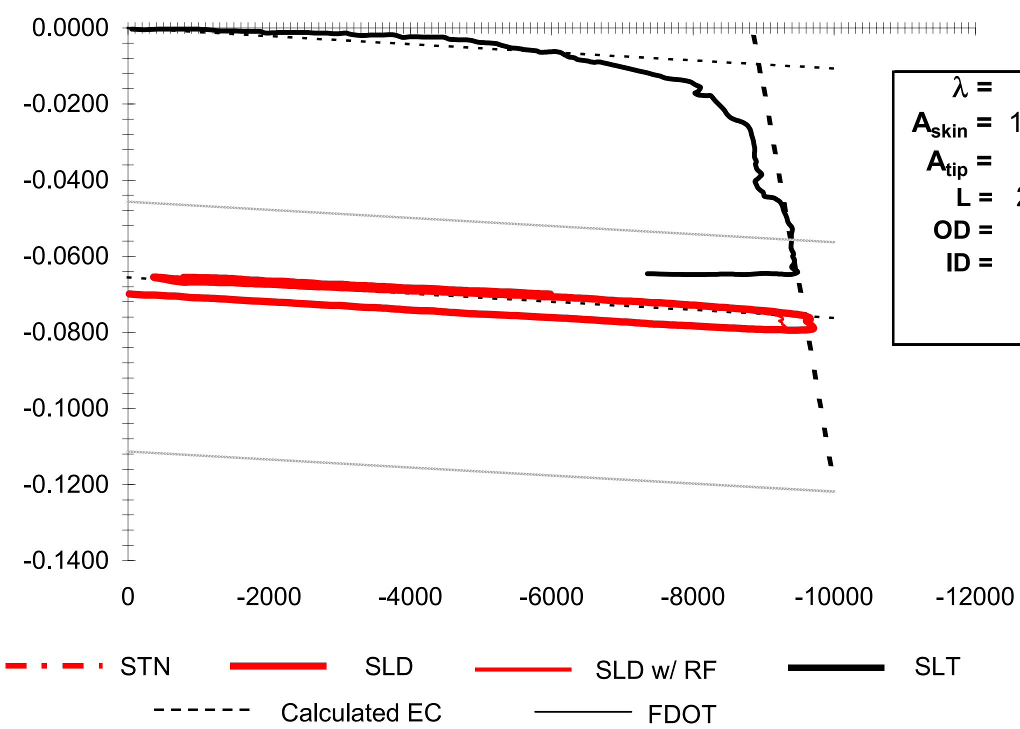
Statnamic testing was carried out 14 months after a cyclic Static Load test was conducted.

STN curve not available

STATNAMIC TEST SUMMARY SHEET

2	1	1	2	4	2	-	USA	STG	LT-1

Job Name: St. George Island Bridge
Job Location: St. George Island, FL



λ = 1.03
A_{skin} = 104.74
A_{tip} = 0.75
L = 24.30
OD = 1.37
ID = 0.97

General Soil Profile

-5-0 m Stickup
0-4 m Water
4-9 m Silty Fine Sand N = WOR
9-14 m Silty Fine Sand N = 5 to 30
14-19.5 m Silty Fine Sand N = 30 to 50
Tipped in 50+ Limestone

Additional Project Information

Concrete Cylinder Driven Piles.

Kentledge Static Load test was conducted prior to Statnamic testing.

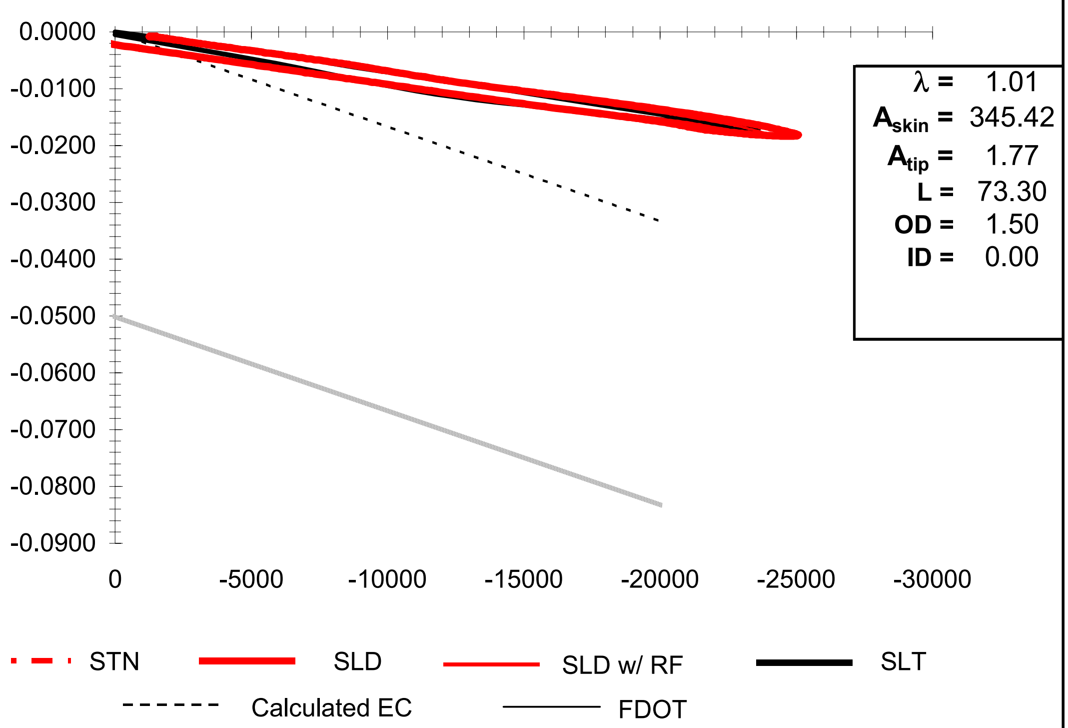
Load Carried primarily in end bearing

STN curve not available.

STATNAMIC TEST SUMMARY SHEET

1	1	1	2	3	1	-	TWN	TFC	241

Job Name: Taipai Financial Center
Job Location: Taiwan

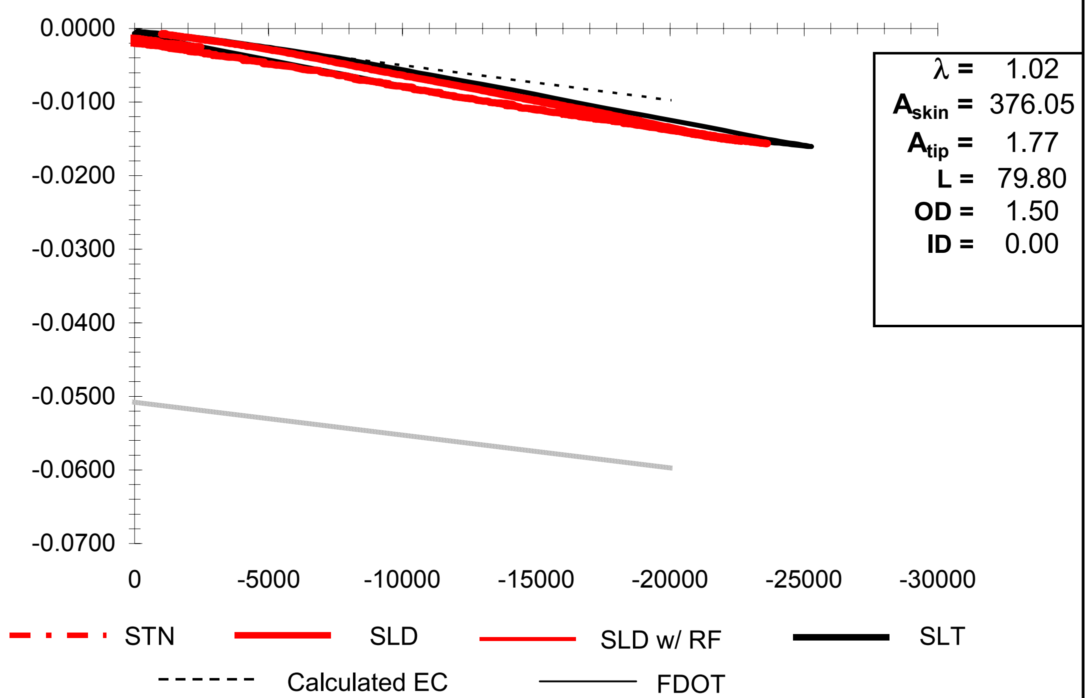


General Soil Profile	**Additional Project Information**
0-6m fill 6-27m grey silty-clay with silty sand layers N<5 27-45m grey silty-clay N<25 45-52m gravel and sand with silt N~80 52-59m mod. weathered sandstone N>100 RQD 50-90% 59-67m mod. Weathered shale N>100 RQD 50-90%	shaft construction - Jan.10, 1999 SLT March 12-14, 1999 STN March 16, 1999 STN curve not available.

STATNAMIC TEST SUMMARY SHEET

1	1	1	1	3	1	-	TWN	TFC	532

Job Name:
Job Location:

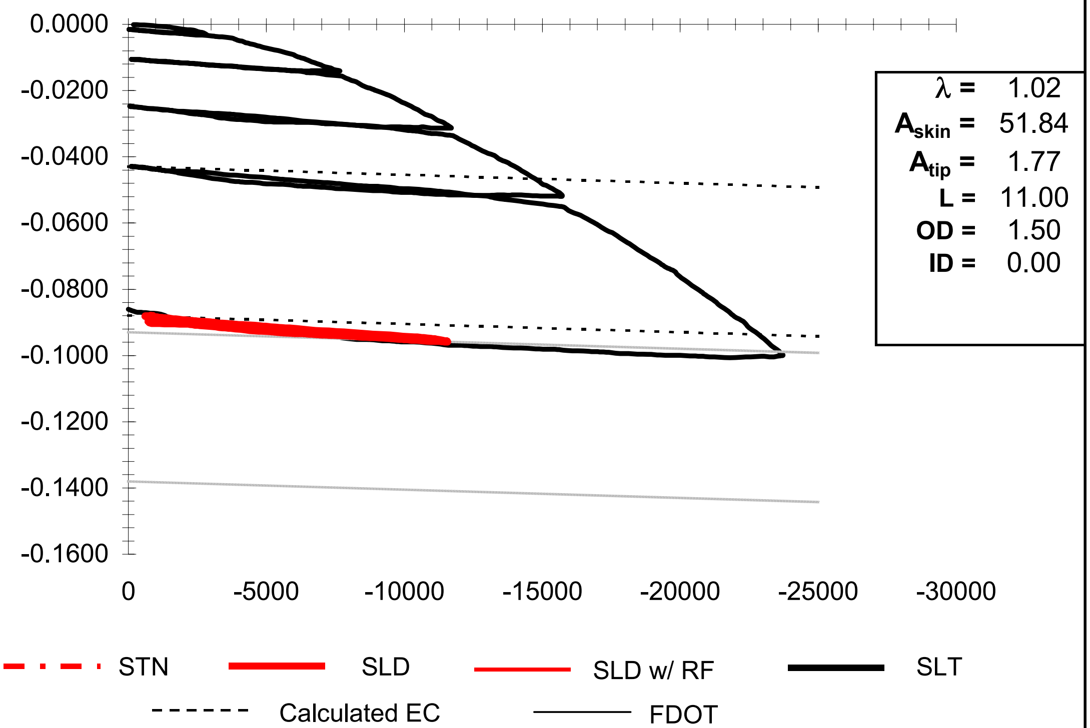


General Soil Profile	Additional Project Information
0-6m fill 6-27m grey silty-clay with silty sand layers N<5 27-45m grey silty-clay N<25 45-52m gravel and sand with silt N~80 52-59m mod. weathered sandstone N>100 RQD 50-90% 59-67m mod. Weathered shale N>100 RQD 50-90%	shaft construction - Jan.4, 1999 SLT March 29-31, 1999 STN March 10, 1999 STN curve not available.

STATNAMIC TEST SUMMARY SHEET

1	2	1	2	3	3	-	JPN	OHITO TEST

Job Name: Ohito Bridge
Job Location: Shizouka Prefecture, Japan



General Soil Profile

10.5 m of sandy gravel with boulders, with intermediate layer of silt and silty sand.

Additional Project Information

The Static Load Test (SLT) was performed four weeks after pile installation.

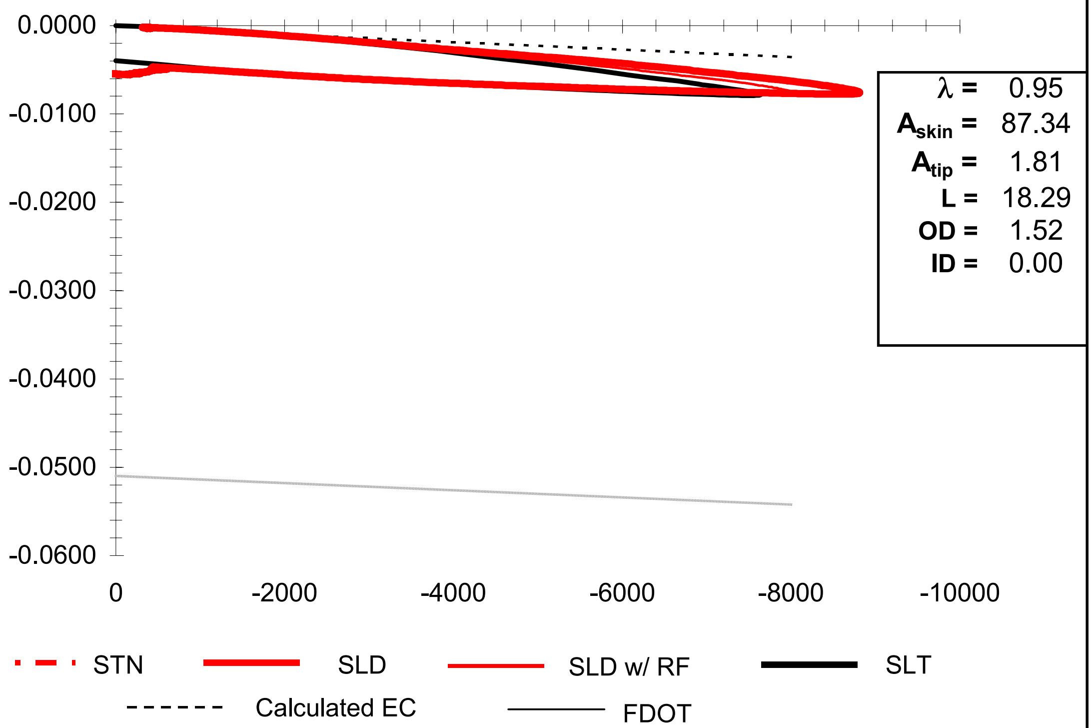
Statnamic testing commenced ten days after full release of SLT load, with target load of 12 MN.

STN curve not available.

STATNAMIC TEST SUMMARY SHEET

1	2	2	0	2	1	-	USA	BQE SA 1&8

Job Name: BQE
Job Location: Queens, NY



General Soil Profile

0 to 1.5 m - Sandy Gravel Avg N=25
1.5 to 13.5 m - Silty Sand w/gravel Avg N=25
13.5 to 18.3 m - Silty fine Sand w/ mica N=30
18.3 to 19.0 m - Silty fine Sand w/ gravel N=100
Shaft Tip at 19 m
19.0 to 27 m - Silty fine Sand w/ gravel N=100

Additional Project Information

Tell tale data from static test indicated Zero end bearing. Strain data from Statnamic test indicated 51 tons of end bearing. Static load test was via anchored reaction beam.

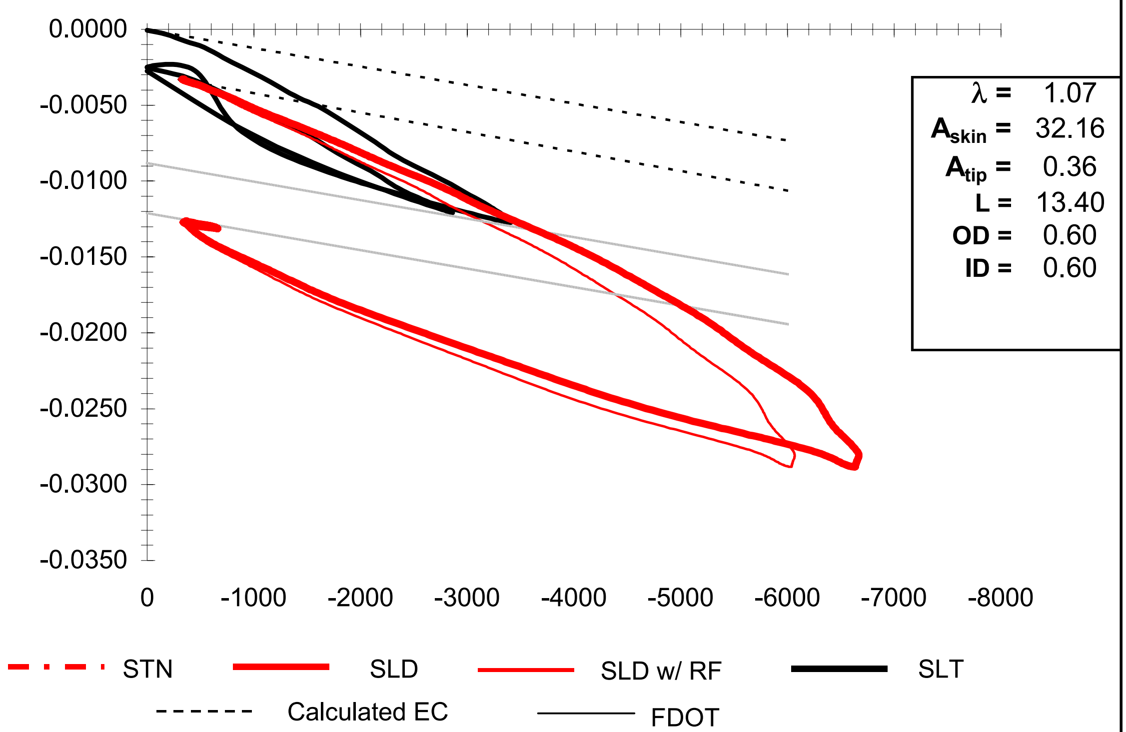
Note: Total skin friction from Statnamic test plotted against static test.

STN curve not available.

STATNAMIC TEST SUMMARY SHEET

2	2	1	2	3	2	-	USA	BC	Pier10

Job Name: Bayou Chico
Job Location: Pensacola, Florida



General Soil Profile

Loose Sand at ground elevation to very dense sand at depths of 12-15 m

Additional Project Information

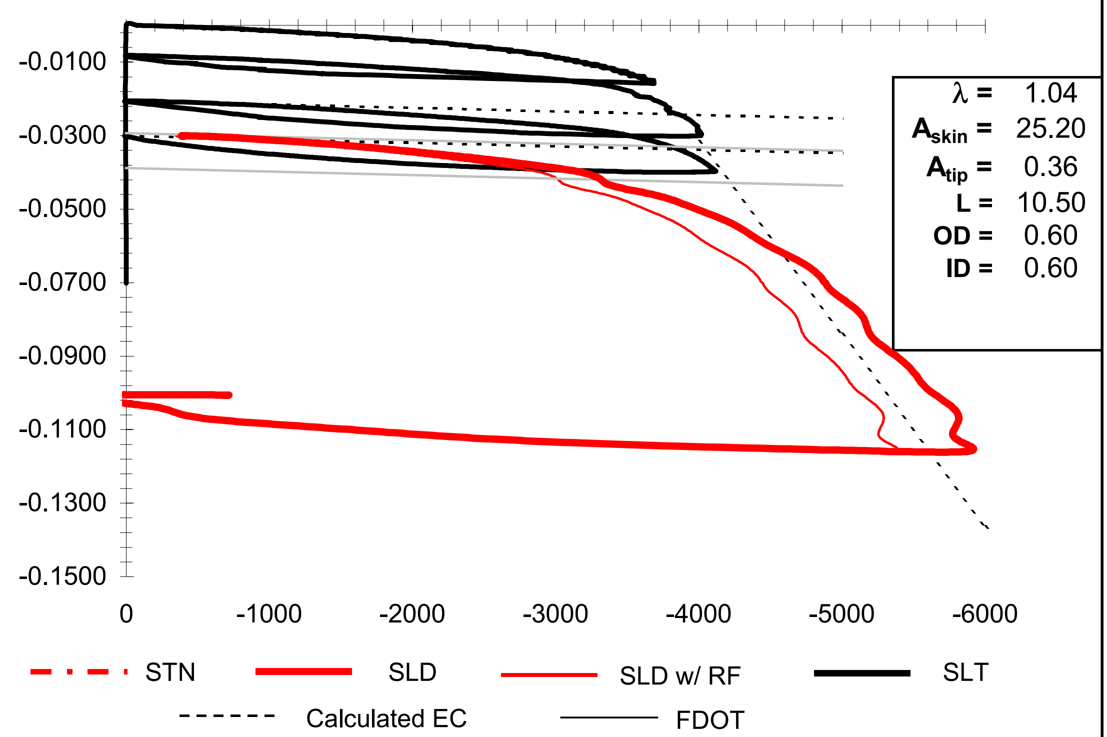
Pile was over water.

STN curve was not available.

