FINAL REPORT

ESTIMATE DAMPING AND QUAKE BY USING TRADITIONAL SOIL TESTINGS

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16. Abstract

Impact pile driving greatly alters the behavior of the soil surrounding the pile. The changes of soil responses make it is very difficult to estimate Smith soil parameters even by means of Pile Driving Analyzer (PDA) monitoring and CAPWAP Analysis. Although GRL, Inc. (1993) had recommended typical values of the Smith damping and quake parameters for different types of soils and pile sizes, many researches indicated that the Smith parameters were not only depended on the soil types and pile sizes, also the pile driving conditions. The ranges of the Smith soil quake and damping from published data were so widely scattered that it was very difficult to select reasonable values for Wave Equation Analysis.

The objectives of this research is to explore the meanings of the Smith soil model in Wave Equation Analysis and identify the key variables affecting the determination of the Smith soil parameters. Using the UF pile database for regression analysis, semi-empirical equations for estimating the Smith soil parameters were developed based on conventional soil properties.

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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)

CHAPTER 1
INTRODUCTION

1.1 General

Since Smith (1960) presented a numerical solution of the one-dimensional wave equation applied to pile driving, Wave Equation Analysis gradually became a primary tool for quality control of pile driving. In performing Wave Equation Analysis, soil parameters are required as input data in addition to the pile and hammer data. However, the Smith soil parameters, soil quake and damping are nonstandard soil mechanics parameters that only can be determined through back analysis of pile driving records and pile load tests.

Impact pile driving greatly alters the behavior of the soil surrounding the pile. The changes of soil responses make it is very difficult to estimate Smith soil parameters even by means of Pile Driving Analyzer (PDA) monitoring and CAPWAP Analysis. Although GRL, Inc. (1993) had recommended typical values of the Smith damping and quake parameters for different types of soils and pile sizes, many researches indicated that the Smith parameters were not only depended on the soil types and pile sizes, also the pile driving conditions. The ranges of the Smith soil quake and damping from published data were so widely scattered that it was very difficult to select reasonable values for Wave Equation Analysis.

1.2 Objectives

The objectives of this research is to explore the meanings of the Smith soil model in Wave Equation Analysis and identify the key variables affecting the determination of the Smith soil parameters, and to develop semi-empirical equations for estimating the Smith soil parameters based on conventional soil properties.

1.3 Scope of work

The overall project will be divided into four major tasks as follows:

Task 1: Identify Representation for Smith soil parameters.

A review of the open literature will be undertaken to identify theoretical representations and published data of Smith soil parameters. Input from the FDOT will also be sought.

Task 2: Database Collection and Analysis.

Pile Database from University of Florida will be used as the primary source in addition to the new cases with appropriate recorded information from the available load test reports. A Excel spreadsheet will be created for statistical analysis.

Task 3: Evaluation of Theoretical Expression of Smith soil parameters.

Based on the database collected in Task 2, theoretical expression collected in Task 1 will be evaluated statistically.

Task 4: Identify or Develop Semi-empirical equations for estimating Smith soil parameters.

Based on the best expression from Task 3, a semi-empirical equation for estimating Smith soil parameters, soil quake and soil damping will be developed. To evaluate the proposed equations, a comparative study by performing Wave Equation Analysis using the Smith soil parameters from the default values recommended by GRL (1993), from CAPWAP analysis and from proposed equations will be conducted.

CHAPTER 2
LITERATURE REVIEW

Smith (1960) presented the one-dimensional wave equation based on solution algorithm for dynamic pile driving. In Smith model, the pile is discretized into lumped masses interconnected by pile "springs." The soil resistance to driving is provided by a series of spring and dashpots. The soil springs are assumed to behave in an elastic-perfectly plastic manner, and the spring stiffness is defined by ratio of the maximum elastic deformation or quake Q. Damping coefficients were introduced to account for the viscous behavior of the soil. The total soil resistance to pile driving is given by

$$R_t = R_s(1 + Jv) = (D_p - D_p')k(1 + Jv) \tag{2.1}$$

where R_t is the total soil resistance to driving; J is the damping coefficient; v is the velocity of the toe of the pile; R_s is the static soil resistance, D_p is the displacement of pile tip, D_p' is the permanent deformation, k is the spring coefficient of soil. The soil spring constant is determined by dividing the resistance assigned to a pile section by the value of soil quake (Q).

Smith quake and damping can only be estimated by means of a load test, or Pile Driving Analyzer (PDA) measurements and Case Pile Wave Analysis Program (CAPWAP) analysis. However, Forehand and Reese (1964) noted that a number of combinations of soil quakes and soil dampings could be used to fit a test point. In addition, driving conditions may result a different combination of Smith soil parameters.

In spite of the question of the uniqueness of the Smith soil parameters, there has been a significant amount of effort performed in the past two decades to understand and compile the numerical values of Smith model parameters (i.e. damping, J, and quake, Q), such as the work by Forehand and Reese (1964), Coyle and Gibson (1970), Coyle, et al (1972), and Litkouchi and Poskitt (1980), among others. However, there has been a lack of understanding about the physical attributes (factors) that affect these constants until recently. Two approaches were used to interpret the Smith damping and quake, theoretical interpretation and in-situ test correlation as follows.

2.1 Theoretical interpretation

1. Lysmer and Richart (1966) modeled the soil resistance at the pile tip as a vertically vibrating rigid disc on the surface of an elastic half-space and derived the following equation for the damping coefficients of pile tip:

$$c_t = \frac{3.4 r_0^2 \sqrt{\rho_t G_s}}{(1 - \upsilon_s)} \tag{2.2}$$

where, ν_s is the soil Poisson's ratio.

2. Novak et al. (1978) derived the soil resistance along the pile shaft using elasto-dynamic theory. The soil resistance before reaching the failure state was expressed in term of pile motion as

$$Q_s = k_s w + c_s \dot{w} \tag{2.3}$$

where Q_s = shaft resistance/unit length of the pile: k_s = soil spring stiffness/unit length of pile; c_s = damping coefficient/unit length of the pile; and w = the pile displacement. The coefficient c_s are given by

$$c_s = \frac{S w_2 G_s r_0}{a_0 V_s} \tag{2.4}$$

where G_s is soil shear modulus, $a_0 = \omega r_0 / V_s$ is a dimensionless frequency ratio, ω is the excitation frequency, r_0 is outer pile radius , V_s is shear wave velocity in the soil, and Sw_2 are functions of the dimensionless frequency ratio a_0. c_s is radiation or geometric damping.

3. Lee and Chow et al (1988) proposed the damping (c_s) coefficients per unit length of the pile shaft based on the work of Novak et al. (1978) as follows:

$$c_s = 2\pi r_0 \sqrt{\rho_t G_s} \tag{2.5}$$

where, G_s is the soil shear modulus, r_0 the pile radius and ρ_t the soil density.

4. Mitwally and Novak (1988) calculated the dynamic skin resistance using one-dimensional shaft model with plane strain conditions as

$$R_d^s = (GS_2/\omega)v \tag{2.6}$$

and for pile tip as

$$R_d^t = (G_b r_0 C_2/\omega)v \tag{2.7}$$

where G and G_b are shear modulus at a shaft and a toe, respectively, S_2 and C_2 are frequency-dependent dimensionless parameters, r_0 is pile radius, and ω is frequency of the pile motion.

5. Nguyen et al. (1988) proposed another equation for the toe quake as a function of shear strain and shear modulus as follows.

$$q = \frac{r_0 \tau_{max}}{2G}\left[\ln\left(\frac{r_m}{r_0}\right) + 2\right] \tag{2.8}$$

where, r_m = 2.5L(1-ν), r_0 is the pile radius, G is the soil shear modulus, τ_{max} is the maximum shear strain and ν is the Poisson's ratio.

6. Liang and Sheng (1992) derived the theoretical expression for Smith toe damping and toe quake based on the dynamic spherical cavity expansion theory and punching theory, respectively. The theoretical expression for Smith skin quake was obtained from the concentric cylinder model originally developed for static load transfer behavior of piles. A semi-empirical rate effect law was used to derive the skin damping.

Toe damping:

$$J_t = \left[\frac{\rho}{3R_s} D \frac{\dot{V}_{pd}}{V_{pd}} + V_{pd}\right] \tag{2.9}$$

Toe quake: $$Q_t = \frac{1+\nu}{2E} p_y \left[\frac{D}{2}\right] \tag{2.10}$$

Skin damping: $$J_s = \frac{K_L}{V_{pd}} \log_{10}\left[\frac{V_{pd}}{V_{ps}}\right] \tag{2.11}$$

Skin quake: $$Q_s = \frac{f_u r_0}{G} \ln\left[\frac{r_m}{r_0}\right] \tag{2.12}$$

where, V_{pd} and V_{ps} are the pile penetration rates under dynamic and quasi-static conditions, respectively. K_L is the soil viscosity coefficient; R_s is static soil resistance, $\dot{V}_{pd}$ is pile penetration acceleration, D is pile size, ρ is soil density, p_y is yield pressure, E the Young's modulus of soil, ν is Poisson's ratio, r_0 and r_m are the radius of pile soil interface and radius of the influence of soil, respectively, G is the soil shear modulus, f_u is the soil shear strength.

The theoretical expression for Smith damping conformed with the experimental findings made by Coyle and Gibson (1970) in which the damping factor varies with the velocity of pile penetration. Furthermore, the Smith damping is a function of static soil resistance, R_s, soil density, penetration acceleration, and the size of the pile D.

As shown in the equations, the toe quake is proportional to the radius of the penetration pile and the skin quake is a function of shear strength, shear modulus, pile radius and disturbance zone.

2.2 In-situ test correlation

1. Svinkin and Abe (1992) developed a simple relationship between Case and hysteretic damping directly from the measured displacement at the pile head. The results suggest that the Case damping coefficient, J_c is not only depend on the soil type but is a

function of the pile-soil system. With the pile-soil system being considered as a dynamic system with measurement at the pile segment near the pile head, the J_c is obtained as

$$J_c = \frac{2\gamma}{\sqrt{4+\gamma^2}} \tag{2.13}$$

where $\gamma = \delta/\pi$ is a dimensionless coefficient of inelastic resistance. This sequence provides calibration of the parameter, J_c, for a particular site and is more reliable than empirical approximation based on a soil type.

2. Abou-matar et al. (1996) proposed a method to estimate the soil damping and soil quake based on the measurements of force and velocity at the top of drill rod during SPT sampler driving. These measurements were used to calculate the transfer energy into the rod. Using the closed form solution of wave propagation theory for linearly elastic rods, the ultimate soil resistance of the SPT sampler was be calculated in addition to the soil damping and soil quake. However, there was not sufficient database to verify the applicability of the Smith soil parameters determined from SPT testing.

3. Malkawi and Mohammad (1996) presented a new simple model for the soil-pile system to determine the effective physical parameters k, c of the soil-pile system, the damping ratio (ξ) and the undamped natural frequency (ω_n). The damping ratio is expressed as

$$\xi = \frac{bc}{2EA} \tag{2.14}$$

where b is damping of the pile (N.sec/m), $c = \sqrt{E/\rho}$ is velocity of wave propagation (m/sec), E is elasticity modulus (N/m^2), ρ is density of pile material (kg/m^3), A is cross sectional area of the pile (m^2).

4. Paikowsky and Chernauskas (1996) indicated that the dynamic soil resistance in term of viscous damping was inadequate and incorrect. The soil damping used in the dynamic analysis cannot correlate to the soil type. Thendean, et al (1996) also indicated

that the Smith soil parameters back calculated from CAPWAP varied strongly within the same soil type and, surprisingly, even the averages showed very little correlation with soil type. The actual physical phenomenon controlling the dynamic resistance is the soil inertia. Examinations of a large data set indicated that under high soil inertia conditions, the performance of the wave matching techniques is extremely poor, while for low soil inertia conditions the performance of the wave matching technique is extremely good.

CHAPTER 3
DATABASE DEVELOPMENT AND EVALUATION

3.1 General

In order to evaluate the theoretical expressions and in-situ test correlation for the Smith soil parameters, i.e., skin damping, toe damping, skin quake and toe quake, a database was developed to include pile length/diameter, hammer data, driving record, Smith soil parameters back-calculated from CAPWAP analysis, related soil properties. Soil properties include weighted average SPT N-value along pile shaft and at pile tip, static soil resistance along pile shaft and at pile tip determined from static load test and/or CAPWAP analysis, pile velocity from PDA measurement.

The database, known as PILEUF developed by the University of Florida was used as the template which was in Lotus 1-2-3 format and was transported into Microsoft Excel for this research. Currently, PILEUF contains 213 pile data including 147 concrete piles, 17 steel pipe piles and 49 steel H-piles. The information of each pile record included:

1. General description of job location, pier or bent number, dates of load tests and name of engineer in-charge or reference.
2. Pile data including pile type, geometry, installing method and soil type.
3. Pile driving information including hammer type and weight, rated energy, depth of penetration and corresponding blow count or set.
4. Load test results including load vs. settlement, failure capacity in term of Davisson, Fuller-Hoy, DeBeer or FDOT failure criteria.
5. In-situ test results including Standard Penetration Test (SPT) data, Cone Penetration Test (CPT) data, and soil description.
6. Dynamic load test data including PDA and CAPWAP results.

However, the Smith quake parameters and driving record were not included or completed in the original UF pile database and were retrieved from the available load test reports. The weighted average SPT N value along pile length is calculated for each pile from available soil boring information.

3.2 Completeness of Data Set

The purpose of the new database is to provide all the necessary information to estimate the Smith soil parameters. Followings are the necessary information of a data set:

1. Soil boring information including SPT data and soil description.
2. Static load test carried to failure and the Davisson failure capacity can be determined.
3. Dynamic load test was performed with PDA monitoring.
4. Pile driving system.
5. CAPWAP results including pile capacity and Smith soil parameters at End of Initial Driving (EOD) and Beginning of Redriving (BOR).

3.3 Database Evaluation

A new database was developed by retrieving necessary information from PILEUF and adding more information from pile load test report for this research as follows. A printout of the database is presented in Appendix A. The database is an Excel spreadsheet consisting of 147 pile records. All of the piles in the database are pre-stressed concrete, driven in the State of Florida.

3.3.1 Soil Classification

Seven (7) soil types were used in PILEUF for soil classification as follows:

1. Plastic clay
2. Silt-sand-clay, silts and marls
3. Clean sand

4. Limestone, very shelly sand
5. Clayey sand
6. Sandy clay
7. Silty clay

However, considering the current practice in correlating the Smith soil parameters with soil types, soil classifications were grouped into three (3) major soil types:

1. Cohesive soil including plastic clay, sandy clay and silty clay.
2. Non-cohesive soil including silty-sand-clay, silts and marls, clean sand and clayey sand.
3. Limestone including limestone and very shelly sand.

3.3.2 Pile Driving Information

Pile information included the width, total length, and embedded length of the pile. The total length and embedded length of the pile were the length at the end of initial driving (EOD) or beginning of redriving (BOR) and in general were obtained from the pile driving records. In addition, the "primary" soil type along the pile shaft and the soil type at the pile tip were included.

Pile driving information including hammer type and weight, rated energy, depth of penetration and the corresponding blow count or set.

3.3.3 Standard Penetration Test

Only very limit of CPT data were available in PILEUF, therefore, SPT data was used as the primary in-situ test information. The weighted average SPT-N value along the pile length and over an interval from 8 times of pile size above to 3.5 time pile size below the pile tip within the same soil layer were calculated as skin-N value (N_s) and tip-N value (N_t), respectively, for each pile data. The maximum SPT-N value was limited to 60 as recommended by Schmertmann (1967)

3.3.4 Load Test and CAPWAP Results

Static soil resistances determined from static load tests, PDA monitoring and CAPWAP analyses were included in the database. Smith damping and quake values from CAPWAP analyses at EOD and BOR for each pile were also included. The Smith soil quakes and the maximum transfer energy during pile driving were not included in the PILEUF and were retrieved from the pile load test reports into the new database.

3.4 Database Summary

Figures 3.1 through 3.4 and Figures 3.5 through 3.8 showed the Smith soil parameters at the end of driving (EOD) and beginning of redriving (BOR), respectively, from the UF database. It can be seen that there were significant deviations between the two different sources. Because of lacking physical meanings, the Smith soil quake and damping can only be back calculated by CAPWAP analysis using the data from Pile Driving Analyzer (PDA) measurements. Although GRL, Inc. (1993) has recommended typical values of the Smith damping and quake parameters for different types of soils and pile sizes, many researches indicated that the Smith parameters were not only depended on the soil types and pile sizes, also the pile driving conditions. The ranges of the Smith soil quakes and dampings from database were so widely scattered as shown in Table 3.1 that it was very difficult to select reasonable values for wave equation analysis. The objective of this research is to identify the key variables affecting the Smith soil parameters and to develop a method to estimate the reasonable Smith soil parameters.

Table 3.1
Summary of Smith Soil Parameters

			Non-Cohesive	Cohesive	Limestone
GRL (1993)	Damping (s/ft)	Skin	0.05	0.20	-
		Toe	0.15	0.15	-
	Quake (in)	Skin	0.10	0.10	-
		Toe	D/120	D/120	-
EOD/ Database	Damping (s/ft)	Skin	0.050-0.421	0.051-0.316	0.105-0.272
		Toe	0.049-0.444	0.035-0.208	0.050-0.439
	Quake (in)	Skin	0.050-0.191	0.020-0.140	0.030-0.150
		Toe	D/45-D/533	D/35-D/200	D/44-D/300
BOR/ Database	Damping (s/ft)	Skin	0.084-0.490	0.074-0.736	0.04-0.535
		Toe	0.002-0.700	0.012-0.649	0.072-0.430
	Quake (in)	Skin	0.055-0.156	0.050-0.200	0.020-0.180
		Toe	D/51-D/333	D/64-D/375	D/39-D/600

Note: D is pile width or diameter in inch.

