NCHRP 24-17

LOAD AND RESISTANCE FACTOR DESIGN (LRFD) FOR DEEP FOUNDATIONS

APPENDIX C
STATIC ANALYSES OF DRIVEN PILES AND DRILLED SHAFTS

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DISCLAIMER

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1. Overview / Static Analyses

1.1 Scope of Attached Appendix

The attached appendix provides the full information regarding databases, methods of analysis, results of analyses and calculated resistance factors for drilled shafts and driven piles based on static methods of analysis.

1.2 Appendix Layout

The appendix is divided into two major parts, Driven Piles and Drilled Shafts. Each part is divided into three major chapters in the following format:

Part I - Summary table of the obtained Resistance Factors

Part II - Detailed of analysis results and Statistics supporting the data summarized in Part I.

Part III - Description of Evaluation Methods - detailed descriptions of the methods used to statistically evaluate the capacity of the driven piles or the drilled shaft, respectively.

1.3 Calculated vs. Measured Capacities

1.3.1 Driven Piles

Davisson's failure criterion was used as the representative static capacity of the pile. A discussion and evaluation of the issue is provided in the report and additional details are provided in Appendix B. Note, the maximum load and settlement of each pile load test are provided as part of tables A2 & A3 of the appendix. Davisson's failure criterion is provided as part of the capacity evaluation of each pile type (e.g., Table 50, p. 108 for plugged H piles).

1.3.2 Drilled Shafts

FHWA failure criterion was used for the capacity evaluation of the drilled shafts, i.e., the load associated with 5% of the shaft's diameter (0.05B), was taken as the shaft's capacity. As no significant amount of data are available to separate side skin from tip (end) resistances, the following procedure was used for the evaluation of the "measured" skin capacities:

The shape of the load-displacement curves was evaluated and the shafts for which more than 80% of the total capacity was mobilized in a displacement less than 2% of the shaft's diameter, were considered as those in which the resistance is based on friction and comparisons were held with the shaft resistance calculated values. See for example, the drilled shaft table of resistance factors related to skin resistance alone.

DRIVEN PILES - STATIC ANALYSIS

SUMMARY TABLES OF RESISTANCE FACTORS

Table 1: All Relevant Factors
Table 1A:Compressed Summary (initial Evaluation)
Table 2: Summary of Correlation Used
Table 3: Soil Properties Correlation from SPT
Table 4: Soil Properties Correlation from CPT

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Table 1 – Summary of Recommended Resistance Factors for Static Analysis of Driven Piles (Skin or Skin and Tip Resistances)

Soil/Method/Condition				Correlation Used (See Table 2)	Resistance Factor					
					β = 2.0		β = 2.5		β = 3.0	
					φ	φ/λ	φ	φ/λ	φ	φ/λ
Clay	**H-Piles**	β-Method	N = 4	11.5 B;T&P(2)	0.23	0.38	0.18	0.29	0.13	0.22
		λ-Method	N = 17	11.5B; T&P(2) 2B; T&P(5)	0.43	0.54	0.34	0.44	0.28	0.36
		α-Tomlinson	N = 17	2B; T&P(2)	0.47	0.57	0.38	0.46	0.31	0.38
		α-API	N = 17	2B; T&P(5)	0.49	0.51	0.39	0.41	0.31	0.32
		Schmertmann's SPT-97	N = 9		0.45	0.35	0.33	0.26	0.25	0.19
	Concrete Piles	λ-Method	N = 19	2B; Hara (5h)	0.52	0.66	0.44	0.56	0.37	0.47
		α-API	N = 19	2B; Hara (5h)	0.56	0.62	0.46	0.52	0.38	0.43
		β-Method	N = 8	2B; Hara (5h)	0.38	0.47	0.30	0.37	0.23	0.29
		α-Tomlinson	N = 19	2B; Hara (5h)	0.41	0.43	0.32	0.34	0.24	0.26
	Pipe Piles	α-Tomlinson	N = 20	2B; T&P (1)	0.28	0.36	0.21	0.27	0.15	0.20
		α-API	N = 20	2B; T&P (1)	0.29	0.32	0.22	0.24	0.16	0.17
		β-Method	N = 13	2B; T&P (1)	0.15	0.28	0.11	0.20	0.08	0.14
		λ-Method	N = 20	2B; T&P (1)	0.26	0.34	0.19	0.26	0.14	0.19
Sand	**H-Piles**	Nordlund	N = 19	36; 11.5B,P(6)	0.53	0.56	0.43	0.46	0.35	0.37
		Meyerhof	N = 19		0.46	0.53	0.37	0.43	0.30	0.35
		β-Method	N = 19	36; 2B; P(5)	0.36	0.46	0.29	0.36	0.22	0.29
		Schmertmann SPT-97	N = 19		0.73	0.51	0.58	0.41	0.46	0.33
	Concrete Piles	Nordlund Method	N = 37	36: 11.5B; P(6)	0.48	0.45	0.37	0.35	0.29	0.27
		β-Method	N = 37	36; 2B; P(5)	0.57	0.47	0.45	0.39	0.36	0.31
		Meyerhof Method	N = 37		0.22	0.35	0.17	0.26	0.12	0.19
		Schmertmann's SPT97	N = 37		0.6	0.48	0.47	0.38	0.37	0.30
	Pipe Piles	Nordlund	N = 20	36; 2B P(5)	0.65	0.41	0.5	0.31	0.38	0.24
		β-Method	N = 20	36; 2B P(5)	0.45	0.38	0.34	0.29	0.26	0.22
		Meyerhof	N = 20		0.37	0.39	0.28	0.30	0.21	0.22
		Schmertmann's SPT97	N = 20		0.70	0.41	0.54	0.31	0.41	0.24
Mixed Soils	**H-Piles**	α-Tomlinson/Nordlund/Thurman	N = 22	36; 2B; P(5)	0.25	0.35	0.19	0.26	0.14	0.20
		α-API/Nordlund/Thurman	N = 37	36; 2B; P(5)	0.35	0.38	0.27	0.29	0.20	0.22
		β-Method/Thurman	N = 35	36; 2B; P(5)	0.20	0.36	0.15	0.30	0.11	0.20
		Schmertmann's SPT-97	N = 41		0.64	0.51	0.51	0.41	0.41	0.32
	Concrete Piles	α-Tomlinson/Nordlund/Thurman	N = 34	36; 2B; P; Hara(5h)	0.45	0.45	0.35	0.35	0.27	0.27
		α-API/Nordland/Thurman	N = 85	36; 11.5B; Sch; T&P	0.42	0.45	0.33	0.35	0.26	0.27
		β-Method/Thurman	N = 85	36; 11.5B; Sch; T&P	0.43	0.49	0.34	0.39	0.27	0.31
		DRIVEN	N = 34		0.43	0.26	0.30	0.18	0.21	0.13
		Schmertmann's SPT97	N = 74		0.75	0.38	0.56	0.29	0.43	0.22
		Schmertmann's CPT	N = 32		0.52	0.58	0.43	0.47	0.35	0.39
	Pipe Piles	All Methods			0.25	--	0.20	--	0.15	--

NOTE: Uplift Capacity to be taken as 0.75 of Compression Capacity

Table 1A – Compressed Summary of Recommended Resistance Factors for Static Analysis of Driven Piles (Skin or Skin and Tip Resistances)

Soil/Method/Condition				Correlation Used (See Table 2)	Resistance Factor		
					β = 2.0	β = 2.5	β = 3.0
					ϕ	ϕ	ϕ
Clay	H-Piles	β-Method	N = 4	11.5 B;T&P(2)	0.23	0.18	0.13
		λ-Method Schmertmann's SPT-97	N = 17 N = 9	11.5B; T&P(2) 2B; T&P(5)	0.44	0.33	0.26
		α-Tomlinson α-API	N = 17 N = 17	2B; T&P(2) 2B; T&P(5)	0.48	0.38	0.30
	Concrete Piles	λ-Method α-API	N = 19 N = 19	2B; Hara (5h) 2B; Hara (5h)	0.54	0.45	0.37
		β-Method α-Tomlinson	N = 8 N = 19	2B; Hara (5h) 2B; Hara (5h)	0.40	0.31	0.27
	Pipe Piles	α-Tomlinson α-API λ-Method	N = 20 N = 20 N = 20	2B; T&P (1) 2B; T&P (1) 2B; T&P (1)	0.30	0.20	0.15
		β-Method	N = 13	2B; T&P (1)	0.15	0.11	0.08
Sand	H-Piles	Nordlund	N = 19	36; 11.5B,P(6)	0.53	0.43	0.35
		Meyerhof	N = 19		0.46	0.37	0.30
		β-Method	N = 19	36; 2B; P(5)	0.36	0.29	0.22
		Schmertmann SPT-97	N = 19		0.73	0.58	0.46
	Concrete Piles	β-Method Schmertmann's SPT97	N = 37 N = 37	36; 2B; P(5)	0.58	0.46	0.36
		Nordlund Method	N = 37	36: 11.5B; P(6)	0.48	0.37	0.29
		Meyerhof Method	N = 37		0.22	0.17	0.12
	Pipe Piles	Nordlund Schmertmann's SPT97	N = 20 N = 20	36; 2B P(5)	0.65	0.50	0.39
		β-Method	N = 20	36; 2B P(5)	0.45	0.34	0.26
		Meyerhof	N = 20		0.37	0.28	0.21
Mixed Soils	H-Piles	α-Tomlinson/Nordlund/Thurman β-Method/Thurman	N = 22 N = 35	36; 2B; P(5) 36; 2B; P(5)	0.20	0.15	0.11
		α-API/Nordlund/Thurman	N = 37	36; 2B; P(5)	0.35	0.27	0.20
		Schmertmann's SPT-97	N = 41		0.64	0.51	0.41
	Concrete Piles	α-Tomlinson/Nordlund/Thurman α-API/Nordland/Thurman β-Method/Thurman	N = 34 N = 85 N = 85	36; 2B; P; Hara(5h) 36; 11.5B; Sch; T&P 36; 11.5B; Sch; T&P	0.43	0.33	0.26
		DRIVEN	N = 34		0.43	0.30	0.21
		Schmertmann's SPT97	N = 74		0.75	0.56	0.43
		Schmertmann's CPT	N = 32		0.52	0.43	0.35
	Pipe Piles	All Methods	---		0.25	0.20	0.15