STATNAMIC TEST SUMMARY SHEET

2	2	1	2	4	2	-	USA	BC	Pier 15

Job Name: Bayou Chico Bridge
Job Location: Pensacola, FL

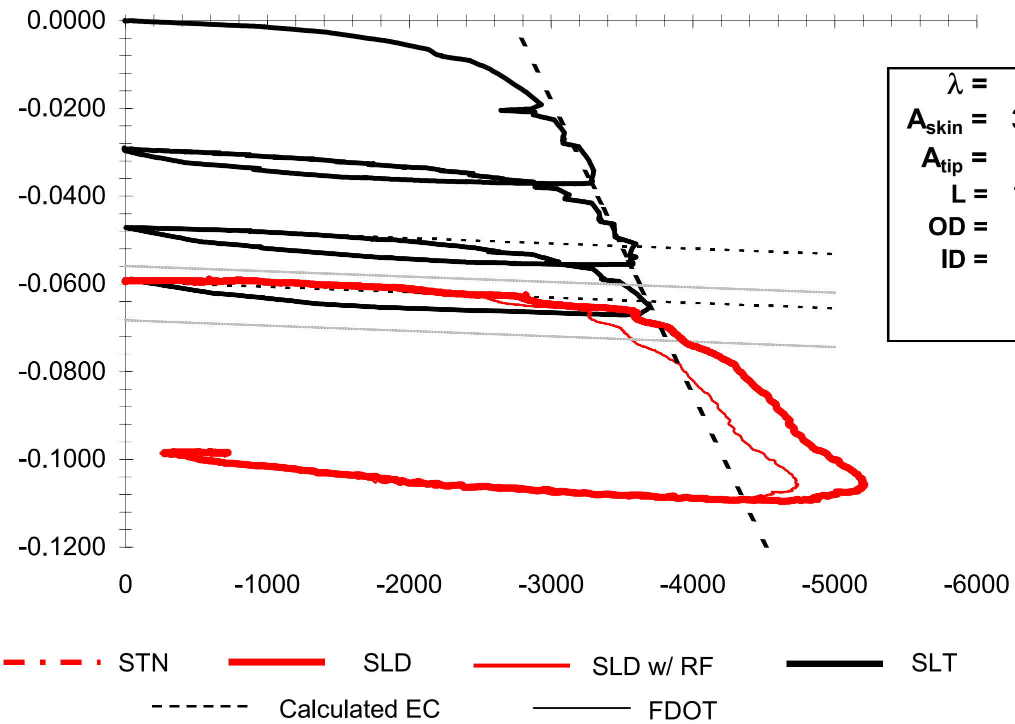


General Soil Profile	**Additional Project Information**
Loose sands at ground elevation to very dense sand at depths of 12-15 m	Three Static load tests were performed two months prior to a single cycle of Statnamic testing. STN curve not available.

STATNAMIC TEST SUMMARY SHEET

2	2	1	2	4	2	-	USA	BC	Pier 5

Job Name: Bayou Chico Bridge
Job Location: Pensacola, FL



λ =	0.91
A_{skin} =	32.16
A_{tip} =	0.36
L =	13.40
OD =	0.60
ID =	0.60

General Soil Profile

Loose Sand at ground elevation to very dense sand at depths of 12-15 m

Additional Project Information

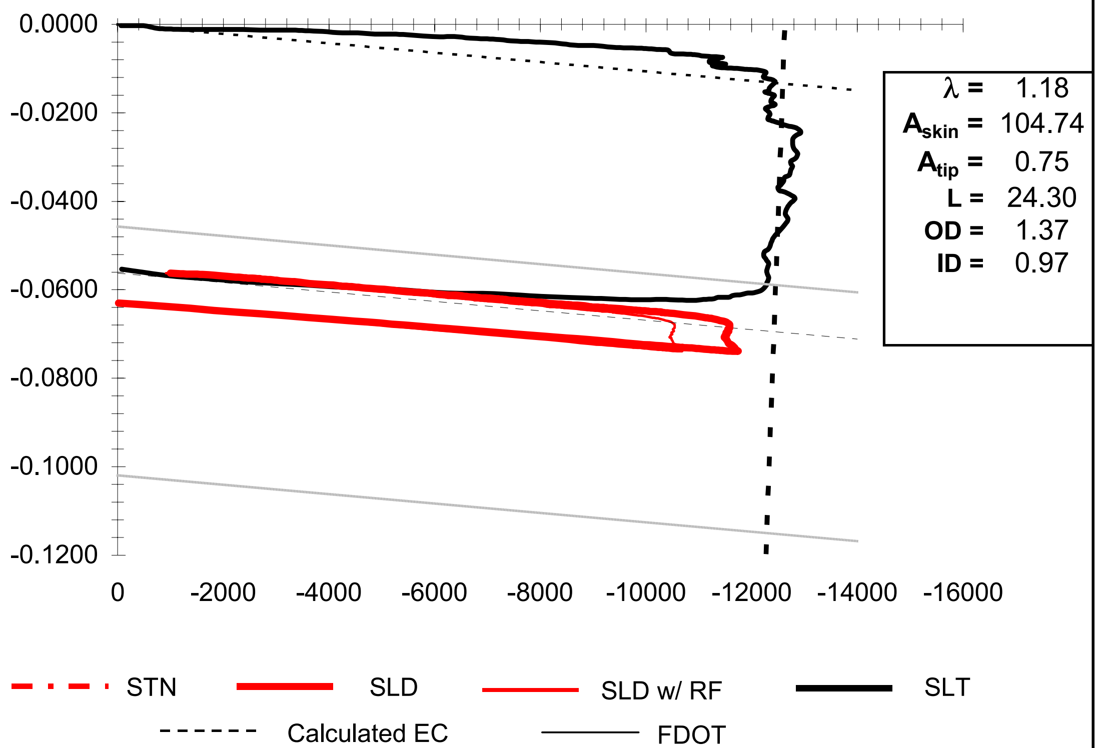
3 cycles of Static Load Testing were conducted prior to 1 cycle of Statnamic Testing

STN curve not available.

STATNAMIC TEST SUMMARY SHEET

2	2	1	2	4	2	-	USA	STG	LT-5

Job Name: St. George Island Bridge
Job Location: St. George Island, FL

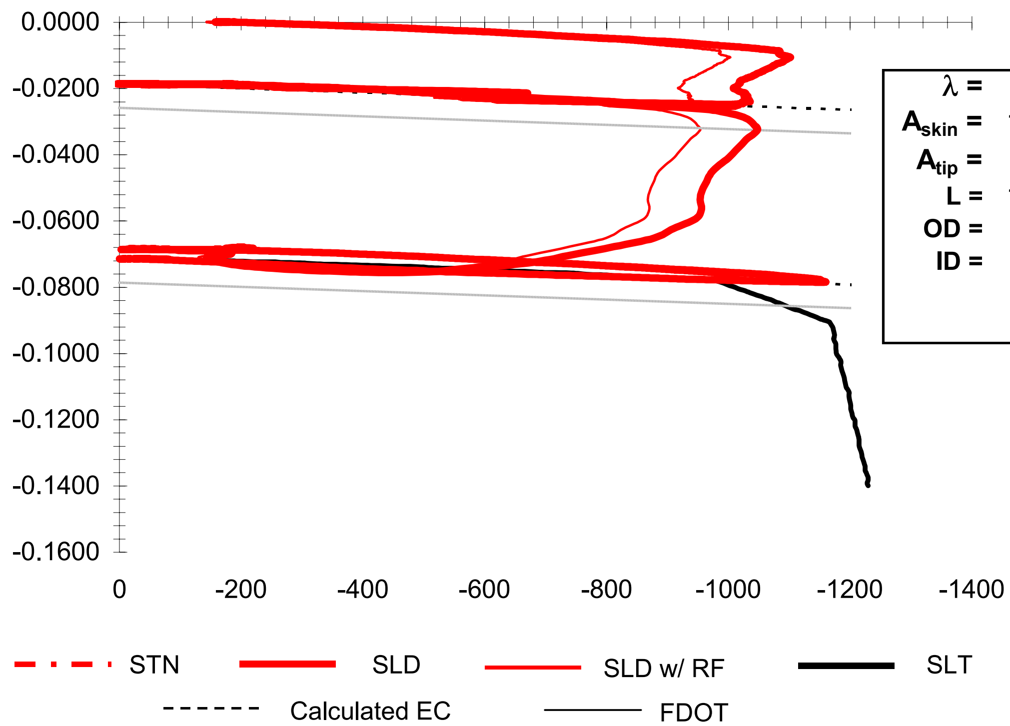


General Soil Profile	Additional Project Information
-4-0 m Stickup 0-6 m Water 6-10 m Silty Fine Sand N = W.O.R. to 5 10-14.5 m Silty Fine Sand N = 20 to 30 14.5-20.5 m Weak Limestone N = 10 to 30	Concrete Cylinder Driven Piles. Kentledge Static Load test was conducted prior to one cycle of Statnamic testing. Load Carried Primarily in side shear. STN curve not available.

STATNAMIC TEST SUMMARY SHEET

4	2	1	1	3	2	-	JPN	;HONAI	T6

Job Name: Shonan Test Site
Job Location: Shonan-Machi, Ciba Prefecture, Japan



λ =	0.97
A_{skin} =	16.34
A_{tip} =	0.01
L =	13.00
OD =	0.40
ID =	0.38

General Soil Profile

0-2 m Loam N = 2
2.5-6.5 m Clay N = 5
6.5-13 m Fine Sand N = 18
13-15 m Fine Sand N = 19
15-20 m Fine Sand N = 30

Additional Project Information

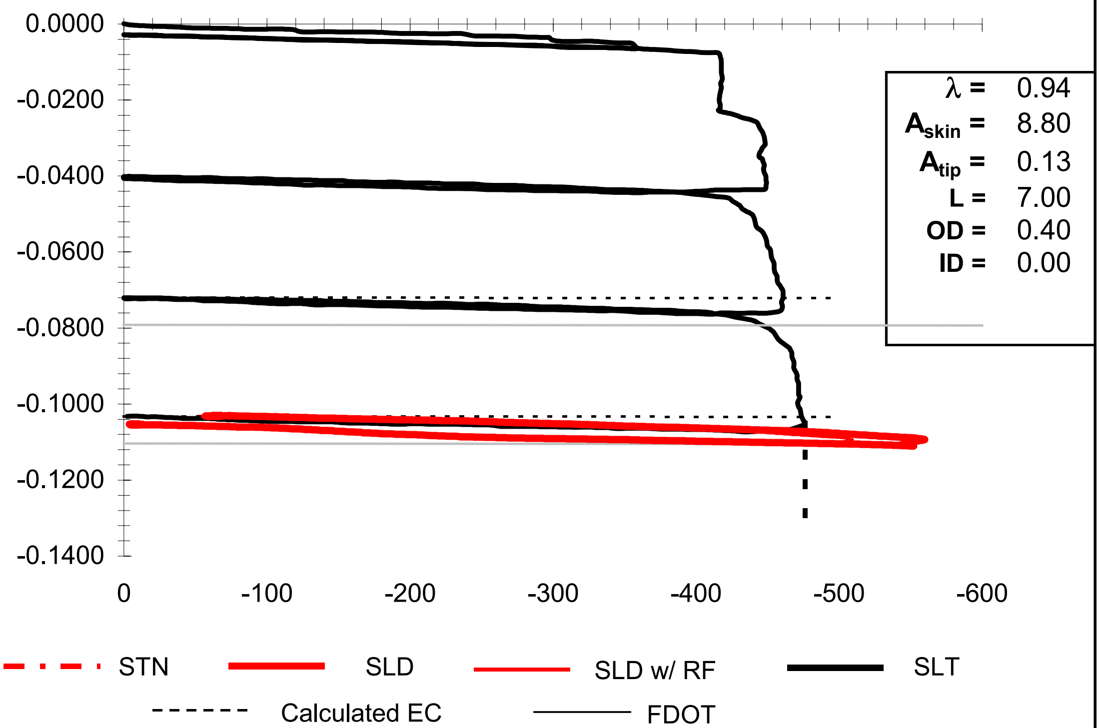
The first Statnamic test was conducted four weeks after installation, with two more Statnamic tests conducted two weeks later. Static load testing was conducted two weeks after final Statnamic test.

STN curve not available.

STATNAMIC TEST SUMMARY SHEET

6	2	1	2	4	2,3	-	JPN	;HONAI	T5

Job Name: Shonan Test Site
Job Location: Shonan-Machi, Ciba Prefecture, Japan

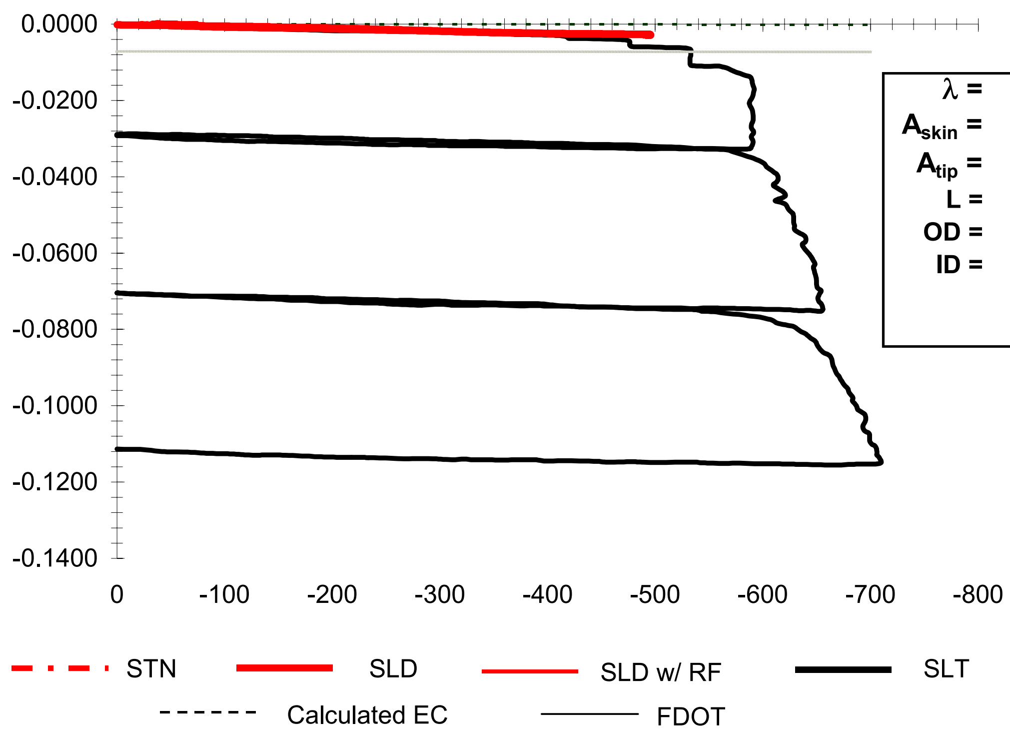


General Soil Profile	**Additional Project Information**
0-2 m Loam N = 2 2.5-6.5 m Clay N = 5 6.5-13 m Fine Sand N = 18 13-15 m Fine Sand N = 19 15-20 m Fine Sand N = 30	Precast High-strength Concrete Pile placed into 5 m deep bored hole, and then driven an additional 2 m. Statnamic conducted after 3 cycles of Static Load testing. STN curve not available.

STATNAMIC TEST SUMMARY SHEET

6	2	1	1	3	3,2	-	JPN	;HONAI	T2

Job Name: Shonan Test Site
Job Location: Shonan-Machi, Chiba Prefecture, Japan



λ = 0.92
A_{skin} = 8.80
A_{tip} = 0.13
L = 7.00
OD = 0.40
ID = 0.00

General Soil Profile

0-2 m Loam N = 2
2.5-6.5 m Clay N = 5
6.5-13 m Fine Sand N = 18
13-15 m Fine Sand N = 19
15-20 m Fine Sand N = 30

Additional Project Information

Precast High-strength Concrete piles were placed into bored and grouted excavation.

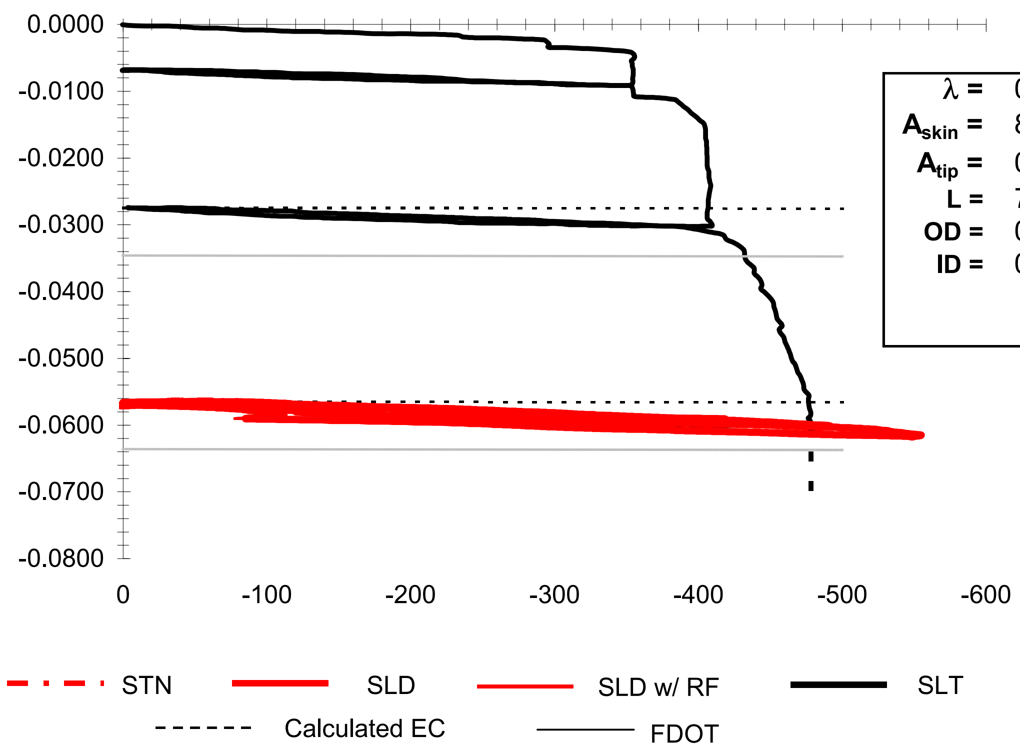
Statnamic testing was conducted prior to three cycles of Static Load testing.

STN curve not available.

STATNAMIC TEST SUMMARY SHEET

6	2	1	2	4	3,2	-	JPN	;HONAI	T1

Job Name: Shonan Test Site
Job Location: Shonan-Machi, Ciba Prefecture, Japan



λ = 0.95
A_{skin} = 8.80
A_{tip} = 0.13
L = 7.00
OD = 0.40
ID = 0.00

General Soil Profile

0-2 m Loam N = 2
2.5-6.5 m Clay N = 5
6.5-13 m Fine Sand N = 18
13-15 m Fine Sand N = 19
15-20 m Fine Sand N = 30

Additional Project Information

Precast High-strength Concrete piles were placed into bored and grouted excavation.

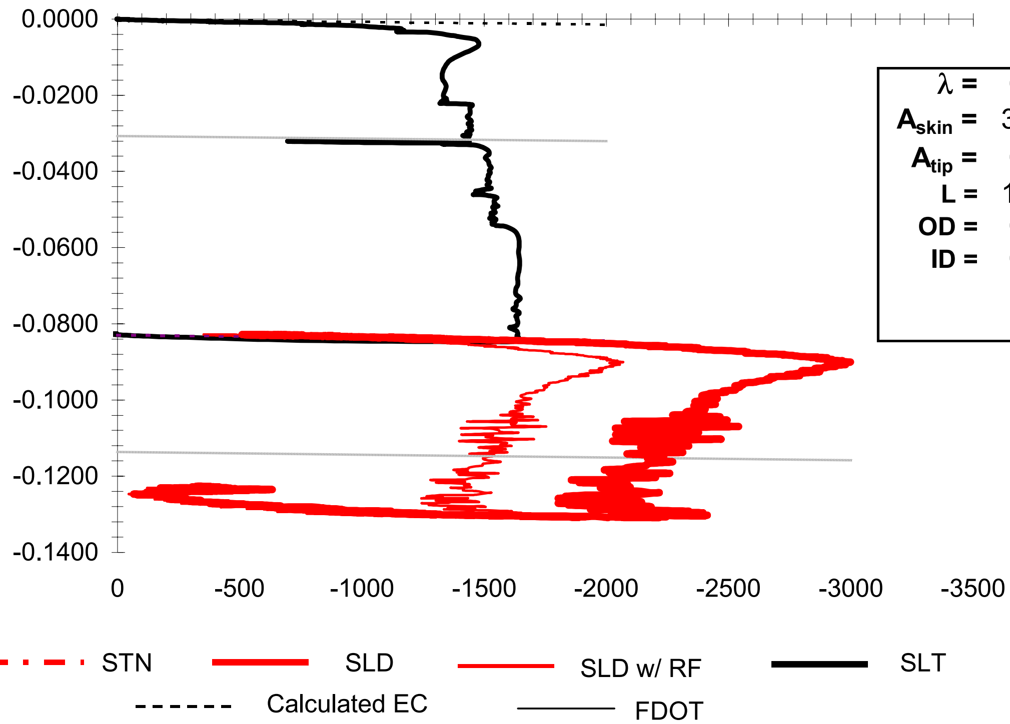
Statnamic testing conducted 2 to 3 days after two cycles of Static Load testing.

STN curve not available.