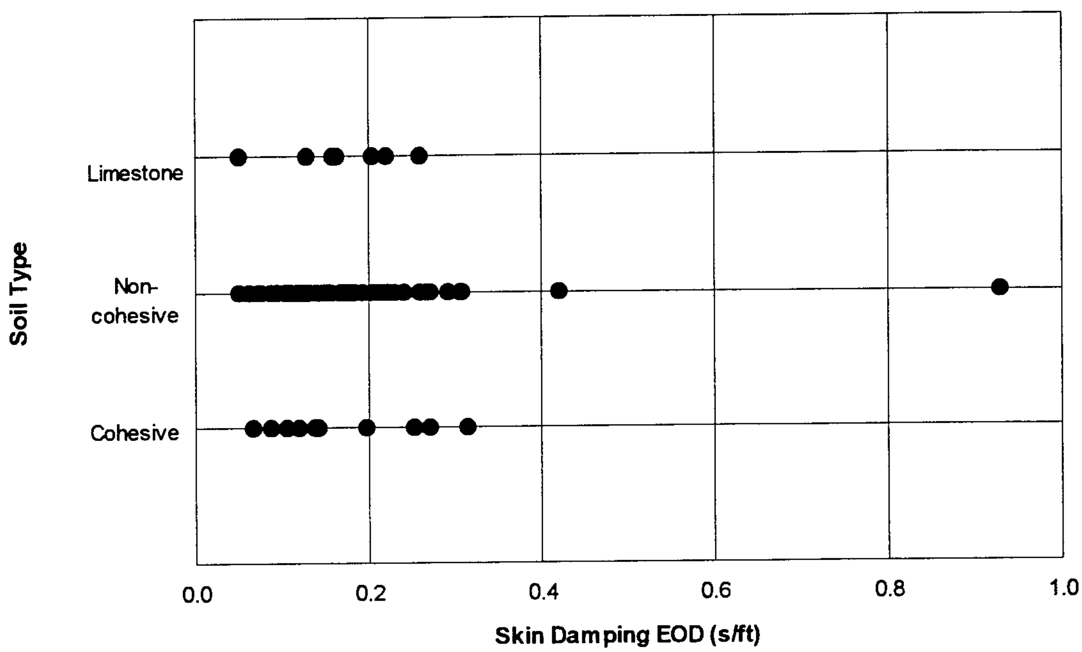

FIGURE 3.1 Skin Damping at EOD vs. Soil Type

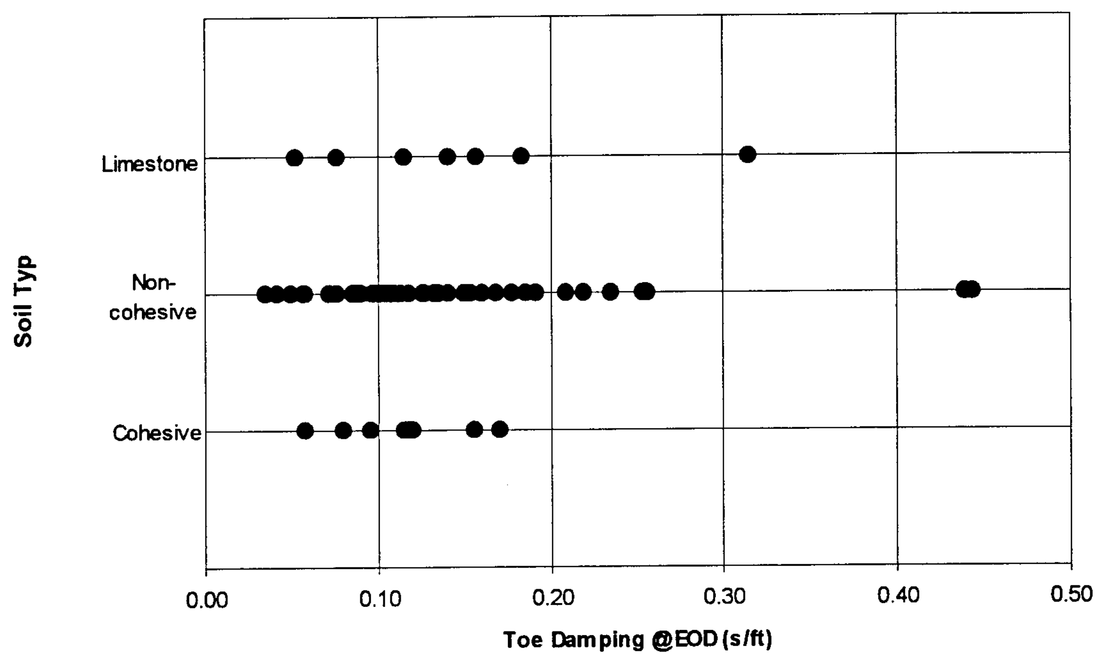

FIGURE 3.2 Toe Damping at EOD vs. Soil type

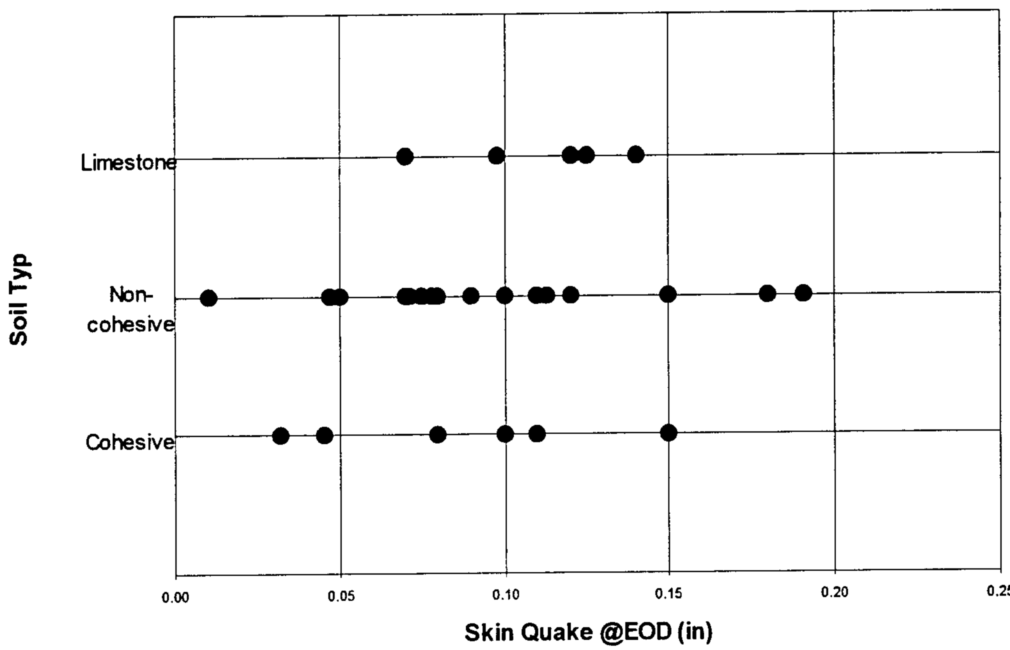

FIGURE 3.3 Skin Quake at EOD vs. Soil Type

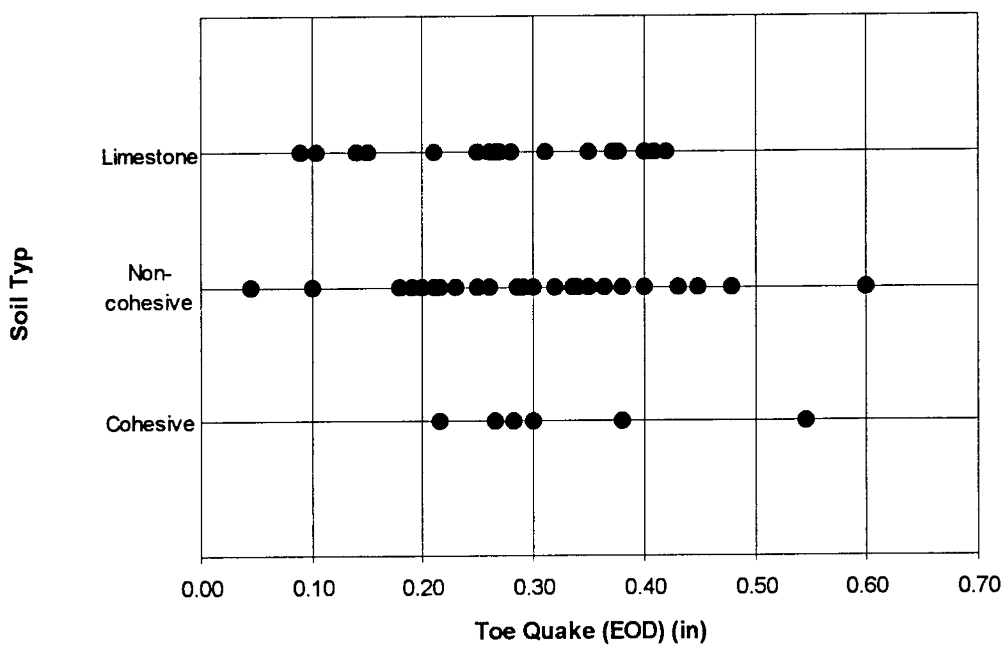

FIGURE 3.4 Toe Quake at EOD vs. Soil Type

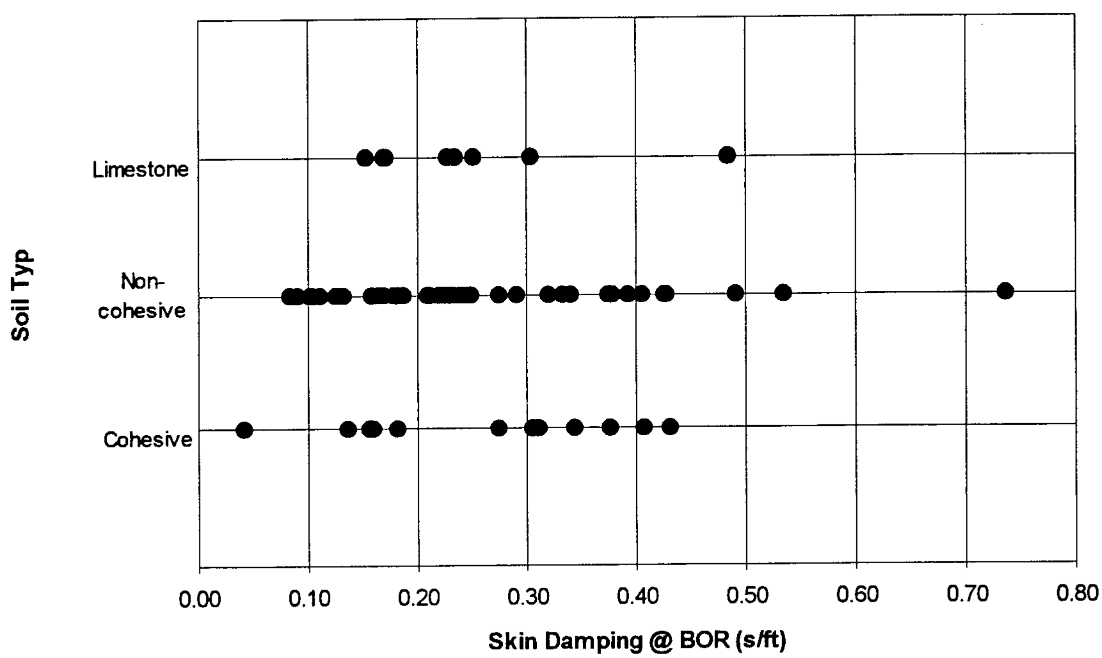

FIGURE 3.5 Skin Damping at BOR vs. Soil Type

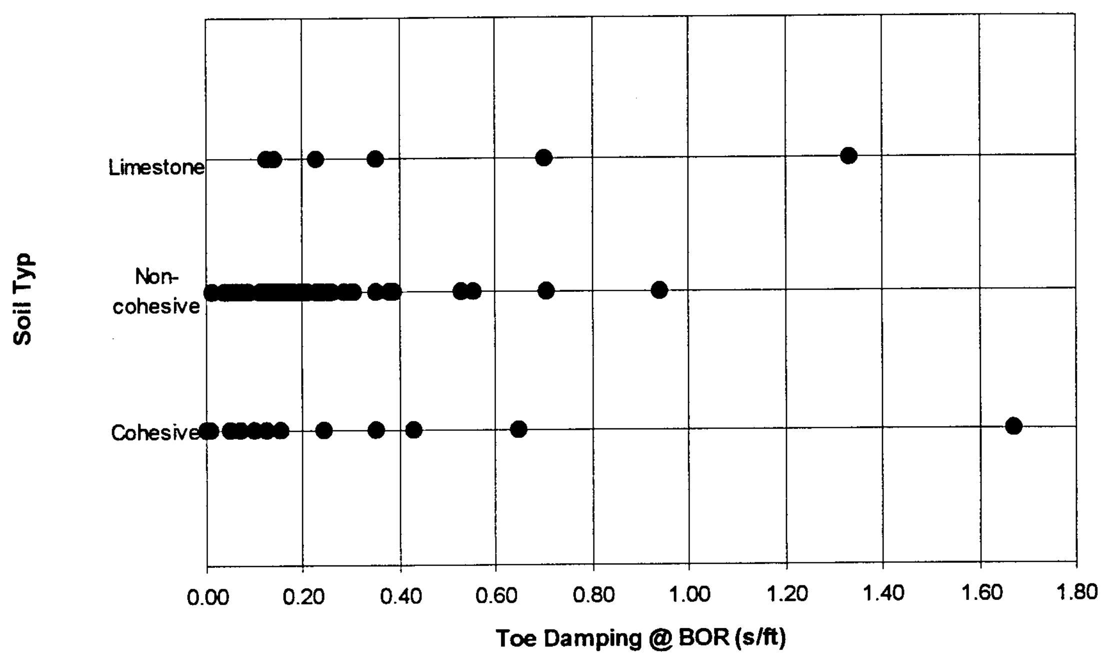

FIGURE 3.6 Toe Damping at BOR vs. Soil Type

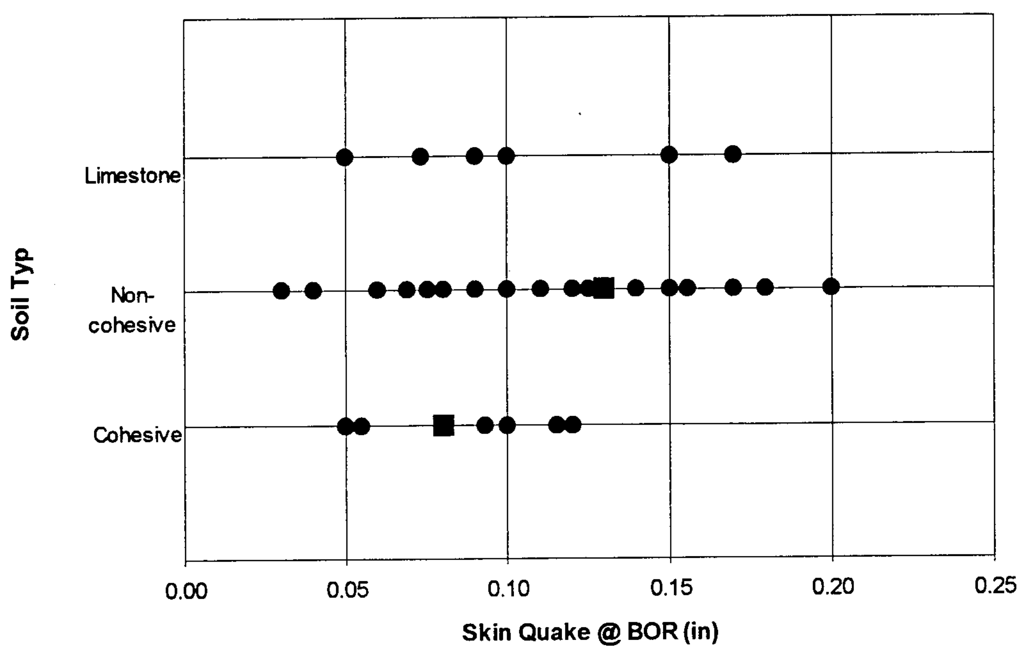

FIGURE 3.7 Skin Quake at BOR vs. Soil Type

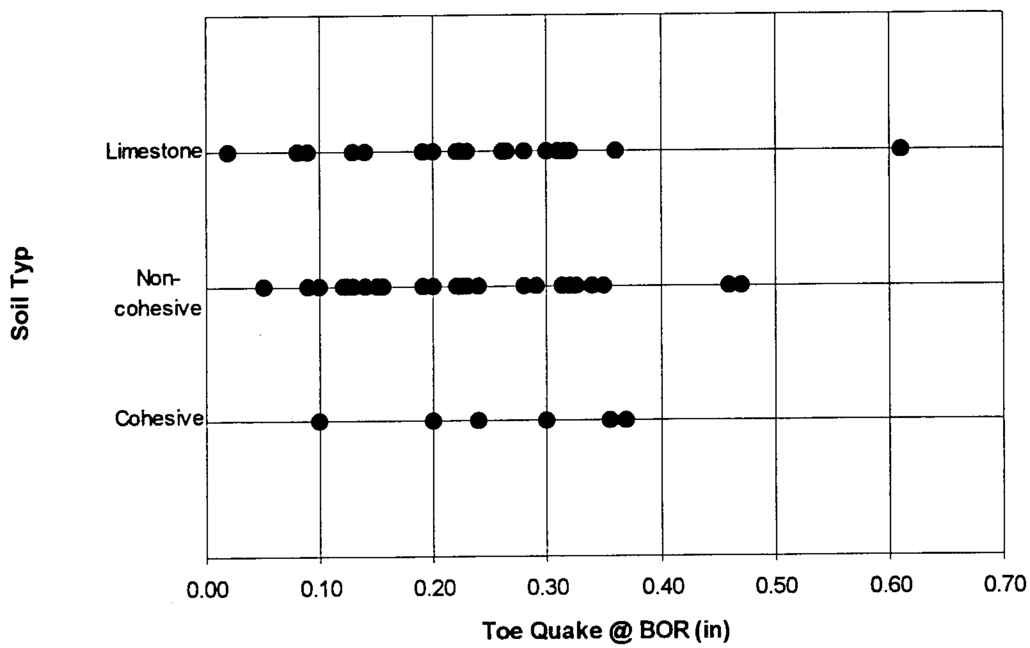

FIGURE 3.8 Toe Quake at BOR vs. Soil Type

CHAPTER 4

REGRESSION ANALYSIS

4.1 Theoretical Expression of Smith Parameters

From the literature review, Liang and Sheng (1992) appeared providing a most complete theoretical expressions for the Smith soil parameters and were summarized as follows.

4.1.1 Toe Damping

Liang and Sheng (1992) derived the theoretical expression of the Smith toe damping based on the dynamic spherical cavity expansion theory.

$$J_t = \left[\frac{\rho}{3R_s} D \frac{\dot{V}_{pd}}{V_{pd}} + V_{pd} \right] \tag{4.1}$$

where, V_{pd} is the pile penetration rate under dynamic conditions, $\dot{V}_{pd}$ is the pile penetration acceleration under dynamic condition, D is the pile size, ρ is the soil density, and R_s is the static soil resistance.

As shown in the equation, the Smith toe damping will increase when the pile penetration rate increase, and will decrease when the soil resistance increases. Also, the bigger size of pile will result larger toe damping.

4.1.2 Skin Damping

The Smith skin damping factor can be expressed as (Liang and Sheng, 1992)

$$J_s = \frac{K_L}{V_{pd}} \log\left[\frac{V_{pd}}{V_{ps}} \right] \tag{4.2}$$

where V_{pd} and V_{ps} are pile penetration rate under dynamic and quasi-static conditions. Quasi-static penetration rate V_{ps} can be estimated from static load test results according to

Davisson's failure criteria, $V_{ps} = x/t$, where $x = 0.15+D/120$ (in) is pile displacement under static load, t is time for load testing, D is the pile diameter. K_L is soil viscosity coefficient.

As shown in the equation, the skin damping has the similar meaning with the soil viscosity coefficient and will be affected by the pile penetration rate.

4.1.3 Toe Quake

According to Smith (1960), the soil quake was defined as the maximum elastic soil deformation. Based on the quasi-static spherical cavity expansion theory, Liang and Sheng (1992) derived the toe quake (Q_t) as follows.

$$Q_t = \frac{1-\nu}{2E} p_y \left[\frac{D}{2}\right] \tag{4.3}$$

where, E is the Young's modulus of soil, ν is the Poisson's ratio, p_y is the soil's yielding stress and D is the pile size.

As shown in the equation, the Smith toe quake will decrease when the Young's modulus of soil increases. Also, the bigger size of pile will result a larger toe quake.

4.1.4 Skin Quake

Liang and Sheng (1992) proposed the following equation for the skin quake based on the load transfer mechanism developed by Kraft, et al (1981)

$$Q_s = \frac{f_u r_0}{G} \ln\left[\frac{r_m}{r_0}\right] \tag{4.4}$$

where G is the soil shear modulus, f_u is the soil shear strength, r_0 and r_m are the radius of pile soil interface and radius of the influence of soil, respectively. $r_m = 2.5L(1-\nu)$.

As shown in the equation, the Smith skin quake will decrease when the shear modulus of soil increases.

4.2 Correlation of Smith Soil Parameters

Although the Smith damping and quake can be estimated from the theoretical expression as shown in the previous section, many variables required in the equations were not available from the database or not measured from PDA monitoring. In addition, the derivations of the equations were based on the assumption that the soils reached the failure state at every hammer blow during pile driving. However, soil resistances may not be completely mobilized due to the insufficient hammer energy. Therefore, different values of soil quakes may be estimated under different hammer energies for the same pile as shown in Figure 4.1. Even for the same set of PDA data, different engineers may result different set of Smith soil parameters from CAPWAP analyses (Bengt Fellenius, 1992) as shown in Figures 4.2 and 4.3 for soil quake and soil damping, respectively.

Due to the uncertainties of pile driving conditions and the lack of unique solution of CAPWAP analysis, the determination of Smith soil parameters became extremely difficult. However, according the expressions proposed by Liang and Sheng (1992) and the definition provided by Smith (1964) for the soil quake and soil damping, the Smith soil parameters are function of soil stiffness, soil strength and hammer transfer energy. Practically, soil stiffness and soil strength can be empirically estimated from SPT-N value, and the pile penetration rate depends on the soil strength and hammer energy. Therefore, the Smith soil parameters logically can be expressed in term of SPT-N value and hammer energy.

4.2.1 Smith Soil Parameters versus SPT-N Value

Figures 4.4 to 4.11 showed the plots of SPT-N values versus Smith soil parameters. According to the theoretical expressions, Smith soil parameters are related to the shear modulus of soil that can be estimated from SPT-N value. However, as shown in the figures, there was no clear correlation observed. It is believed that in addition to the SPT-N value, the transfer energy should have major impacts on the determination of

Smith soil parameters. It is because that the mobilization of the pile capacity depends on the energy transferred into the pile from the pile driving system.

4.2.2 Smith Damping versus Pile Penetration Rate

Figure 4.12 showed the Smith skin dampings and the pile penetration rate at EOD condition. In general, the damping increased with the penetration rate increased. However, the variation was relatively high.

4.3 Linear Regression Evaluation

Although the Smith damping and quake can be estimated from the theoretical expression proposed by various researchers, many variables in the equations are not available from the current database to evaluate these equations. However, according to the theoretical expressions and in-situ test correlation of Smith soil parameters discussed in the previous sections, the Smith soil parameters can be expressed in term of soil type, SPT-N value (N), pile size (D), and transfer energy (E) of driving system as the following relations:

- Skin Quake (Q_s) $\propto$ {Soil type (Cohesive, Non-Cohesive, Limestone), N_s, E}
- Toe Quake (Q_t) $\propto$ {Soil type (Cohesive, Non-Cohesive, Limestone), D, N_t, E}
- Skin Damping (J_s) $\propto$ {Soil type (Cohesive, Non-Cohesive, Limestone), N_s, E}
- Toe Damping (J_t) $\propto$ {Soil type (Cohesive, Non-Cohesive, Limestone), N_t, E}

Where N_t and N_s are the average SPT-N values at the pile tip and along the shaft, respectively.

Using the database described in chapter 3, different regressions were evaluated to reach the best correlation. The following basic format was selected:

For skin damping and quake:

$$\ln(X) = A\frac{E}{N_s}\ln(X) + B \tag{4-5}$$

for toe damping and quake:

$$\ln(X) = A\frac{E}{N_t}\frac{D}{12}\ln(X) + B \tag{4-6}$$

where X = the Smith soil parameter(in for quake, sec/ft for damping)

A, B = the regression constants

N_s, N_t = weighted average skin and toe SPT-N value

E = rated energy of hammer (kips-ft)

D = pile size(in)

Figures 4.13 to 4.18 showed the regression of skin quakes at the end of initial driving (EOD) and beginning of resdriving (BOR) for different soil types, each correlation graph includes a first order best fit line. Figures 4.19 to 4.24 showed the regression of toe quake at EOD and BOR for different soil types. Figures 4.25 to 4.30 showed the regression of skin damping, and Figures 4.31 to 4.36 showed the toe damping at EOD and BOR for different soil types.

Equation 4-5 and 4-6 can be rewritten as:

For skin damping and skin quake

$$X = e^{\left(\frac{B}{A\cdot\frac{E}{Ns}-1}\right)} \tag{4-7}$$

for toe damping and toe quake

$$X = e^{\left(\frac{B}{A\cdot\frac{E}{Nt}\cdot\frac{D}{12}-1}\right)} \tag{4-8}$$

Based on the results of regression evaluation and Equation (4-7) and (4-8), the constants A and B of the Smith Soil parameters and corresponding coefficient of determination, R^2 are summarized in Table 4-1.