NOTE: Uplift Capacity to be taken as 0.75 of Compression Capacity

Table 2: Summary of Correlations Used

Symbol	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
limit ϕ below	40^0				36^0			
contributed zone to tip resistance	2B	11.5B	2B	11.5B	2B	11.5B	2B	11.5B
ϕ, if from SPT, is correlated by	Peck, Hanson and Thornburn(1)		Schmertmann(1)		Peck, Hanson and Thornburn(1)		Schmertmann(1)	
S_u, if from SPT, is correlated by	Terzaghi and Peck							

Symbol	(1h)	(2h)	(3h)	(4h)	(5h)	(6h)	(7h)	(8h)
limit ϕ below	40^0				36^0			
contributed zone to tip resistance	2B	11.5B	2B	11.5B	2B	11.5B	2B	11.5B
ϕ, if from SPT, is correlated by	Peck, Hanson and Thornburn(1)		Schmertmann(1)		Peck, Hanson and Thornburn(1)		Schmertmann(1)	
S_u, if from SPT, is correlated by	Hara(1)							

(1) See Tables 3 and 4 for Correlation Details

Table 3: Correlations of soil properties from SPT

Properties	From SPT	Reference	
ϕ	Peck, Hanson and Thornburn: $\approx 54 - 27.6034 \exp(-0.014N')$	Figure 4.12	Kulhawy and Mayne, 1990
	Schmertmann ϕ' $\approx \tan^{-1} [N / (12.2 + 20.3\sigma')]^{0.34}$	Figure 4.13 and Equation 4.11	
S_u (bar)	Terzaghi and Peck (1967): 0.06 N	Equation 4.59	
	Hara 1974: $0.29 N^{0.72}$	Equation 4.60	
OCR for clay	Mayne and Kemper $\approx 0.5 N / \sigma'_o$ (σ'_o in bar)	Figures 3.9 and 3.18	
Dr	Gibbs and Holtz's Figures	Figures 2.13 and 2.14	

Table 4. Correlations of soil properties from CPT

Properties	From CPT	Reference	
ϕ	Robertson and Campanella: $\text{atan}(0.1+0.38*\text{Log}(q_c/\sigma'))$	Figure 4.14 and Equation 4.12	Kulhawy and Mayne, 1990
S_u (bar)	Theoretical: $(q_c - \sigma_o) / Nk$ q_c and σ_o in bars.	Equation 4.61	
OCR for clay	Mayne: $0.29\, q_c / \sigma'_o$ q_c and σ_o in bars.	Figure 3.10	
Dr	Jamiolkowski: $68 \log(q_{cn}) - 68$ $q_{cn} = \dfrac{q'_c}{\sqrt{P_a \sigma'_0}}$ (dimensionless) $q'_c = q_c / K_q$ $K_q = 0.9 + Dr/300$ q_c and σ'_o in bars.	Figure 2.24 and Equation 2.20	

DRIVEN PILES - STATIC ANALYSIS

RESULTS AND STATISTICS

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Table 1: Summary of the methods

Methods	Side resistance	Tip resistance	Parameters required	Constraints
α-Tomlinson (Tomlinson, 1980/1995)	$q_s = \alpha S_u$	$q_p = 9\ S_u$	S_u; D_b (bearing embedment)	+Bearing layer must be stiff cohesive + Number of soil layers ≤ 2
α-API (Reese et al., 1998)			S_u	
β in cohesive (AASHTO, 1996/2000)	$q_s = \beta\sigma'$		OCR	
λ (US Army Corps of Engineers, 1992)	$q_s = \lambda(\sigma'+2S_u)$		S_u	Only for cohesive soils
β in cohesionless (Bowles, 1996)	$\beta\sigma'$		D_r	
Nordlund and Thurman (Hannigan et al., 1995)	$q_s = K_\delta C_F \sigma' \frac{\sin(\delta+\varpi)}{\cos\varpi}$	$q_p = \alpha_t N'_q \sigma'$	ϕ	
Meyerhof SPT (Meyerhof, 1976/1981)	$q_s = k\ N$	$q_p = 0.4D/BN'$	N	+ For cohesionless soils + SPT data
Schmertmann SPT (Lai and Graham, 1995)	q_s = function(N)	q_p = fn(N)	N	SPTdata
Schmertmann CPT (McVay and Townsend, 1989)	q_s = function(f_s)	q_p = fn(q_c)	q_c, f_s	CPT data

Table 2: Correlations of soil properties from SPT

Properties	From SPT	Reference	
ϕ	Peck, Hanson and Thornburn: $\approx 54 - 27.6034 \exp(-0.014N')$	Figure 4.12	
	Schmertmann φ' $\approx \tan^{-1} [N / (12.2 + 20.3\ \sigma')]^{0.34}$	Figure 4.13 and Equation 4.11	
S_u (bar)	Terzaghi and Peck (1967): 0.06 N	Equation 4.59	
	Hara 1974: $0.29\ N^{0.72}$	Equation 4.60	Kulhawy and Mayne, 1990
OCR for clay	Mayne and Kemper $\approx 0.5\ N / \sigma'_o$ (σ'_o in bar)	Figures 3.9 and 3.18	
Dr	Gibbs and Holtz's Figures	Figures 2.13 and 2.14	

Table 3. Correlations of soil properties from CPT

Properties	From CPT	Reference	
ϕ	Robertson and Campanella: $\mathrm{atan}(0.1+0.38{*}\mathrm{Log}(q_c/\sigma'))$	Figure 4.14 and Equation 4.12	
S_u (bar)	Theoretical: $(q_c - \sigma_o) / Nk$ q_c and σ_o in bars.	Equation 4.61	
OCR for clay	Mayne: $0.29\ q_c / \sigma'_o$ q_c and σ_o in bars.	Figure 3.10	Kulhawy and Mayne, 1990
Dr	Jamiolkowski: $68 \log(q_{cn}) - 68$ $q_{cn} = \dfrac{q'_c}{\sqrt{P_a \sigma'_0}}$ (dimensionless) $q'_c = q_c / K_q$ $K_q = 0.9 + Dr/300$ q_c and σ'_o in bars.	Figure 2.24 and Equation 2.20	

THE LRFD CALIBRATION RESULTS AND ANALYSIS

1.1. Capacity--Results for Concrete Piles

Table 4: Symbols represented the soil parameters used.

Symbol	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
limit ϕ below	40^0				36^0			
contributed zone to tip resistance	2B	11.5B	2B	11.5B	2B	11.5B	2B	11.5B
ϕ, if from SPT, is correlated by	Peck, Hanson and Thornburn		Schmertmann		Peck, Hanson and Thornburn		Schmertmann	
S_u, if from SPT, is correlated by	Terzaghi and Peck							

Symbol	(1h)	(2h)	(3h)	(4h)	(5h)	(6h)	(7h)	(8h)
limit ϕ below	40^0				36^0			
contributed zone to tip resistance	2B	11.5B	2B	11.5B	2B	11.5B	2B	11.5B
ϕ, if from SPT, is correlated by	Peck, Hanson and Thornburn		Schmertmann		Peck, Hanson and Thornburn		Schmertmann	
S_u, if from SPT, is correlated by	Hara							