STATNAMIC TEST SUMMARY SHEET

1	3	1	2	4	2	-	USA	Aub	10

Job Name: Auburn Test Series
Job Location: Opelika, AL



λ =	0.80
A_{skin} =	33.35
A_{tip} =	0.66
L =	11.54
OD =	0.92
ID =	0.00

General Soil Profile

Micaceous sandy or clayey silt, ML-SM, with seams of sand which are remnants of igneous quartz seams.
WC = 34%
Grain size: 47% sand, 33% silt, 10% clay.
LL = 46
PI = 10
SPT: N (avg) = 12, ranging from 8 to 14 blows/ft
CPT: qc = 3 to 4 MPa, friction ratio = 4 to 6%
CU/CD tests: effective cohesion = 17 kPa, phi = 32, Su = 92 kPa

Additional Project Information

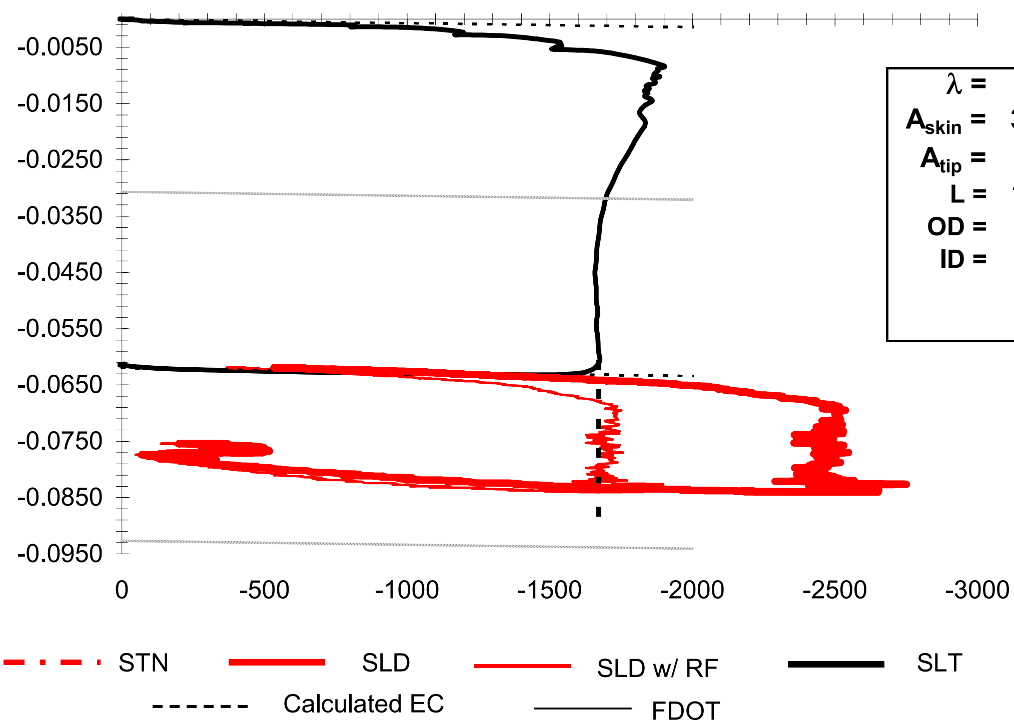
- Statnamic test date: 4/15/97
- Static test date: 4/8/97

STN curve not available.

STATNAMIC TEST SUMMARY SHEET

1	3	1	2	4	2	-	USA	Aub	8

Job Name: Auburn Test Series
Job Location: Opelika, AL



λ =	0.88
A_{skin} =	33.35
A_{tip} =	0.66
L =	11.54
OD =	0.92
ID =	0.00

General Soil Profile

Micaceous sandy or clayey silt, ML-SM, with seams of sand which are remnants of igneous quartz seams.
WC = 34%
Grain size: 47% sand, 33% silt, 10% clay.
LL = 46
PI = 10
SPT: N (avg) = 12, ranging from 8 to 14 blows/ft
CPT: qc = 3 to 4 MPa, friction ratio = 4 to 6%
CU/CD tests: effective cohesion = 17 kPa, phi = 32, Su = 92 kPa

Additional Project Information

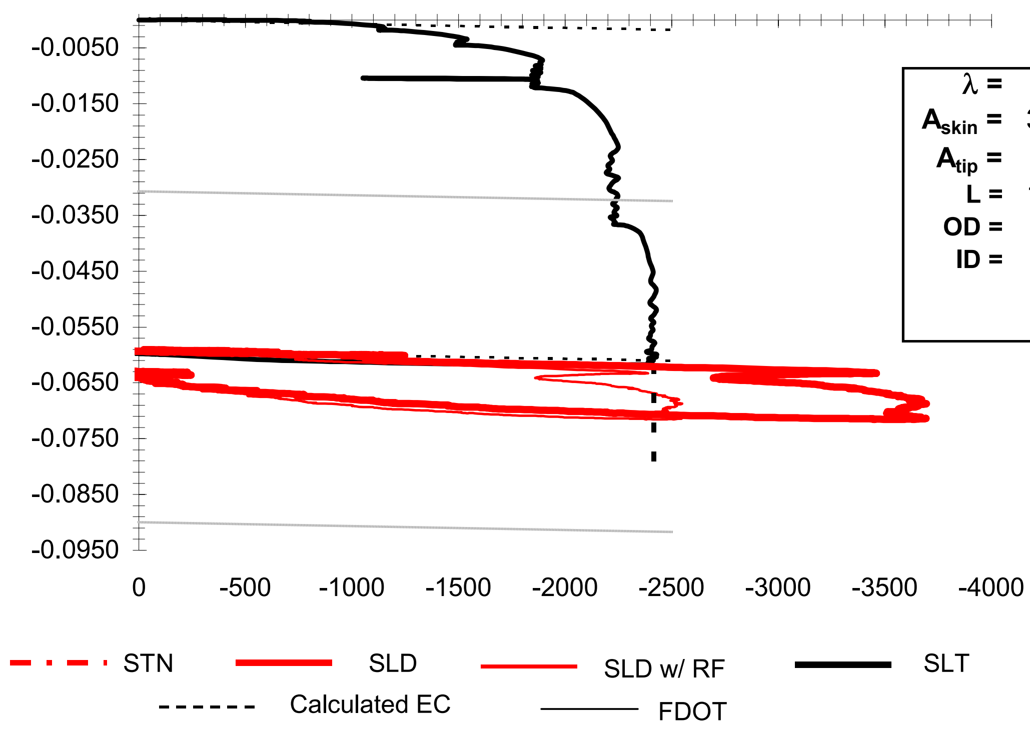
- Statnamic test date: 4/16/97
- Static test date: 4/7/97

STN curve not available.

STATNAMIC TEST SUMMARY SHEET

1	3	1	2	4	2	-	USA	Aub	7

Job Name: Auburn Test Series
Job Location: Opelika, AL



λ =	0.95
A_{skin} =	33.35
A_{tip} =	0.66
L =	11.54
OD =	0.92
ID =	0.00

General Soil Profile

Micaceous sandy or clayey silt, ML-SM, with seams of sand which are remnants of igneous quartz seams.
WC = 34%
Grain size: 47% sand, 33% silt, 10% clay.
LL = 46
PI = 10
SPT: N (avg) = 12, ranging from 8 to 14 blows/ft
CPT: qc = 3 to 4 MPa, friction ratio = 4 to 6%
CU/CD tests: effective cohesion = 17 kPa, phi = 32, Su = 92 kPa

Additional Project Information

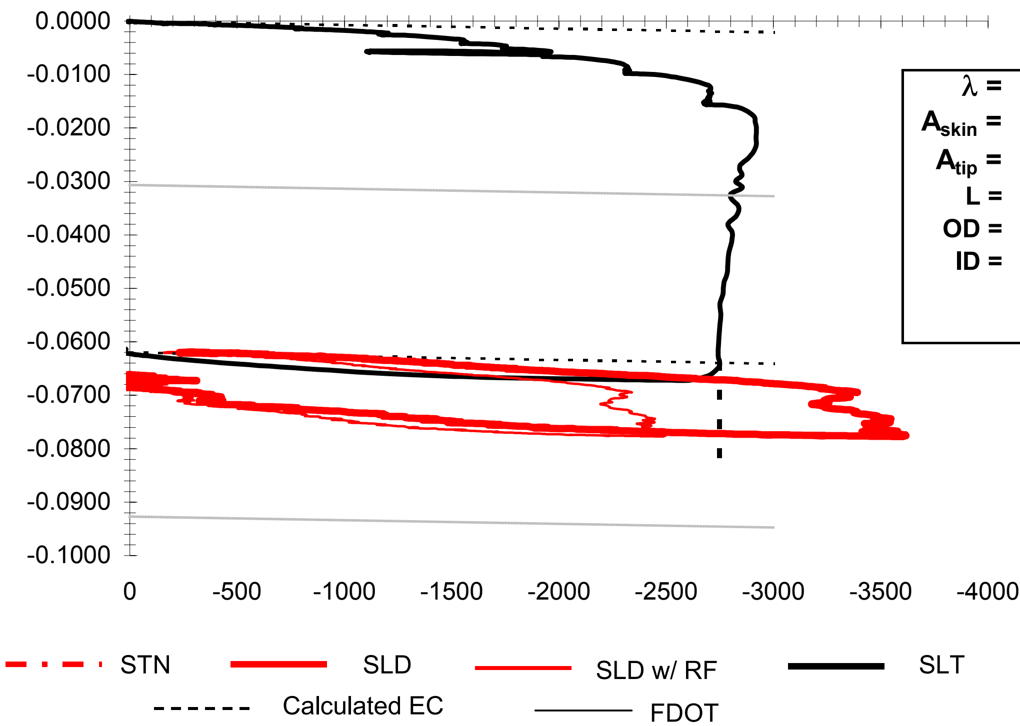
- Statnamic test date: 4/25/97
- Static test date: 4/4/97

STN curve not available.

STATNAMIC TEST SUMMARY SHEET

1	**3**	**1**	**2**	**4**	**2**	**-**	**USA**	**Aub**	**5**

Job Name: Auburn Test Series
Job Location: Opelika, AL



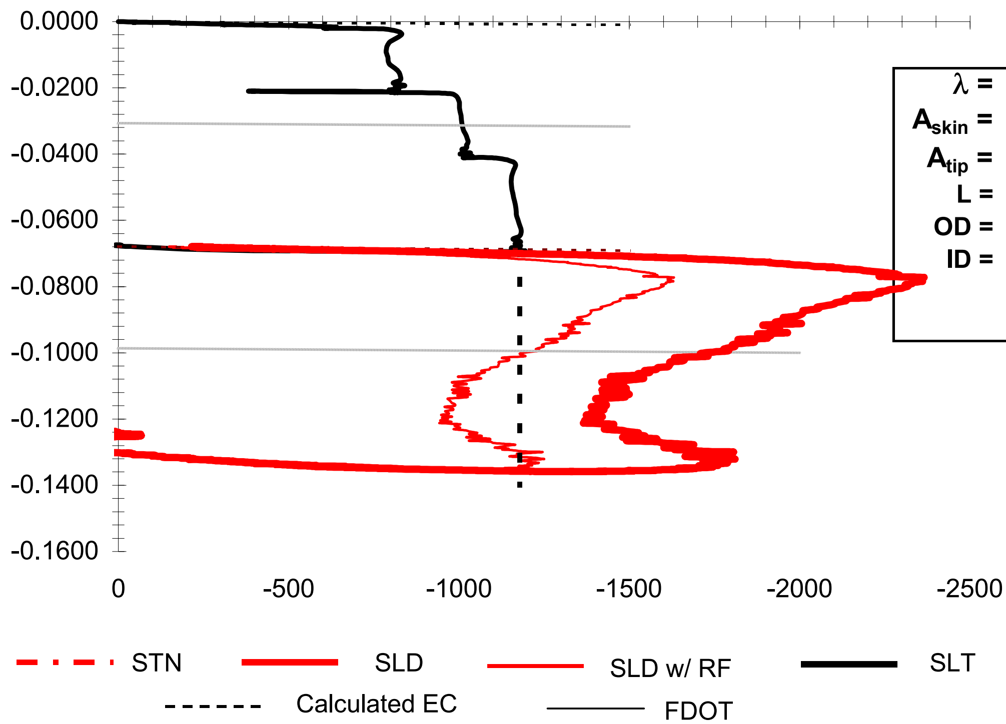
λ =	1.08
A_{skin} =	33.35
A_{tip} =	0.66
L =	11.54
OD =	0.92
ID =	0.00

General Soil Profile	Additional Project Information
Micaceous sandy or clayey silt, ML-SM, with seams of sand which are remnants of igneous quartz seams. WC = 34% Grain size: 47% sand, 33% silt, 10% clay. LL = 46 PI = 10 SPT: N (avg) = 12, ranging from 8 to 14 blows/ft CPT: qc = 3 to 4 MPa, friction ratio = 4 to 6% CU/CD tests: effective cohesion = 17 kPa, phi = 32, Su = 92 kPa	- Statnamic test date: 4/15/97 - Static test date: 4/8/97 STN curve not available.

STATNAMIC TEST SUMMARY SHEET

1	3	1	2	4	2	-	USA	Aub	3

Job Name: Auburn Test Series
Job Location: Opelika, AL



λ =	0.96
A_{skin} =	33.35
A_{tip} =	0.66
L =	11.54
OD =	0.92
ID =	0.00

General Soil Profile

Micaceous sandy or clayey silt, ML-SM, with seams of sand which are remnants of igneous quartz seams.
WC = 34%
Grain size: 47% sand, 33% silt, 10% clay.
LL = 46
PI = 10
SPT: N (avg) = 12, ranging from 8 to 14 blows/ft
CPT: qc = 3 to 4 MPa, friction ratio = 4 to 6%
CU/CD tests: effective cohesion = 17 kPa, phi = 32, Su = 92 kPa

Additional Project Information

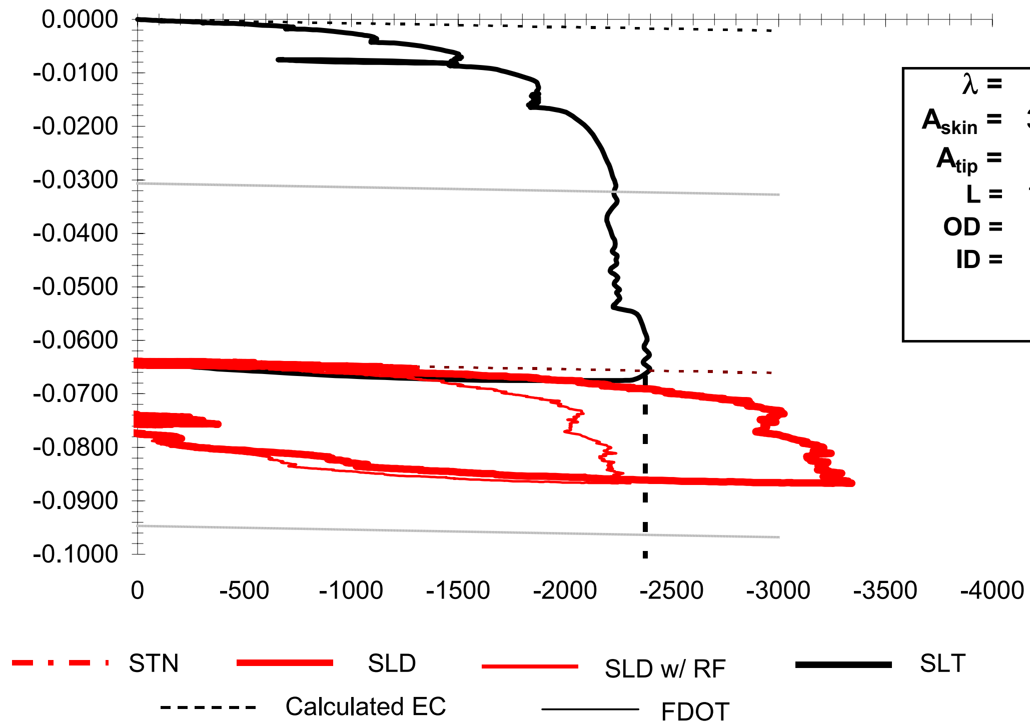
- Statnamic test date: 4/18/97
- Static test date: 4/7/97

STN curve not available.

STATNAMIC TEST SUMMARY SHEET

1	3	1	2	4	2	-	USA	Aub	2

Job Name: Auburn Test Series
Job Location: Opelika, AL



λ =	1.04
A_{skin} =	33.35
A_{tip} =	0.66
L =	11.54
OD =	0.92
ID =	0.00

General Soil Profile
Micaceous sandy or clayey silt, ML-SM, with seams of sand which are remnants of igneous quartz seams.
WC = 34%
Grain size: 47% sand, 33% silt, 10% clay.
LL = 46
PI = 10
SPT: N (avg) = 12, ranging from 8 to 14 blows/ft
CPT: qc = 3 to 4 MPa, friction ratio = 4 to 6%
CU/CD tests: effective cohesion = 17 kPa, phi = 32, Su = 92 kPa

Additional Project Information

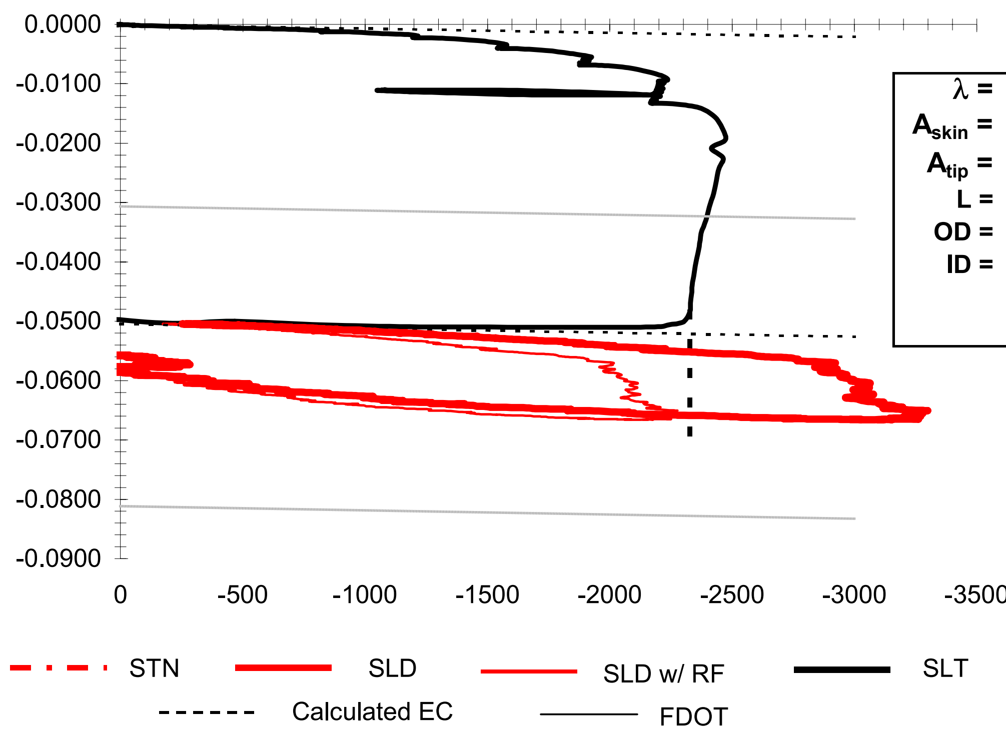
- Statnamic test date: 4/24/97
- Static test date: 4/7/97

STN curve not available.

STATNAMIC TEST SUMMARY SHEET

1	3	1	2	4	2	-	USA	Aub	1

Job Name: Auburn Test Series
Job Location: Opelika, AL



λ =	1.01
A_{skin} =	33.35
A_{tip} =	0.66
L =	11.54
OD =	0.92
ID =	0.00

General Soil Profile

Micaceous sandy or clayey silt, ML-SM, with seams of sand which are remnants of igneous quartz seams.
WC = 34%
Grain size: 47% sand, 33% silt, 10% clay.
LL = 46
PI = 10
SPT: N (avg) = 12, ranging from 8 to 14 blows/ft
CPT: qc = 3 to 4 MPa, friction ratio = 4 to 6%
CU/CD tests: effective cohesion = 17 kPa, phi = 32, Su = 92 kPa

Additional Project Information

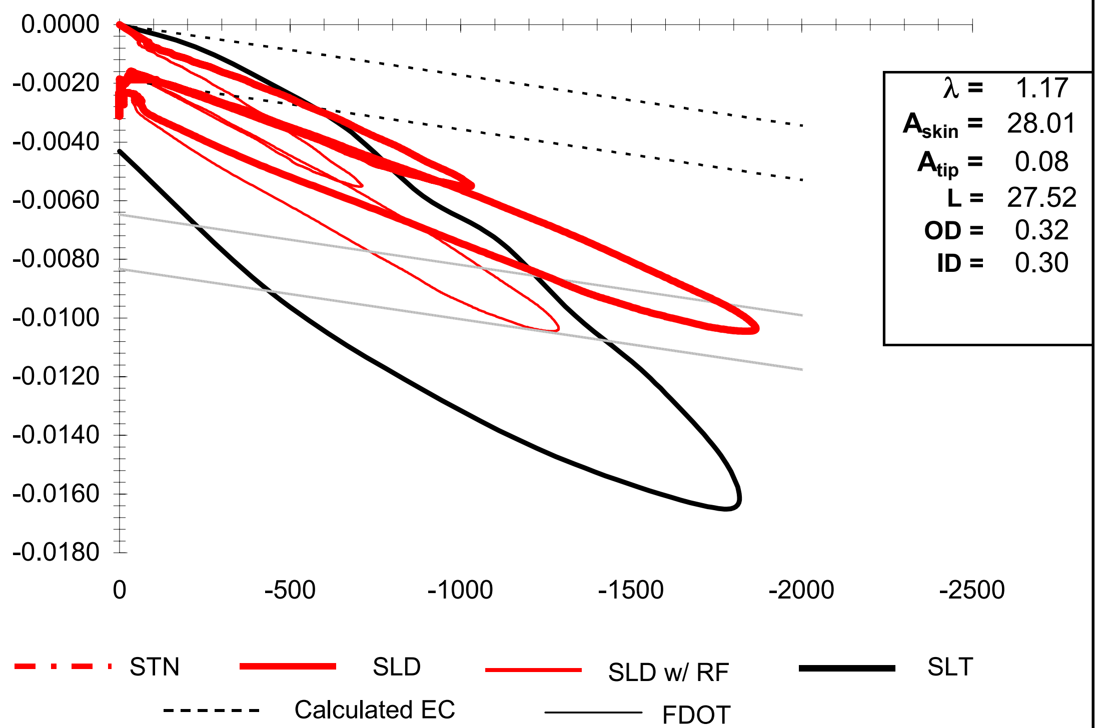
- Statnamic test date: 4/18/97
- Static test date: 4/3/97

STN curve not available.