Table 4-1 Regression Analysis Constants of Smith Soil Parameters

Parameter	Soil Type	A	B	R^2
EOD Skin Quake	Non-Cohesive	0.0209	2.1486	0.0973
	Cohesive	0.0024	2.3258	0.0056
	Limestone	0.4114	-0.0301	0.996
BOR Skin Quake	Non-Cohesive	0.0138	2.057	0.1867
	Cohesive	0.0171	1.9058	0.3231
	Limestone	0.2374	1.3498	0.8354
EOD Toe Quake	Non-Cohesive	0.0358	0.914	0.4236
	Cohesive	0.0106	1.0476	0.7365
	Limestone	0.0261	1.1989	0.0971
BOR Toe Quake	Non-Cohesive	0.0289	1.1802	0.2538
	Cohesive	-0.0196	2.3603	0.8813
	Limestone	0.081	1.1567	0.5837
EOD Skin Damping	Non-Cohesive	0.0251	1.7143	0.1722
	Cohesive	0.0247	1.4873	0.269
	Limestone	0.0402	1.9008	0.0563
BOR Skin Damping	Non-Cohesive	0.0219	1.2902	0.1191
	Cohesive	0.0326	1.0556	0.5241
	Limestone	0.2996	0.8214	0.9835
EOD Toe Damping	Non-Cohesive	0.0088	1.9217	0.0447
	Cohesive	0.0043	1.8556	0.0544
	Limestone	0.0257	2.1241	0.2244
BOR Toe Damping	Non-Cohesive	0.0371	1.1763	0.2322
	Cohesive	0.006	1.2628	0.2227
	Limestone	0.0062	2.6315	0.0029