1.1.1. Figures of Capacity Prediction--Concrete Piles in Cohesionless Soils (1)

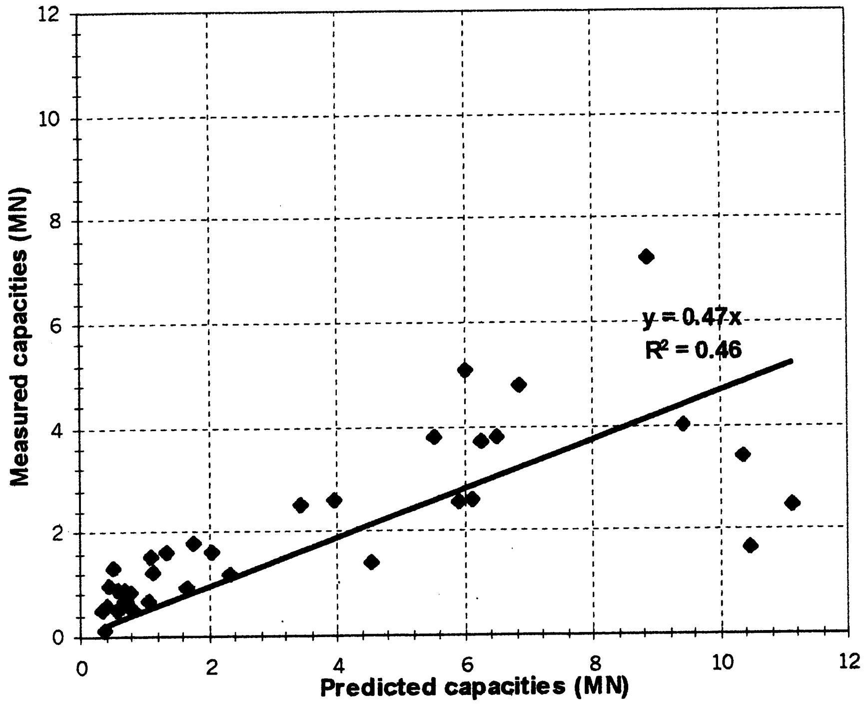

Figure 1. Concrete piles in Cohesionless Soils: β method

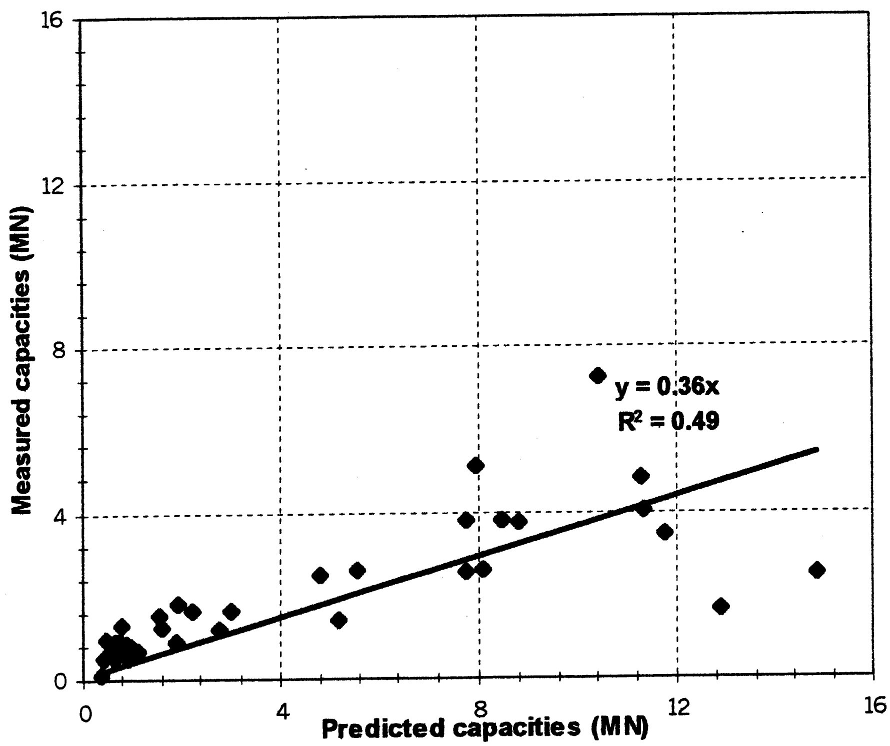

Figure 2: Concrete piles in Cohesionless Soils: Nordlund method

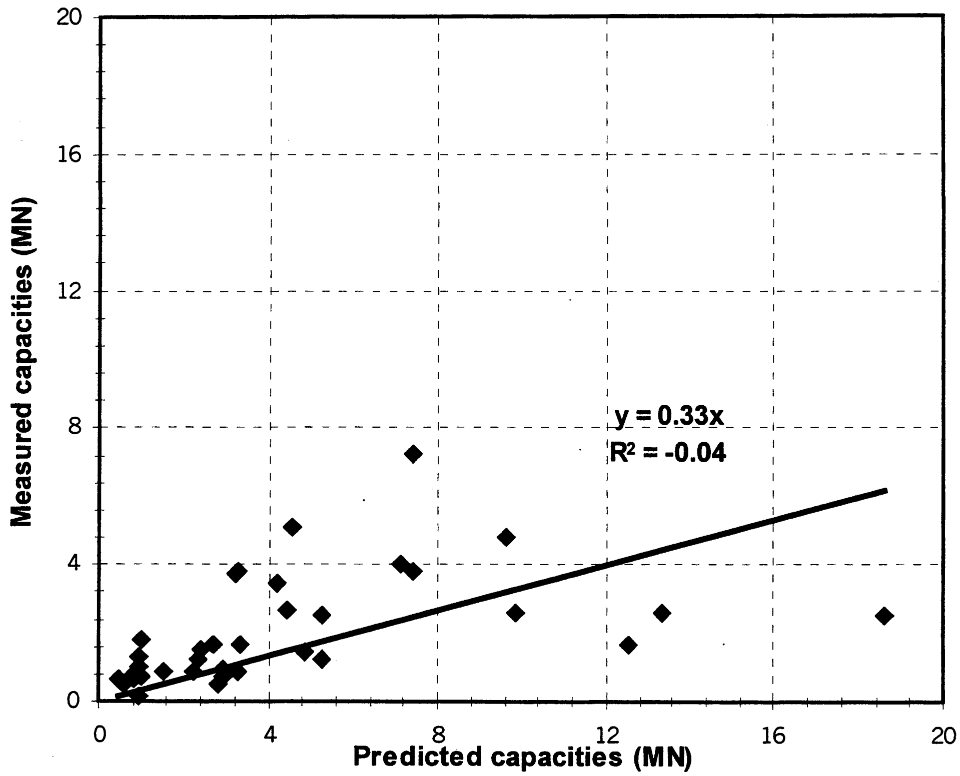

Figure 3: Concrete piles in Cohesionless Soils: Meyerhof method

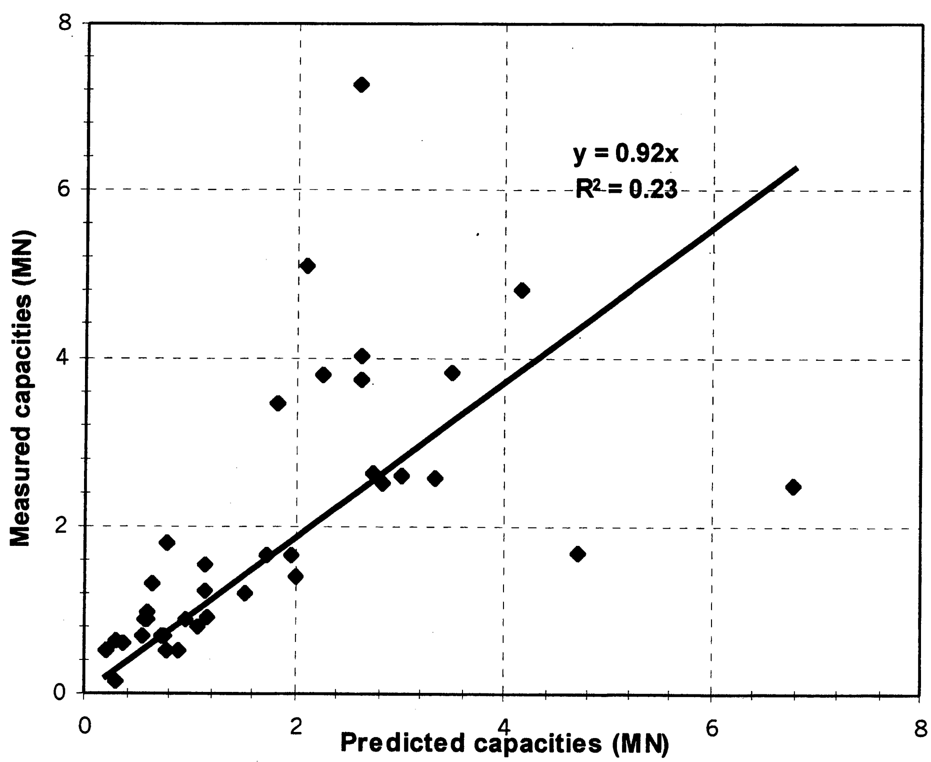

Figure 4: Concrete piles in Cohesionless Soils: Schmertmann SPT mobilized