STATNAMIC TEST SUMMARY SHEET

4	3	2	0	3	2	-	USA	NIA	TP-1&2a

Job Name: Newark Airport Parking Garage @ Lot "E"
Job Location: Newark, New Jersey

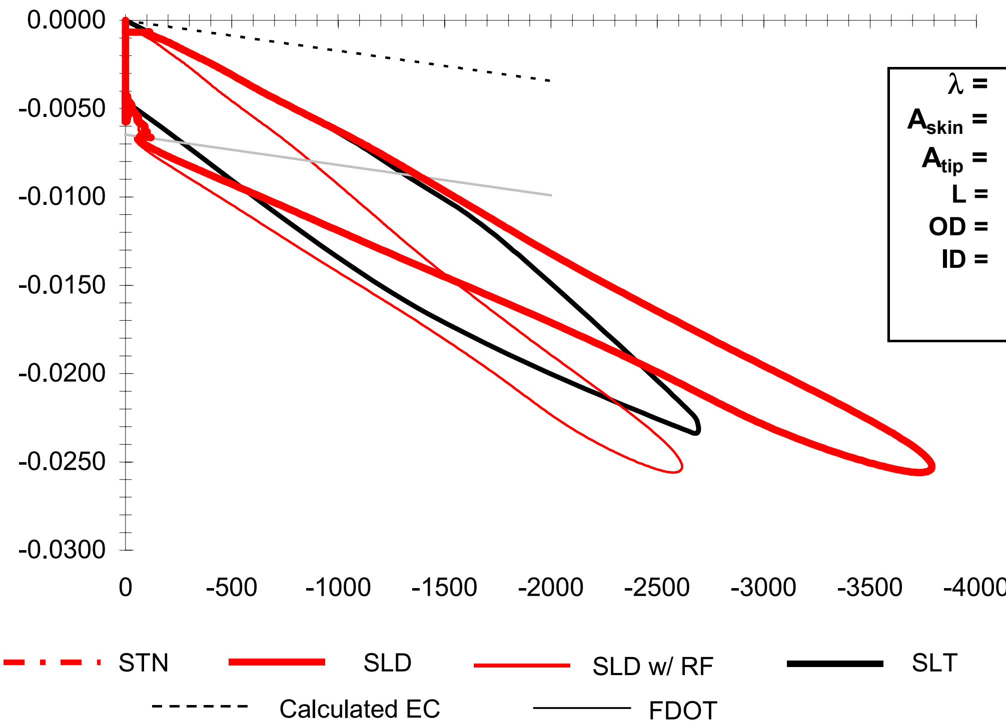


General Soil Profile	Additional Project Information
0-4.5 m - fill (sand,gravel,glass,wood,metal,cinders, slag and garbage) 4.5 to 5.5 m - Peat N=W.O.R. 5.5 to 7.5 m - Sandy Silt Avg N=10 7.5 to 26.0 m - Reddish Brown Clayey Silt Avg N=10 26.0 m - Red Shale Qc=10,000psi	Closed ended piles driven to refusal in rock and filled with concrete. Strain data from STN shows 60% skin and 30% EB STN curve not available.

STATNAMIC TEST SUMMARY SHEET

4	3	2	0	3	2	-	USA	NIA	TP-1&2b

Job Name: Newark Airport Parking Garage @ Lot "E"
Job Location: Newark, New Jersey



λ =	1.07
A_{skin} =	28.01
A_{tip} =	0.08
L =	27.52
OD =	0.32
ID =	0.30

General Soil Profile

0-4.5 m - fill (sand,gravel,glass,wood,metal,cinders, slag and garbage)
4.5 to 5.5 m - Peat N=W.O.R.
5.5 to 7.5 m - Sandy Silt Avg N=10
7.5 to 26.0 m - Reddish Brown Clayey Silt Avg N=10
26.0 m - Red Shale Qc=10,000psi

Additional Project Information

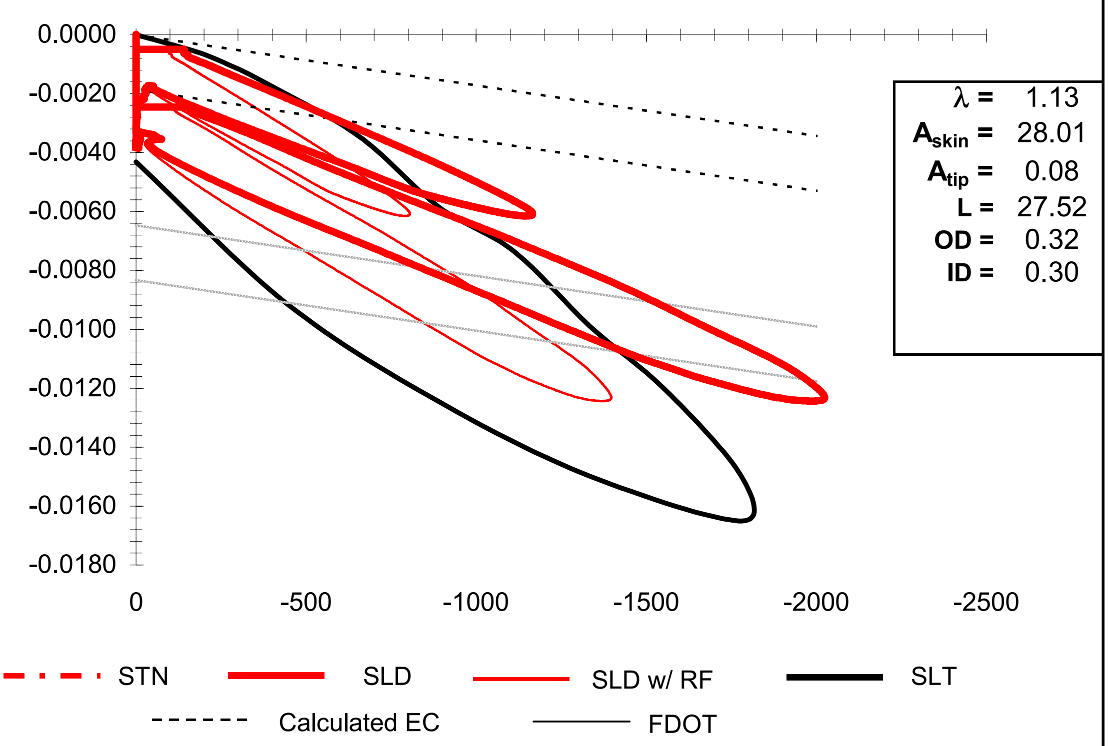
Closed ended piles driven to refusal in rock and filled with concrete. Strain data from STN shows 30% skin and 60% EB

STN curve not available.

STATNAMIC TEST SUMMARY SHEET

4	3	2	0	3	2	-	USA	NIA	TP-1&3a

Job Name: Newark Airport Parking Garage @ Lot "E"
Job Location: Newark, New Jersey

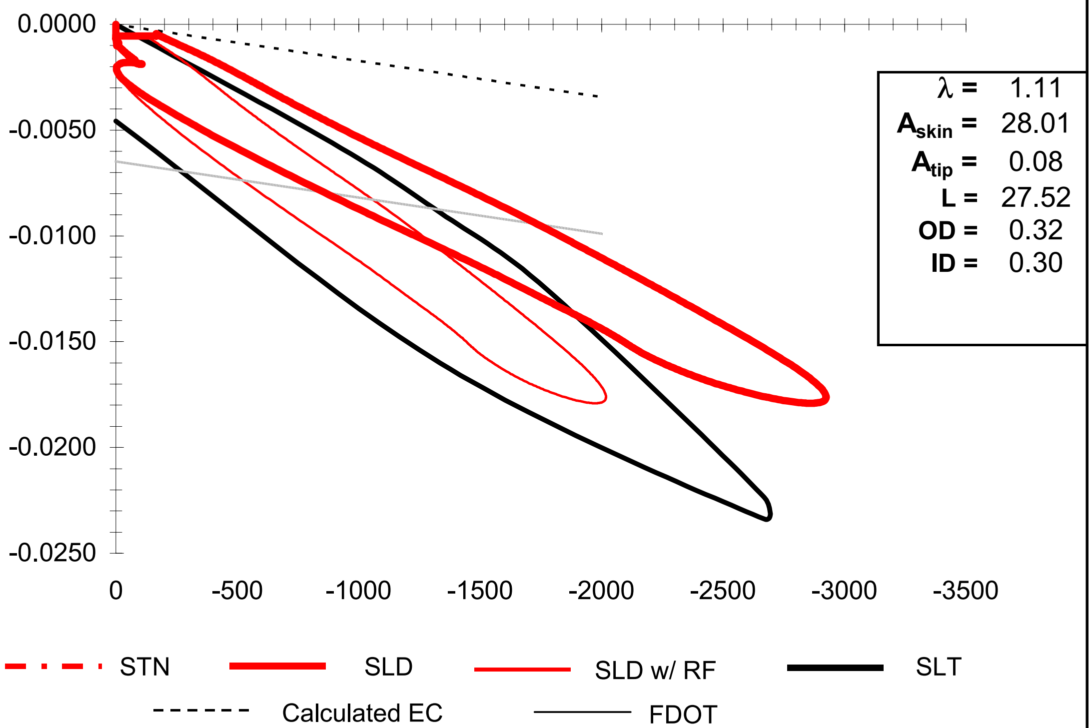


General Soil Profile	**Additional Project Information**
0-4.5 m - fill (sand,gravel,glass,wood,metal,cinders, slag and garbage) 4.5 to 5.5 m - Peat N=W.O.R. 5.5 to 7.5 m - Sandy Silt Avg N=10 7.5 to 26.0 m - Reddish Brown Clayey Silt Avg N=10 26.0 m - Red Shale Qc=10,000psi	Closed ended piles driven to refusal in rock and filled with concrete. Strain data from STN shows 55% skin and 45% EB STN curve not available.

STATNAMIC TEST SUMMARY SHEET

4	3	2	0	4	2	-	USA	NIA	TP-1&3b

Job Name: Newark Airport Parking Garage @ Lot "E"
Job Location: Newark, New Jersey

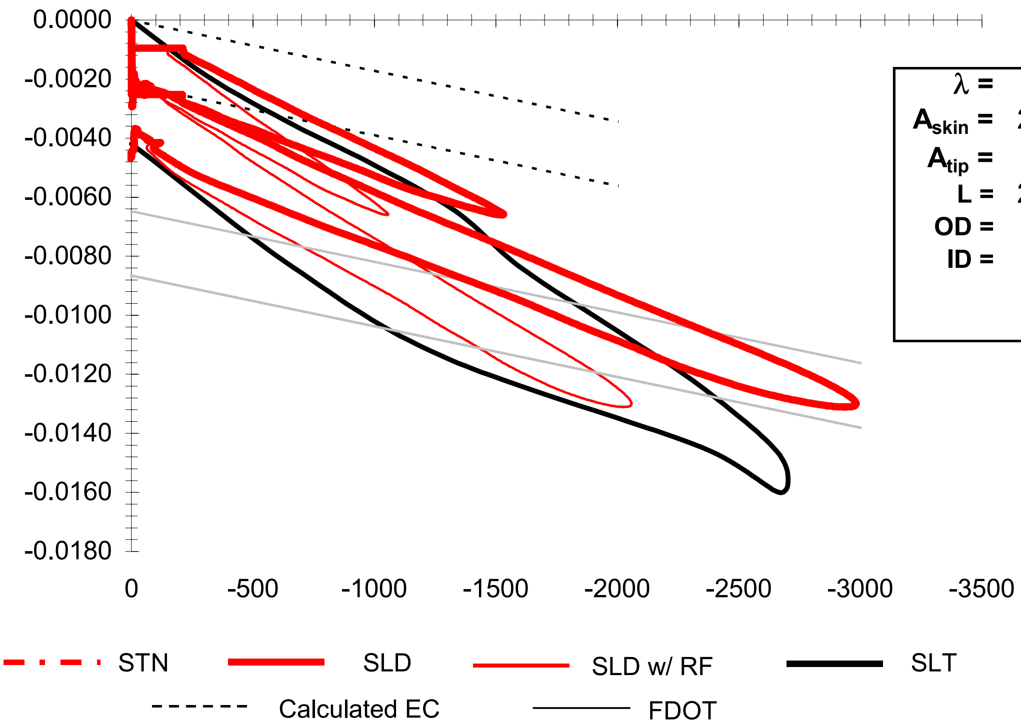


General Soil Profile	Additional Project Information
0-4.5 m - fill (sand,gravel,glass,wood,metal,cinders, slag and garbage) 4.5 to 5.5 m - Peat N=W.O.R. 5.5 to 7.5 m - Sandy Silt Avg N=10 7.5 to 26.0 m - Reddish Brown Clayey Silt Avg N=10 26.0 m - Red Shale Qc=10,000psi	Closed ended piles driven to refusal in rock and filled with concrete. Strain data from STN shows 30% skin and 60% EB STN curve not available.

STATNAMIC TEST SUMMARY SHEET

4	3	2	0	3	2	-	USA	NIA	P-9&10

Job Name: Newark Airport Parking Garage @ Lot "E"
Job Location: Newark, New Jersey



λ =	1.18
A_{skin} =	28.01
A_{tip} =	0.08
L =	27.52
OD =	0.32
ID =	0.30

General Soil Profile - Boring 3-945

0-4 m - fill (sand,gravel,glass,wood,metal,cinders, slag and garbage)
4 to 6 m - Organic Clay N=W.O.R.
6 to 17 m - Reddish Brown Silty Sand Avg N=15
17 to 26 m - Reddish Brown Silty Clay Avg N=12
26.0 m - Red Shale Qc=10,000psi

Additional Project Information

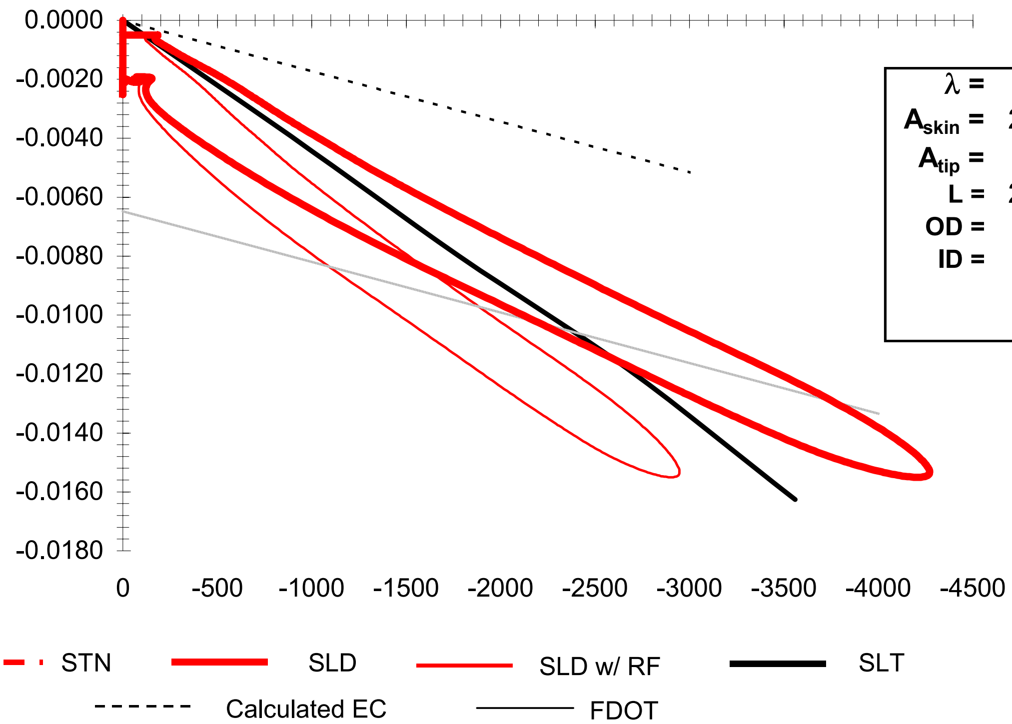
Closed ended piles driven to refusal in rock and filled with concrete. Strain data from STN shows 55% skin and 45% EB. static tests were via kentledge with 1 hour holds and 24 hour hold at max load.

STN curve not available.

STATNAMIC TEST SUMMARY SHEET

4	3	2	0	3	2	-	USA	NIA	P-9&10

Job Name: Newark Airport Parking Garage @ Lot "E"
Job Location: Newark, New Jersey



λ =	1.14
A_{skin} =	28.01
A_{tip} =	0.08
L =	27.52
OD =	0.32
ID =	0.30

General Soil Profile - Boring 3-945

0-4 m - fill (sand,gravel,glass,wood,metal,cinders, slag and garbage)
4 to 6 m - Organic Clay N=W.O.R.
6 to 17 m - Reddish Brown Silty Sand Avg N=15
17 to 26 m - Reddish Brown Silty Clay Avg N=12
26.0 m - Red Shale Qc=10,000psi

Additional Project Information

Closed ended piles driven to refusal in rock and filled with concrete. Strain data from STN shows 48% skin and 52% EB. static tests were via kentledge with 1 hour holds and 24 hour hold at max load.

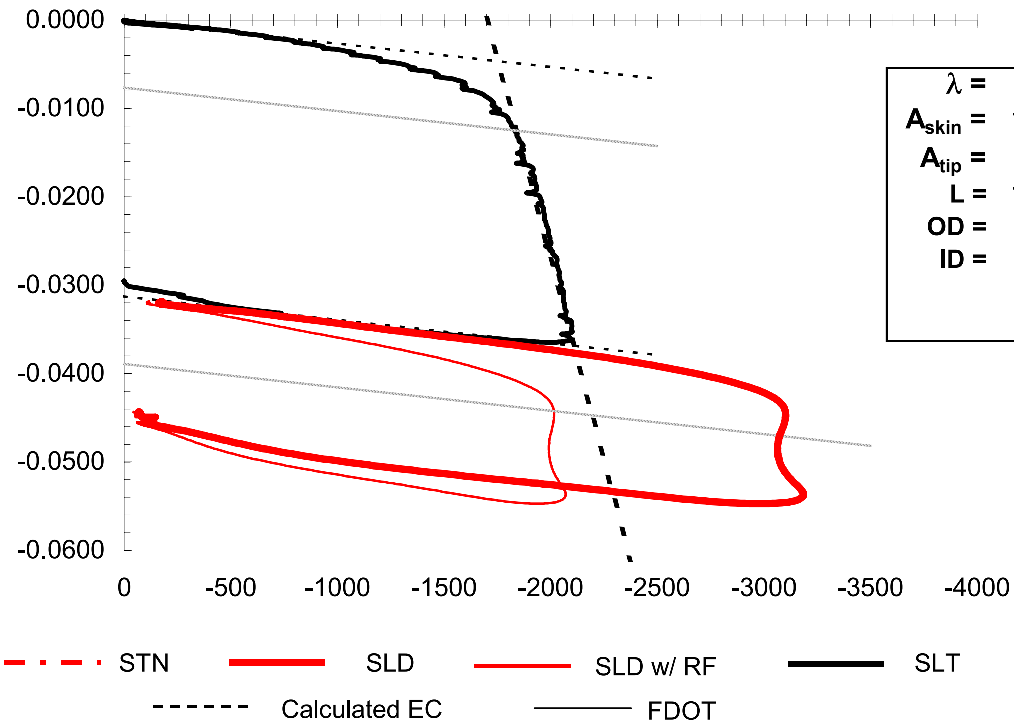
NOTE: No unloading data available for SLT.

STN curve not available.

STATNAMIC TEST SUMMARY SHEET

2	4	1	2	3	1	-	USA	LC	T-114.5

Job Name: Contraband Bayou
Job Location: Lake Charles, LA



λ =	1.11
A_{skin} =	18.59
A_{tip} =	0.16
L =	12.95
OD =	0.46
ID =	0.00

General Soil Profile - Boring B-8

0-2.6 m - Silty Sand (SM) N=2
2.6 to 7.0 m - Organic Clay (OH) N=2
(UU Triax Qc = 0.6 tsf, Dry Density=24 pcf, MC=225, LL=301, PI=195)
7 to 8.5 m - Sandy Silt (ML) N=24 (-200=56%)
8.5 to 20.4 m - Sand (SP) Avg N=28 (-200=4%, MC=25%)

** Pile Tip @ 12.95 m**

Additional Project Information

455mm diameter PSC piles. Pile was instrumented with sisterbar gages. Strain data from STN shows 35% skin. Strain data from SLT shows 35% skin. SLT performed first, STN performed 12 hours after SLT. Static tests were via anchored reaction beam using QUICK method. Also note that reaction piles were in place prior to performing STN.

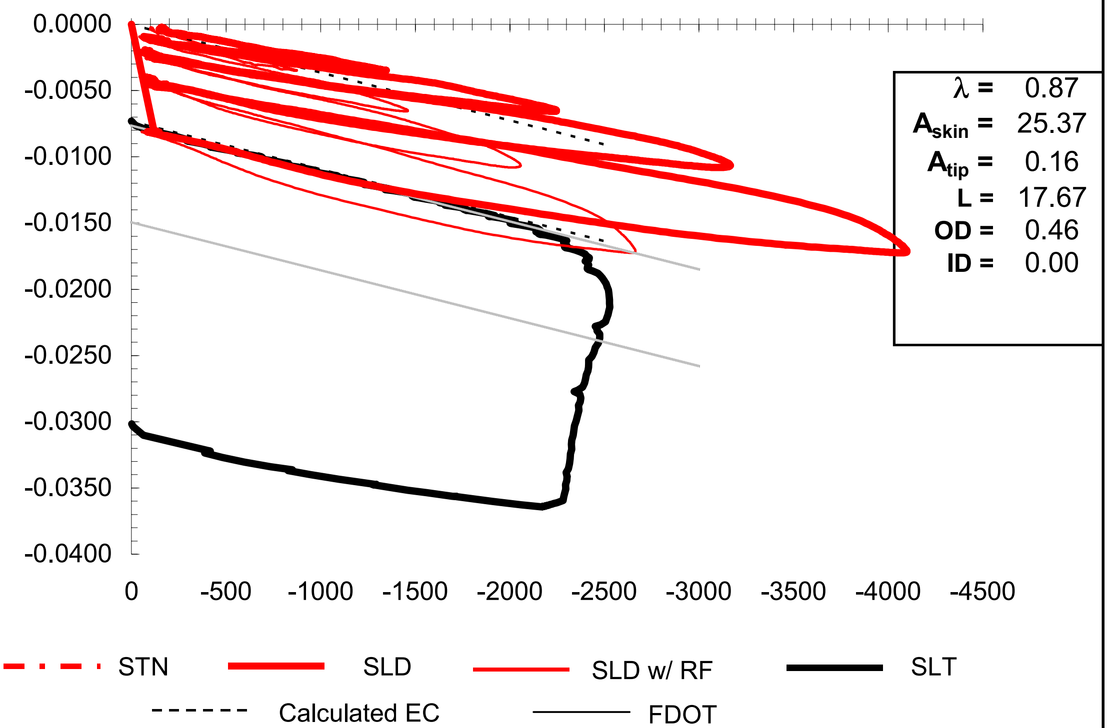
0.64 rate factor used in AFT Report.