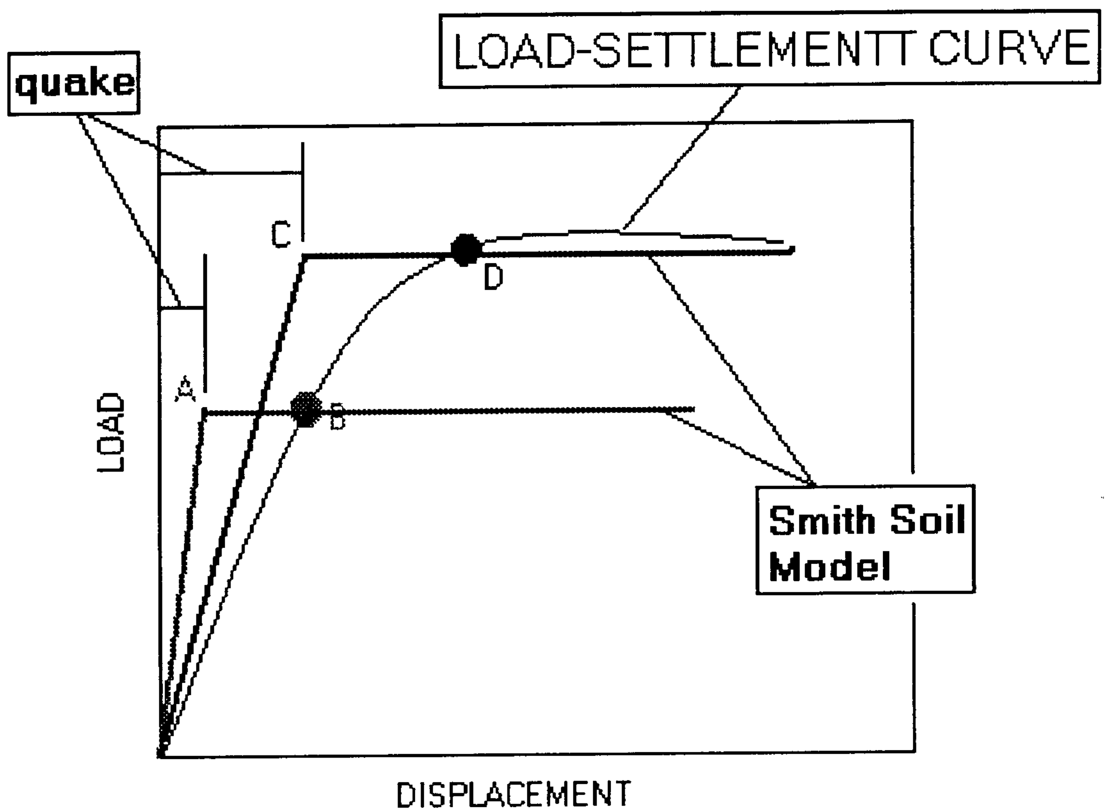

FIGURE 4.1 The Impact of Capacity Mobilization on Determining Soil Quake

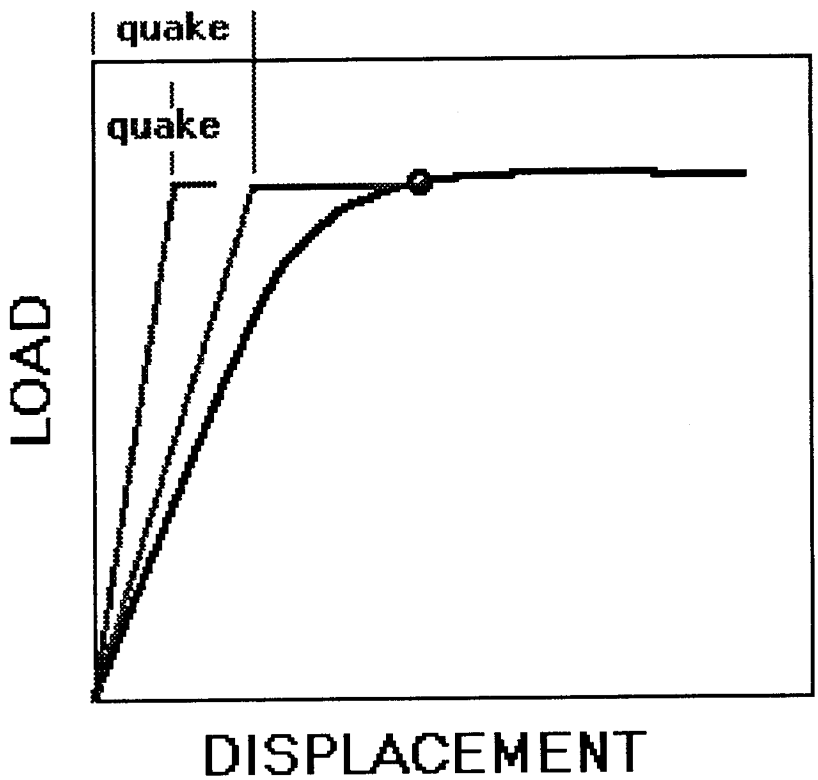

FIGURE 4.2 The Impact of Different Engineers on Determining Soil Quake

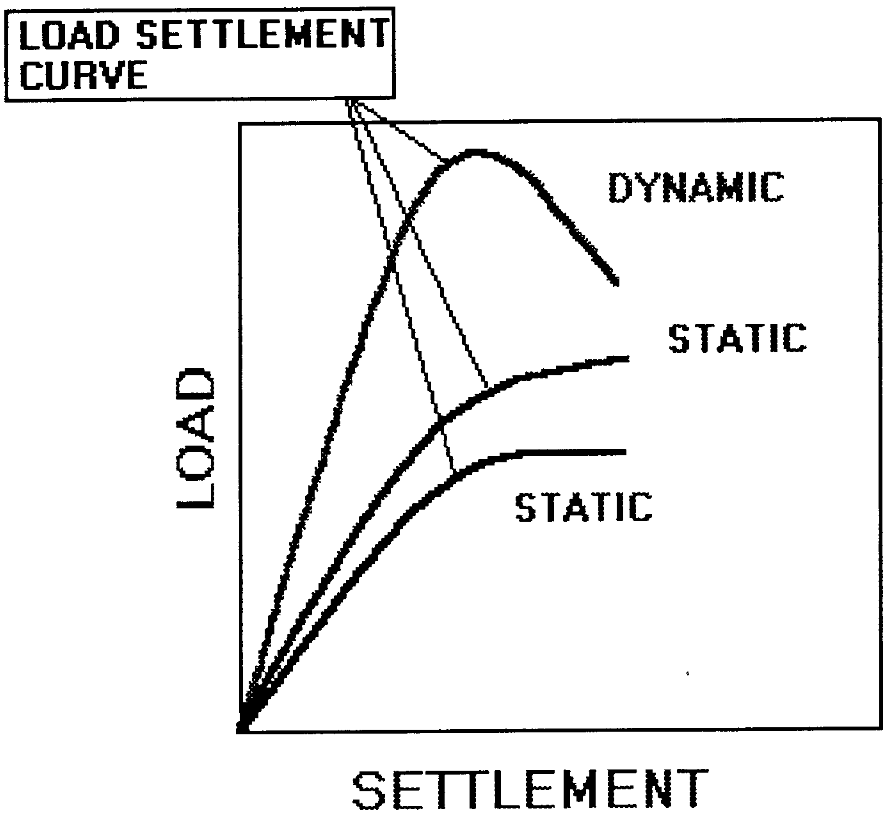

FIGURE 4.3 The Impact of Different Engineers on Determining Soil Damping

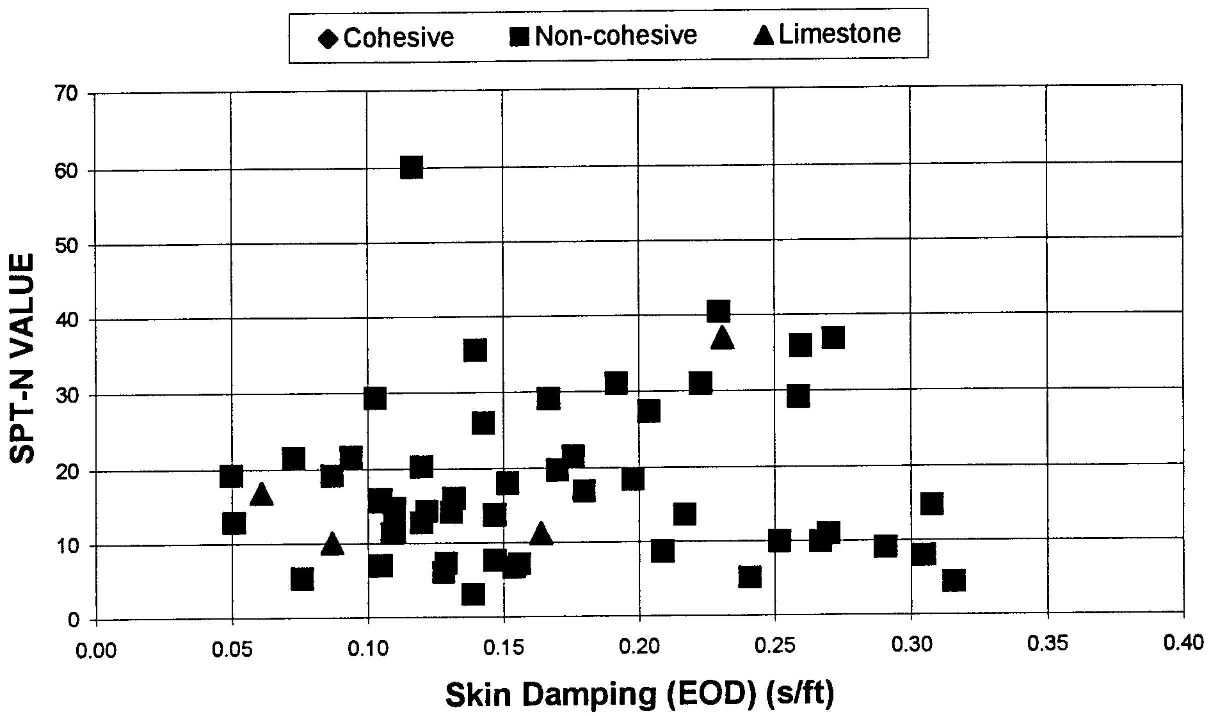

FIGURE 4.4 SPT-N Value vs. Skin Damping @ EOD

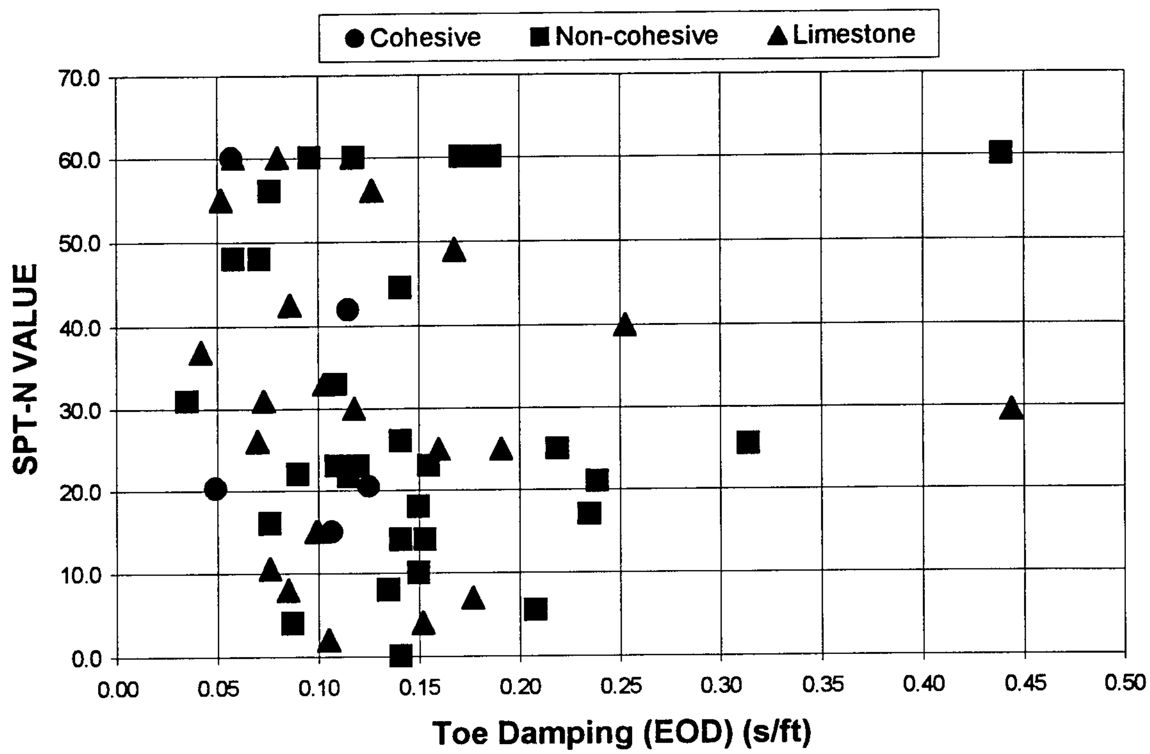

FIGURE 4.5 SPT-N Value vs. Toe Damping @ EOD

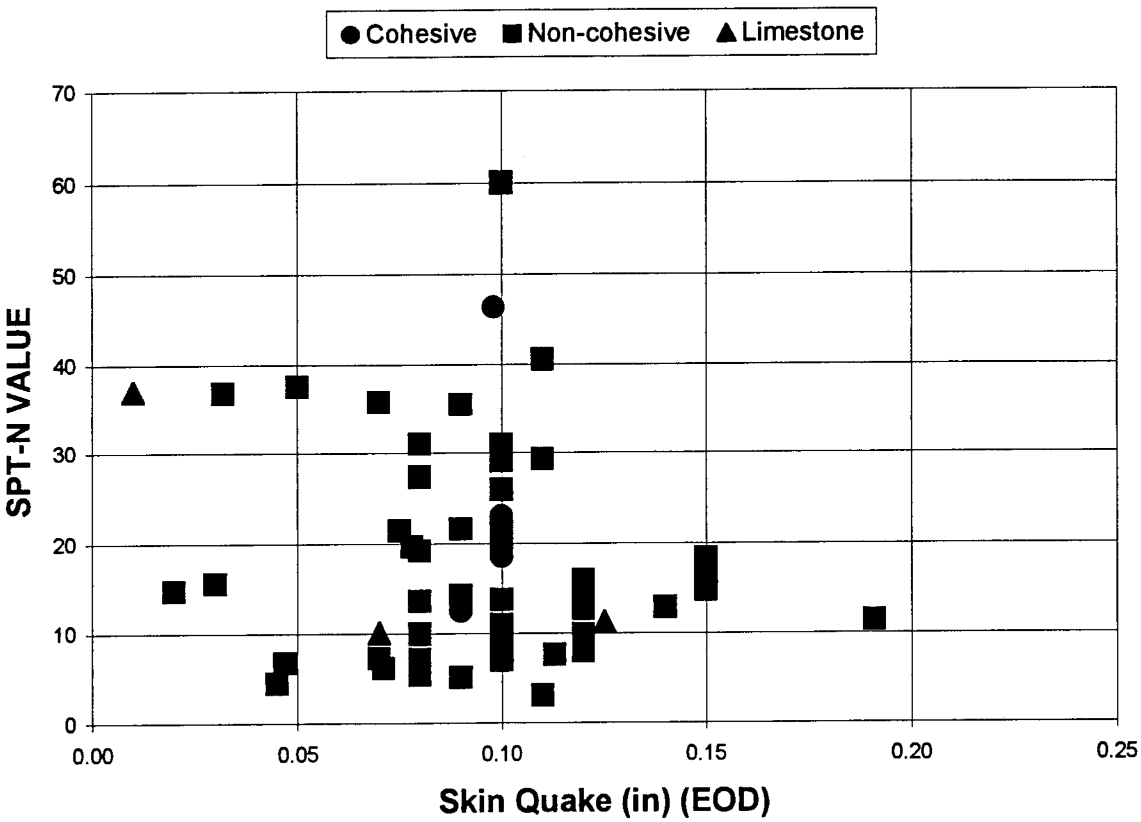

FIGURE 4.6 SPT-N Value vs. Skin Quake @ EOD

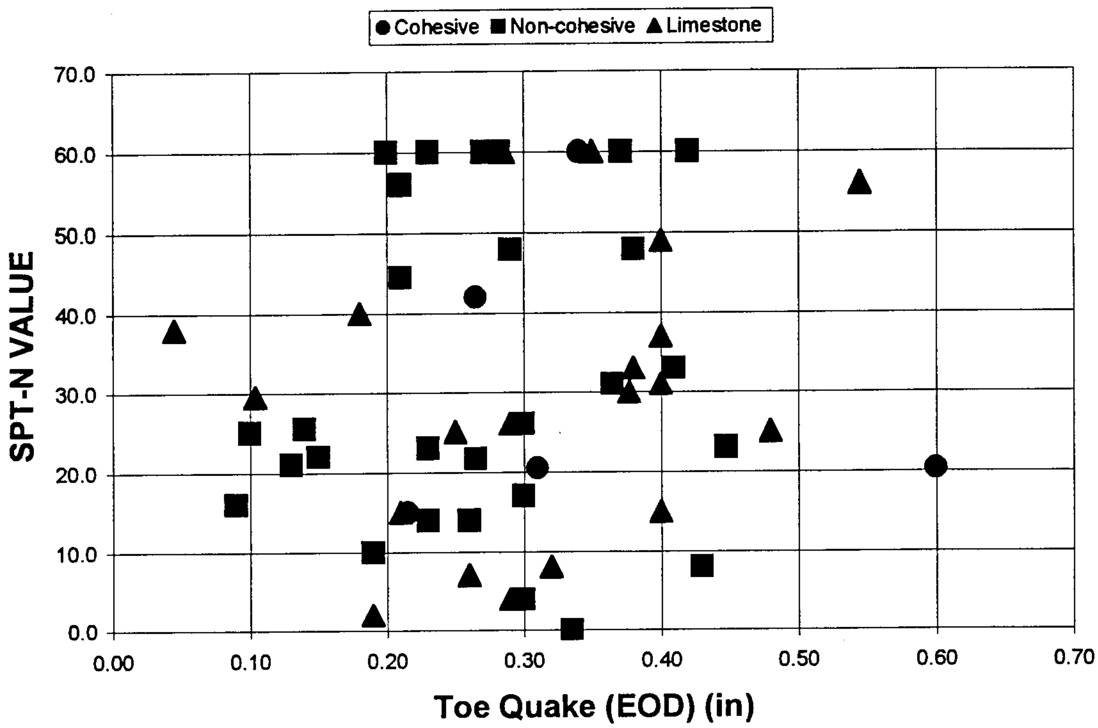

FIGURE 4.7 SPT-N Value vs. Toe Quake @ EOD

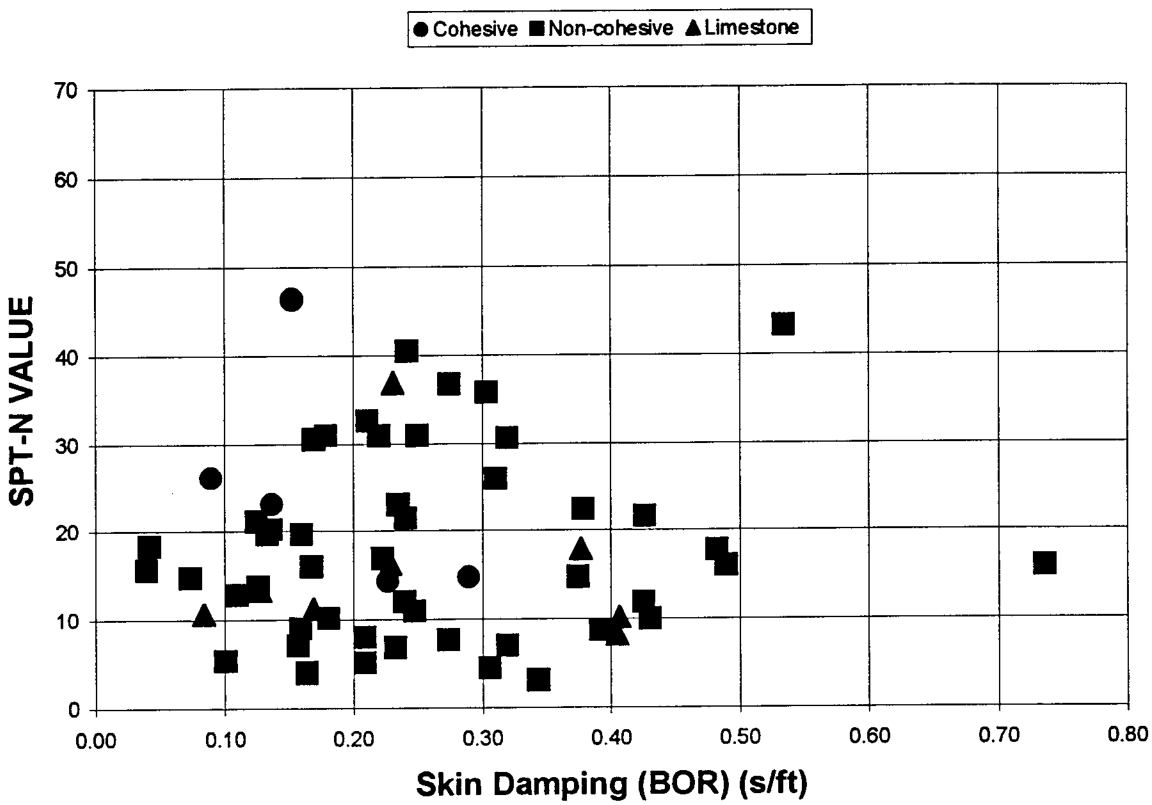

FIGURE 4.8 SPT-N Value vs. Skin Damping @ BOR

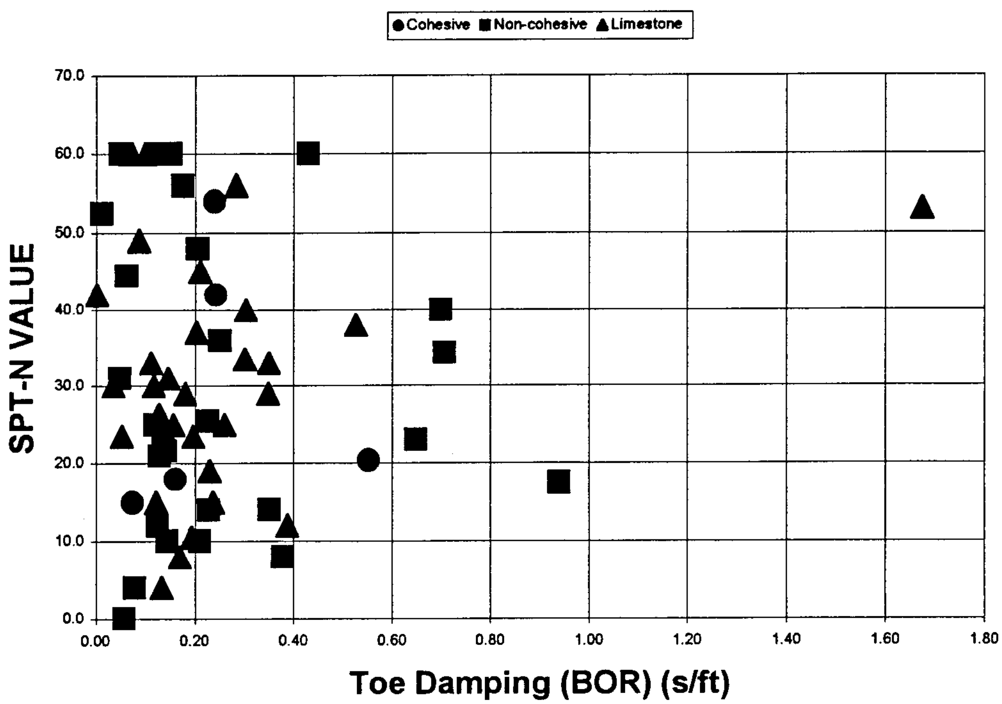

FIGURE 4.9 SPT-N Value vs. Toe Damping @ BOR

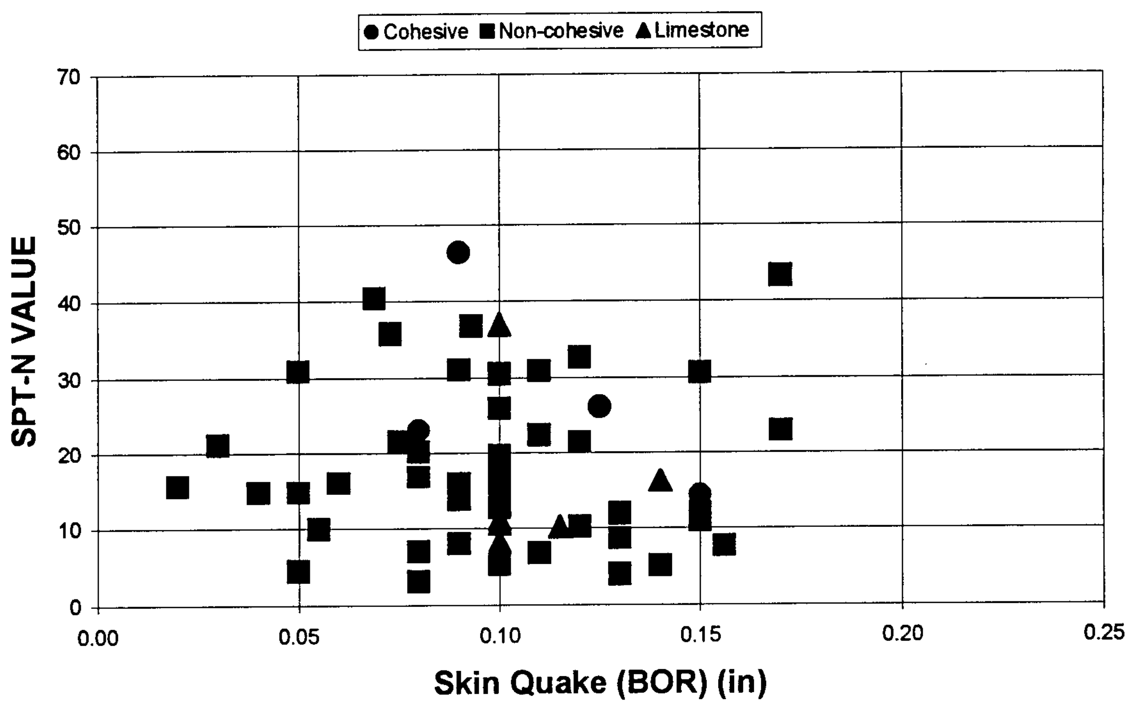

FIGURE 4.10 SPT- N Value vs. Skin Quake @ BOR

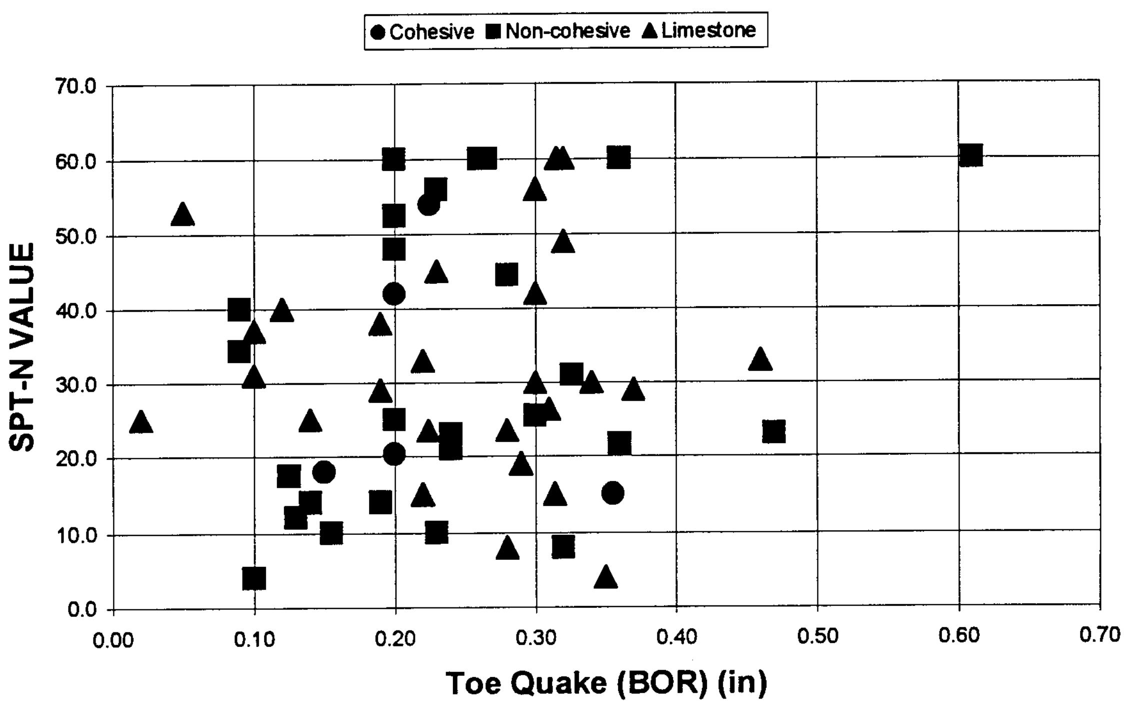

FIGURE 4.11 SPT-N Value vs. Toe Quake @ BOR

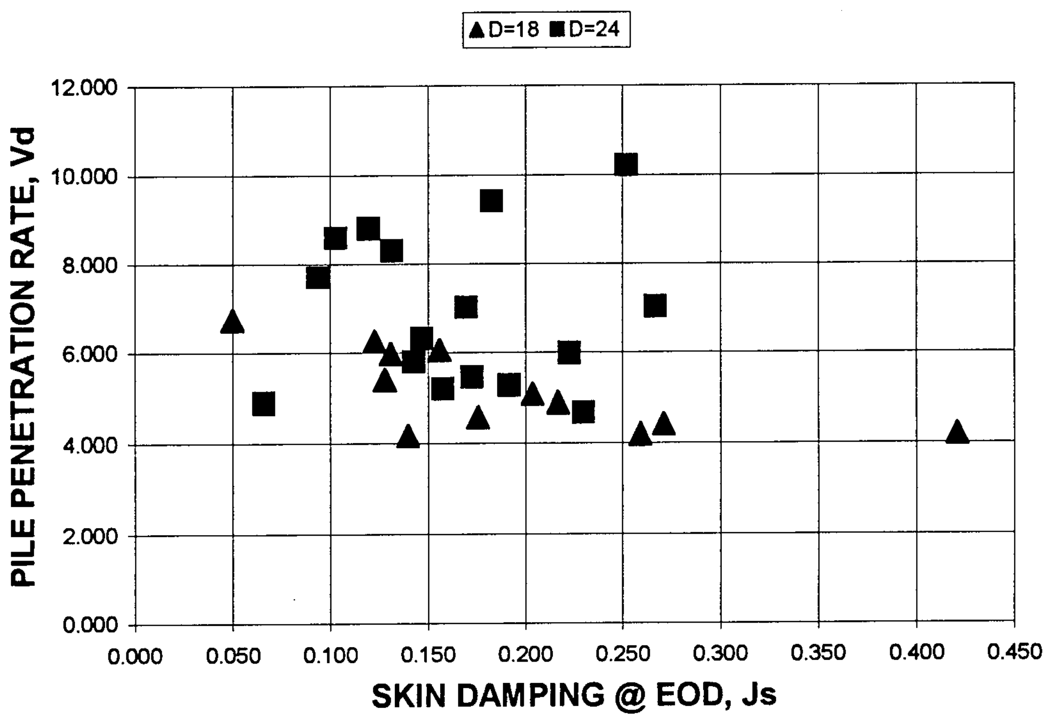

FIGURE 4.12 Pile Penetration Rate vs. Skin Damping @ EOD

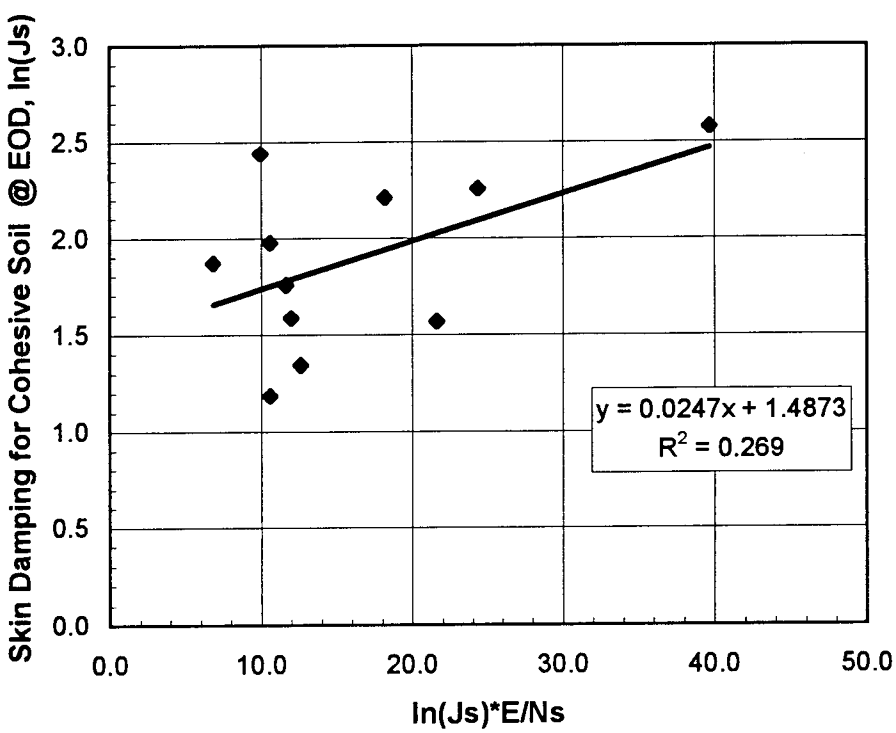

FIGURE 4.13 Skin Damping for Cohesive Soil @ EOD

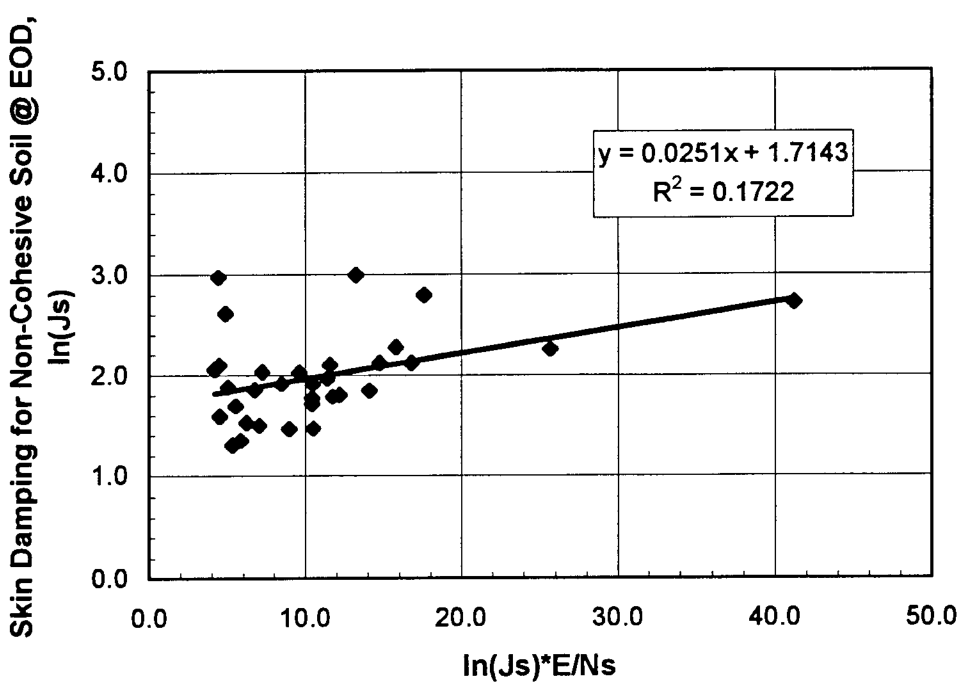

FIGURE 4.14 Skin Damping for Non-Cohesive Soil @ EOD

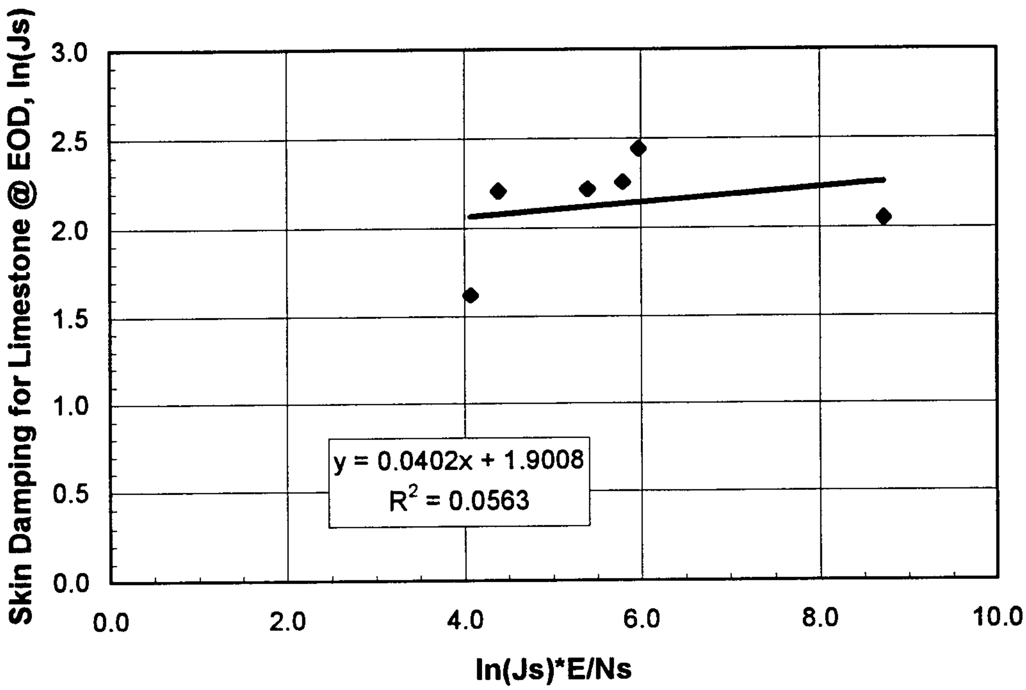

FIGURE 4.15 Skin Damping for Limestone @ EOD

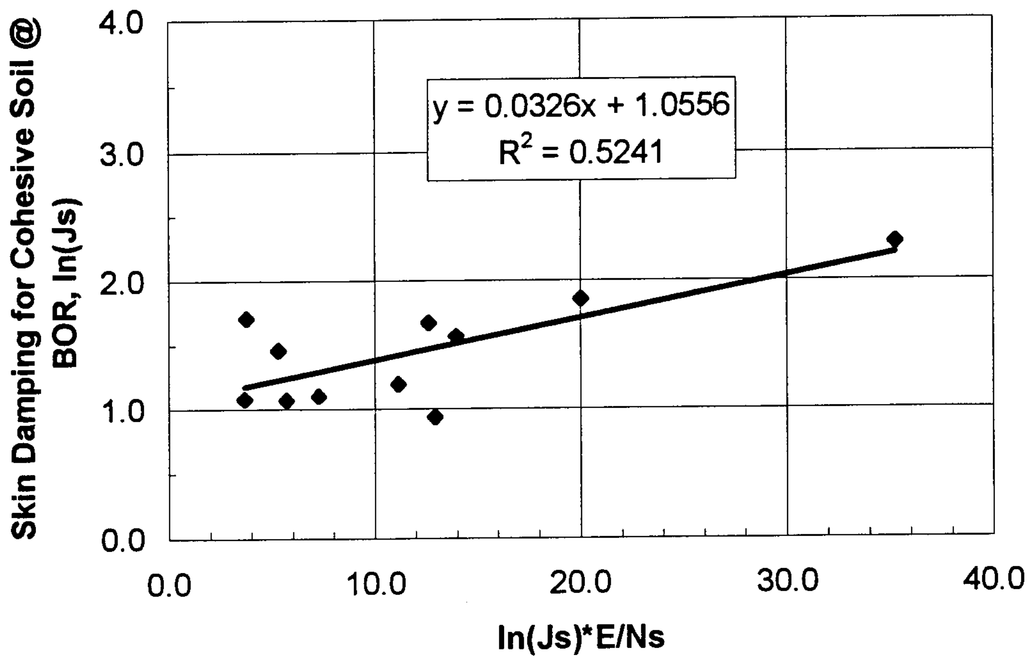

FIGURE 4.16 Skin Damping for Cohesive Soil @ BOR

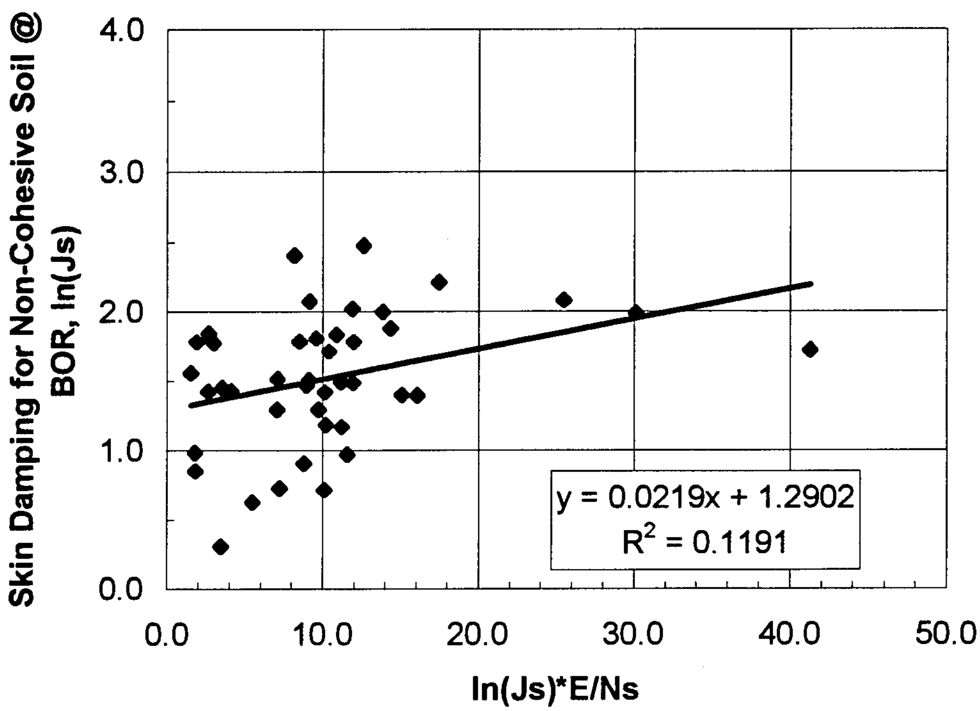

FIGURE 4.17 Skin Damping for Non-Cohesive Soil @ BOR

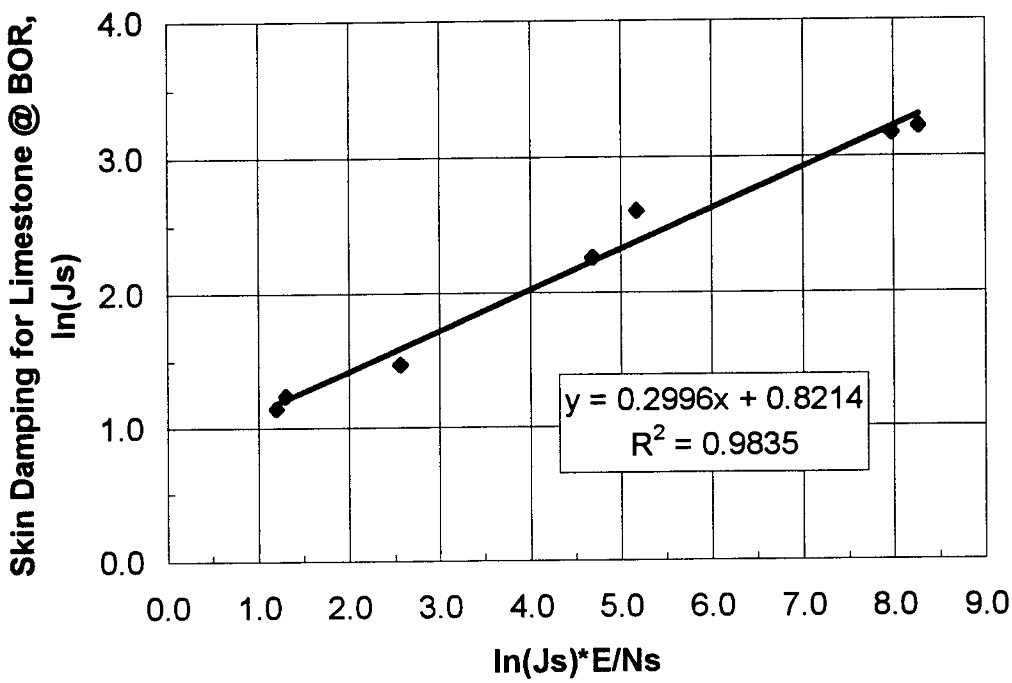

FIGURE 4.18 Skin Damping for Limestone @ BOR

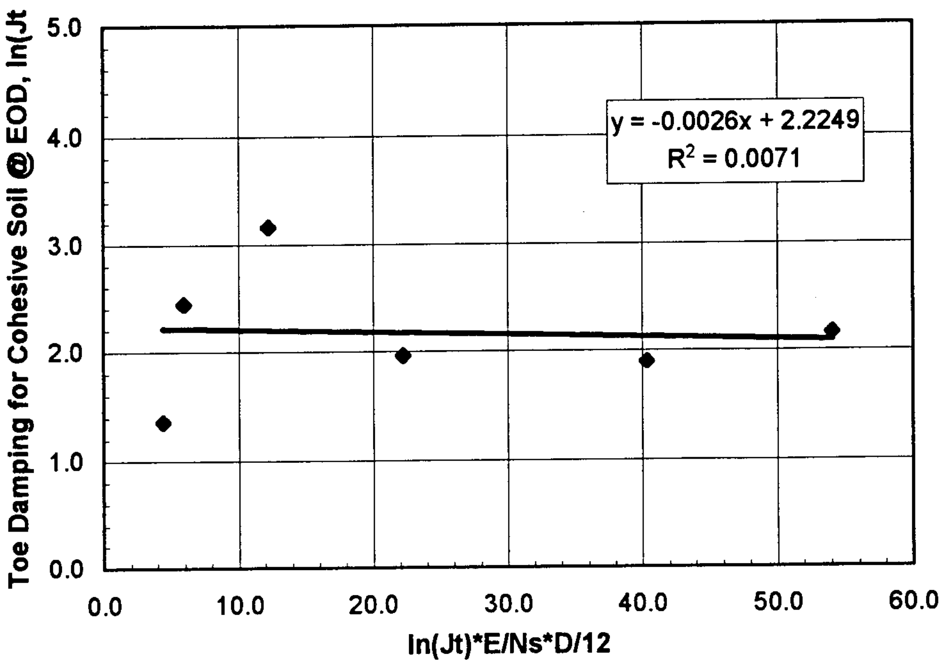

FIGURE 4.19 Toe Damping for Cohesive Soil @ EOD

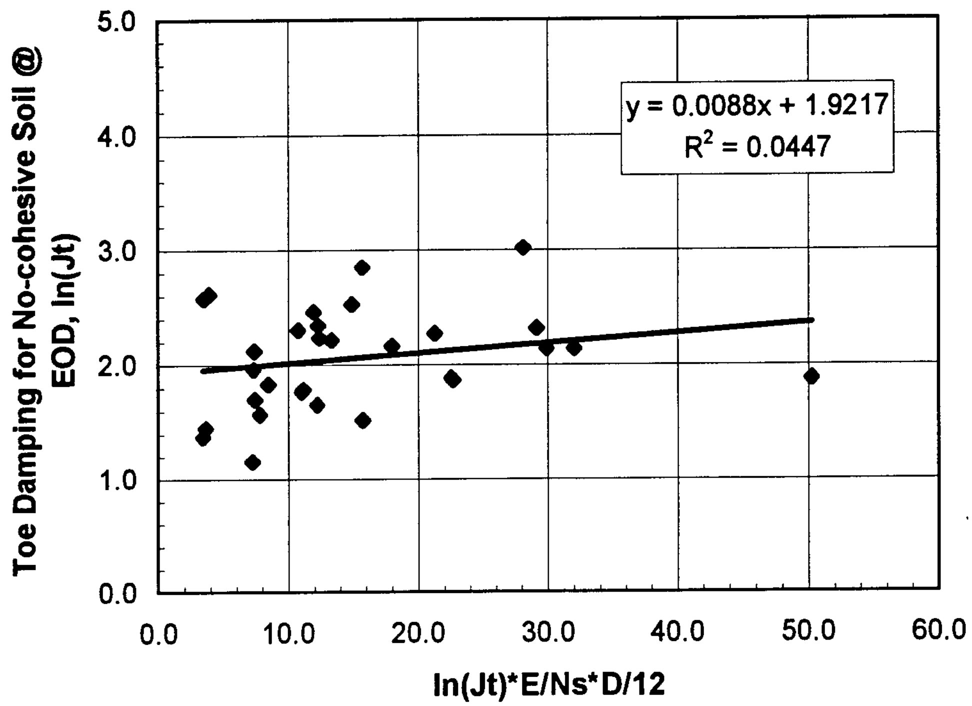

FIGURE 4.20 Toe Damping for Non-Cohesive @ EOD

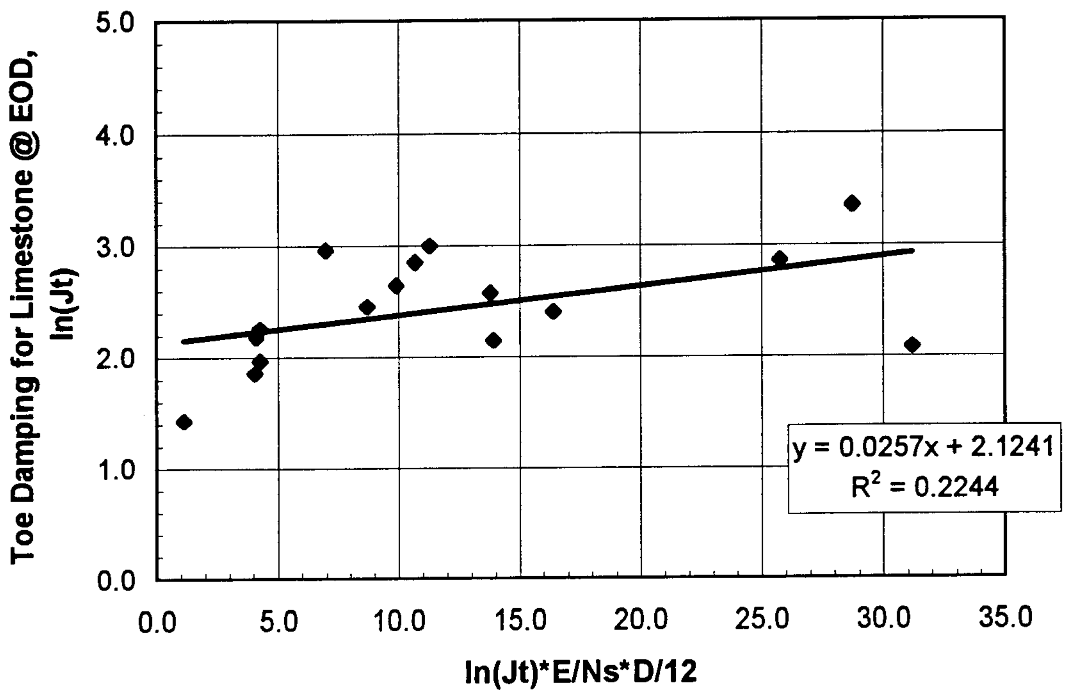

FIGURE 4.21 Toe Damping for Limestone @ EOD

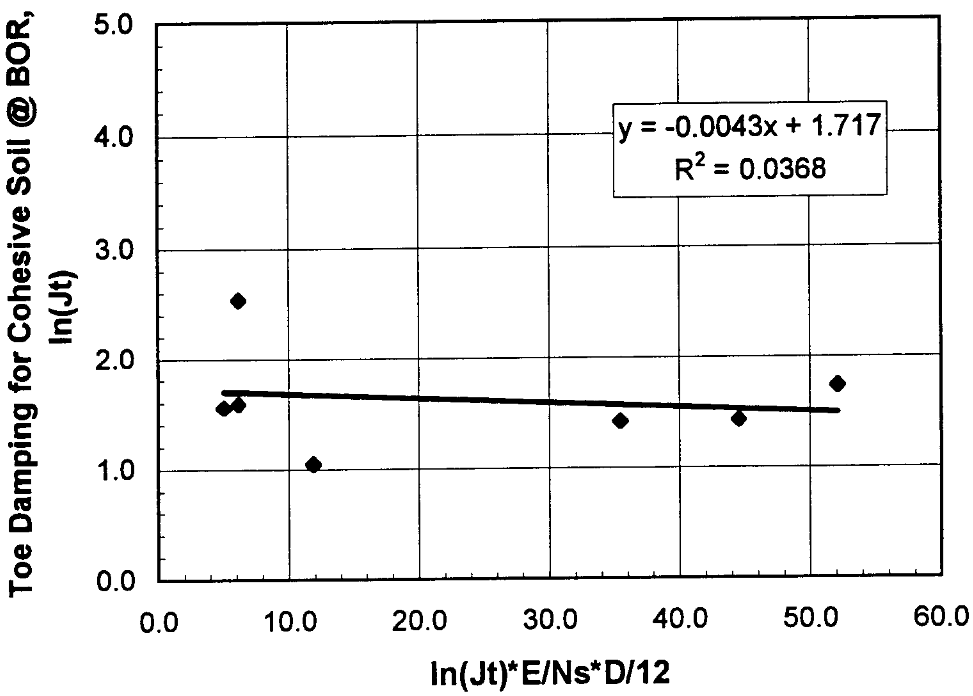

FIGURE 4.22 Toe Damping for Cohesive Soil @ BOR

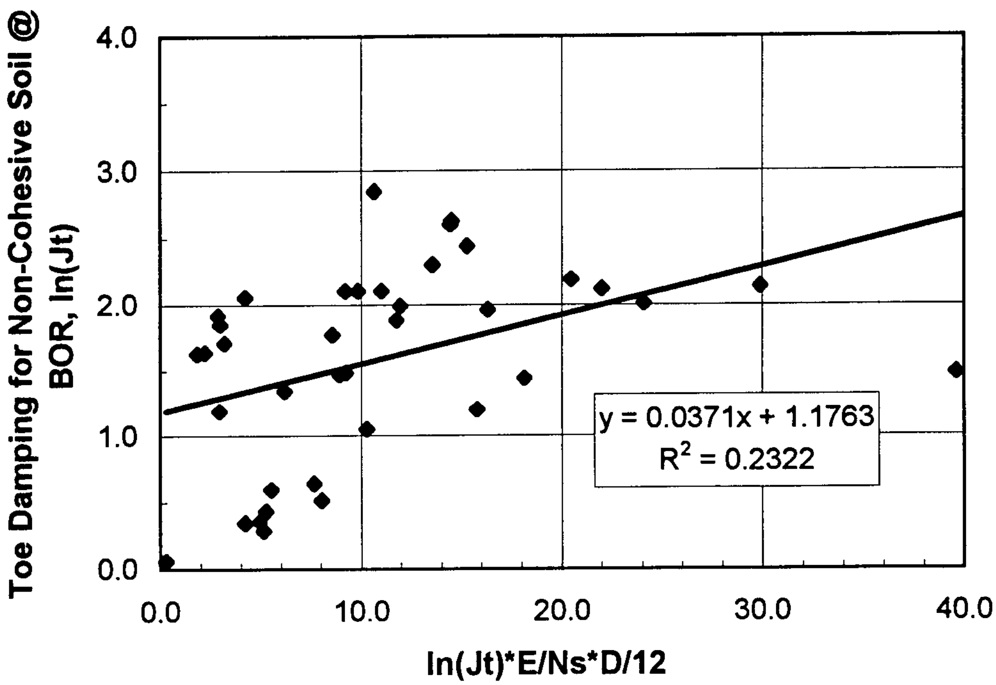

FIGURE 4.23 Toe Damping for Non-Cohesive Soil @ BOR

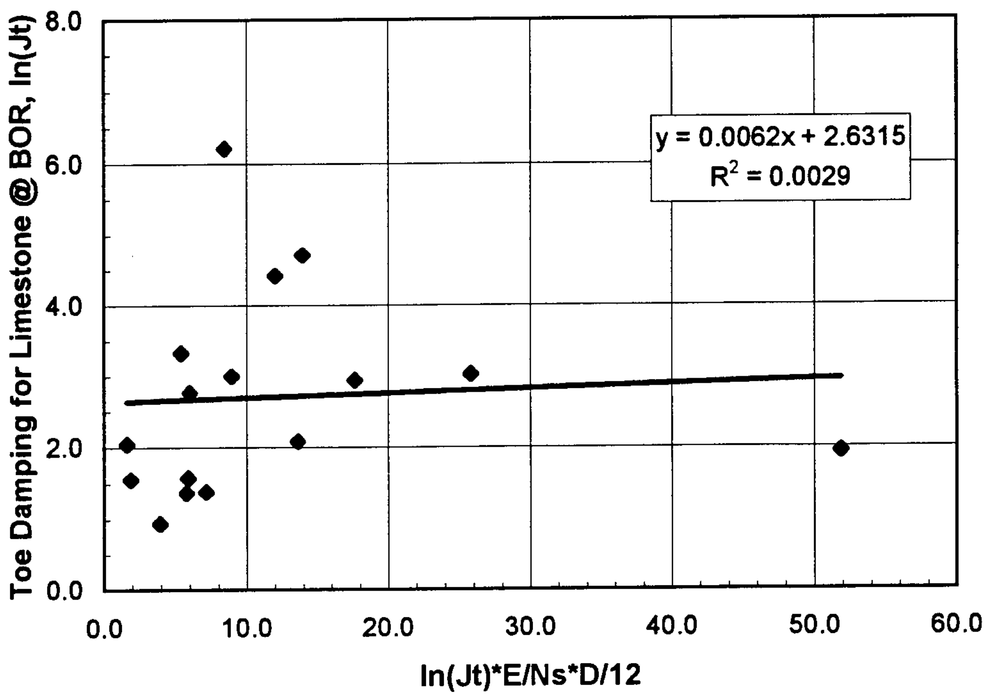

FIGURE 4.24 Toe Damping for Limestone @ BOR

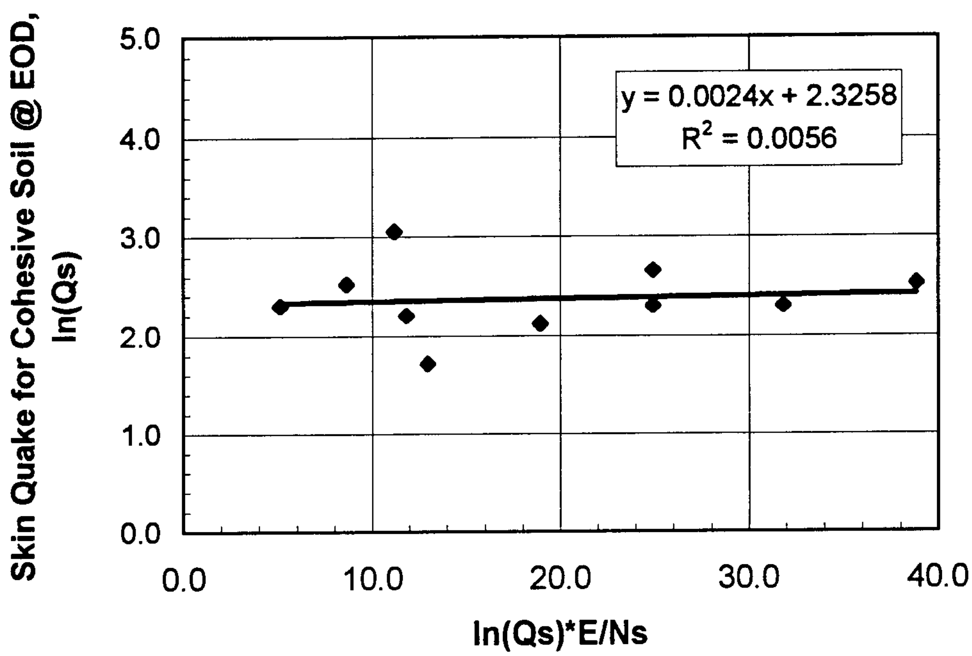

FIGURE 4.25 Skin Quake for Cohesive Soil @ EOD

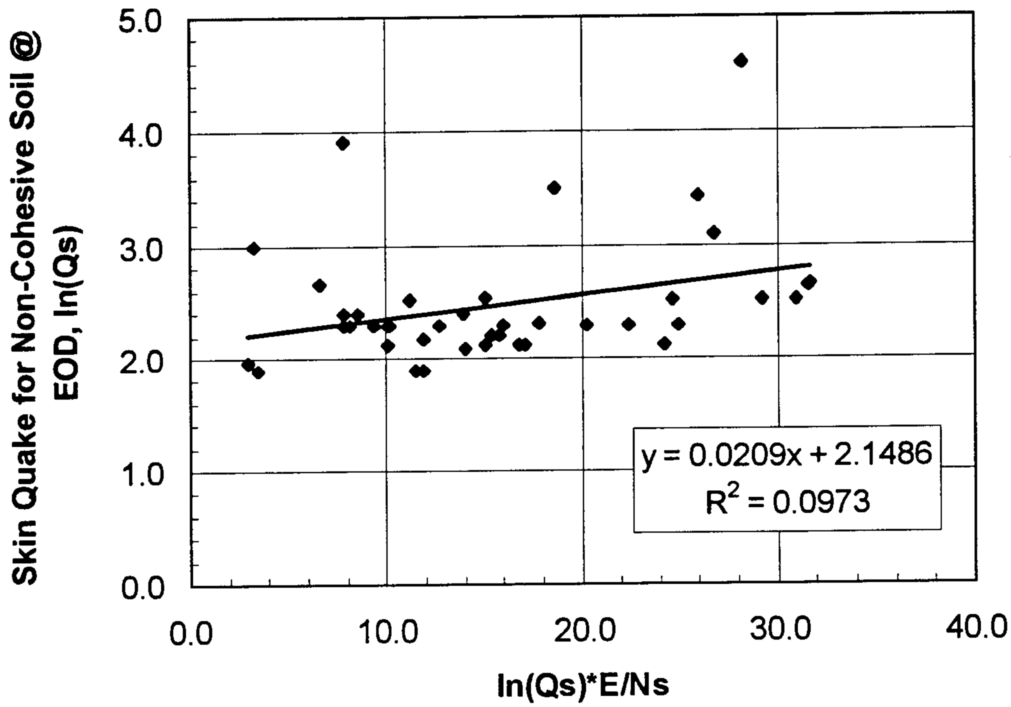

FIGURE 4.26 Skin Quake for Non-Cohesive Soil @ EOD

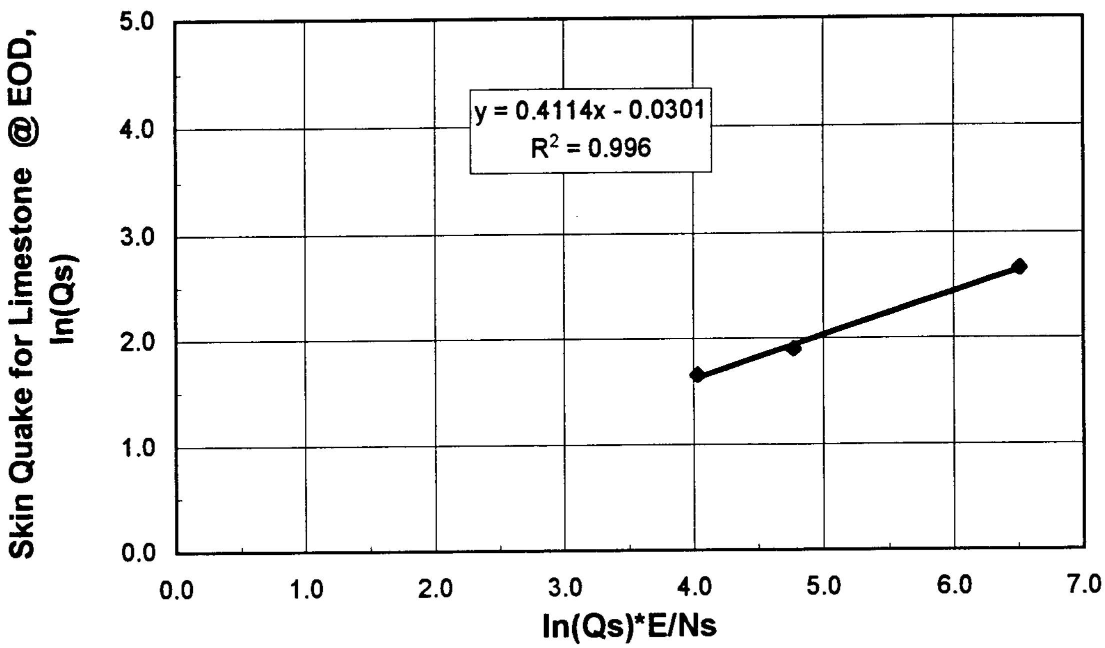

FIGURE 4.27 Skin Quake for Limestone @ EOD

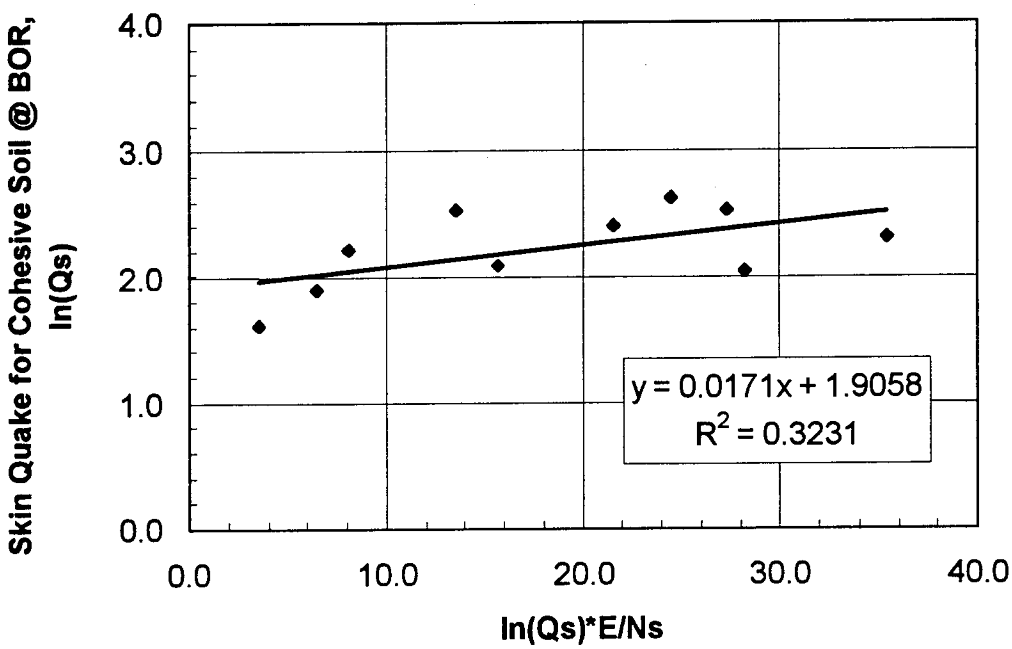

FIGURE 4.28 Skin Quake for Cohesive Soil @ BOR

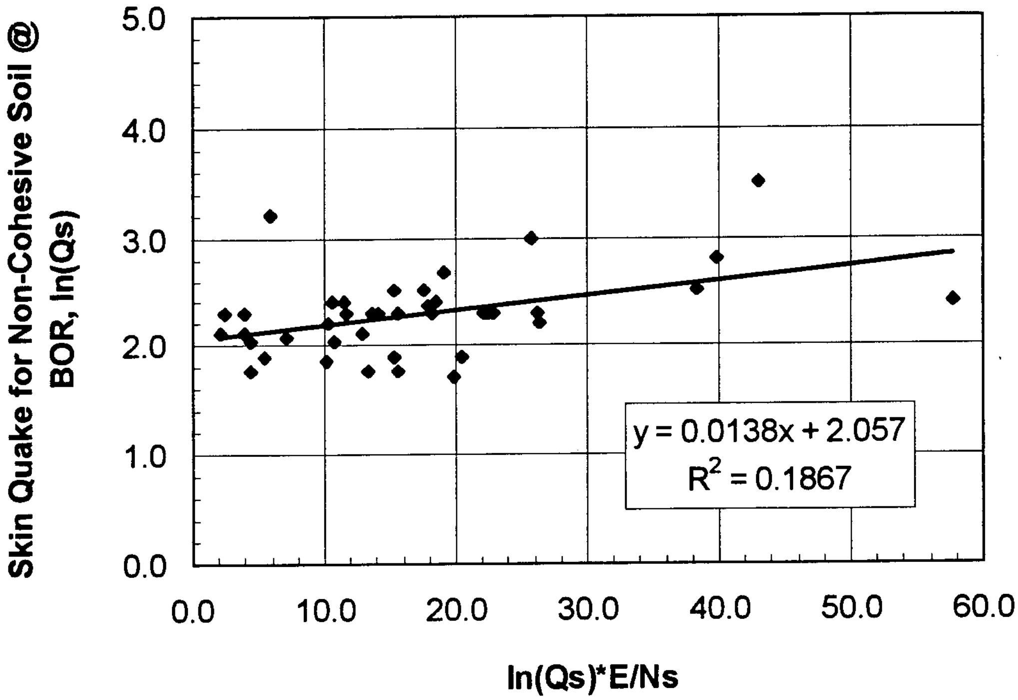

FIGURE 4.29 Skin Quake for Non-Cohesive Soil @ BOR

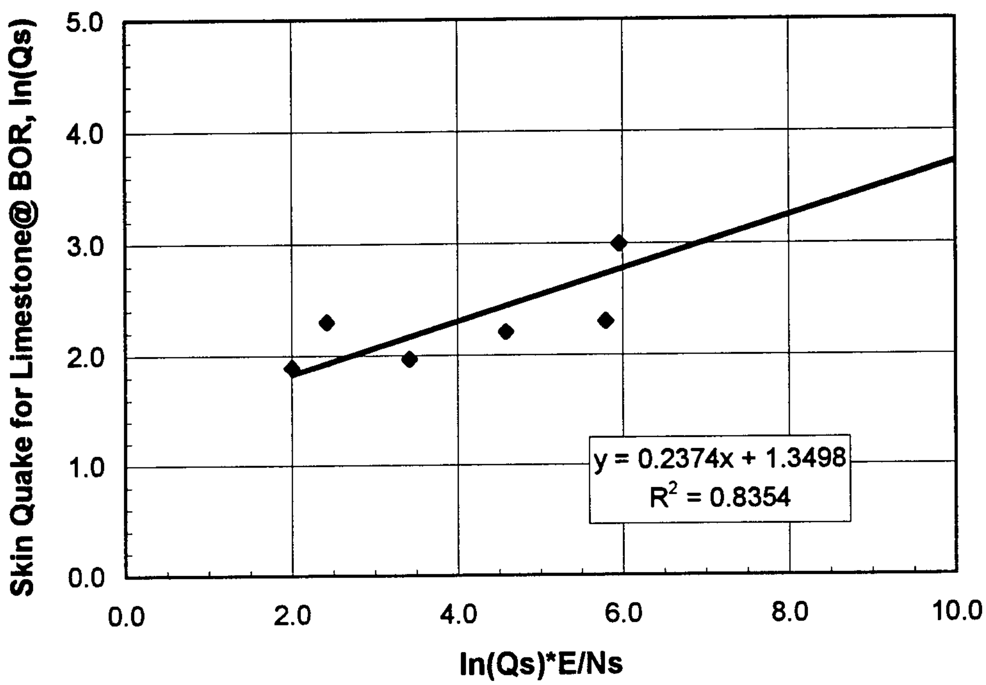

FIGURE 4.30 Skin Quake for Limestone @ BOR

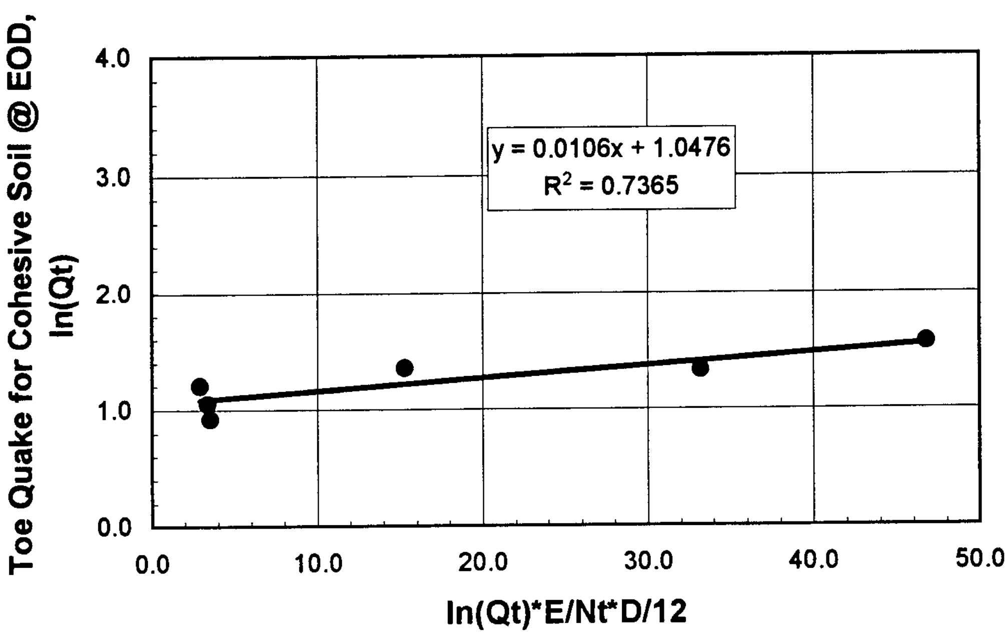

FIGURE 4.31 Toe Quake for Cohesive Soil @ EOD

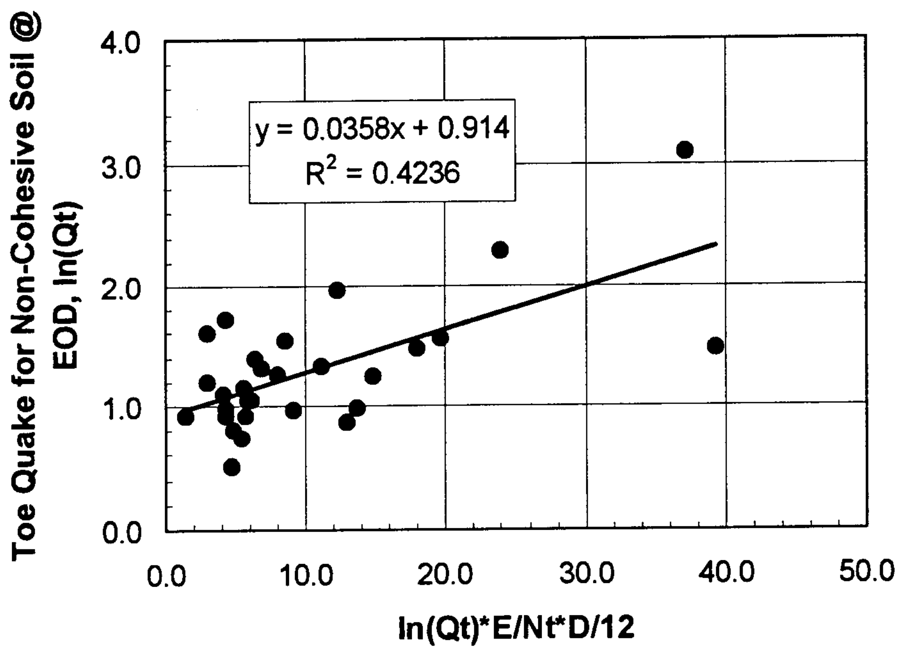

FIGURE 4.32 Toe Quake for Non-Cohesive Soil @ EOD

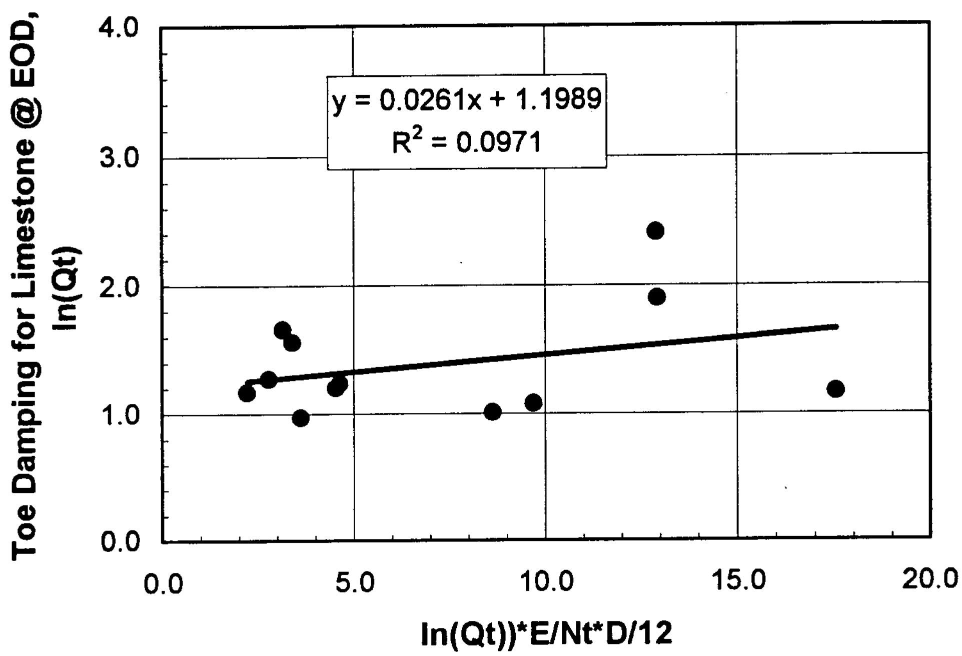

FIGURE 4.33 Toe Quake for Limestone @ EOD