1.1.2. Figures of Capacity Prediction--Concrete Piles in Cohesive Soils (1)

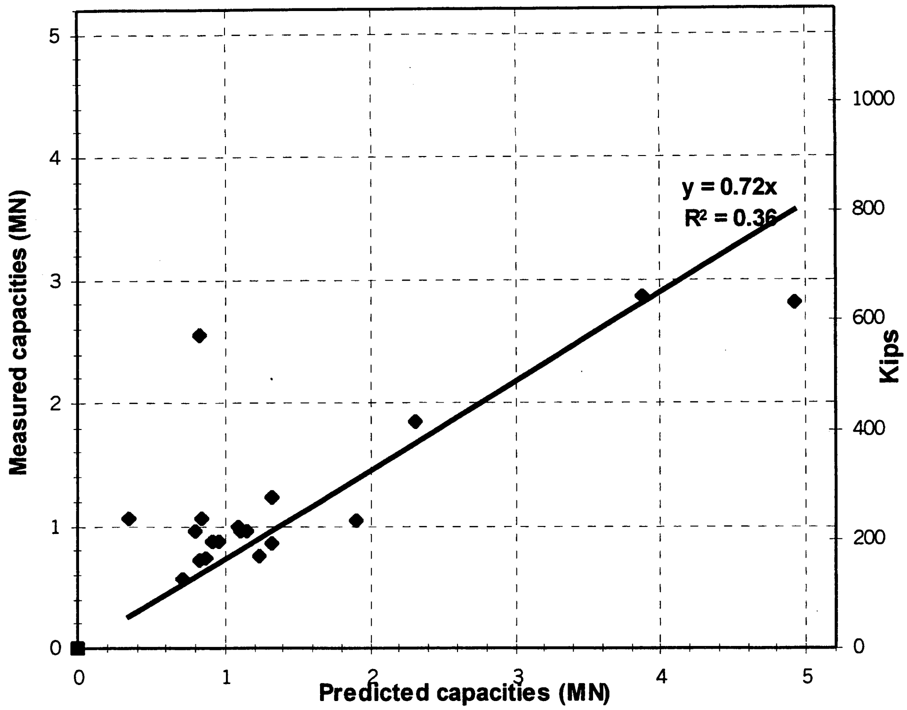

Figure 5: Concrete piles in Cohesive Soils: α-API method

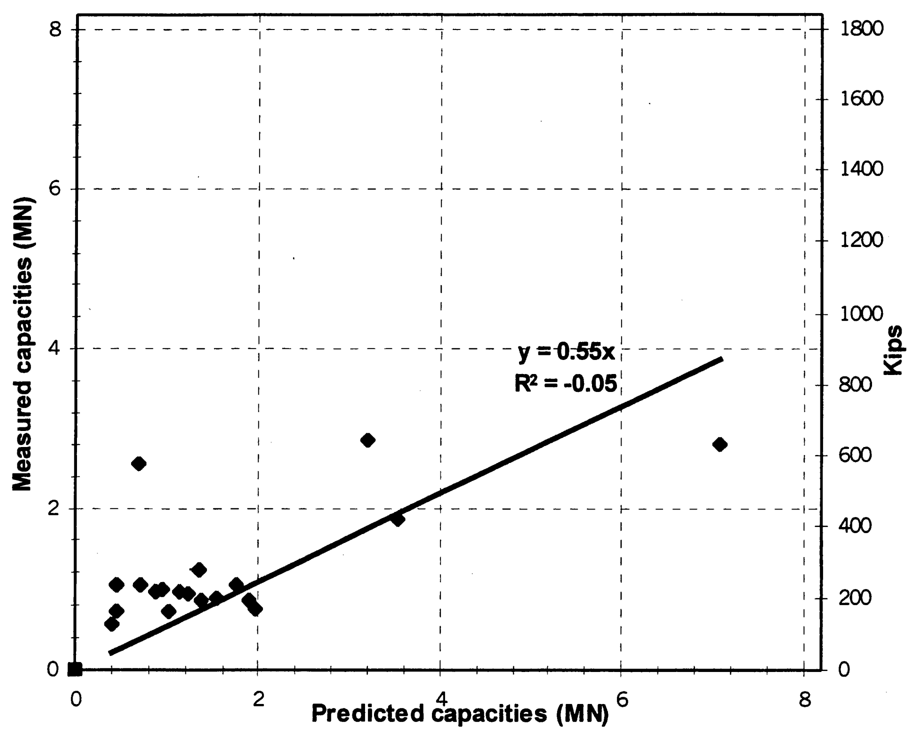

Figure 6: Concrete piles in Cohesive Soils: α-Tomlinson method

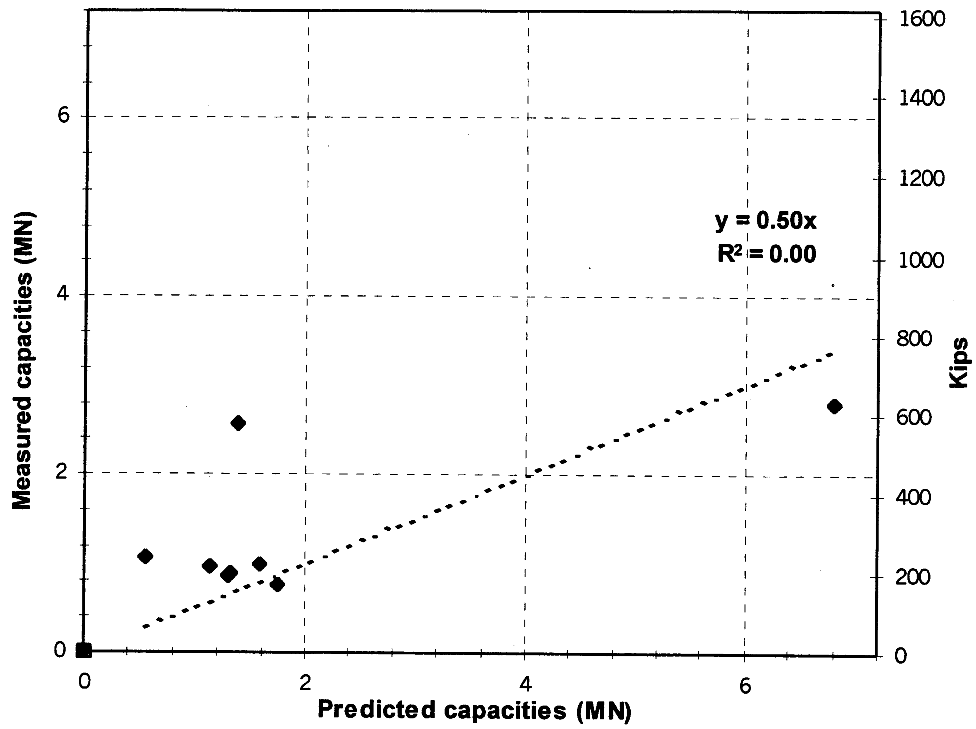

Figure 7: Concrete piles in Cohesive Soils: β method

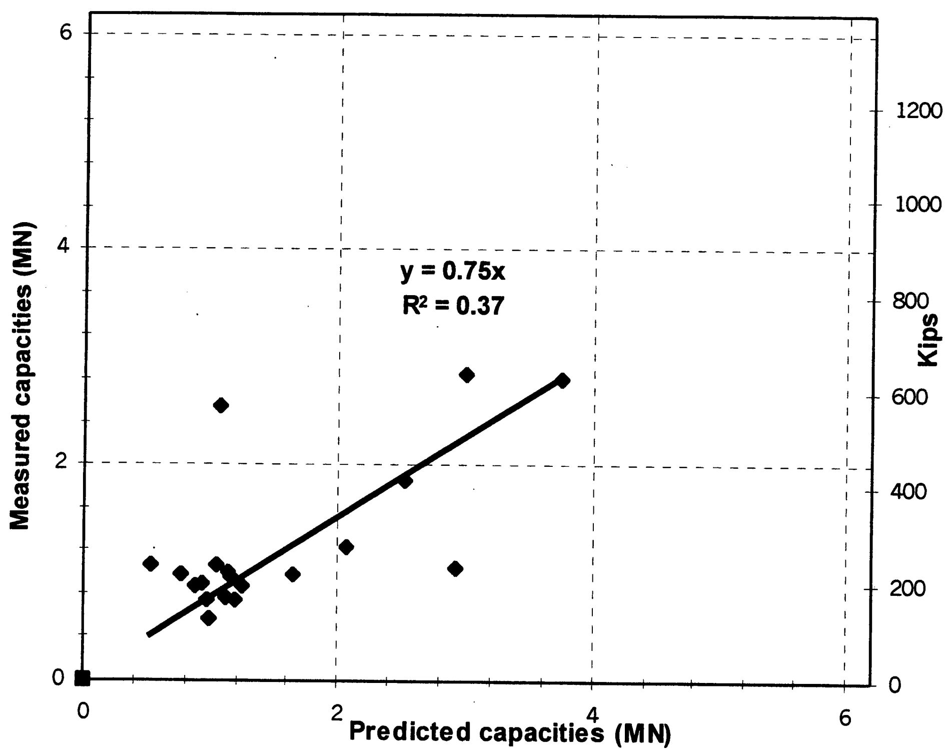

Figure 8: Concrete piles in Cohesive Soils: λ method