STN curve not available.

STATNAMIC TEST SUMMARY SHEET

2	4	1	1	3	1	-	USA	LC	X-123

Job Name: Contraband Bayou
Job Location: Lake Charles, LA

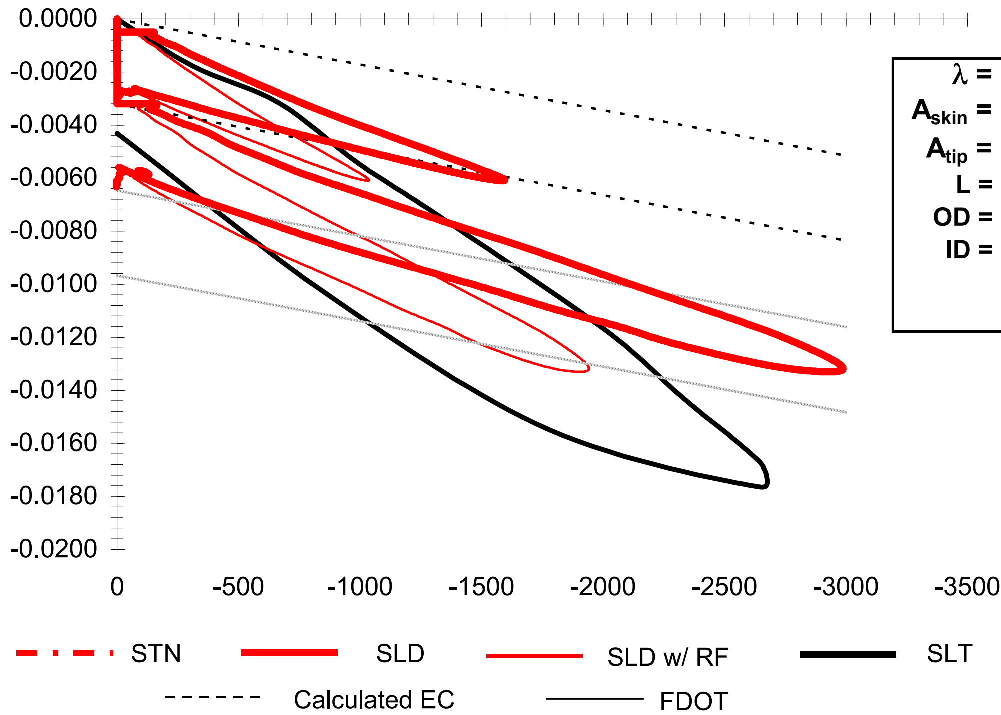


General Soil Profile - Boring b-7	**Additional Project Information**
0-2.1 m - Sandy Silt (ML) w/ wood & concrete debris 2.1 to 3.6 m - Sand (SP) N=10 3.6 to 5.2 m - Organic Clay (OH) N=W.O.R. 5.2 to 7 m - Silty Sand (SM) N=W.O.R. (-200=29%) 7 to 12.2 m - Sand (SP) Avg N=48 (-200=8%, MC=24%) 12.2 to 29 m - Clay (CH) Avg N=20 ** Pile Tip @ 17.8 m** Lab data on clay: UU Triax Qc = 2.0 tsf	455mm diameter PSC piles. Pile was instrumented with sisterbar gages. Strain data from STN shows 84% skin. Strain data from SLT shows 83% skin. STN performed first, SLT performed 12 hours after STN. Static tests were via anchored reaction beam using QUICK method. Also note that reaction piles were in place prior to performing STN. STN curve not available.

STATNAMIC TEST SUMMARY SHEET

4	4	2	0	3	2	-	USA	NIA	ΓP-5&6a

Job Name: Newark Airport Parking Garage @ Lot "E"
Job Location: Newark, New Jersey



λ =	1.13
A_{skin} =	28.01
A_{tip} =	0.08
L =	27.52
OD =	0.32
ID =	0.30

General Soil Profile

0-5 m - fill (sand,gravel,glass,wood,metal,cinders, slag and garbage)
5 to 6 m - Peat N=W.O.R.
6 to 15 m - Reddish Brown Silt Avg N=45
7.5 to 26.0 m - Reddish Brown Silty Clay Avg N=12
26.0 m - Red Shale Qc=10,000psi

Additional Project Information

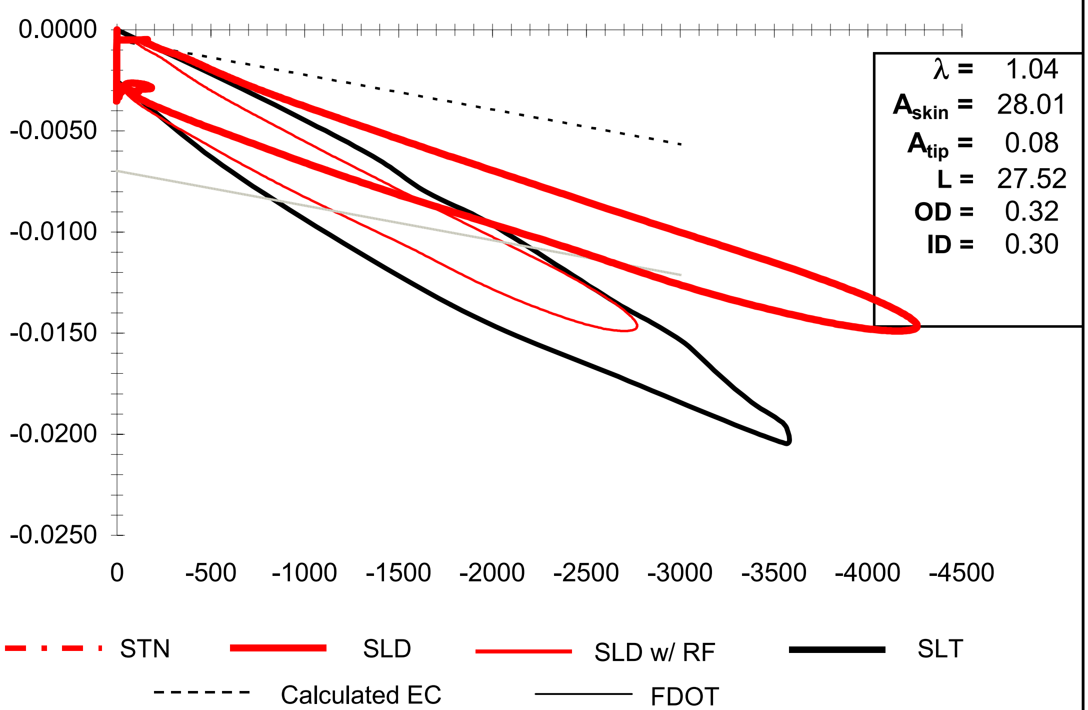
Closed ended piles driven to refusal in rock and filled with concrete. Strain data from STN shows 35% skin and 65% EB. Static tests were via kentledge with 1 hour holds and 24 hour hold at max load.

STN curve not available.

STATNAMIC TEST SUMMARY SHEET

4	4	2	0	3	2	-	USA	NIA	TP-5&6b

Job Name: Newark Airport Parking Garage @ Lot "E"
Job Location: Newark, New Jersey

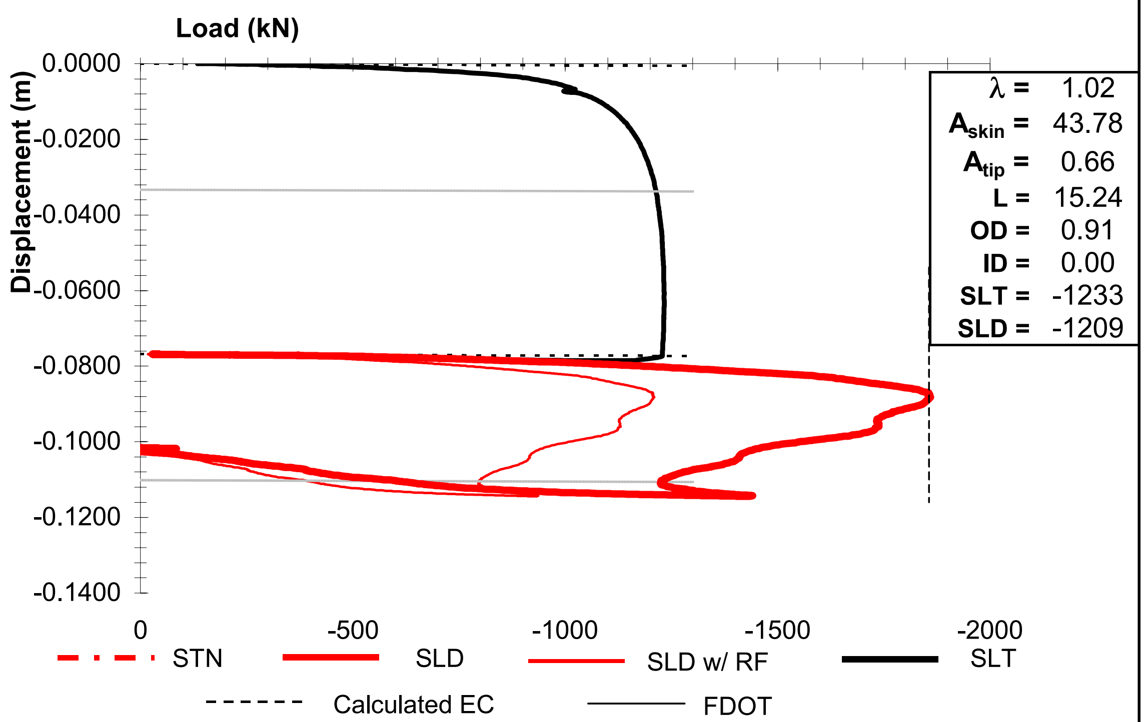


General Soil Profile	Additional Project Information
0-5 m - fill (sand,gravel,glass,wood,metal,cinders, slag and garbage) 5 to 6 m - Peat N=W.O.R. 6 to 15 m - Reddish Brown Silt Avg N=45 7.5 to 26.0 m - Reddish Brown Silty Clay Avg N=12 26.0 m - Red Shale Qc=10,000psi	Closed ended piles driven to refusal in rock and filled with concrete. Strain data from STN shows 25% skin and 75% EB. Static tests were via kentledge with 1 hour holds and 24 hour hold at max load. STN curve not available.

STATNAMIC TEST SUMMARY SHEET

1	4	1	2	4	4	-	USA	UMass	2

Job Name: Amherst
Job Location: University of Massachusetts



General Soil Profile

0 to 1.5m - miscellaneous clay/silt fill
1.5m to 3.5m - sandy silt
3.5m - varved clay

Additional Project Information

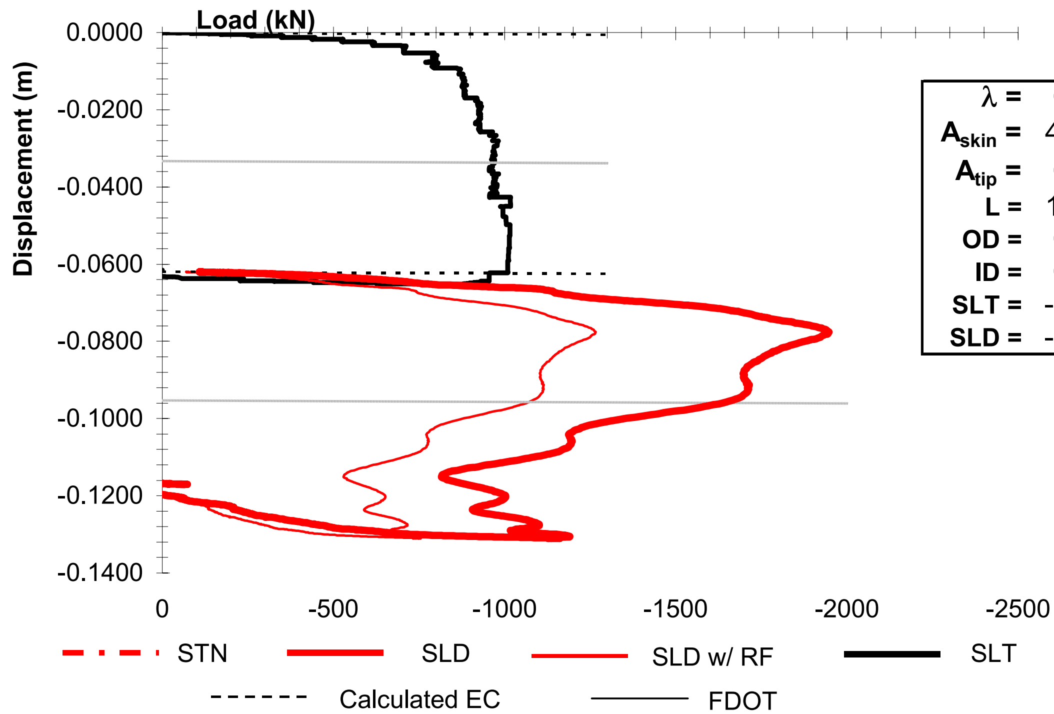
- Static Load Test: 7/22/00 17:47
- Statnamic Test: 7/25/00 10:00

STN curve not available.

STATNAMIC TEST SUMMARY SHEET

1	4	1	2	4	1	-	USA	Umass	4

File Name: Amherst
Job Location: University of Massachusetts



λ =	0.80
A_{skin} =	43.78
A_{tip} =	0.66
L =	15.24
OD =	0.91
ID =	0.00
SLT =	-1017
SLD =	-1266

General Soil Profile

0 to 1.5m - miscellaneous clay/silt fill
1.5m to 3.5m - sandy silt
3.5m - varved clay

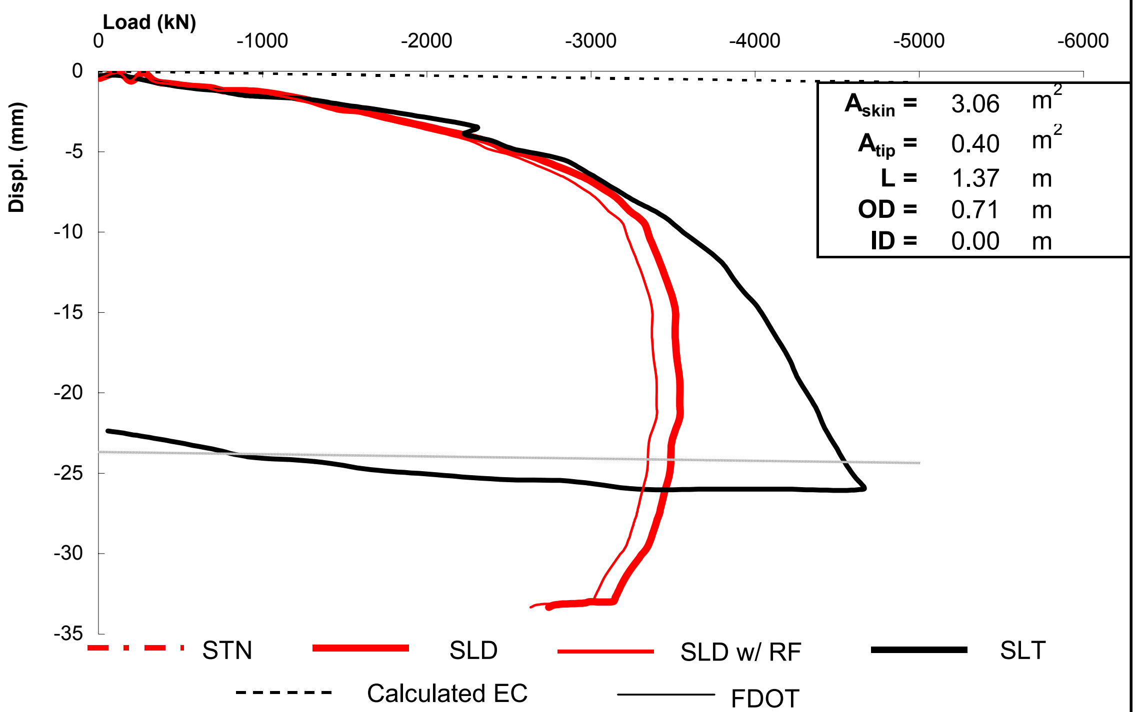
Additional Project Information

- Static Load Test: 7/21/00 10:00
- Statnamic Test: 7/25/00

STN curve not available.

STATNAMIC TEST SUMMARY SHEET

Job Name: King Road
Job Location: Burlington, Ontario



General Soil Profile
shale

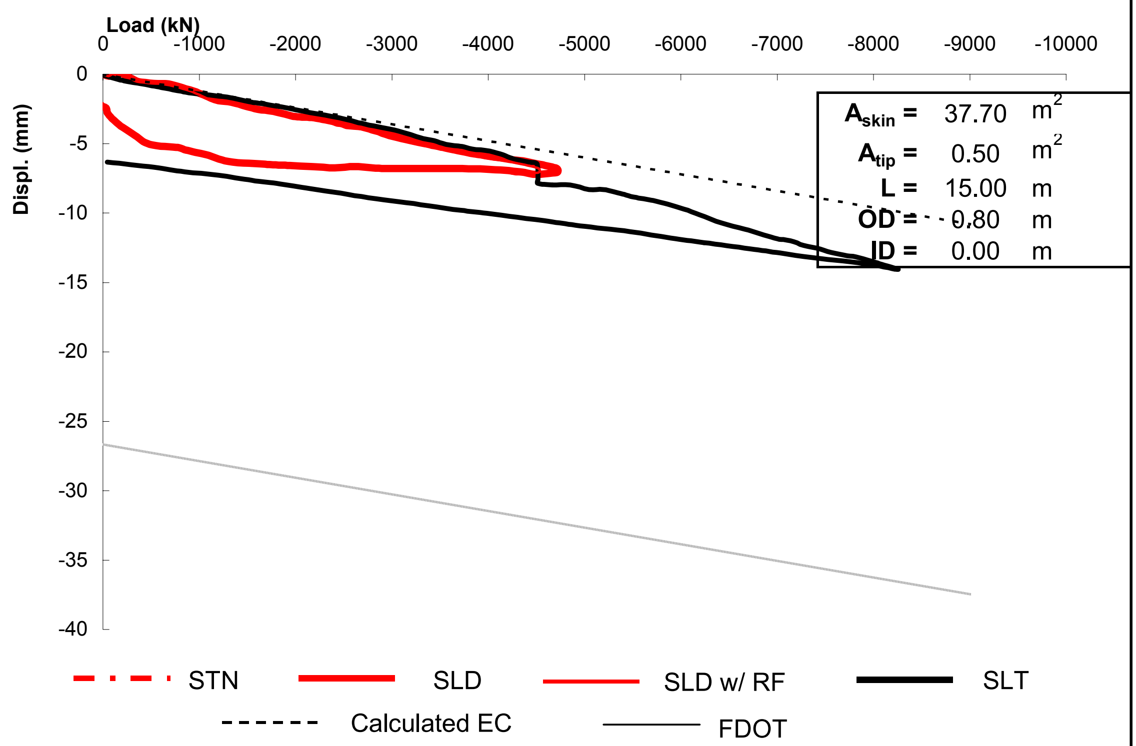
Additional Project Information

static QML (1980)

STN curve not available.

STATNAMIC TEST SUMMARY SHEET

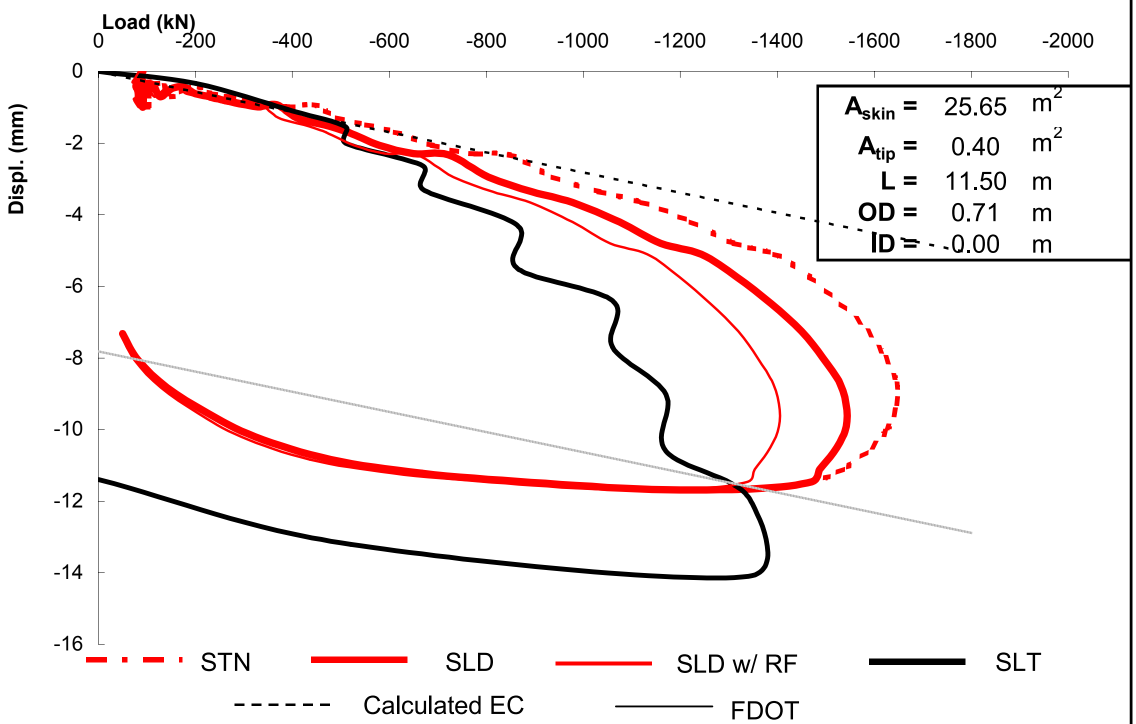
Job Name: McMaster Universtiy
Job Location: Hamilton, Ontario



General Soil Profile	**Additional Project Information**
shale	static QML (1980) STN curve not available.