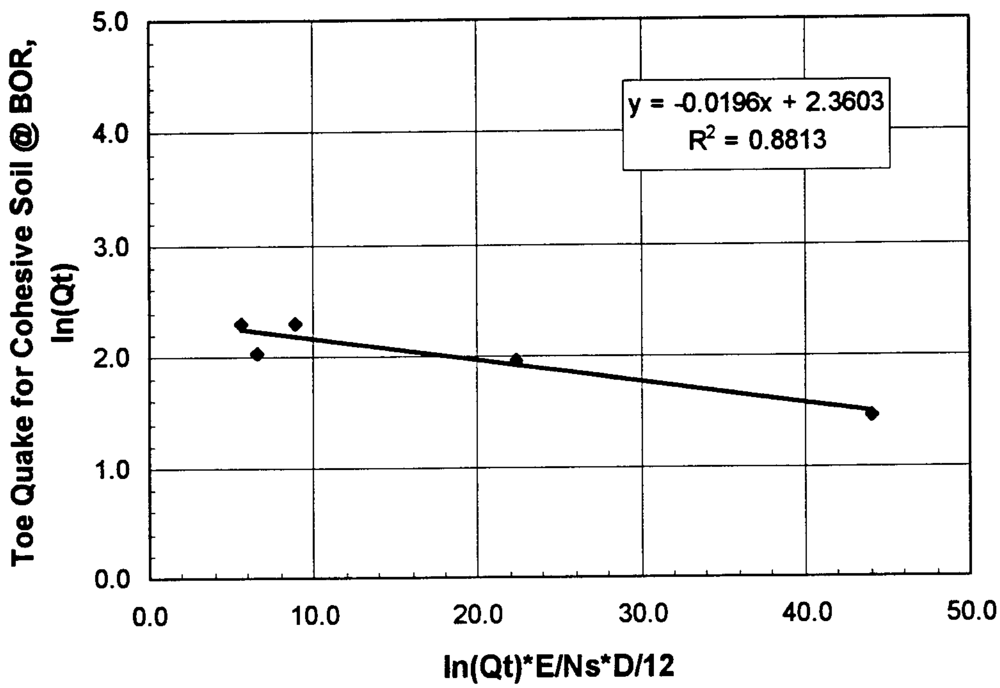

FIGURE 4.34 Toe Quake for Cohesive Soil @ BOR

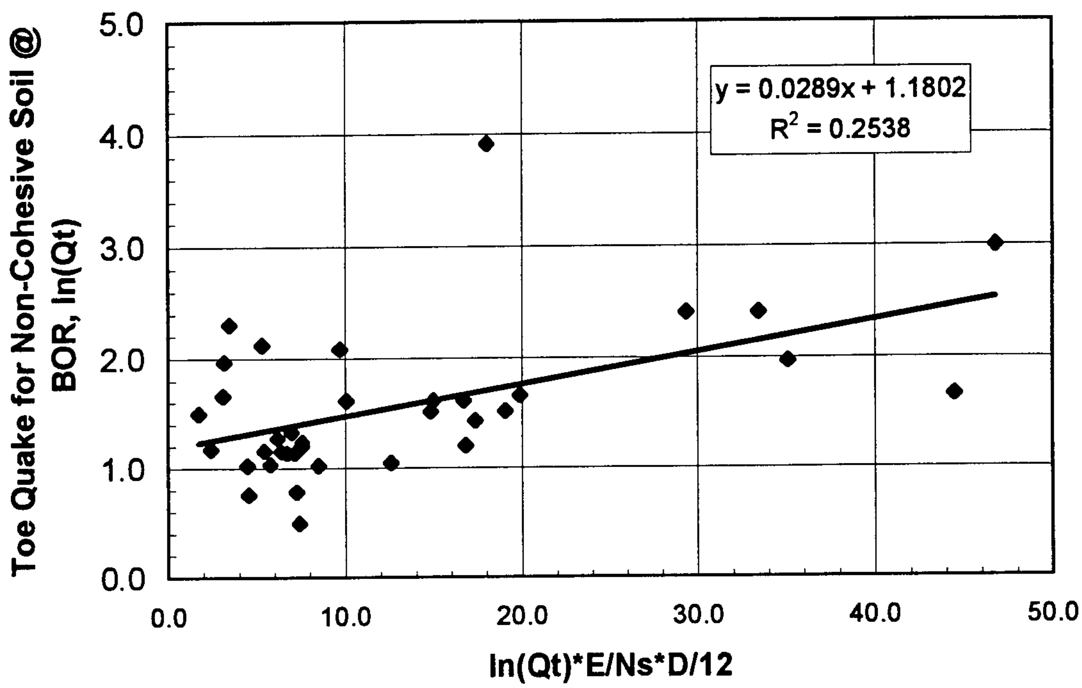

FIGURE 4.35 Toe Quake for Non-Cohesive Soil @ BOR

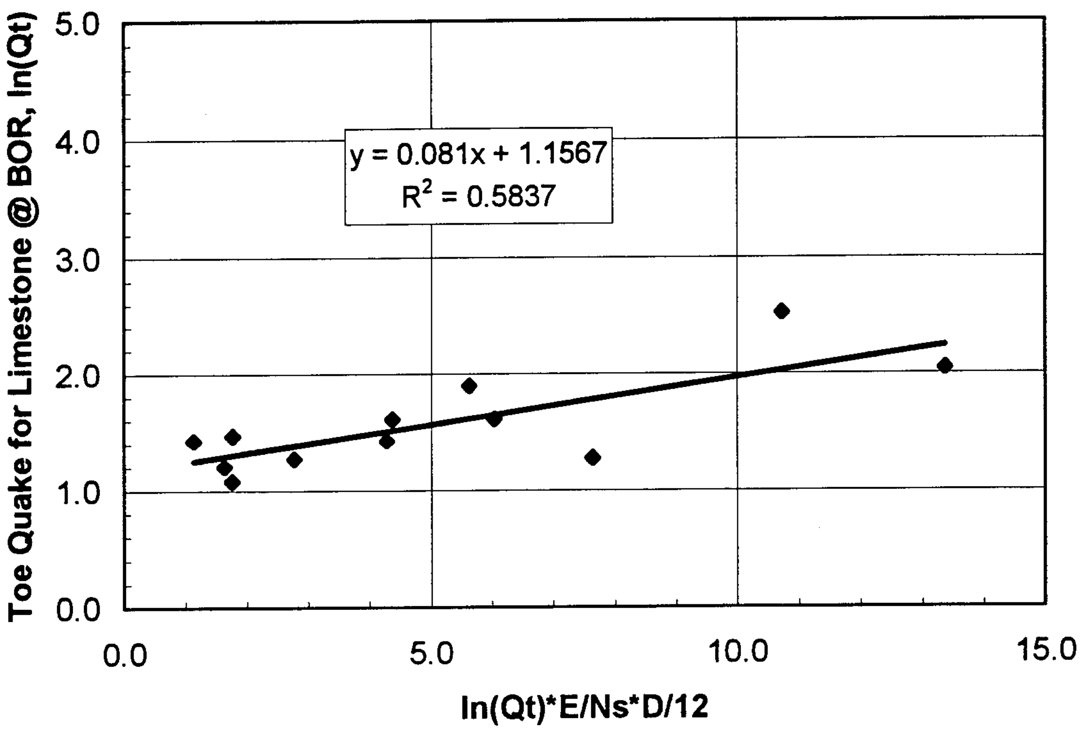

FIGURE 4.36 Toe Quake for Limestone @ BOR

CHAPTER 5
VERIFICATION AND COMPARISION

5.1 General

To evaluate the reliability of using the Smith soil parameters determined from the proposed empirical equations to estimate the pile capacity, wave equation analyses were performed using computer program GRLWEAP. Three different sets of Smith soil parameters, i.e., default values recommended by GRL, Inc., results from CAPWAP analysis, and suggested values from proposed equations for each load test data.

These predicted pile capacity (Q_p) from GRLWEAP analysis at End of Initial Driving (EOD) and Beginning of Redriving (BOR) were compared with measured pile capacity (Q_m) from static pile load tests using Davisson failure criteria (Davisson, 1972).

5.2 Wave Equation Analysis

5.2.1 Design Parameters

Twenty-one (21) load test cases were used for comparison study. Each set of data at least included:

1. Davisson failure capacity determined from static load test;
2. Smith soil damping and soil quake determined from CAPWAP results at end of initial driving (EOD) and/or beginning of redriving (BOR).
3. Hammer and pile data and
4. Blow counts at EOD and/or BOR conditions from Driving records.
5. Soil profiles including SPT data.

For each load test case, five (5) different sets of Smith soil parameters, i.e., the default values recommended by GRL, Inc. the results from CAPWAP analyses at EOD and BOR, and the estimates from the proposed equations at EOD and BOR, were used in wave equation analyses using the computer program GRLWEAP developed by GRL, Inc. (1997). However, majority of the hammer data were not completed. Therefore, the default values of the weight, size and material properties of helmet and hammer cushion from GRLWEAP hammer file were used in analysis. The shaft resistance in percent of the ultimate resistance (R_{ut}) was obtained from CAPWAP results. In addition, the efficiency of hammer performance was unknown at the pre-construction analysis, therefore, the default values of hammer efficiency and maximum hammer stroke from GRLWEAP were used in the comparison study.

The bearing graph in term of capacity versus blow counts was determined for each set of Smith soil parameters from GRLWEAP analysis. The pile capacity at EOD or BOR condition was interpolated from the bearing graph corresponding to blow counts recorded during pile driving.

5.2.2 Results of Comparison Analysis