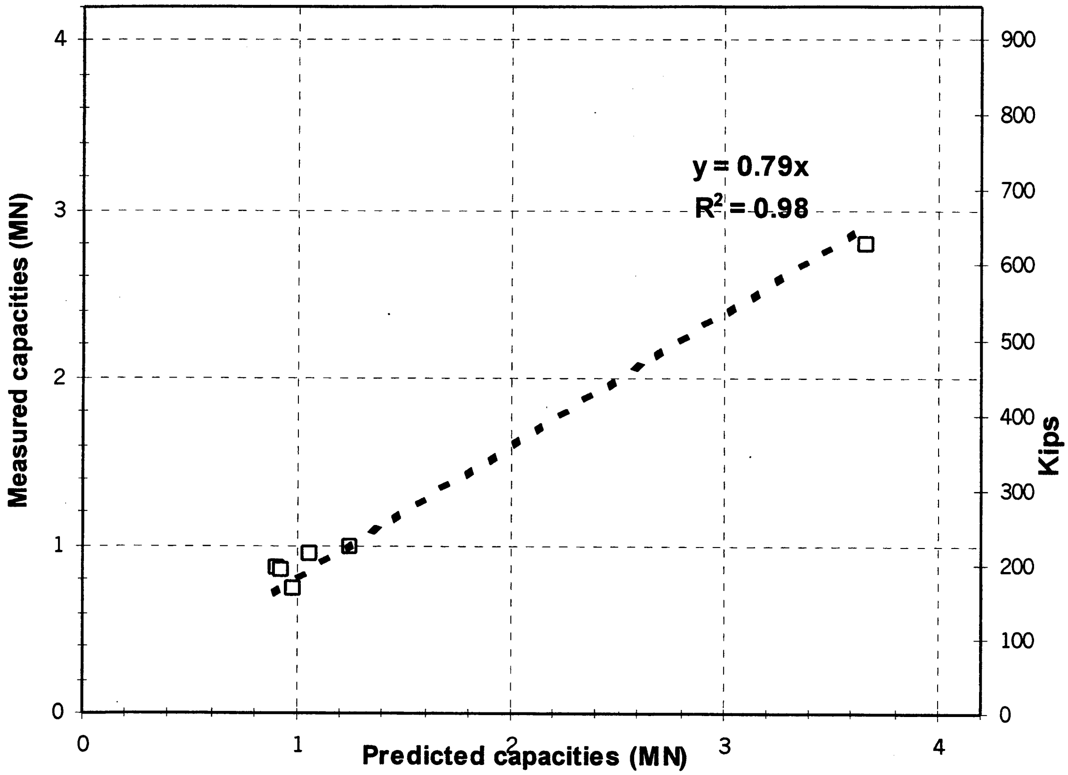

Figure 9: Concrete piles in Cohesive Soils: Schmertmann CPT method

1.1.3. Figures of Capacity Prediction--Concrete Piles in Mixed Soils (1)

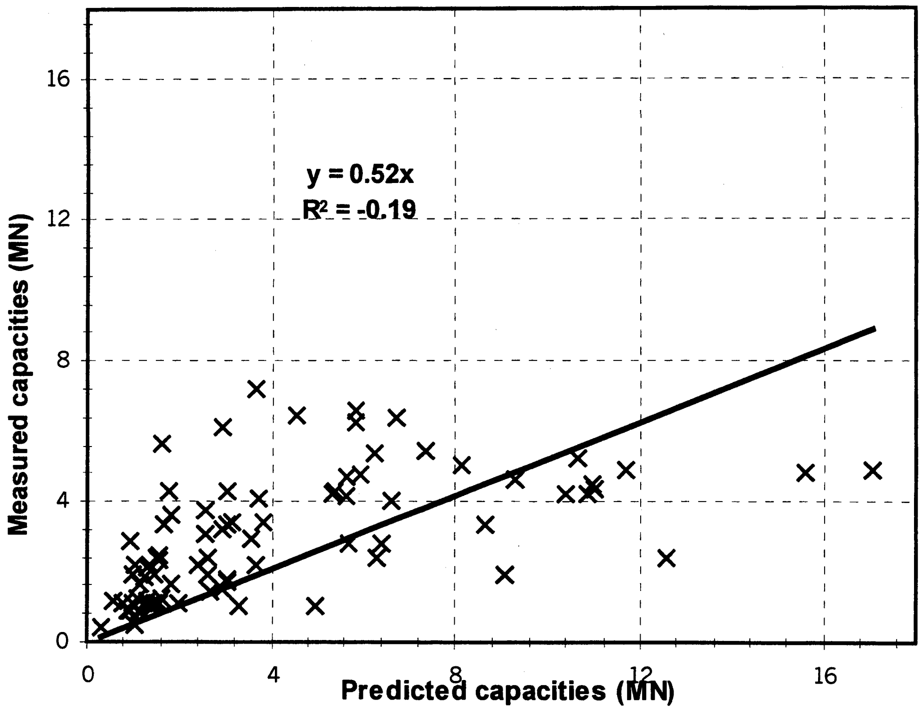

Figure 10: Concrete piles in Mixed Soil: α-API, Nordlund, Thurman methods

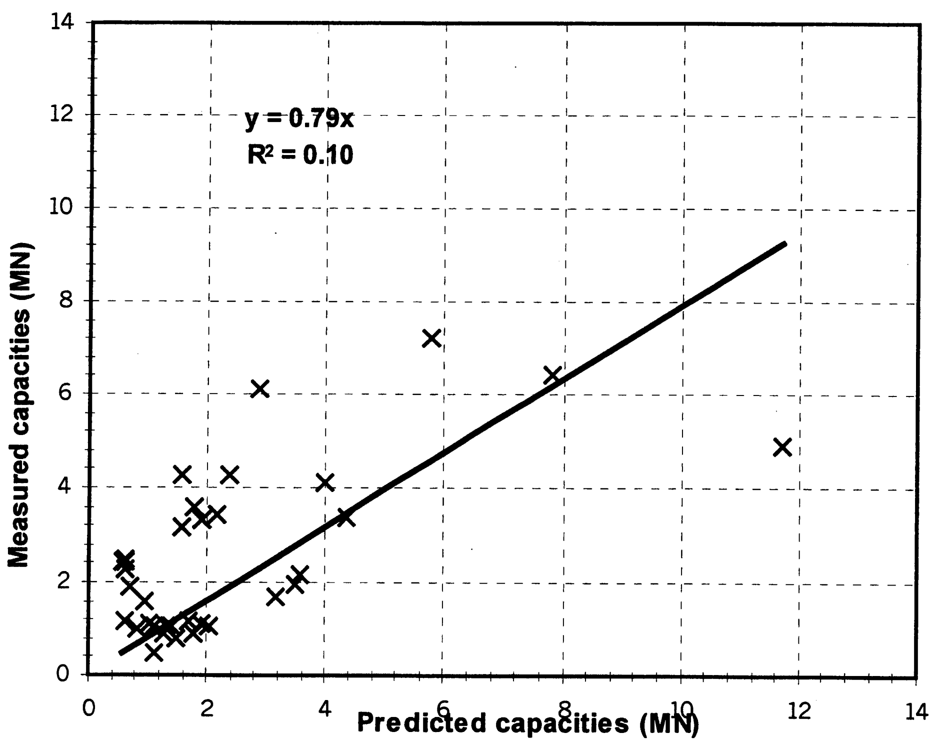

Figure 11: Concrete piles in Mixed Soil: α-Tomlinson, Nordlund, Thurman methods

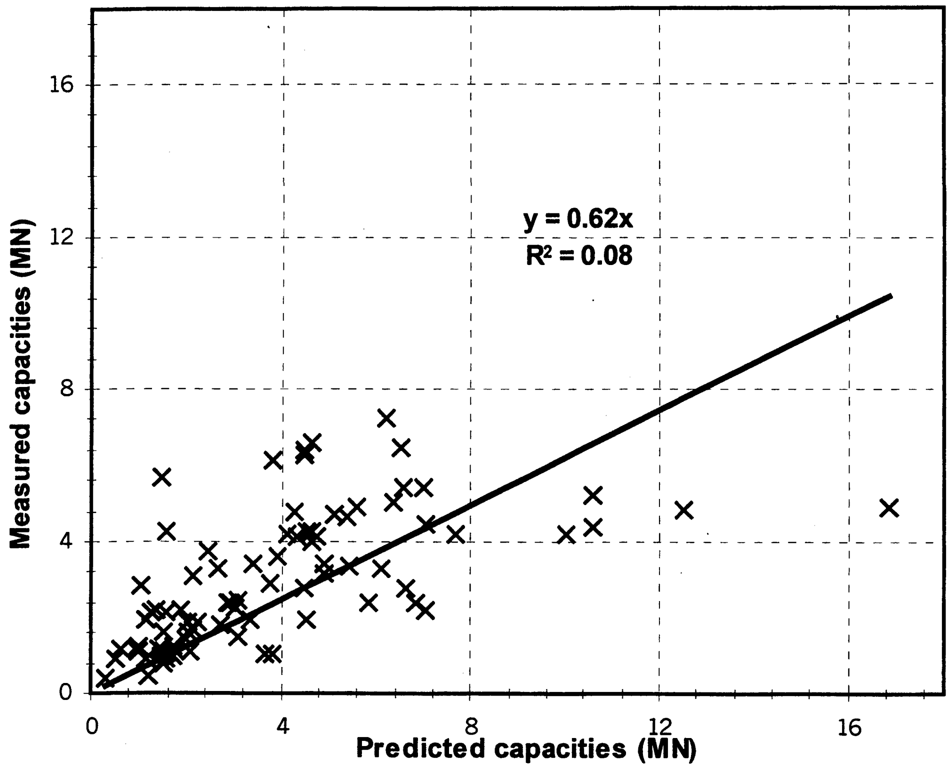