STATNAMIC TEST SUMMARY SHEET

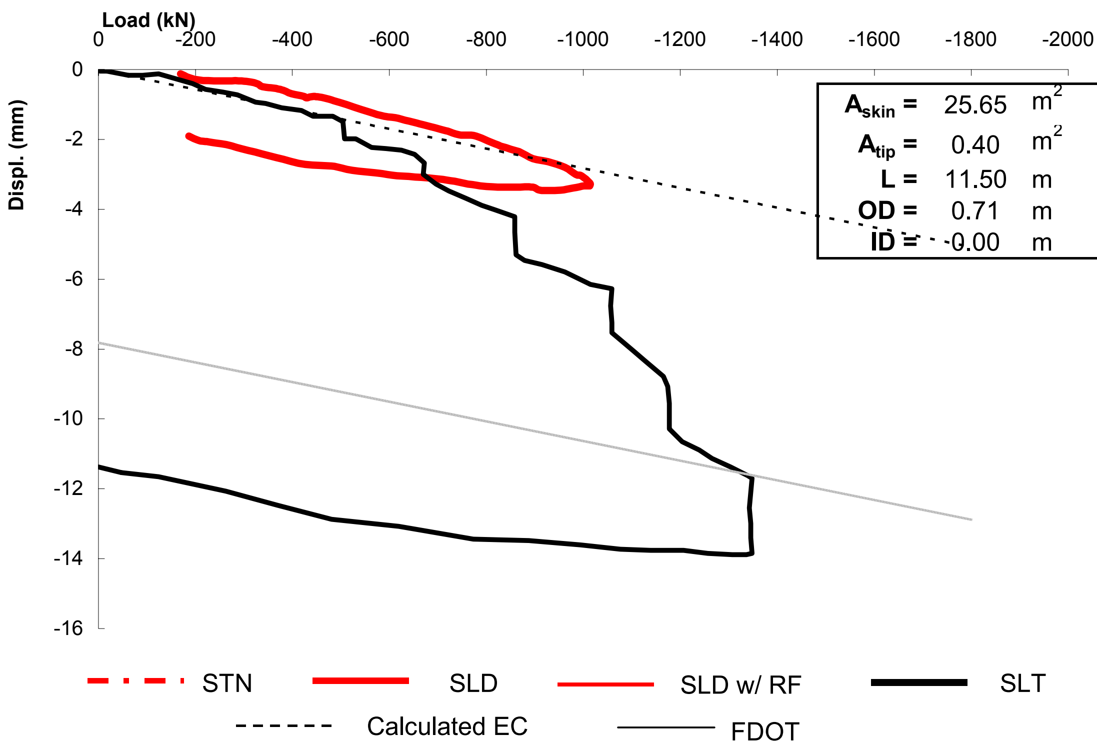
Job Name: Deep Foundation Contractors
Job Location: Barrie, Ontario - Pile 30



General Soil Profile	**Additional Project Information**
Heterogeneous fill overlying peat overlying silty sand with silt and clayey pockets. GWT - 3m from groud surface.	Static Load Test on pile 1. SLD derived from UPM using SAWR4

STATNAMIC TEST SUMMARY SHEET

Job Name: Deep Foundation Contractors
Job Location: Barrie, Ontario Pile 44



A_{skin} =	25.65	m^2
A_{tip} =	0.40	m^2
L =	11.50	m
OD =	0.71	m
ID =	0.00	m

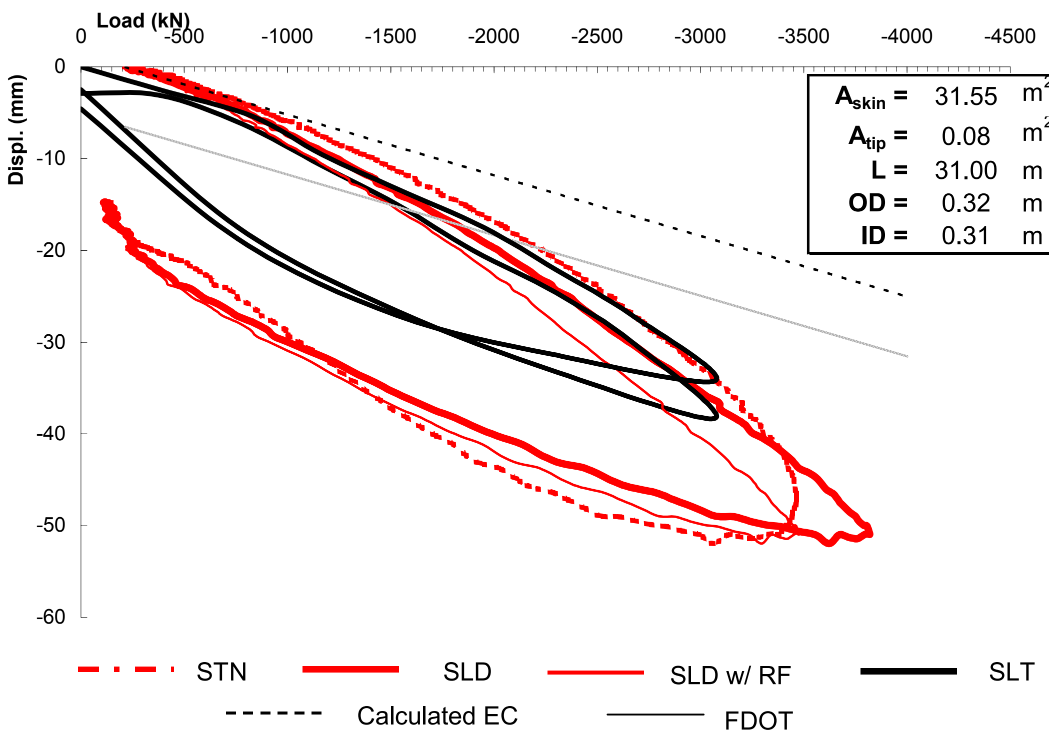
General Soil Profile
Heterogeneous fill overlying peat overlying silty sand with silt and clayey pockets.
GWT - 3m from groud surface.

Additional Project Information

Static Load Test on pile 1 May 1990

STATNAMIC TEST SUMMARY SHEET

Job Name: Golder & Associates Hunt Club Road
Job Location: Ottawa, Ontario

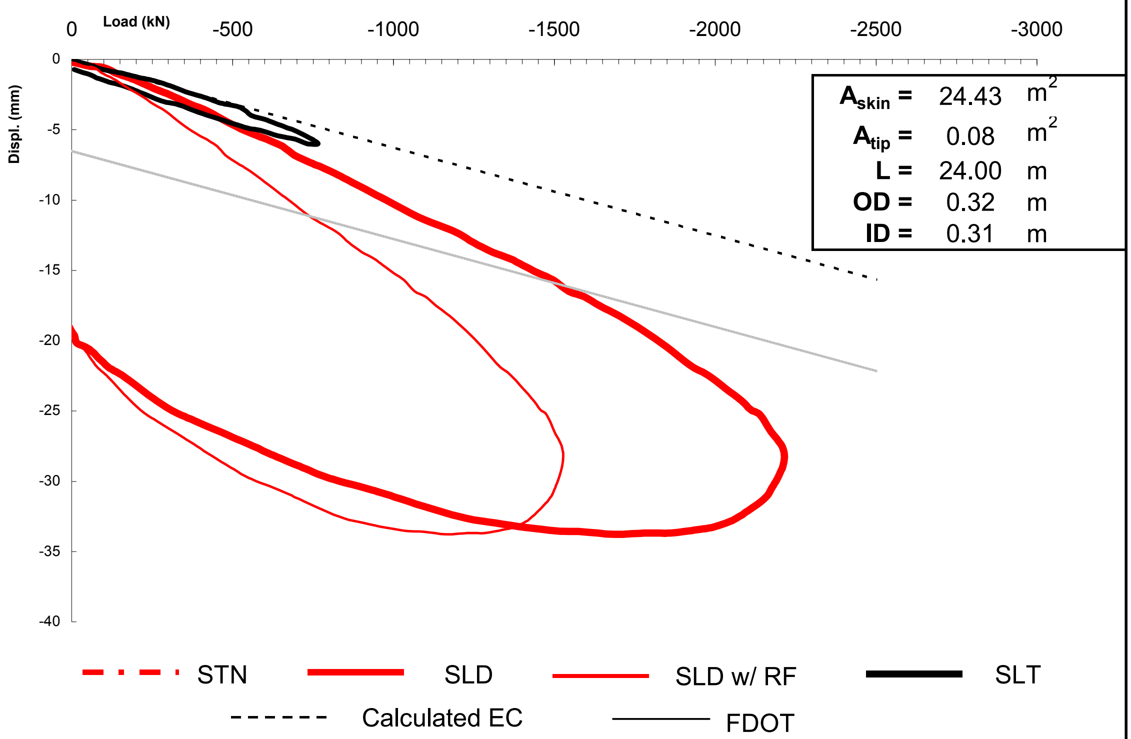


A_{skin} =	31.55	m^2
A_{tip} =	0.08	m^2
L =	31.00	m
OD =	0.32	m
ID =	0.31	m

General Soil Profile	Additional Project Information
Surface deposits of sand (0.5 to 2m thick) underlain by sensitive clays (10 to 13 m thick). Beneath the clay a dense, variable sequence of sands, silts, clays and glacial till overlies bedrock at 30m depth.	.

STATNAMIC TEST SUMMARY SHEET

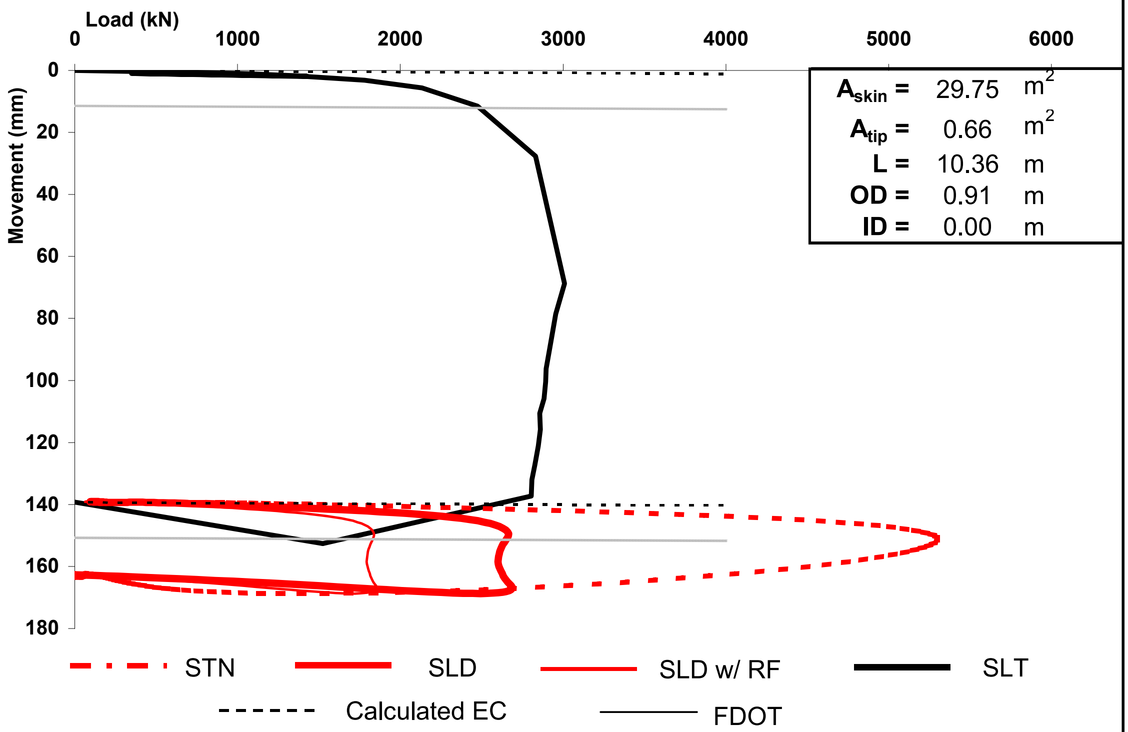
Job Name: UBC
Job Location: Vancouver, British Columbia



General Soil Profile	**Additional Project Information**
Silt	static (March 1, 1986)

STATNAMIC TEST SUMMARY SHEET

Job Name: Texas A&M FHWA
Job Location: College Station, Texas - Pile 4



A_{skin} =	29.75	m^2
A_{tip} =	0.66	m^2
L =	10.36	m
OD =	0.91	m
ID =	0.00	m

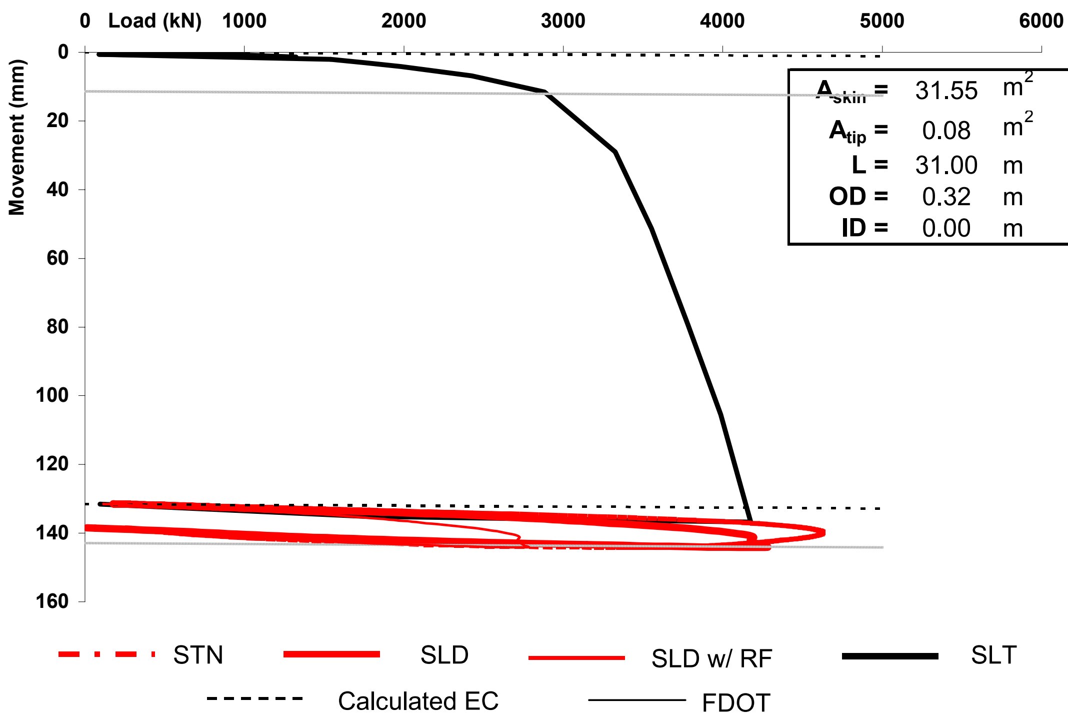
General Soil Profile

Silty Sand (0 to 4m)
Clean Sand (4m to 8m)
Clayey Sand (8m to 12.5m)
Hard Clay (Shale) (12.5m to 18m)
Water Table @ 7.5m

Additional Project Information

STATNAMIC TEST SUMMARY SHEET

Job Name: Texas A&M FHWA
Job Location: College Station, Texas - Pile 7



A_{skin} =	31.55	m^2
A_{tip} =	0.08	m^2
L =	31.00	m
OD =	0.32	m
ID =	0.00	m

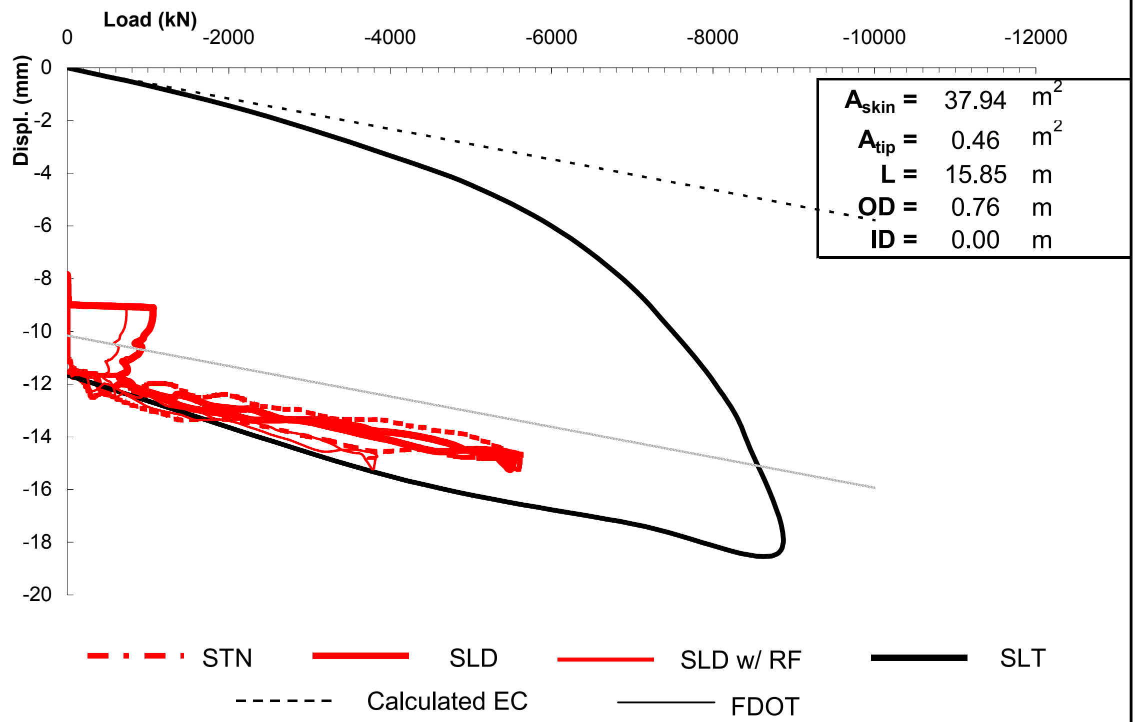
General Soil Profile

Very Stiff Clay (0 to 5.5m)
Sand (5.5m to 6.5m)
Very Stiff Clay (6.5m to 12.5m)
Hard Clay (Shale) (12.5m to 18m)
Water Table @ 6m.

Additional Project Information

STATNAMIC TEST SUMMARY SHEET

Job Name: FHWA University of Louisiana
Job Location: Shreveport, Louisiana - Pile 2

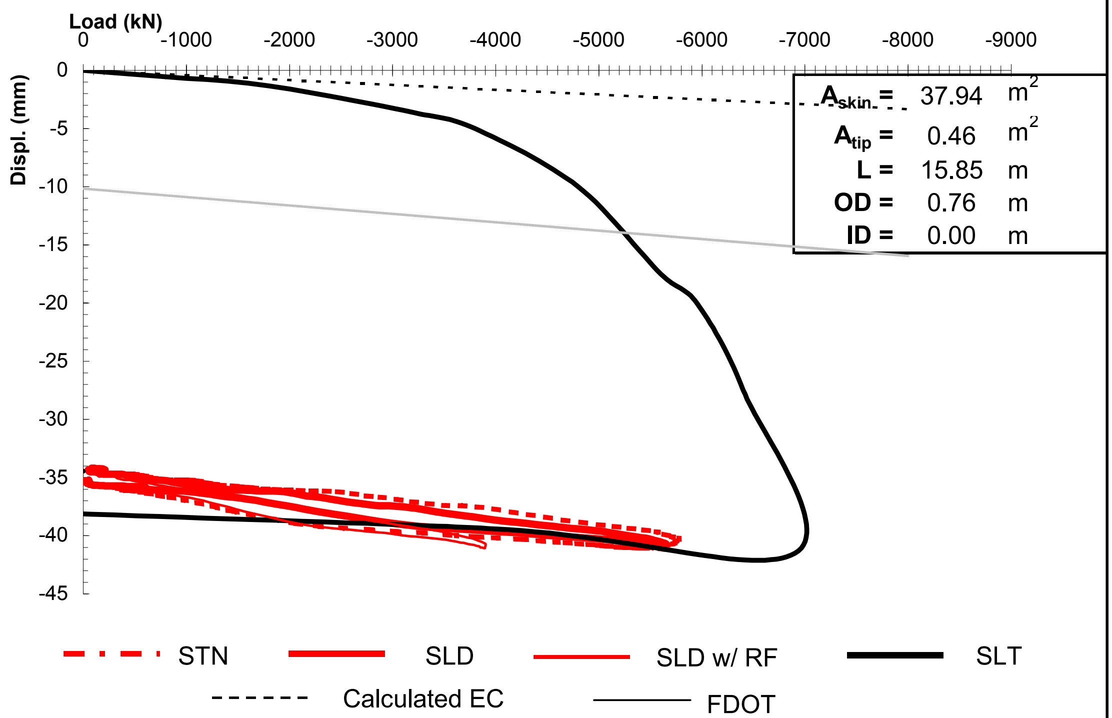


A_{skin} =	37.94	m^2
A_{tip} =	0.46	m^2
L =	15.85	m
OD =	0.76	m
ID =	0.00	m

General Soil Profile	**Additional Project Information**
Tan, gray silty clay over gray fine silty sand w/ seepage water @ bottem of pile.	