Figures 5.1 through 5.4 showed the pile capacities predicted by GRLWEAP analyses using five (5) different sets of Smith soil parameters versus the measured capacities from static load tests. Each graph also included a solid line representing the best fit of first order, the corresponding equation and the coefficient of determination, R^2, and a set of dashed lines representing the different ratios of the predicted capacity over the measurements. Table 5-1 summarized the statistical descriptors, i.e., bias factor, standard deviation, and coefficient of variation, of the ration of predicted capacity to measured capacity (Q_p /$Q_{m)}$. It should be noticed that the default Smith soil parameters from GRL were primarily back-calculated to fit the static pile load test results, therefore, the GRLWEAP results using default values should be considered as beginning of redriving conditions.

As shown in Figures 5.1 and 5.2, the predicted pile capacities of GRLWEAP analyses using the Smith soil parameters from CAPWAP results and Proposed equation at the end of initial driving condition (EOD) were about 40 to 80% of the measured capacity from static load tests with an average of 66 and 70%, respectively.

Figures 5.3 through 5.5 presented the predicted pile capacities of GRLWEAP analyses from the default values, CAPWAP results and Proposed equations at the beginning of redriving (BOR), respectively. The results indicated that the predicted pile capacities using the Smith soil parameters from CAPWAP results and proposed equations ranged from 60 to 125% of the measured capacities with an average of 95 and 109%, respectively. The predicted pile capacities using the default Smith soil parameters from GRLWEAP ranged from 40 to 100% of the measured capacities with an average of 77%. As shown in the figures, all cases tends to over-predict for the capacity piles less than 1000 kips and under-predict for large capacity piles.

The results of the statistical analysis of the GRLWEAP prediction were presented in Table 5-1. The average and coefficient of variation of Q_p/Q_m at EOD condition were 0.66 and 0.43 using CAPWAP results, and 0.7 and 0.41 for the proposed equation, respectively. The average and coefficient of variation of Q_p/Q_m at BOR condition were 0.95 and 0.35 using CAPWAP results, and 1.09 and 0.43 for the proposed equation, respectively, while the average and coefficient of variation of Q_p/Q_m for GRL default values were 0.77 and 0.55. In general, GRLWEAP prediction using the Smith soil parameters from the proposed equation provided the best results, and that using GRL default values was the worst. However, the coefficient of variation for all the cases were relatively high that indicated the reliability of the GRLWEAP prediction is low.

Table 5-1

Statistical analysis of Q_m/Q_p for different Smith soil parameters

Smith parameters	No. of Cases	Bias factor	Standard Dev.	COV
CAPWAP result (EOD)	21	0.661	0.282	0.426
Proposed equation (EOD)	21	0.699	0.289	0.413
GRL Defaultr	21	0.768	0.383	0.498
CAPWAP result (BOR)	21	0.954	0.335	0.351
Proposed equation (BOR)	21	1.091	0.474	0.434

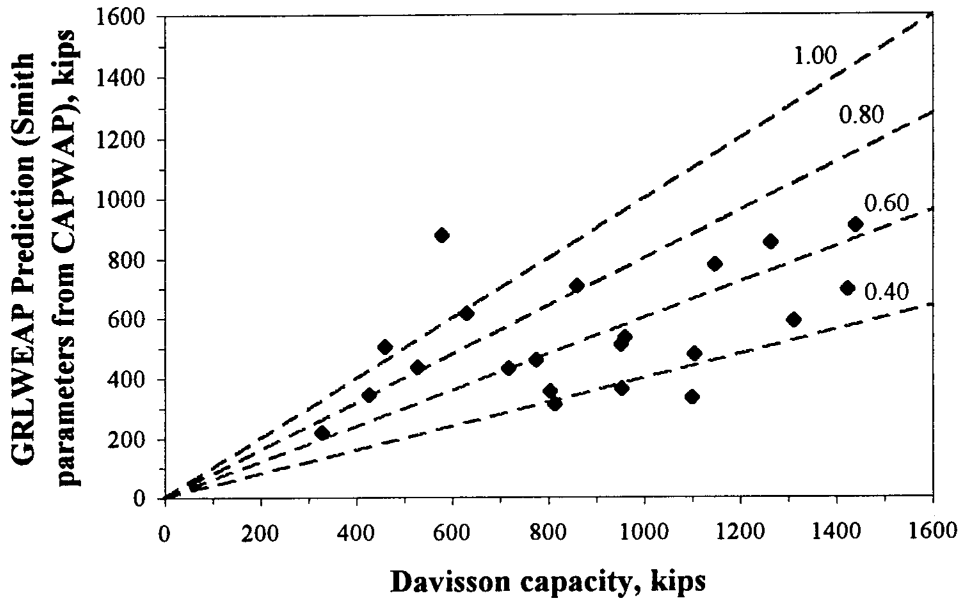

FIGURE 5.1 Davisson Capacity vs. GRLWEAP Prediction @ EOD using Smith Soil Parameters from CAPWAP Results

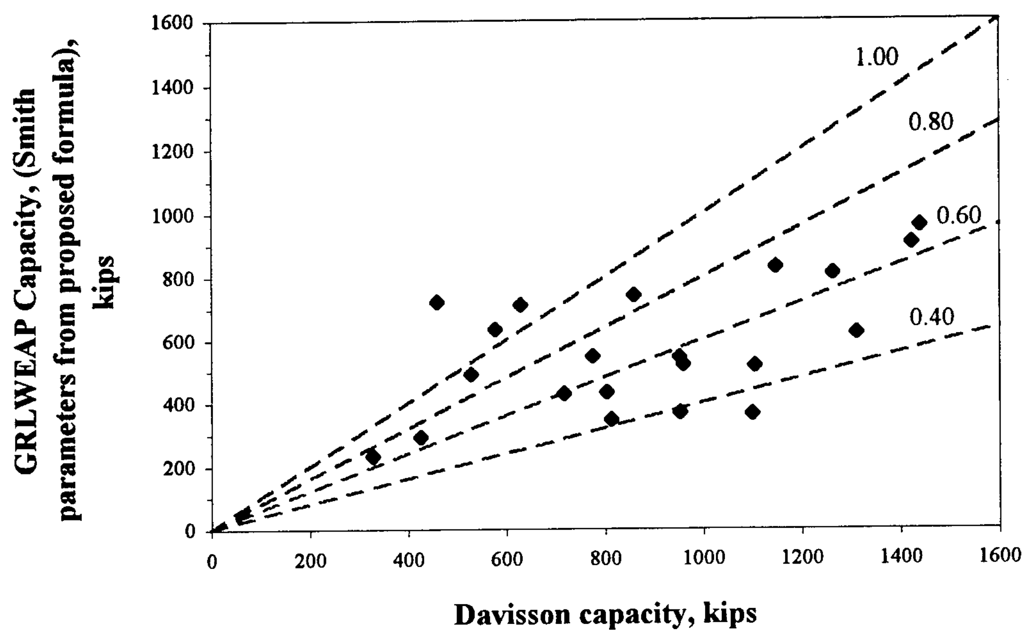

FIGURE 5.2 Davisson Capacity vs. GRLWEAP Prediction @ EOD using Smith Soil Parameters from Proposed Equation