Figure 12: Concrete piles in Mixed Soil: β method

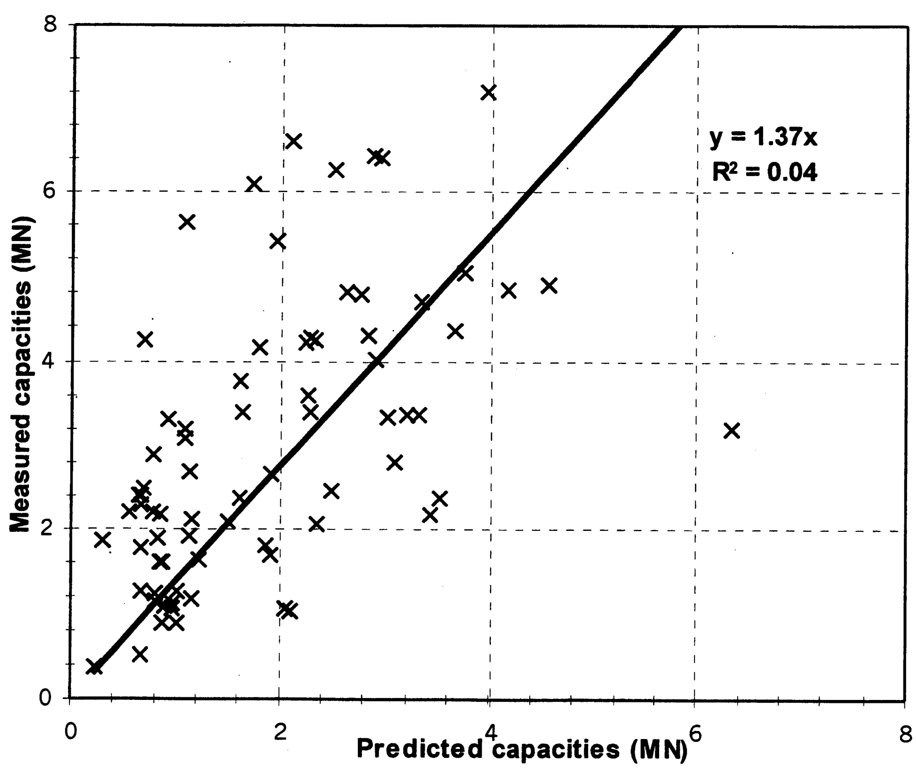

Figure 13: Concrete piles in Mixed Soil: Schmertmann SPT mobilized

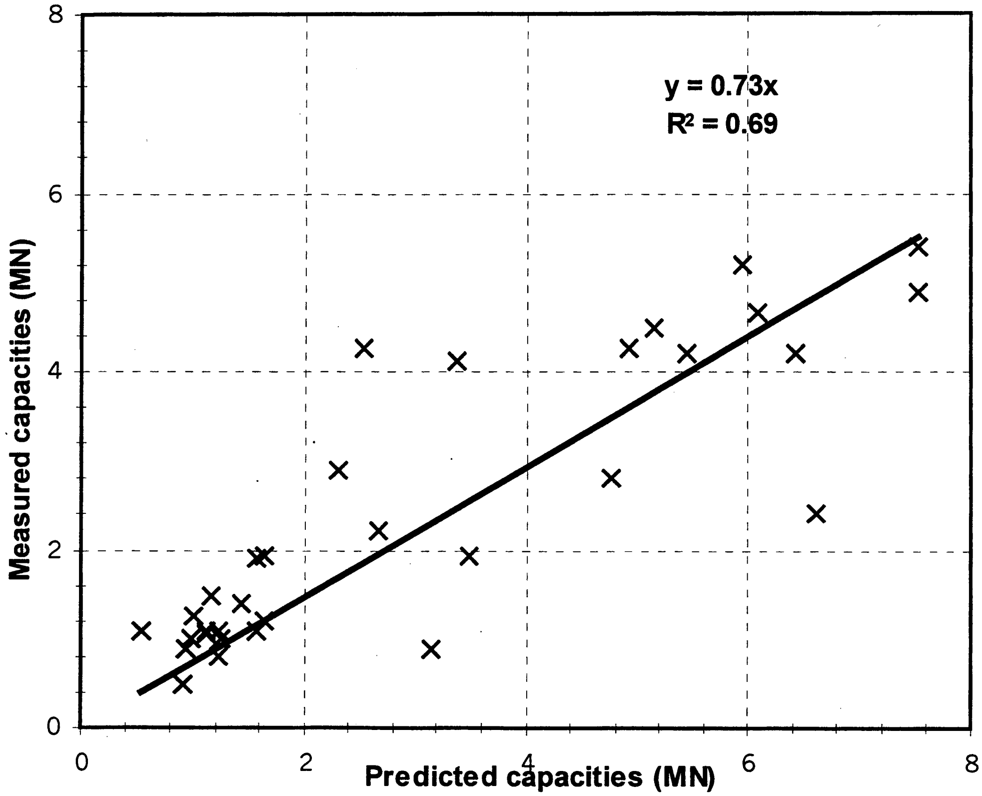

Figure 14: Concrete piles in Mixed Soil: Schmertmann CPT method

1.1.4. Histogram--Concrete Piles in Cohesionless Soils (5)

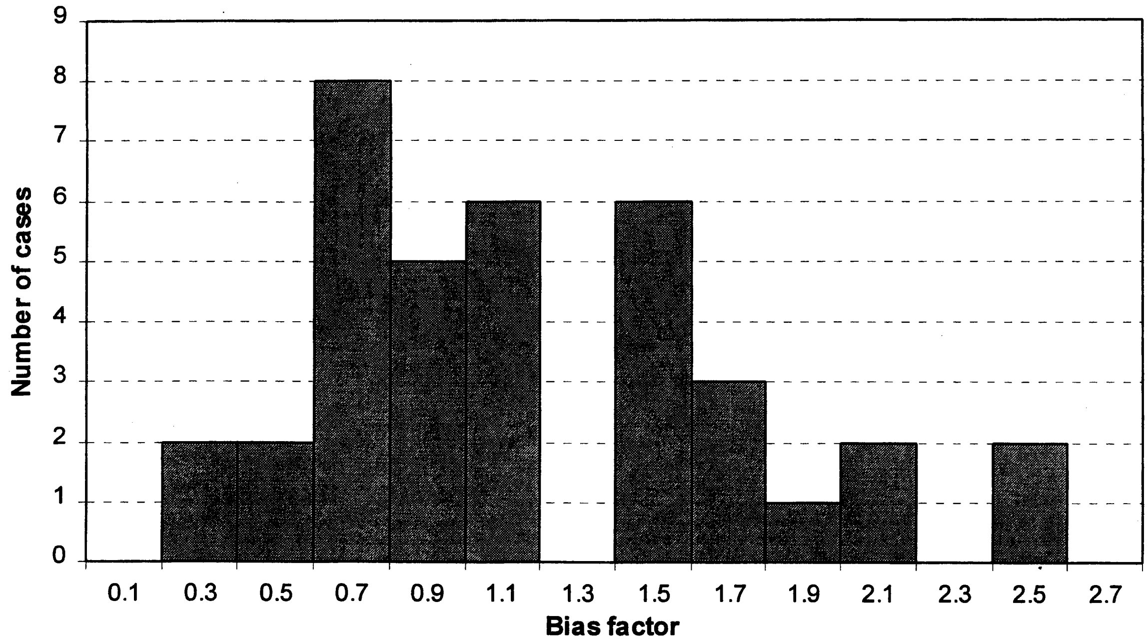

Figure 15: Histogram--β method (5): $\lambda_R = 1.17$

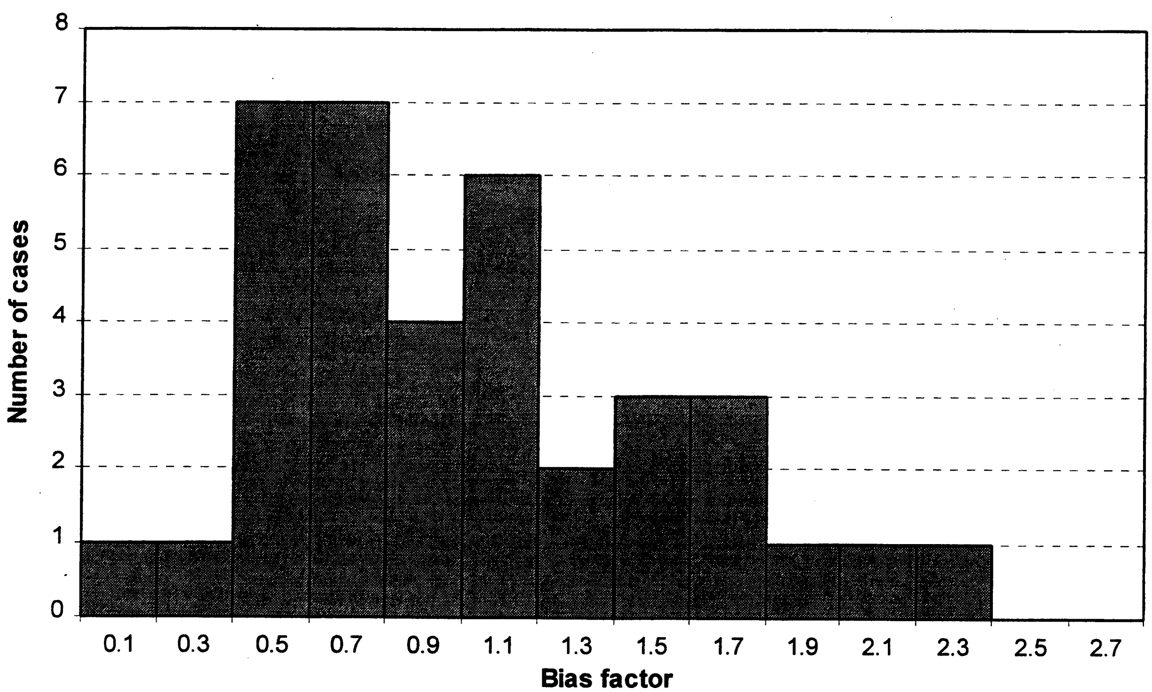

Figure 16: Histogram--Nordlund method (5): $\lambda_R = 1.00$