STATNAMIC TEST SUMMARY SHEET

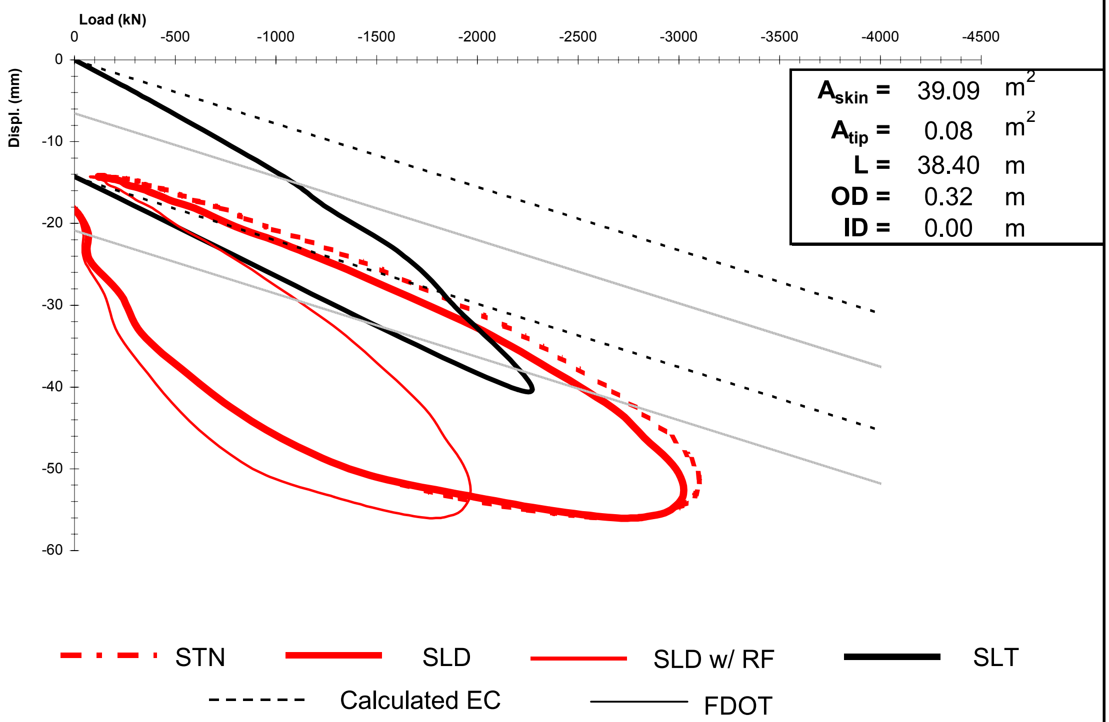
Job Name: FHWA University of Louisiana
Job Location: Shreveport, Louisiana - Pile 3



General Soil Profile	**Additional Project Information**
Tan, gray silty clay over gray fine silty sand w/ seepage water @ bottom of pile.	

STATNAMIC TEST SUMMARY SHEET

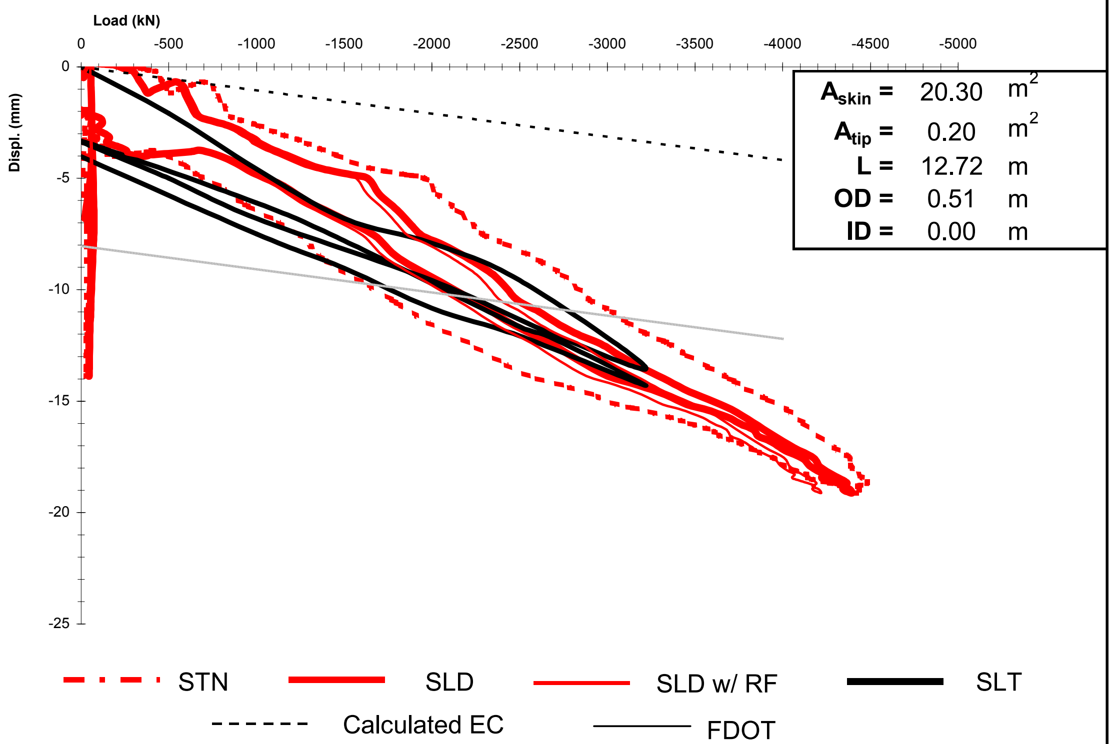
Job Name: Ontario Hydro Lambton Generating Station
Job Location: Courtright, Ontario



General Soil Profile	**Additional Project Information**
clayey silt to silty clay glacial till	

STATNAMIC TEST SUMMARY SHEET

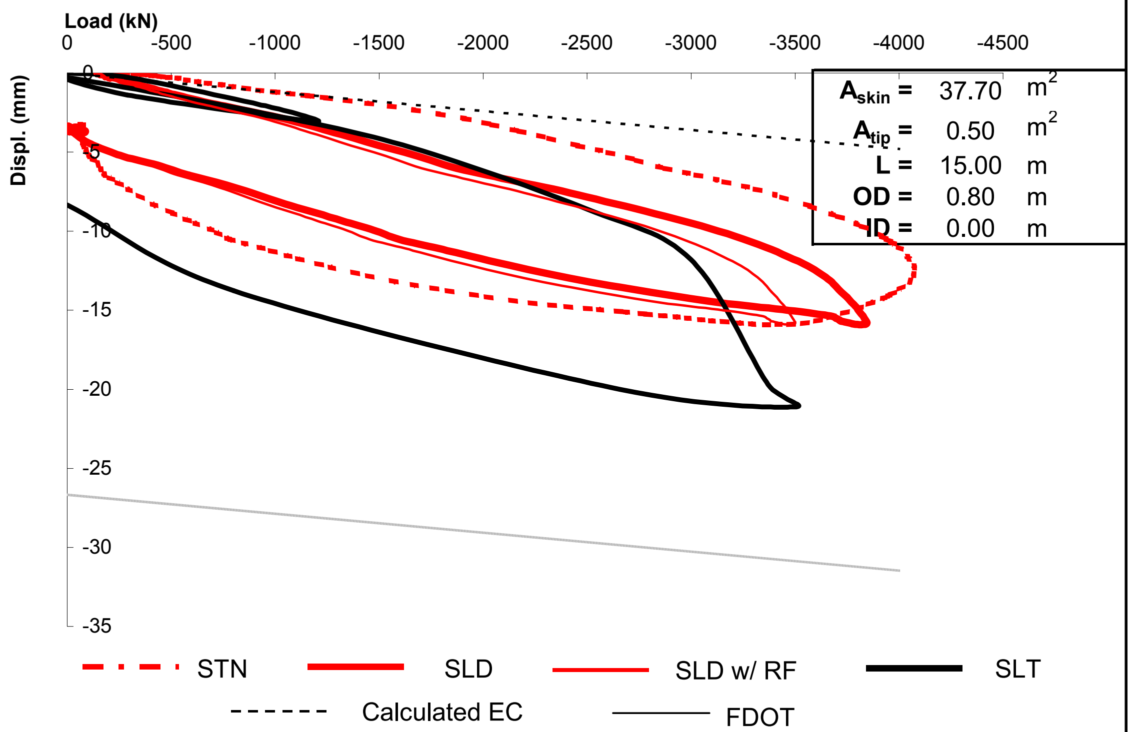
Job Name: Public Works Canada
Job Location: Les Mechins, Quebec



General Soil Profile	**Additional Project Information**
Appx. 10m of fill consisting of cobbles and boulders and containing pieces of wood. Beneath the fill a 3.5m thick layer of gravel and sand with some silt, clay and cobbles overlying bedrock. The bedrock was grey-black mudstone.	

STATNAMIC TEST SUMMARY SHEET

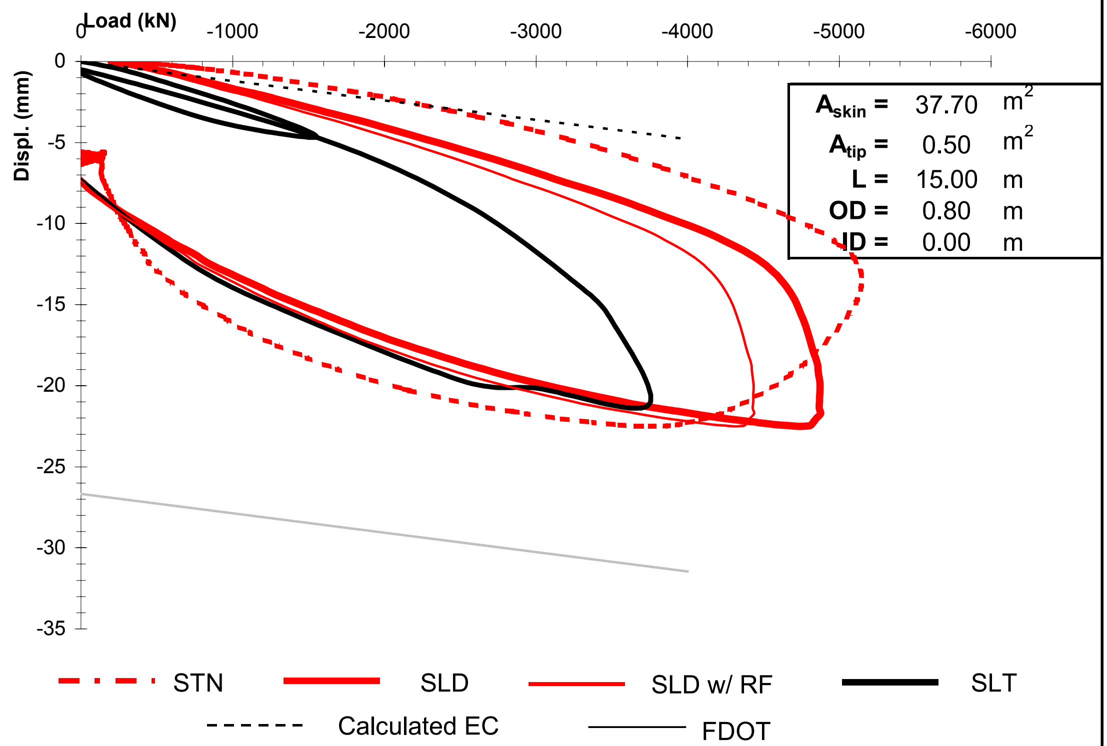
Job Name: Franki Grundbau
Job Location: Emden, Germany - Pile 6



General Soil Profile	**Additional Project Information**
clay over Holocene Sand	

STATNAMIC TEST SUMMARY SHEET

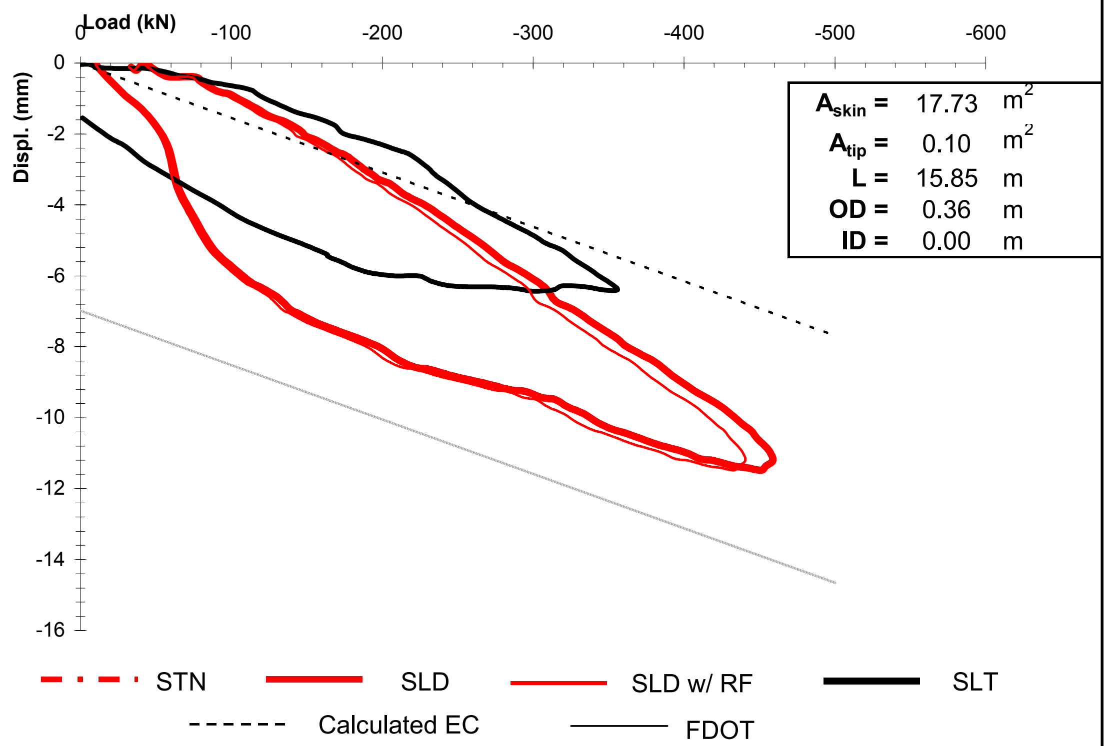
Job Name: Franki Grundbau
Job Location: Emden, Germany - Pile 5



General Soil Profile	**Additional Project Information**
clay over Holocene Sand	

STATNAMIC TEST SUMMARY SHEET

Job Name: Onandoga Co-Generating Facility
Job Location: Syracuse, New York



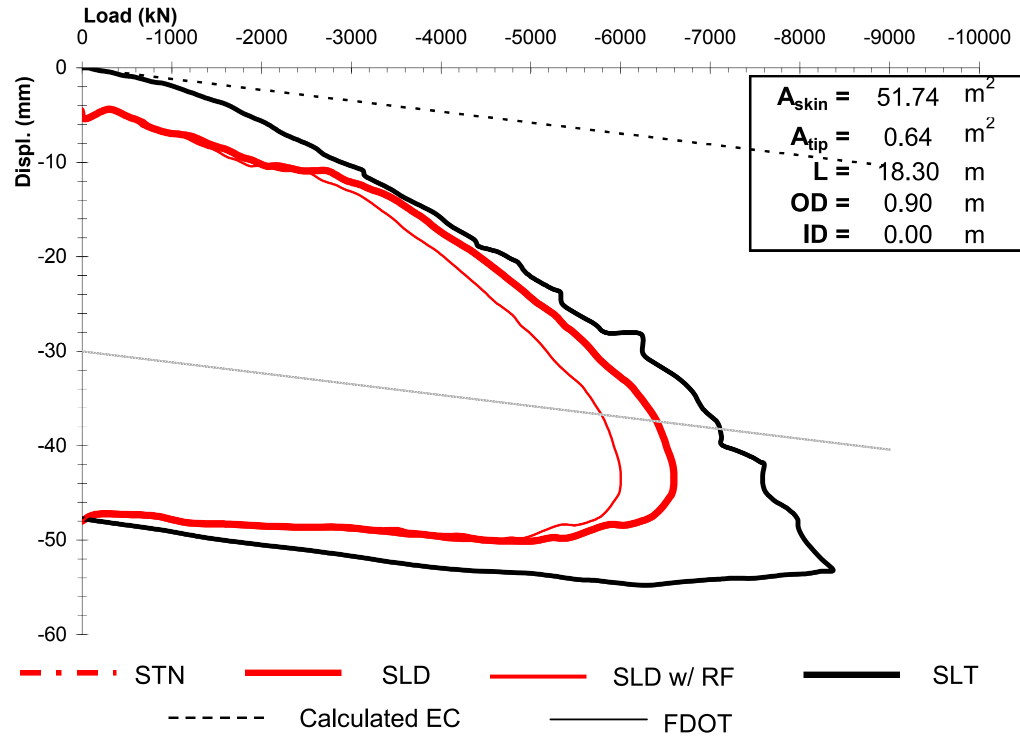
A_{skin} =	17.73	m^2
A_{tip} =	0.10	m^2
L =	15.85	m
OD =	0.36	m
ID =	0.00	m

General Soil Profile
weak soil overlying shale bedrock

Additional Project Information
timber pile

STATNAMIC TEST SUMMARY SHEET

Job Name: New Mexico Department of Transportation
Job Location: Albuquerque, New Mexico pile T1



A_{skin} =	51.74	m^2
A_{tip} =	0.64	m^2
L =	18.30	m
OD =	0.90	m
ID =	0.00	m

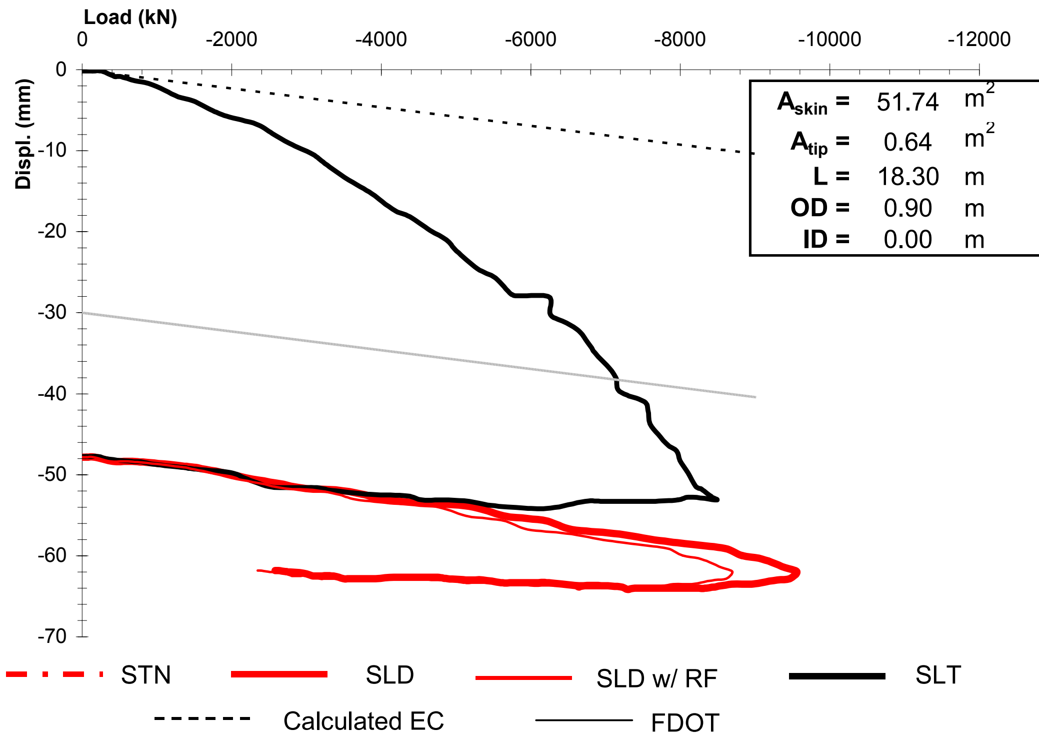
General Soil Profile

sand

Additional Project Information

STATNAMIC TEST SUMMARY SHEET

Job Name: New Mexico Department of Transportation
Job Location: Albuquerque, New Mexico pile T2