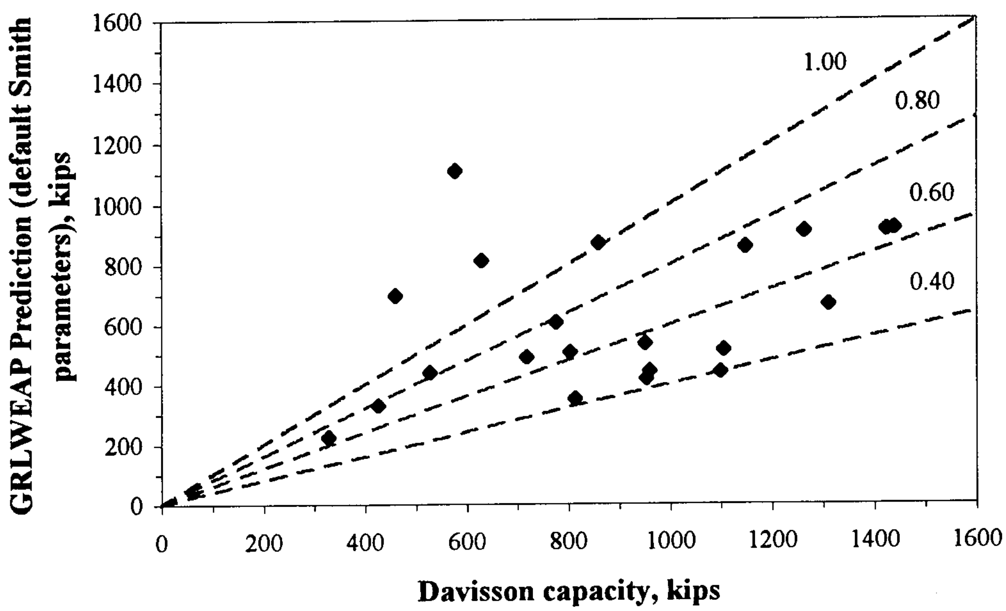

FIGURE 5.3 Davisson Capacity vs. GRLWEAP Prediction using Smith Soil Parameters from GRL Default

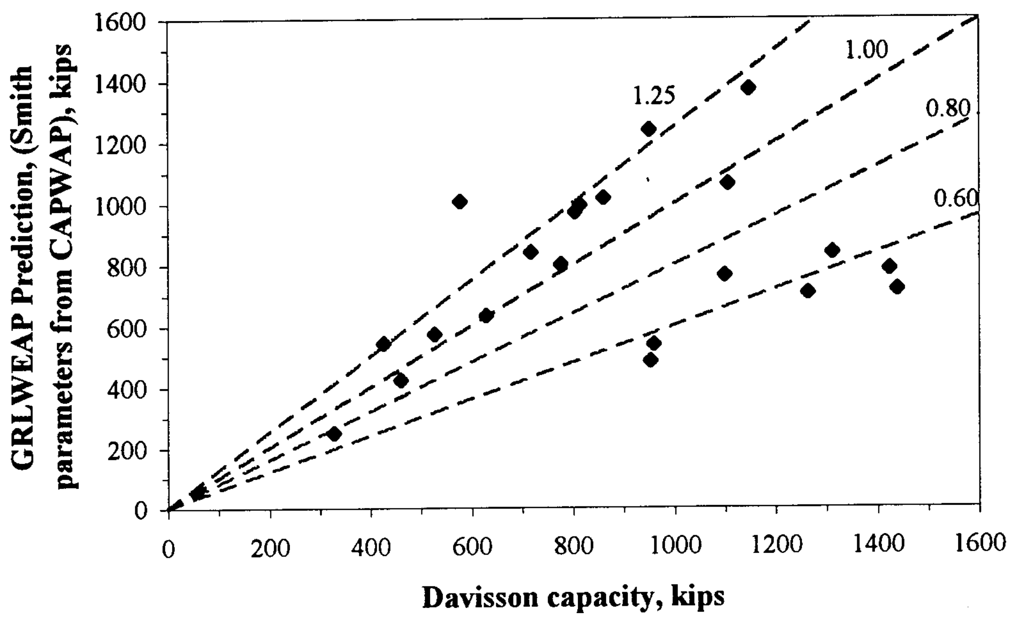

FIGURE 5.4 Davisson Capacity vs. GRLWEAP Prediction @ BOR using Smith Soil Parameters from CAPWAP Results

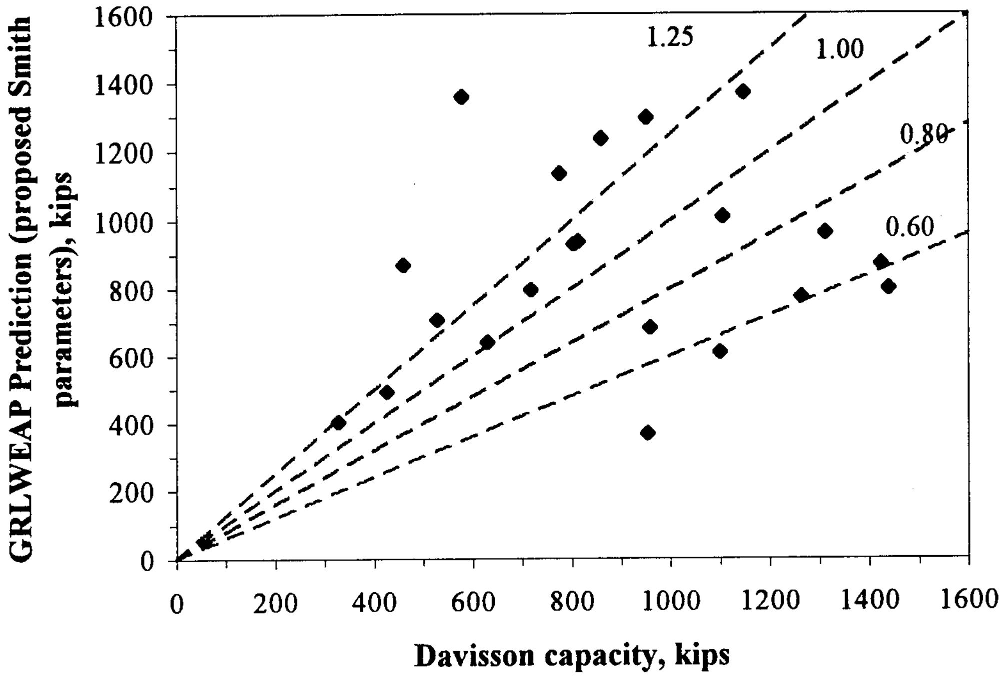

FIGURE 5.5 Davisson Capacity vs. GRLWEAP Prediction @ BOR using Smith Soil Parameters from Proposed Equation

CHAPTER 6
CONCLUSIONS AND RECOMMENDATIONS

6.1 General

Wave equation was developed more than 100 years by Saint Venant and Boussinesq (1866) for end impact on rods. In 1938, E. N. Fox published a solution for the wave equation applied to pile driving. From 1955 to 1962, E. A. L. Smith presented a mathematical model for the pile driving problems that could be solved by computer. He suggested using soil quake and soil damping to model soil behavior subject to impact loading. However, Smith soil parameters, soil quake and soil damping are nonstandard soil mechanics parameters that only can be determined through back analysis of pile driving records and pile load tests. Forehand and Reese (1964) noted that a number of combinations of the quake and damping could be used to fit a test data.

Currently the best approach to determine the Smith soil parameters is to perform CAPWAP analysis by using the PDA monitoring data. However, CAPWAP analysis is a Linear Algebra process to determine the best-fit solution and does not has unique solution. In addition, driving conditions have significant impacts on the PDA results. This makes the determination of Smith soil parameters even more difficult. There are significant amount of efforts performed in the past two decades to compile numerical values of Smith soil parameters. However, the data were widely scattered, it was very difficult to establish an empirical relation to estimate the Smith soil parameters. The database from UF as discussed in Chapter 3 also indicated the same trend. Although many researchers worked on developing the theoretical expression of the Smith soil parameters, no predicted values were available to validate the proposed equations.

6.2 Summary

Based on the evaluation of the UF database, it indicated that the ranges of Smith soil parameters did not have significant difference for different soil types. There is no one-to-one correlation between Smith soil parameters and SPT-N values. However, according to the theoretical expressions proposed by many researchers, Smith soil parameters may be influenced by the SPT-N value and hammer energy. Therefore, a regression analysis was perform on the UF database in term of SPT-N value and hammer energy. The results were presented in Chapter 4. A comparison study of wave equation analysis using program GRLWEAP was performed using the Smith soil parameters from the default values of GRL, Inc (1993), CAPWAP results and the proposed empirical relations between Smith soil parameters and SPT-N value and Hammer energy. The results were summarized as follows.

1. GRLWEAP prediction at the end of initial driving (EOD) condition under-estimated the pile capacity compared to the measured Davisson capacity of static load tests using the Smith soil parameters CAPWAP results and proposed equations. The average ratios of Q_p/Q_m were 0.66 and 0.70 for CAPWAP result case and proposed equation case, respectively. It was because the Smith soil parameters were determined from end of initial driving (EOD) condition which did not consider the pile freeze effect.
2. GRLWEAP slightly under predicts measured capacity for beginning of redriving (BOR) condition using the Smith soil parameters of CAPWAP results, and the average Q_p/Q_m was 0.95. The prediction of GRLWEAP at the beginning of redriving (BOR) using the Smith soil parameters of proposed equations slightly over-estimated the measured capacity, and the average Q_p/Q_m was 1.09.
3. The GRLWEAP prediction using the GRL default values of Smith soil parameters has an average Q_p/Q_m of 0.76. This indicated that a better prediction for EOD condition compared to the other two methods. However, the coefficient of variation in the highest among the three methods. Also, comparing to the BOR condition, the GRL default value greatly underestimated the pile capacity

6.3 Conclusions and Recommendations

The pile capacity estimated by GRLWEAP using the Smith soil parameters determined by the proposed empirical equations was in reasonable agreement with the load test results and was the best compared to the other two methods. However, there are significant variations compared to the load test results. The main reasons are:

1. The quality of database. The results of CAPWAP analysis were used as basic source for regression study. However, the prediction of CAPWAP analysis already posed significant scattered.
2. Lack of unique solution of CAPWAP analysis. Although CAPWAP can be used to back calculate the Smith soil parameters, the lack of unique solution make it very difficult to select a site-specific parameters.
3. Dependence of Smith soil parameters. Smith soil parameters in conjunction with the estimated total capacity and resistance distribution are a set of parameters that can't be separated in CAPWAP analysis. Therefore, a regression analysis using one parameter at a time may significantly increase the scatter of the results.
4. Uncertainty of the efficiency of hammer and transfer energy. Mobilization of the soil resistance has significant impact on determining the Smith soil parameters. However, the performance of the driving system sometimes is unpredictable.

To overcome the problems, a rational model using soil mechanics parameters, such as shear modulus and finite element analysis may provide a better approach to solve the pile driving problems. However, the uncertainty of driving conditions during pile installation still make the prediction of the pile capacity using wave equation analysis difficult. Exploration of other alternatives to estimate pile capacity without solving the wave equation analysis is encouraged.

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APPENDIX

UF DATABASE SUMMARY

PILEUF File No.	Location	Pile Width D (in)	Total Pile Length (ft)	Embedded Pile Length L (ft)	Soil Type (along pile)	Soil Type (at tip)	Weighted Avrage SPT N (along pile)	Weighted Avrage SPT N (at tip)
1	HOWARD FRANKLAND / LS1	24.00	85.6	54.8	2	2	32.5	48.0
2	HOWARD FRANKLAND / LS3	30.00	67.7	39.6	2	3	60.0	60.0
3	HOWARD FRANK. / LS4 SHORT	30.00	52.9	24.6	2	3	14.3	25.0
4	HOWARD FRANK. / LS4 LONG	30.00	101.8	73.5	3	3	35.8	60.0
5	APPALACHICOLA RIVER PIER 3	24.00	93.2	90.63	2	2	9.9	26.0
6	APPALACHICOLA RIVER PIER14	30.00	83.9	58.8	2	2	11.2	21.7
7	APPALACHICOLA RIVER PIER25	24.00	66.3	55.45	2	2	7.6	20.3
8	APP. RIVER BRIDGE FSB16	18.00	65.2	61.02	1	2	3.0	4.0
9	APPALACHICOLA BAY BENT 41	24.00	69.2	52.3	1	2	3.9	10.0
10	APP. BAY BRIDGE BENT 101	24.00	80.5	62.1	1	1	6.7	31.0
11	APP. BAY BRIDGE BENT 133	24.00	123.7	104.87	1	1	7.9	37.0
12	APP. BAY BRIDGE BENT 145	24.00	121.5	102.98	2	2	43.4	#N/A
13	APP. BAY FSB22	18.00	68.2	64.02	1	1	7.0	23.0
14	BLOUNT ISLAND TERM. B-20	20.00	55.0	46.2	2	1	11.5	45.0
15	BLOUNT ISLAND TERM. B-21	20.00	40.0	36.42	2	2	30.5	26.3
16	ORLANDO D-22	14.00	94.0	90	2	2	16.6	21.0
17	DODGE ISLAND 3-E-18	30.00	65.0	49.43	3	3	36.9	42.0
18	DODGE ISLAND 4-E-18	30.00	75.0	52.8	3	3	36.8	15.0
19	DODGE ISLAND 6-E-20	30.00	110.0	97.2	3	3	#N/A	#N/A
20	DODGE ISLAND 8-E-20	30.00	110.0	39.78	3	3	21.1	33.0
21	DODGE ISLAND 9-E-20	30.00	65.0	29	3	3	43.2	53.0
22	DODGE ISLAND LTP (STATIC)	30.00	110.0	39.8	2	3	17.7	34.3
23	CHOCTAWHATCHEE FSB-3	24.00	83.9	77.74	2	2	5.0	10.0
24	CHOCTAWHATCHEE P-5	30.00	71.1	53.86	2	2	10.3	40.0
25	CHOCTAWHATCHEE P-11	30.00	106.0	85.51	2	2	8.6	14.0
26	CHOCTAWHATCHEE P-17	30.00	102.0	77.8	2	2	8.2	25.5
27	CHOCTAWHATCHEE P-23	30.00	101.0	82.53	2	2	11.4	16.0
28	CHOCTAWHATCHEE P-29	30.00	103.6	84.35	2	2	10.0	20.5
29	CHOCTAWHATCHEE P-35	30.00	98.5	78.96	2	2	13.3	18.0
30	CHOCTAWHATCHEE P-41	30.00	85.0	65.19	1	1	7.2	22.0
31	CHOCTAWHATCHEE FSB-26	24.00	69.0	64.84	2	1	5.3	4
32	CHOCTAWHATCHEE FSB-26	24.00	125.0	87.2	2	2	#N/A	25.0
33	CAPE CANAVERAL T-1	14.00	77.2	76.28	2	2	21.4	31.0
34	CAPE CANAVERAL T-6	18.00	53.1	53.08	2	2	18	53
35	CAPE CANAVERAL T-7	14.00	76.2	76.19	3	2	23.0	29.0
36	CAPE CANAVERAL T-14	14.00	69.7	69.5	2	2	16.3	25.0
37	WHITE CITY BRIDGE TP1	24.00	125.6	50.1	1	2	12.7	60.0

PILEUF File No.	Location	Pile Width D (in)	Total Pile Length (ft)	Embedded Pile Length L (ft)	Soil Type (along pile)	Soil Type (at tip)	Weighted Avrage SPT N (along pile)	Weighted Avrage SPT N (at tip)
38	WHITE CITY BRIDGE TP2	24.00	51.3	40	1	2	15.8	56.0
39	WHITE CITY BRIDGE TP3	24.00	40.3	37.2	2	2	6.9	33.5
40	WHITE CITY BRIDGE TP4	24.00	34.8	29.5	2	2	22.4	38.0
41	WHITE CITY BRIDGE TP5	24.00	37.8	29.3	2	2	4.4	15.0
42	WHITE CITY BRIDGE TP6	24.00	31.0	28.5	2	2	10.9	14.0
43	WHITE CITY BRIDGE TP7	24.00	43.7	37.5	2	2	14.7	40.0
44	WHITE CITY BRIDGE TP8	24.00	37.5	29.3	2	2	13.7	48.0
45	ACOSTA BRIDGE PEIR F6	24.00	67.0	58.5	2	2	19.7	23.0
46	ACOSTA BRIDGE PEIR G13	24.00	62.0	46.13	2	3	20.2	8
47	ACOSTA BRIDGE PEIR H2	24.00	39.0	35.91	1	3	29.3	25.0
48	WEST BAY BRIDGE TP9	30.00	130.0	128.42	1	1	#N/A	#N/A
49	WEST BAY BRIDGE TP15	30.00	105.0	103.62	2	1	10.6	12.0
50	ESCAMBIA RIVER BENT5	24.00	92.0	85.71	2	2	12.4	17.0
51	ESCAMBIA RIVER BENT77	24.00	65.0	61.32	2	2	9.9	8.0
52	ROOSEVELT BRIDGE A-30	30.00	72.0	53.38	3	3	18.3	48.0
53	ROOSEVELT BRIDGE B-30-W	30.00	62.5	43.8	2	3	16.8	60.0
54	BUCKMAN BRIDGE TS-13	30.00	121.0	94.52	1	2	12.8	48.0
55	BUCKMAN BRIDGE TS-19	30.00	116.9	89.28	2	2	17.8	36.0
56	BUCKMAN BRIDGE TS-24	30.00	110.6	80.8	2	1	19.6	12.0
57	BUCKMAN BRIDGE TS-29	30.00	104.5	79.99	2	2	15.9	54.0
58	JUL. CRK BENT 55-P4 1OF2	24.00	80.0	51	2	2	15.9	15.0
59	JULING. CRK BT 55-P4 2OF2	24.00	80.0	72	2	2	21.6	56.0
60	JULING. CRK BENT 47-P4 #1	24.00	95.0	74	2	2	10.1	29.0
61	JULING. CRK BENT 47-P4 #2	24.00	95.0	76	2	2	10	19
62	JULING. CRK BENT 37-P4	24.00	95.0	72	2	2	12.0	23.5
63	JULINGTON CRK BENT 28-P8	24.00	95.0	90	2	2	#N/A	#N/A
64	JULINGTON CRK BENT 22-P3	24.00	90.0	74	2	2	14.8	30.0
65	JULINGTON CRK BENT 18-P4	24.00	90.0	84	2	2	#N/A	#N/A
66	JULINGTON CRK BENT 32-P4	24.00	76.0	72.5	2	2	11.5	23.0
67	JULINGTON CRK BENT 31-P10	24.00	90.0	90	2	2	#N/A	#REF!
68	I295 SR21 PIER 3R-37	18.00	65.0	53	2	2	11.9	23.5
69	I295/ORTEGA RIV PIER 3R-14	18.00	40.0	33	2	3	17.6	31.0
70	I295/CSX BENT 2R-16	18.00	65.0	27	2	3	30.7	52.0
71	I295/SR 17 PIER 1L-11	18.00	50.0	34	2	3	13.4	18.0
72	I295/103RD ST. PIER 1R-P1	18.00	60.0	45	2	1	21.3	8.0
73	I295/WILSON PIER 2W-P3	18.00	90.0	66	2	1	29.1	5.5
74	I295/SR 228 PIER 6E-P2	18.00	70.0	51	2	3	14.2	33.0

PILEUF File No.	Location	Pile Width D (in)	Total Pile Length (ft)	Embedded Pile Length L (ft)	Soil Type		Weighted Avrage SPT N	
					(along pile)	(at tip)	(along pile)	(at tip)
75	I295/SR 228 BENT 1W-P1	20.00	70.0	45	1	3	19.1	55.0
76	I295/MEM. PK BENT 2W-P12	18.00	50.0	50	#REF!	#REF!	#N/A	#N/A
77	I295/MELVIN RD PIER 2E-P1	18.00	85.0	64	2	2	29.1	26.0
78	I295/I10 SB PIER 1-P21	18.00	40.0	39	2	3	18.6	#N/A
79	I295/I10 NB PIER 1-P11	18.00	40.0	40	2	3	17.4	#N/A
80	I295/US 90 PIER 5-P42	18.00	70.0	44	2	3	37.5	60.0
81	I295/US 90 PIER 6-P66	18.00	70.0	43	2	3	21.4	#N/A
82	I295/US 90 BENT 2-P17	20.00	70.0	42	2	3	27.3	#N/A
83	I295/US90 PIER 5-P55	18.00	70.0	48	2	3	19.1	#N/A
84	I295/I10 RAMP A PIER 1-P12	18.00	60.0	53	2	3	35.5	60.0
85	MARCO ISLAND TP10	14.00	65.0	63	2	3	15.1	24.0
86	MARCO ISLAND TP2	14.00	48.0	33	2	3	15.7	16.5
87	DU CHARME RESIDENCE TP3	14.00	45.0	43	2	3	8.6	14.5
88	ST. LAURENT TOWER 106	14.00	70.0	70	2	3	8.9	19.0
89	MARINA BAY CLUB TP7	14.00	85.0	83	2	3	9.5	26
90	ST. MARISSA CONDO. TP8	14.00	65.0	50	2	3	10.8	60.0
91	ST. MARISSA CONDO. PILE 20	14.00	50.0	50	2	3	10.8	41.3
92	STONEBURNER TP-SW-2-14	14.00	55.0	49	2	3	10.8	40.0
93	SR 580 OLDSMAR, FLORIDA	20.00	50.0	47	1	3	39.7	60.0
94	GEORGIA/FLORIDA BOUNDARY	10.00	43.0	43	2	3	13.9	35.7
95	JACKSONVILLE SITE B	14.00	33.0	33	2	3	23.0	60.0
96	JACKSONVILLE SITE D	14.00	62.0	62	2	3	14.8	60.0
97	SAINT JOHN RIVER SITE F	18.00	51.0	35	2	3	14	43
98	DUPONT CENTER, JACKSONVILLE	12.00	31.0	31	2	3	25.4	31.0
99	LONGBOAT KEY - SARASOTA	12.00	50.0	49.1	3	3	20.8	36.0
100	SAINT JOHN'S (ASCE) - 3A	20.00	36.5	36.5	2	2	29	20
101	49th STREET BRIDGE TP1	24.00	45.0	41.2	2	3	23.1	49
102	49th STREET BRIDGE TP37	30.00	59.0	23.4	1	3	31	60
103	49th STREET BRIDGE TP38	24.00	59.0	23.6	1	3	30.9	60.0
104	SURFRIDER CONDOMINIUM	12.00	35.0	29	2	2	37.2	29.0
105	KARIDAS CONDOMINUM #1	12.00	12.0	12	2	2	20.0	28.3
106	KARIDAS CONDOMINIUM #2	12.00	12.0	8.5	2	2	17.9	15.5
107	KARIDAS CONDOMINIUM #3	14.00	13.0	8.5	2	2	19.0	26.0
108	BEACHES OF LONGBOAT	12.00	22.0	14	2	2	28.4	40.0
109	VIENTA CONDOMINIUM	12.00	14.0	13	2	2	43.5	50.0
110	ARVIDA HOTEL	12.00	35.0	35	2	2	34.7	60.0
111	VERANDA HOTEL, SARASOTA	12.00	20.0	18	2	2	37.8	45.5