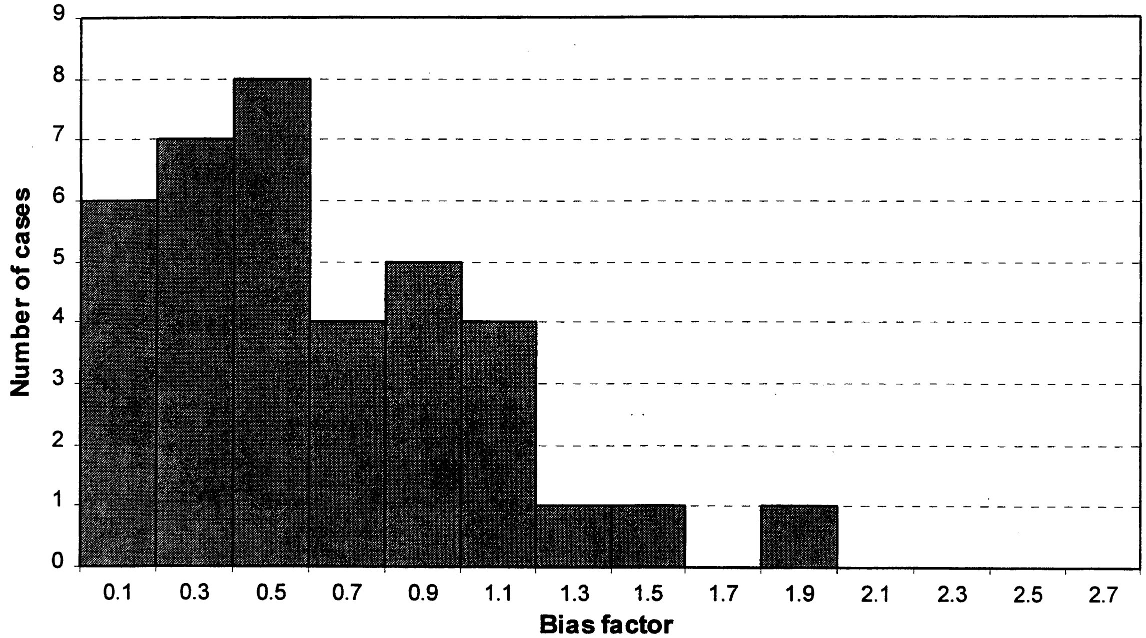

Figure 17: Histogram--Meyerhof method: $\lambda_R = 0.64$

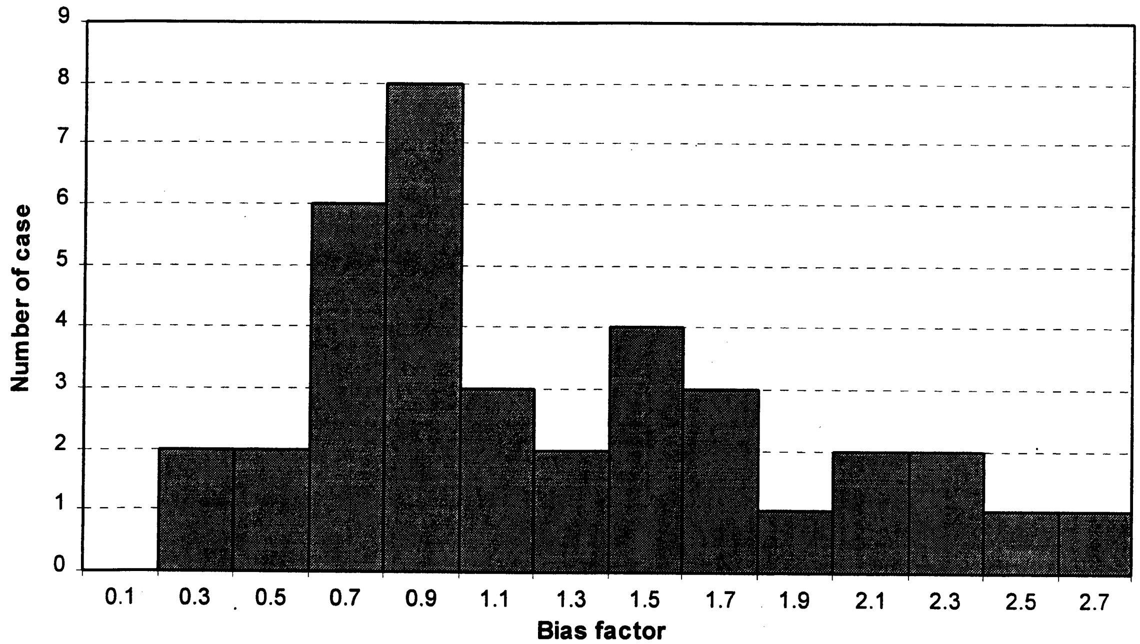

Figure 18: Histogram--Schmertmann SPT mobilized method: $\lambda_R = 1.25$

1.1.5. Histogram--Concrete Piles in Cohesive Soils (5h)

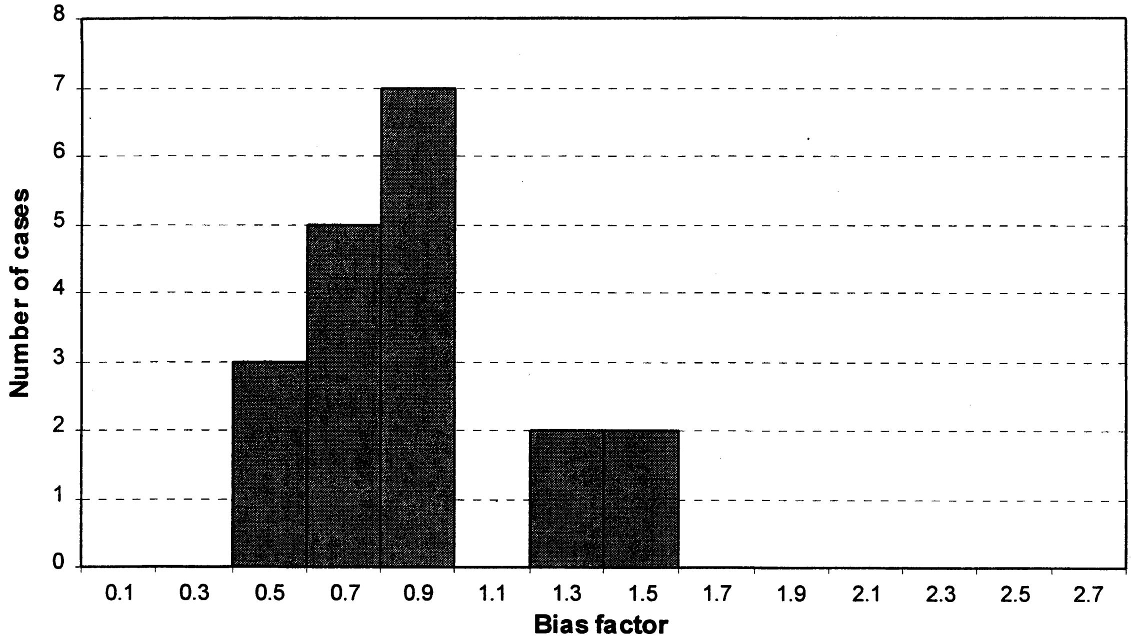

Figure 19: Histogram--α-API method (5h): $\lambda_R = 0.89$

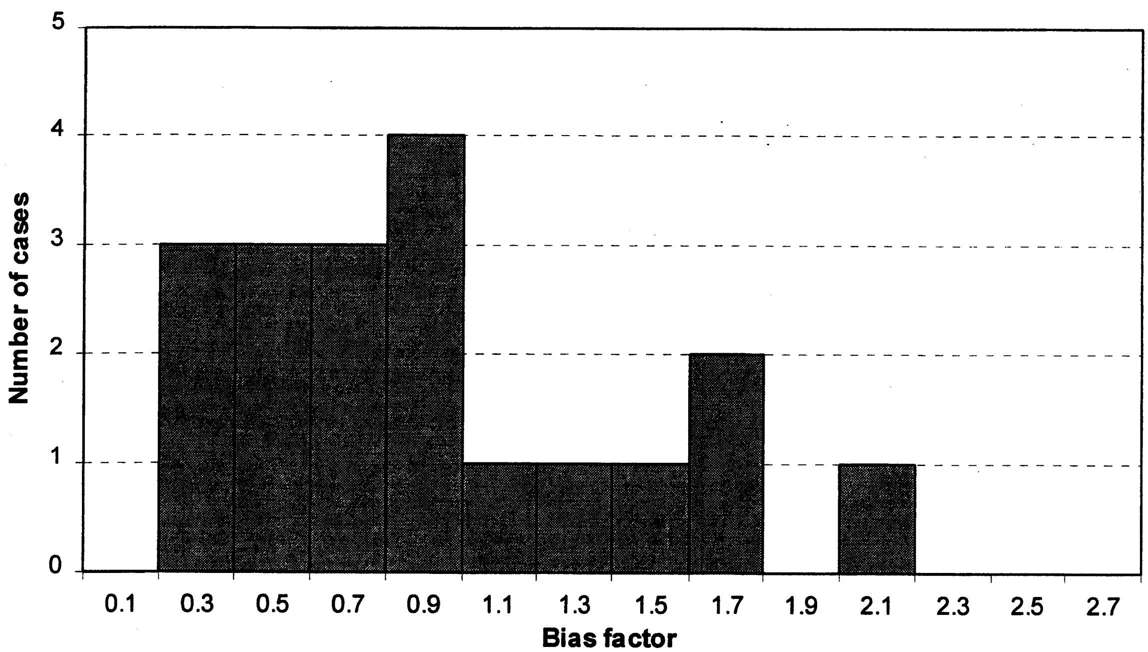

Figure 20: Histogram--α-Tomlinson method (5h) : $\lambda_R = 0.94$