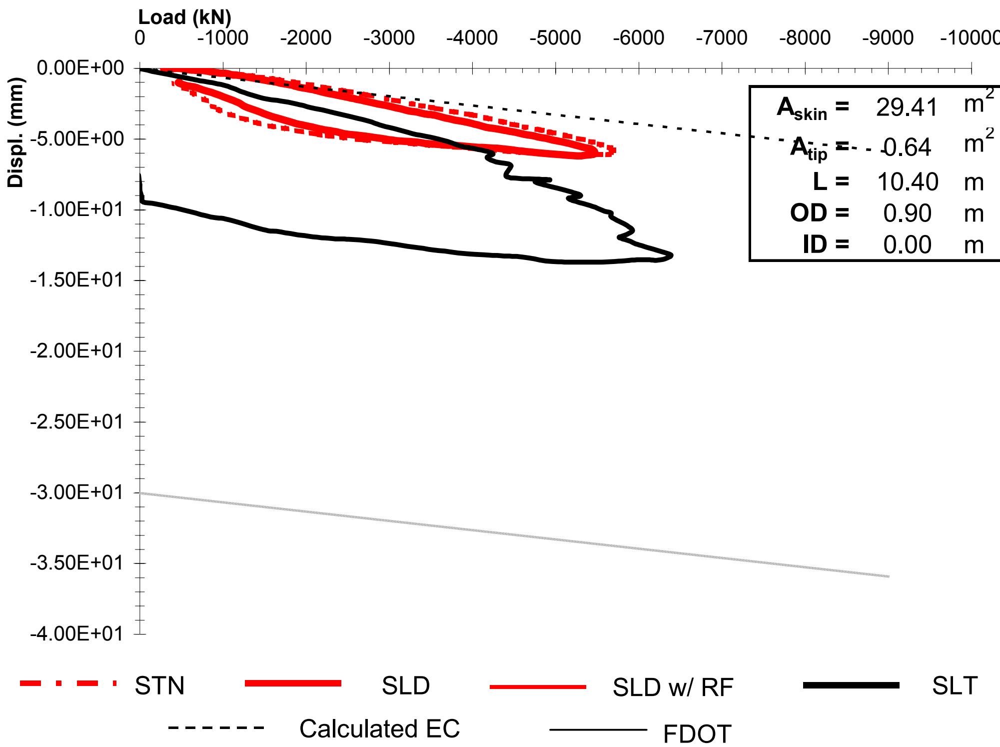


A_{skin} =	51.74	m^2
A_{tip} =	0.64	m^2
L =	18.30	m
OD =	0.90	m
ID =	0.00	m

General Soil Profile	**Additional Project Information**
sand	

STATNAMIC TEST SUMMARY SHEET

Job Name: FHWA
Job Location: Cupertino, Calirfornia - pile 4

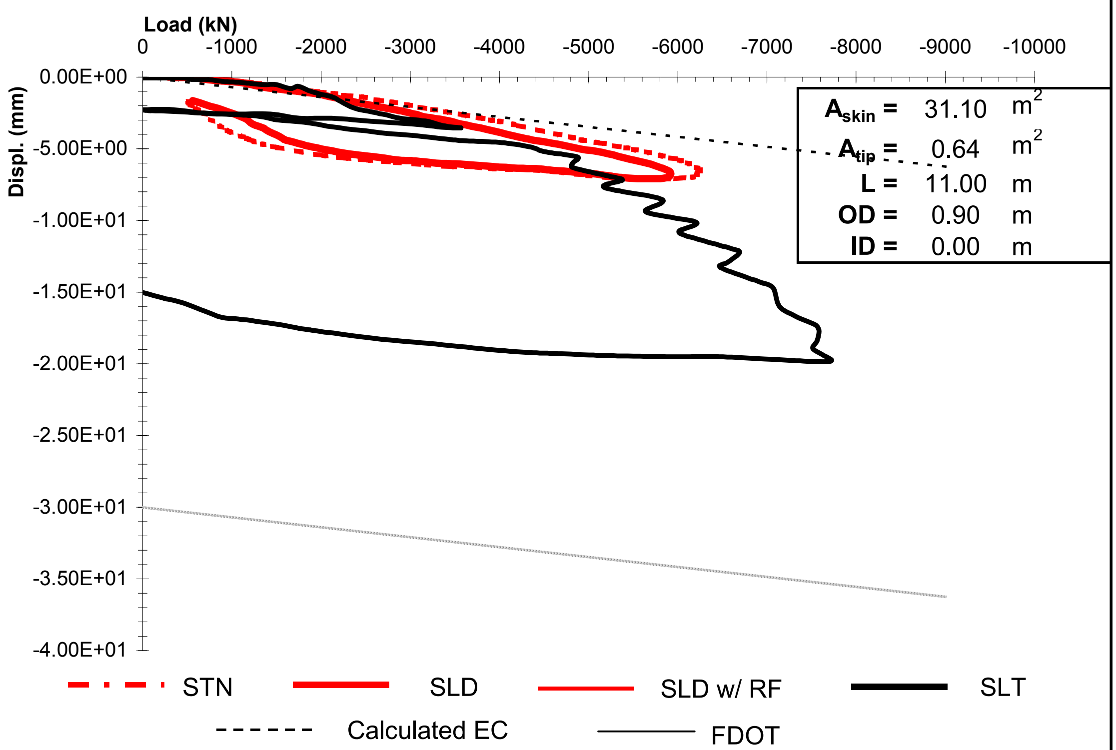


A_{skin} =	29.41	m^2
A_{tip} =	0.64	m^2
L =	10.40	m
OD =	0.90	m
ID =	0.00	m

General Soil Profile	Additional Project Information
gravelly soil	

STATNAMIC TEST SUMMARY SHEET

Job Name: FHWA
Job Location: Cupertino, Calirfornia - pile 2



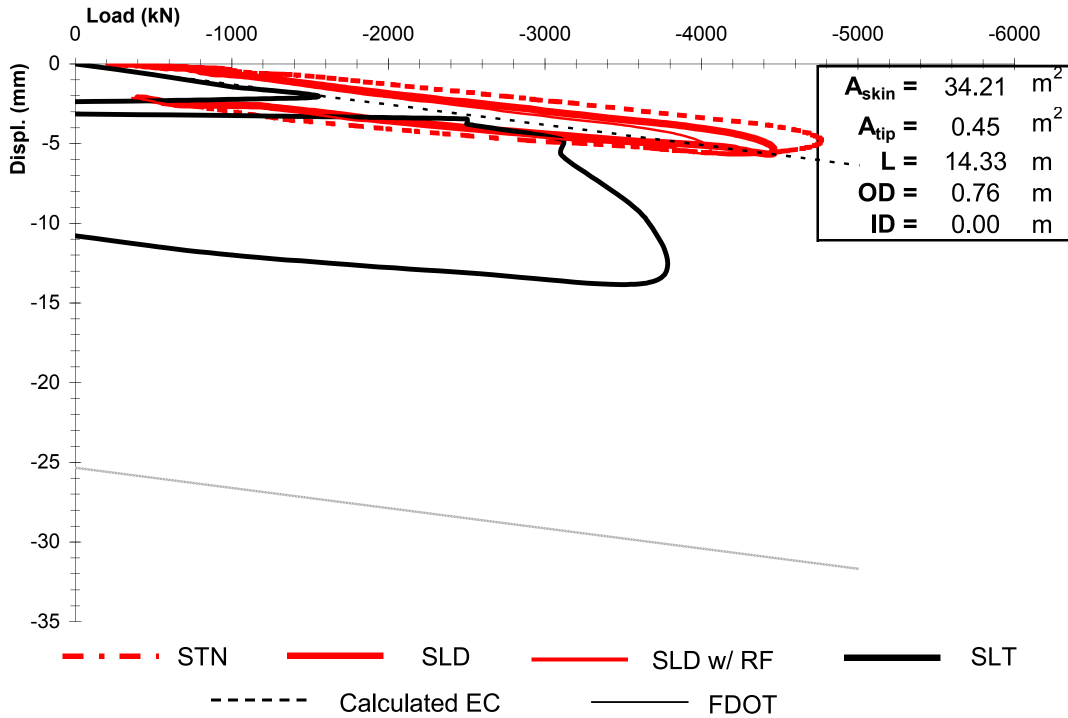
General Soil Profile

gravelly soil

Additional Project Information

STATNAMIC TEST SUMMARY SHEET

Job Name: FHWA NM DOT
Job Location: Gallup, New Mexico



A_{skin} =	34.21	m^2
A_{tip} =	0.45	m^2
L =	14.33	m
OD =	0.76	m
ID =	0.00	m

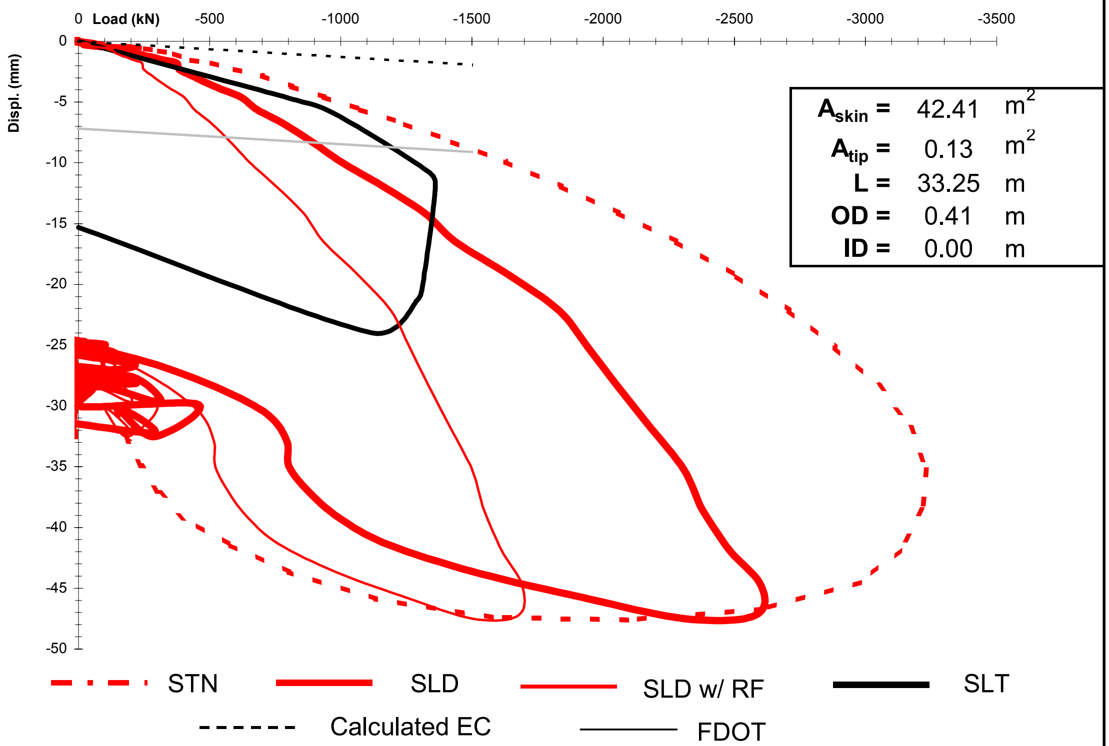
General Soil Profile

Silty Sand

Additional Project Information

STATNAMIC TEST SUMMARY SHEET

Job Name: FHWA Caltrans
Job Location: San Francisco, California



A_{skin} =	42.41	m^2
A_{tip} =	0.13	m^2
L =	33.25	m
OD =	0.41	m
ID =	0.00	m

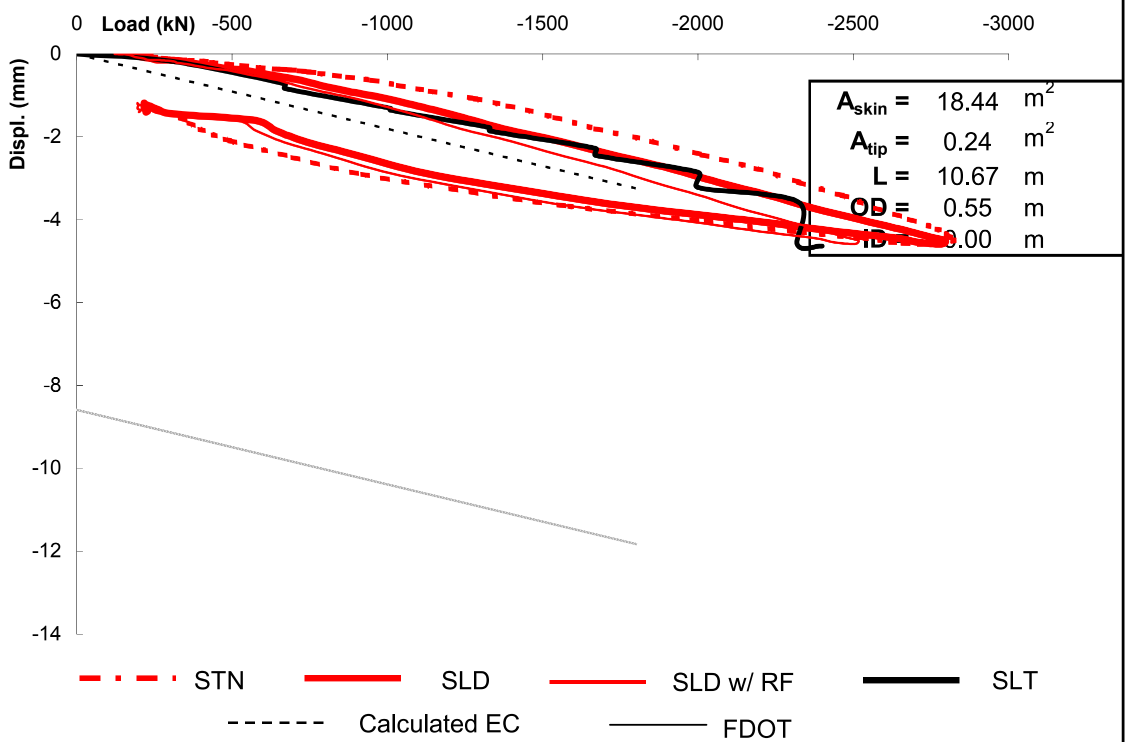
General Soil Profile

Very Soft Bay Mud

Additional Project Information

STATNAMIC TEST SUMMARY SHEET

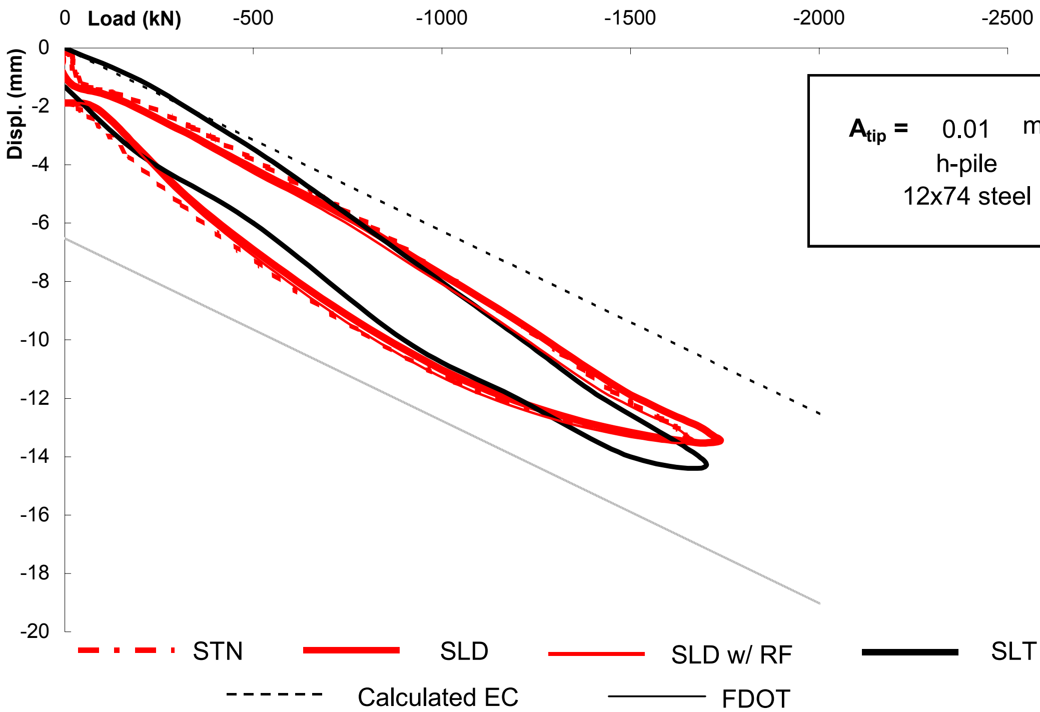
Job Name: 21 St. Joseph St. Anchor Shoring & Caissons
Job Location: Toronto, Ontario



General Soil Profile	**Additional Project Information**
dry sandy soil	

STATNAMIC TEST SUMMARY SHEET

Job Name: Pittsburgh Courthouse
Job Location: Pittsburgh, Pennsylvania

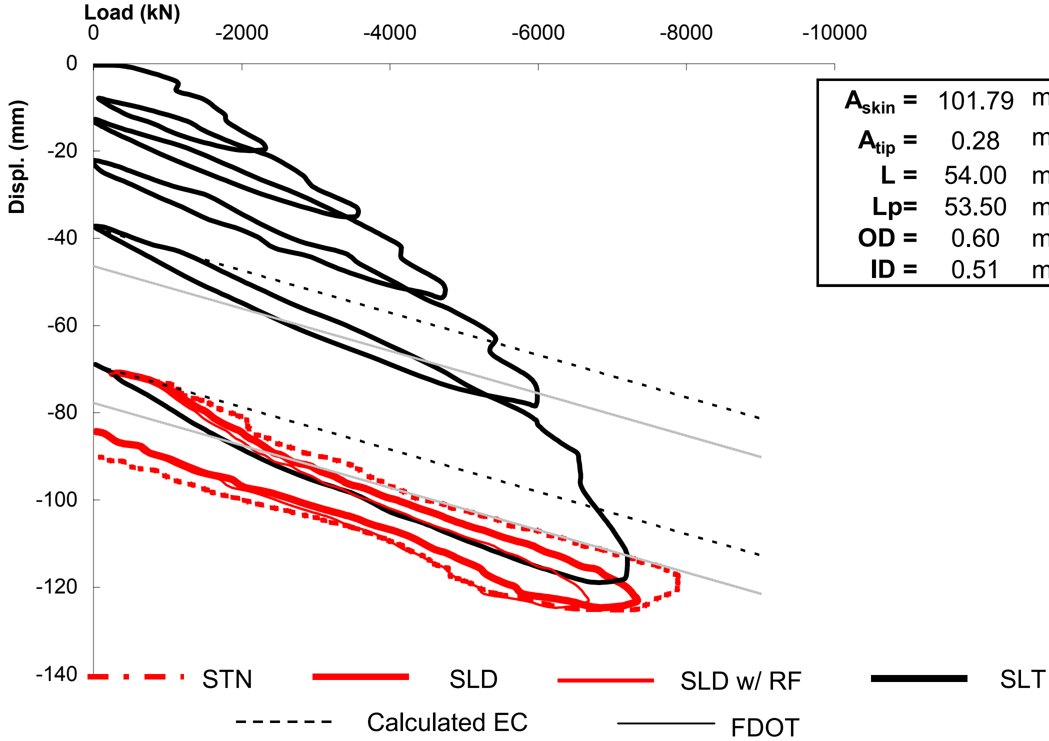


A_{tip} = 0.01 m^2
h-pile
12x74 steel

General Soil Profile	**Additional Project Information**
fill and weak alluvium to end-bearing in rock.	

STATNAMIC TEST SUMMARY SHEET

Job Name: YKN
Job Location: Japan



A_{skin} =	101.79	m^2
A_{tip} =	0.28	m^2
L =	54.00	m
Lp=	53.50	m
OD =	0.60	m
ID =	0.51	m

General Soil Profile

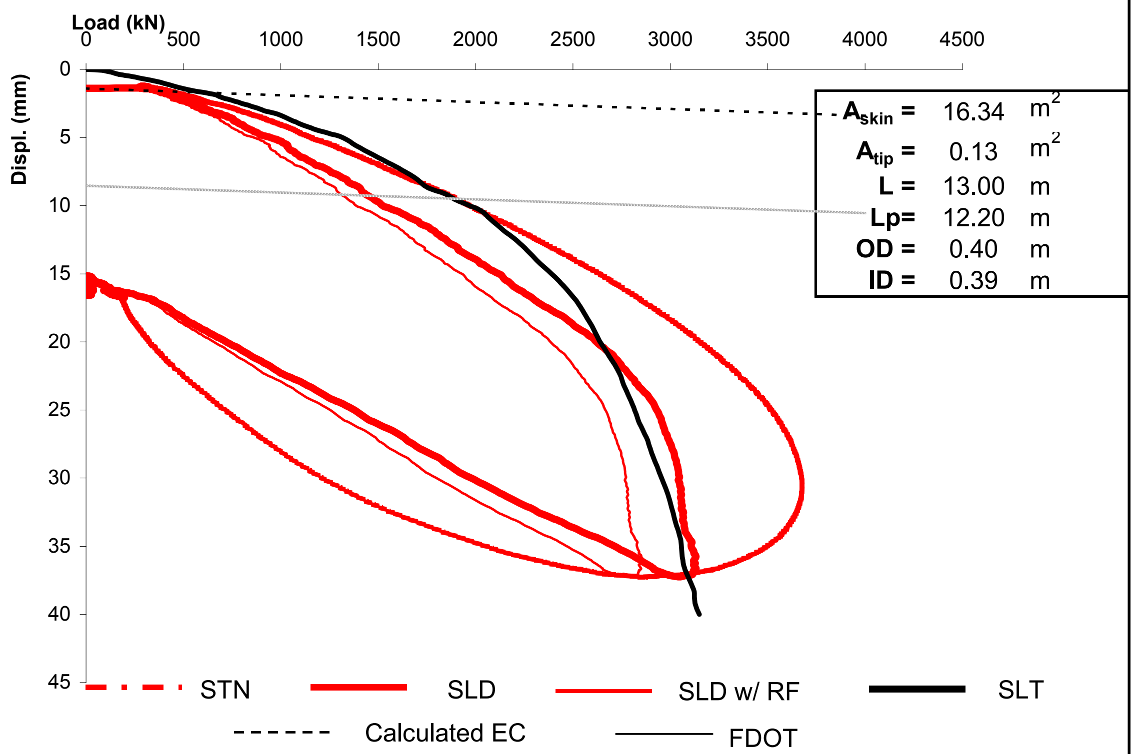
Additional Project Information

SLD by UPM using SAWR4.
$E=3.92E4\ MN/m^2$
$\gamma_\tau=2.6\ t/m^3$
V=3880m/s
W=20.23t
8MN

Source: Report

STATNAMIC TEST SUMMARY SHEET

Job Name: HASAKI
Job Location: JAPAN



General Soil Profile	Additional Project Information
	SLD from UPM. E=2.05E5 γ_t=7.85t/m^3 V=5120m/s