PILEUF File No.	Location	Pile Width D (in)	Total Pile Length (ft)	Embedded Pile Length L (ft)	Soil Type (New system)		Weighted Avrage SPT N	
					(along pile)	(at tip)	(along pile)	(at tip)
112	LONGBOAT COVE, SARASOTA	12.00	16.0	16	2	2	45.8	60.0
113	I-95 WEST PALM BEACH #1	18.00	35.0	26.5	2	2	6	2
114	I-95 WEST PALM BEACH #2	18.00	45.0	37.2	2	2	7	7
115	BLOUNT ISLAND SITE 215	10.00	70.0	68	2	2	11.7	22.0
116	BLOUNT ISLAND SITE 316	14.00	52.0	52	2	2	9.0	10.5
117	BLOUNT ISLAND SITE 348	18.00	49.0	49	2	2	13.6	29.5
118	I-275 34th ST. PINELLAS	18.00	70.0	69	2	1	18	18
119	MAYPORT N.A.S. JACKSONVILLE	14.00	40.0	40	2	2	18.5	21.0
120	PLAYERS CLUB VILLAS BRIDGE	14.00	56.0	44	2	1	10.7	13.0
121	SIESTA KEY SARASOTA	12.00	35.0	16.3	2	2	47.1	58.0
122	DeSOTA CONDOMINIUM MS.	16.00	24.9	23.8	2	2	15.8	38.0
123	WASHINGTON CONDOMINIUM	14.00	54.5	52.5	2	2	43.4	58.0
124	SUNSET RESORT HOTEL	12.00	65.0	64	2	2	20.6	60.0
125	SUNSHINE SKYWAY SITE 1 A	24.00	68.8	49.21	2	3	31.0	30.0
126	SUNSHINE SKYWAY SITE 1 B	20.00	68.0	47.31	2	3	28.8	30
127	SUNSHINE SKYWAY SITE 3	24.00	79.6	48	1	3	25.9	60.0
128	SUNSHINE SKYWAY SITE 10	24.00	60.5	27.9	1	3	40.5	60.0
129	SUNSHINE SKYWAY SITE 13 A	20.00	38.2	20.63	3	3	46.4	60
130	SUNSHINE SKYWAY SITE 13 B	24.00	43.5	26.91	3	3	46.4	60.0
131	SUNSHINE SKYWAY SITE 15	20.00	49.7	32	2	2	14.4	17.5
132	ST. JOHN'S RIVER (ASCE)-3B	20.00	46.0	46	2	2	30.6	53
133	ST. JOHN'S RIVER (ASCE) 3C	14.00	60.0	60	2	3	15.3	21.0
134	ST. AUGUSTINE (ASCE) 4A	12.00	28.0	28	2	2	12.0	19.0
135	DOWNTOWN ORLANDO ARENA	14.00	86.0	86	2	2	14.8	29.5
136	TALMADGE MEMORIAL BRIDGE	14.00	75.0	73	2	2	15.4	30.0
137	JACKSONVILLE INDUSTRIAL #1	20.00	46.0	46	2	2	30.6	42.0
138	JACKSONVILLE INDUSTRIAL #2	20.00	36.0	36	2	2	26.1	17.5
139	FORT MYERS	14.00	67.0	67	2	2	9.0	6.0
140	FLORENCE/MARION 3 ASD	18.00	65.0	25	2	2	24	30
141	FLORENCE / MARION 3 BSD	18.00	70.0	40	2	2	11	31
142	FLORENCE / MARION 3 CSD	18.00	70.0	38	2	2	15	45
143	NORTHEAST VILLA MIRADA - 6	14.00	9.0	8.8	3	1	11.5	18.5
144	SARASOTA MEM. HOSPITAL	12.00	26.7	25	2	3	19.3	60.0
145	PORT ORANGE BENT 19 PILE 9	5.00	34.3	30.87	3	3	15.6	21.0
146	PORT ORANGE BENT 2 PILE 6	18.00	32.8	30.11	3	3	20.1	27.5
147	SEAWAY HOTELS, SAND KEY	14.00	30.0	29.8	2	3	24.3	36.0

PILEUP File No.	Hammer Type	Rated Energy	Smith Damping (EOD)		Smith Quake (EOD)		Smith Damping (BOR)		Smith Quake (BOR)	
			Skin	Toe	Skin	Toe	Skin	Toe	Skin	Toe
		(kips-ft)	(s/f)	(s/f)	(in)	(in)	(s/f)	(s/f)	(in)	(in)
1	VULCAN 020	60.0	0.308	0.253	0.150	0.180	0.375	0.304	0.040	0.120
2	CONMACO C300 AIR/STEAM HAMMER	90.0	0.051	0.058	0.140	0.290	#N/A	#N/A	#N/A	#N/A
3	CONMACO C300 AIR/STEAM HAMMER	90.0	0.061	0.057	0.150	0.340	#N/A	#N/A	#N/A	#N/A
4	CONMACO C300 AIR/STEAM HAMMER	90.0	0.198	0.071	0.150	0.380	0.042	0.206	0.100	0.200
5	VULCAN 020 AIR/STEAM HAMMER	60.0	0.180	0.160	0.150	0.250	0.223	0.261	0.080	0.020
6	CONMACO 300 AIR/STEAM HAMMER	90.0	0.220	0.219	0.120	0.100	0.227	0.120	0.150	0.200
7	VULCAN 020 AIR/STEAM HAMMER	60.0	0.120	0.080	0.120	0.350	0.110	0.100	0.100	0.320
8	VULCAN 010 AIR/STEAM HAMMER	32.5	0.105	0.155	0.100	0.230	0.157	0.649	0.080	0.240
9	VULCAN 020 AIR/STEAM HAMMER	60.0	0.076	0.152	0.080	0.290	0.101	0.134	0.100	0.350
10	VULCAN 020 AIR/STEAM HAMMER	60.0	0.305	0.042	0.120	0.400	0.209	0.204	0.090	0.100
11	VULCAN 020 AIR/STEAM HAMMER	60.0	0.205	0.256	0.180	0.350	0.188	0.210	0.125	0.130
12	VULCAN 020 AIR/STEAM HAMMER	60.0	0.241	0.150	0.090	0.190	0.209	0.210	0.140	0.155
13	CONMACO 115 AIR/STEAM HAMMER	37.4	0.139	0.087	0.110	0.300	0.344	0.079	0.080	0.100
14	VULCAN 510	32.5	#N/A	#N/A	#N/A	#N/A	0.211	0.211	0.120	0.230
15	VULCAN 510	32.5	#N/A	#N/A	#N/A	#N/A	0.169	0.128	0.100	0.310
16	VULCAN 80C	24.5	0.291	0.076	#N/A	#N/A	0.159	0.194		
17	CONMACO 300	90.0	0.109	0.076	0.191	0.090	#N/A	#N/A	#N/A	#N/A
18	CONMACO 300	90.0	0.087	0.125	0.070	0.310	#N/A	#N/A	#N/A	#N/A
19	CONMACO C300	90.0	#N/A	#N/A			0.128	0.162	0.100	0.150
20	CONMACO 300	90.0	0.129	0.090	0.070	0.150	#N/A	#N/A		
21	CONMACO	90.0	#N/A	#N/A			0.105	0.254	0.110	0.080
22	CONMACO 300	90.0	#N/A	#N/A	#N/A	#N/A	0.084	0.125	0.100	0.130
23	VULCAN 020 AIR/STEAM HAMMER	60.0	#N/A	#N/A		0.045	0.379	0.527	0.110	0.190
24	ICE 200S DIESEL HAMMER	100.0	0.929	0.314	0.100	0.140	0.405	0.227	0.100	0.300
25	ICE 200S DIESEL HAMMER	100.0	#N/A	#N/A			0.249	1.335	0.180	0.140
26	ICE 200S DIESEL HAMMER	100.0	#N/A	#N/A			0.125	0.350	0.030	0.220
27	ICE 200S DIESEL HAMMER	100.0	#N/A	#N/A			0.535	1.674	0.170	0.050
28	ICE 200S DIESEL HAMMER	100.0	#N/A	#N/A			0.483	0.707	0.100	0.090
29	ICE 200S DIESEL HAMMER	100.0	#N/A	#N/A			0.226	0.700	0.170	0.090
30	ICE 200S DIESEL HAMMER	100.0	0.209	0.141	0.100	0.260	0.392	0.351	0.130	0.140
31	VULCAN 020	60.0	0.105	0.076	0.120	0.210	0.736	0.176	0.100	0.230
32	DELMAG D 62-12	165.0	#N/A	#N/A	#N/A	#N/A	0.320	0.302		
33	ICE 640 CLOSED ENDED DIESEL HAMMER	40.0	0.073	0.073	0.100	0.400	0.241	0.147	0.120	0.100
34	ICE 640 CLOSED ENDED DIESEL HAMMER	40.0	#N/A	#N/A			0.426	0.197	0.130	0.224
35	ICE 640 CLOSED ENDED DIESEL HAMMER	40.0	#N/A	#N/A			0.229	0.157	0.140	0.140
36	ICE 640 CLOSED ENDED DIESEL HAMMER	40.0	#N/A	#N/A			0.235	0.181	0.170	0.190
37	DELMAG D46-02 OPEN ENDED DIESEL HAMI	105.0	0.110	0.120			#N/A	#N/A		

PILEUP File No.	Hammer Type	Rated Energy	Smith Damping (EOD)		Smith Quake (EOD)		Smith Damping (BOR)		Smith Quake (BOR)	
			Skin	Toe	Skin	Toe	Skin	Toe	Skin	Toe
		(kips-ft)	(s/f)	(s/f)	(in)	(in)	(s/f)	(s/f)	(in)	(in)
38	DELMAG D46-02 OPEN ENDED DIESEL HAM	105.0	0.173	0.141	0.120	0.335	0.333	0.058		
39	DELMAG D46-02 OPEN ENDED DIESEL HAM	105.0	0.066	0.168	0.100	0.400	0.137	0.087	0.080	0.320
40	DELMAG D46-02 OPEN ENDED DIESEL HAM	105.0	0.223	0.058	0.100	0.350	0.220	0.072	0.110	0.315
41	DELMAG D46-02 OPEN ENDED DIESEL HAM	105.0	0.192	0.118	0.080	0.377	0.179	0.118	0.090	0.300
42	DELMAG D46-02 OPEN ENDED DIESEL HAM	105.0	0.143	0.118	0.100	0.420	0.311	0.048	0.100	0.610
43	DELMAG D46-02 OPEN ENDED DIESEL HAM	105.0	0.230	0.096	0.110	0.270	0.242	0.122	0.069	0.265
44	DELMAG D46-02 OPEN ENDED DIESEL HAM	105.0	0.158	0.183	0.098	0.371	0.153	0.122	0.090	0.360
45	DELMAG D46-32	107.2	0.147	0.049	0.113	0.600	0.274	0.553	0.156	0.200
46	DELMAG D46-32	107.2	#N/A	#N/A			0.164	0.144	0.130	0.230
47	DELMAG D46-32	107.2	0.154	0.035	0.047	0.365	0.233	0.049	0.110	0.326
48	CONMACO 300E5	150.0	#N/A	#N/A			0.250	0.430	0.050	0.260
49	CONMACO 300E5	150.0	#N/A	#N/A			0.490	0.240	0.060	0.225
50	DELMAG 46-32	107.2	0.316	0.099	0.045	0.210	0.306	0.237	0.050	0.220
51	DELMAG 46-32	107.2	0.270	0.153	0.100	0.230	0.248	0.228	0.150	0.190
52	ICE 200S SINGLE ACTING DEISEL HAMMER	100.0	#N/A	#N/A			0.377	0.251		
53	ICE 200S SINGLE ACTIN DEISEL HAMMER	100.0	#N/A	#N/A			0.160	0.388		
54	CONMACO 300E5 AIR HAMMER	120.0	0.260	0.171	0.070	0.282	0.304	0.152	0.073	0.200
55	CONMACO 300E5	120.0	0.164	0.115	0.125	0.265	0.169	0.141	0.100	0.360
56	CONMACO 300E5	120.0	0.231	0.115	0.010	0.265	0.231	0.242	0.100	0.200
57	CONMACO 300E5	120.0	0.272	0.107	0.032	0.215	0.275	0.074	0.093	0.355
58	DELMAG 46-32	70.2	0.147	0.103	0.100	0.380	0.126	0.112	0.090	0.460
59	DELMAG 46-32	70.2	0.183	0.235	0.090	0.300	#N/A	#N/A		
60	DELMAG 46-32	70.2	0.120	0.085	0.100	0.320	0.136	0.170	0.080	0.280
61	DELMAG 46-32	70.2	0.103	0.191	0.110	0.480	#N/A	#N/A		
62	DELMAG 46-32	70.7	0.170	0.109	0.078	0.448	0.133	0.137	0.100	0.470
63	delmag 46-32	70.2	0.094	0.127	0.090	0.545	0.427	0.284	0.075	0.300
64	DELMAG 46-32	70.2	0.132	0.100	0.120	0.400	0.168	0.122	0.090	0.314
65	DELMAG 46-32	70.2	0.252	0.135	0.080	0.430	0.431	0.379	0.055	0.320
66	DELMAG 46-32	70.2	#N/A	#N/A			0.181	0.231	0.120	0.290
67	DELMAD 46-32	70.2	#N/A	#N/A			0.407	0.349	0.115	0.370
68	BERMING B4505 OED	78.0	0.167	0.208			#N/A	#N/A		
69	BERMING B4505 OED	77.9	0.050	0.050	0.080	0.300	#N/A	#N/A		
70	BERMING B4505 OED	75.9	0.204	0.157	0.070	0.280	#N/A	#N/A		
71	BERMING B4505 OED	77.9	0.140	0.117	0.090	0.285	#N/A	#N/A		
72	BERMING B4505 OED	75.9	0.131	0.109	0.090	0.410	#N/A	#N/A		
73	BERMING B4505 OED	78.0	0.152	0.150			#N/A	#N/A		
74	BERMING B4505 OED	78.0	0.122	0.086			#N/A	#N/A		

PILEUF File No.	Hammer Type	Rated Energy	Smith Damping (EOD)		Smith Quake (EOD)		Smith Damping (BOR)		Smith Quake (BOR)	
			Skin	Toe	Skin	Toe	Skin	Toe	Skin	Toe
		(kips-ft)	(s/f)	(s/f)	(in)	(in)	(s/f)	(s/f)	(in)	(in)
75	BERMING B4505 OED	78.0	0.087	0.052			#N/A	#N/A		
76	BERMING B4505 OED	32.6	0.176	0.097	0.075	0.215	#N/A	#N/A		
77	BERMING B4505 OED	32.0	0.421	0.439	0.050	0.200	#N/A	#N/A		
78	BERMING B4505 OED	75.9	0.271	0.089	0.100	0.260	#N/A	#N/A		
79	BERMING B4505 OED	75.9	0.259	0.070	0.100	0.290	#N/A	#N/A		
80	BERMING B4505 OED	75.9	0.128	0.105	0.071	0.190	#N/A	#N/A		
81	BERMING B4505 OED	78.0	0.156	0.177	0.080	0.260	#N/A	#N/A		
82	BERMING B4505 OED	78.0	0.204	0.132	0.080	0.250	#N/A	#N/A		
83	BERMING B4505 OED	78.0	0.217	0.444	0.080	0.104	#N/A	#N/A		
84	BERMING B4505 OED	75.9	0.123	0.113	0.100	0.311	#N/A	#N/A		
85	ICE 520-30	30.0	#N/A	#N/A			#N/A	#N/A		
86	ICE 520-30	31.5	#N/A	#N/A			#N/A	#N/A		
87	VULCAN 01	15.0	#N/A	#N/A			#N/A	#N/A		
88	ICE 640	40.0	#N/A	#N/A			#N/A	#N/A		
89	ICE 520	31.0	#N/A	#N/A			#N/A	#N/A		
90	LINKBELT 520	25.0	#N/A	#N/A			#N/A	#N/A		
91	ICE 520-30	30.0	#N/A	#N/A			#N/A	#N/A		
92	LINKBELT 520	23.6	#N/A	#N/A			#N/A	#N/A		
93	MKT DE-70B	49.0	#N/A	#N/A			#N/A	#N/A		
94	VULCAN 1	15.0	#N/A	#N/A			#N/A	#N/A		
95	LINKBELT 520	25.5	#N/A	#N/A			#N/A	#N/A		
96	RAYMOND 65C	19.5	#N/A	#N/A			#N/A	#N/A		
97	VULCAN 010	32.5	#N/A	#N/A			#N/A	#N/A		
98	LINKBELT 520	26.3	#N/A	#N/A			#N/A	#N/A		
99	MKT DA35B	38.0	#N/A	#N/A			#N/A	#N/A		
100	VULCAN 510	32.5	#N/A	#N/A			#N/A	#N/A		
101	DELMAG D46-32	28.0	#N/A	#N/A			#N/A	#N/A		
102	DELMAG D62-32	#N/A	0.117	0.186	0.100	0.230	#N/A	#N/A		
103	DELMAG D46-32	#N/A	0.267	0.141	0.120	0.300	#N/A	#N/A		
104	MKT DA 35C	21.0	#N/A	#N/A			#N/A	#N/A		
105	LINK BELT 312	15.0	#N/A	#N/A			#N/A	#N/A		
106	LINK BELT 312	15.0	#N/A	#N/A			#N/A	#N/A		
107	LINK BELT 312	15.0	#N/A	#N/A			#N/A	#N/A		
108	MKT DA 35C	21.0	#N/A	#N/A			#N/A	#N/A		
109	MKT DA 35C	21.0	#N/A	#N/A			#N/A	#N/A		
110	MKT-DA 35C	21.0	#N/A	#N/A			#N/A	#N/A		
111	MKT DA 35C	21.0	#N/A	#N/A			#N/A	#N/A		

PILEUP File No.	Hammer Type	Rated Energy	Smith Damping (EOD)		Smith Quake (EOD)		Smith Damping (BOR)		Smith Quake (BOR)	
			Skin	Toe	Skin	Toe	Skin	Toe	Skin	Toe
		(kips-ft)	(s/f)	(s/f)	(in)	(in)	(s/f)	(s/f)	(in)	(in)
112	MKT-DA 35C	21.0	#N/A	#N/A			#N/A	#N/A		
113	MOD. VULCAN #2	#N/A	#N/A	#N/A			#N/A	#N/A		
114	MOD. VULCAN #2	#N/A	#N/A	#N/A			#N/A	#N/A		
115	RAYMOND M65-C	19.5	#N/A	#N/A			#N/A	#N/A		
116	RAYMOND M65-C	19.5	#N/A	#N/A			#N/A	#N/A		
117	RAYMOND M 65-C	19.5	#N/A	#N/A			#N/A	#N/A		
118	DELMAG DE 46-02	75.8	#N/A	#N/A			#N/A	#N/A		
119	ICE MODEL 520	26.3	#N/A	#N/A			#N/A	#N/A		
120	VULCAN NO. 1	15.0	#N/A	#N/A			#N/A	#N/A		
121	ICE 520	30.0	#N/A	#N/A			#N/A	#N/A		
122	FAIRCHILD	20.0	#N/A	#N/A			#N/A	#N/A		
123	MKT	28.0	#N/A	#N/A			#N/A	#N/A		
124	#N/A	#N/A	#N/A	#N/A			#N/A	#N/A		
125	CONMACO 300	90.0	#N/A	#N/A			0.240	0.053	0.150	0.280
126	CONMACO 300	48.9	#N/A	#N/A			0.170	0.012	0.100	0.200
127	CONMACO 300	89.0	#N/A	#N/A			0.340	0.009	0.150	0.150
128	CONMACO 300	90.0	#N/A	#N/A			0.181	0.050	0.200	0.240
129	CONMACO 300	49.0	#N/A	#N/A			0.320	0.002	0.150	0.300
130	CONMACO 300	49.0	#N/A	#N/A			0.290	0.036	0.100	0.340
131	CONMACO 300	49.0	#N/A	#N/A			0.090	0.940	0.125	0.125
132	VULCAN 510	32.5	#N/A	#N/A			#N/A	#N/A		
133	VULCAN 80C	24.5	#N/A	#N/A			#N/A	#N/A		
134	ICD 440	19.8	#N/A	#N/A			#N/A	#N/A		
135	ICE 640	40.0	#N/A	#N/A			#N/A	#N/A		
136	ICE 1070	34.0	#N/A	#N/A			#N/A	#N/A		
137	#N/A	#N/A	#N/A	#N/A			#N/A	#N/A		
138	#N/A	#N/A	#N/A	#N/A			#N/A	#N/A		
139	#N/A	#N/A	#N/A	#N/A			#N/A	#N/A		
140	MKT DE 70B	#N/A	#N/A	#N/A			#N/A	#N/A		
141	MKT DE 70B	#N/A	#N/A	#N/A			#N/A	#N/A		
142	MKT DE 70B	#N/A	#N/A	#N/A			#N/A	#N/A		
143	MKT DA 35 B	21.0	#N/A	#N/A			#N/A	#N/A		
144	MKT DA 35 B	21.0	#N/A	#N/A			#N/A	#N/A		
145	ICE 640	40.0	0.105	0.239	0.030	0.130	0.040	0.129	0.020	0.240
146	ICE 640	40.0	0.110	0.141	0.020	0.210	0.074	0.063	0.050	0.280
147	ICE 520	32.0	#N/A	#N/A			#N/A	#N/A		

UFPILE	Measured		CAPWAP RESULTS				PROPOSED FORMULA			
File No.	Davisson's Capacity (Total) (tons)	Blow Count (BOR) (bls/ft)	Smith Damping (BOR)		Smith Quake (BOR)		Smith Damping (BOR)		Smith Quake (BOR)	
			Skin (s/f)	Toe (s/f)	Skin (in)	Toe (in)	Skin (s/f)	Toe (s/f)	Skin (in)	Toe (in)
1	550	120	0.375	0.304	0.040	0.120	0.2607	0.2735	0.1211	0.2802
4	430	80	0.042	0.206	0.100	0.200	0.0354	0.0676	0.0350	0.1899
5	477	48	0.223	0.261	0.080	0.020	0.2257	0.2419	0.1059	0.2562
6	476	240	0.227	0.120	0.150	0.200	0.2088	0.1476	0.0989	0.1852
7	359	240	0.110	0.100	0.100	0.320	0.2100	0.2218	0.0993	0.2410
8	164	92	0.157	0.649	0.080	0.240	0.1955	0.1168	0.0964	0.1617
9	264	96	0.101	0.134	0.100	0.350	0.1202	0.1200	0.0753	0.1642
10	407	576	0.209	0.204	0.090	0.100	0.2254	0.2745	0.1054	0.1115
11	402	672	0.188	0.210	0.125	0.130	0.2463	0.2759	0.1120	0.1087
13	213	144	0.344	0.079	0.080	0.100	0.2782	0.2776	0.1227	0.1051
24	712	192	0.405	0.227	0.100	0.300	0.1939	0.2162	0.0929	0.2368
30	720	105	0.392	0.351	0.130	0.140	0.1462	0.2579	0.0824	0.1451
31	480	144	0.736	0.176	0.100	0.230	0.1791	0.2144	0.0871	0.2262
39	315	80	0.137	0.087	0.080	0.320	0.1449	0.2159	0.0742	0.2366
42	230	80	0.311	0.048	0.100	0.610	0.1950	0.0705	0.0933	0.1245
45	388	216	0.274	0.553	0.156	0.200	0.2310	0.1656	0.1081	0.1988
47	289	168	0.233	0.049	0.110	0.326	0.3016	0.0621	0.1309	0.0227
54	553	120	0.304	0.152	0.073	0.200	0.2187	0.2162	0.1033	0.2368
55	656	96	0.169	0.141	0.100	0.360	0.2201	0.1822	0.1035	0.2113
56	574	348	0.231	0.242	0.100	0.200	0.2253	0.2264	0.1057	0.2051
57	632	75	0.275	0.074	0.093	0.355	0.2133	0.2273	0.1007	0.2451