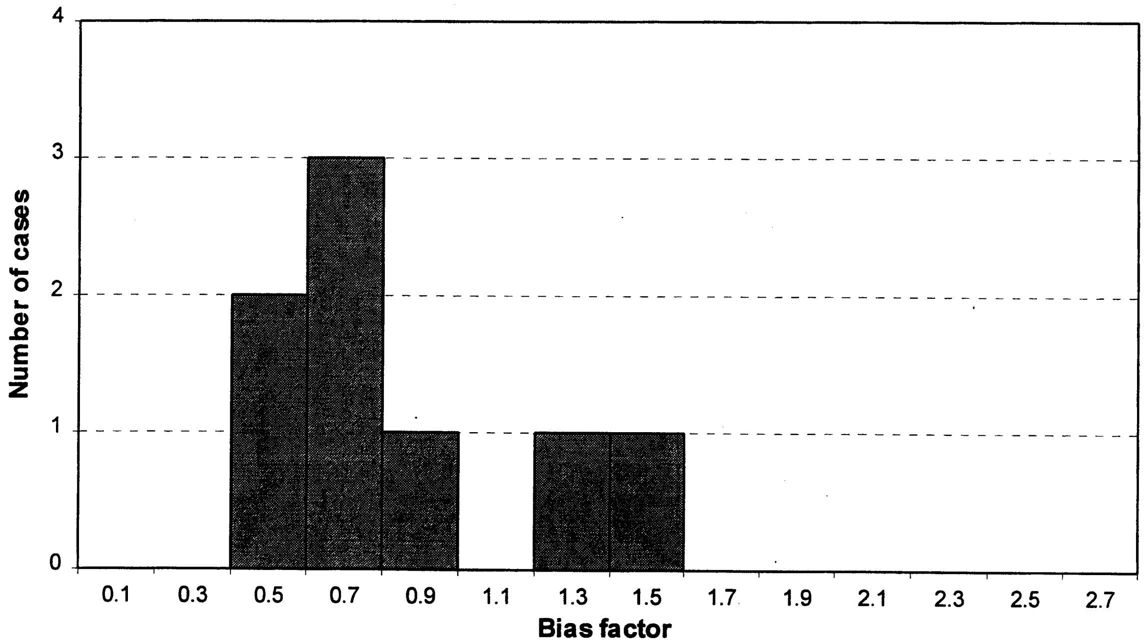

Figure 21: Histogram--β method (5h) : $\lambda_R = 0.81$

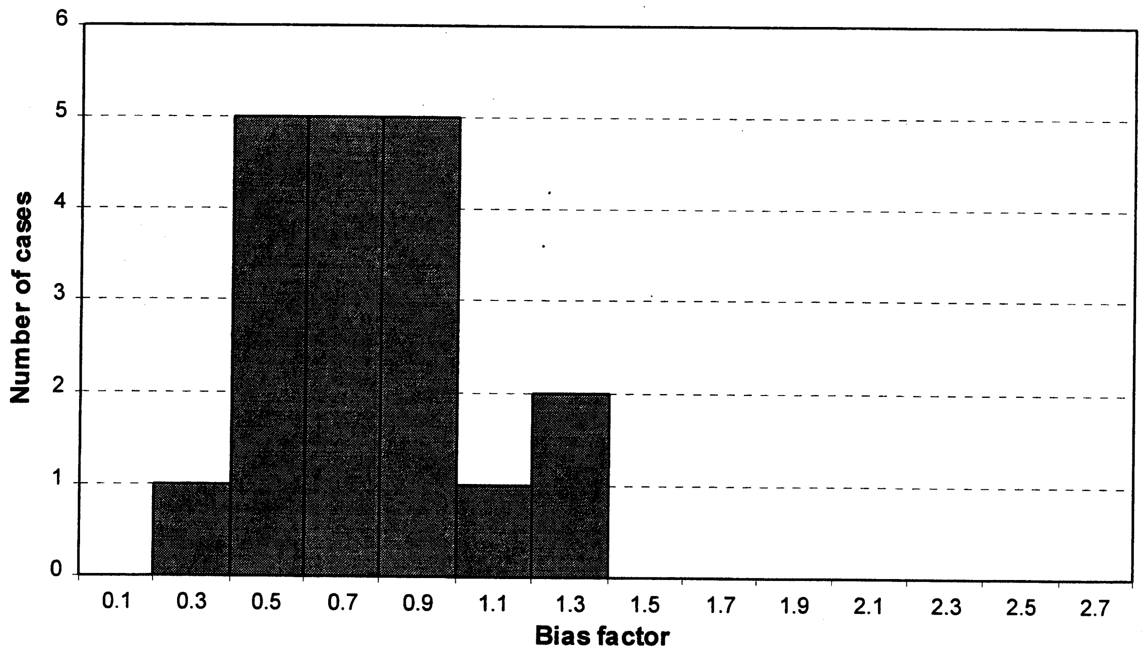

Figure 22: Histogram--λ method (5h) : $\lambda_R = 0.79$

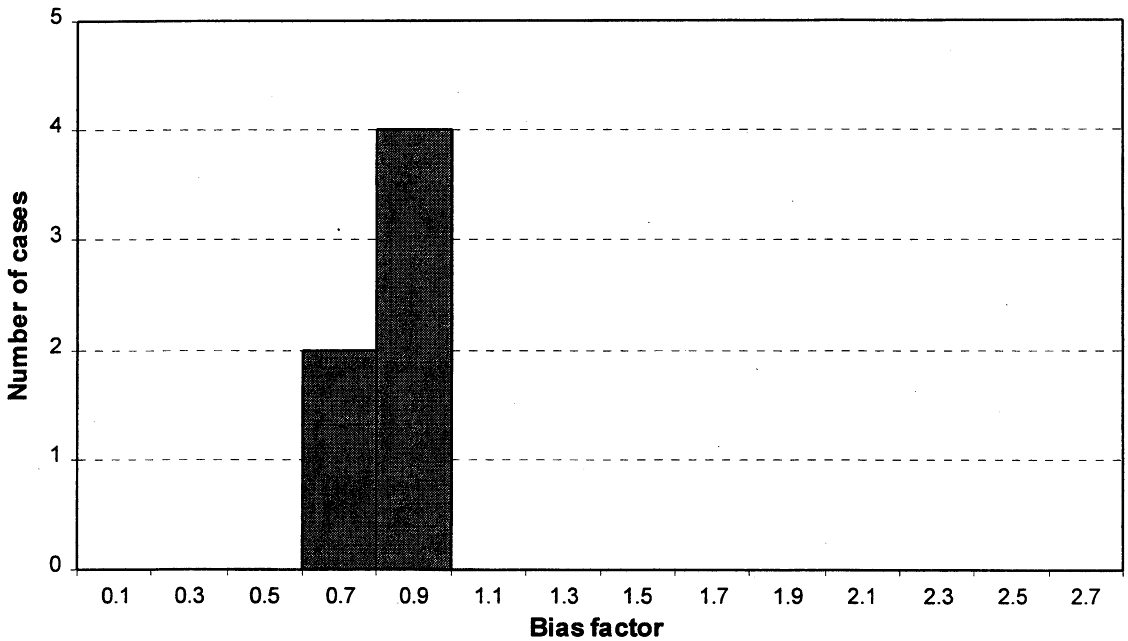

Figure 23: Histogram--Schmertmann CPT method: $\lambda_R = 0.86$

1.1.6. Histogram--Concrete Piles in Mixed Soils (5h)

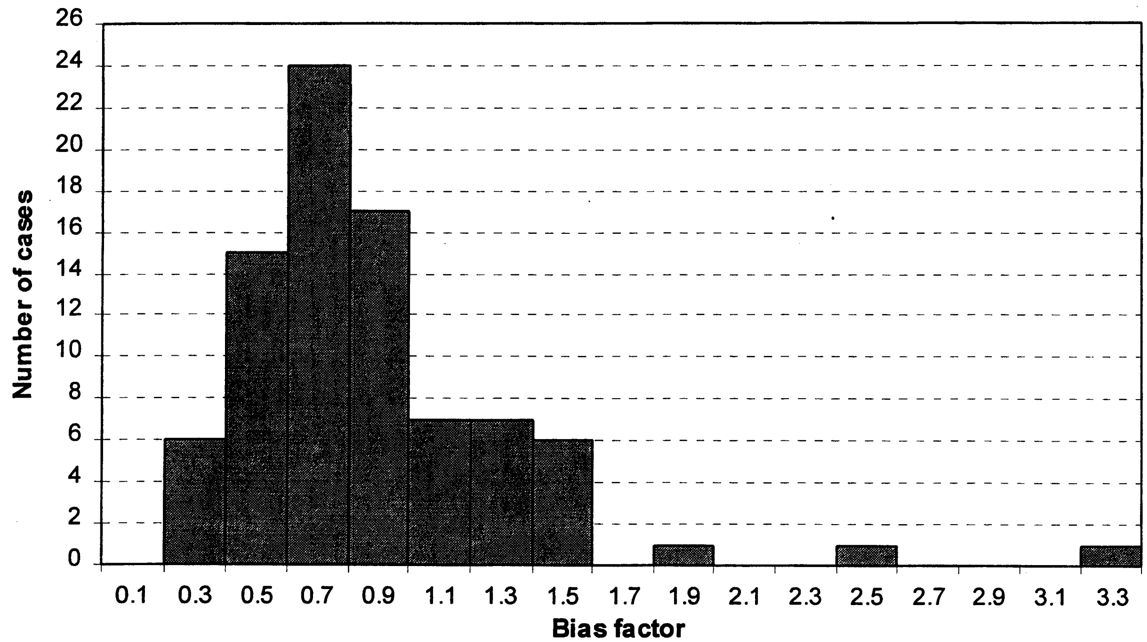

Figure 24: Histogram--α-API/ Nordlund method (5h) : $\lambda_R = 0.88$

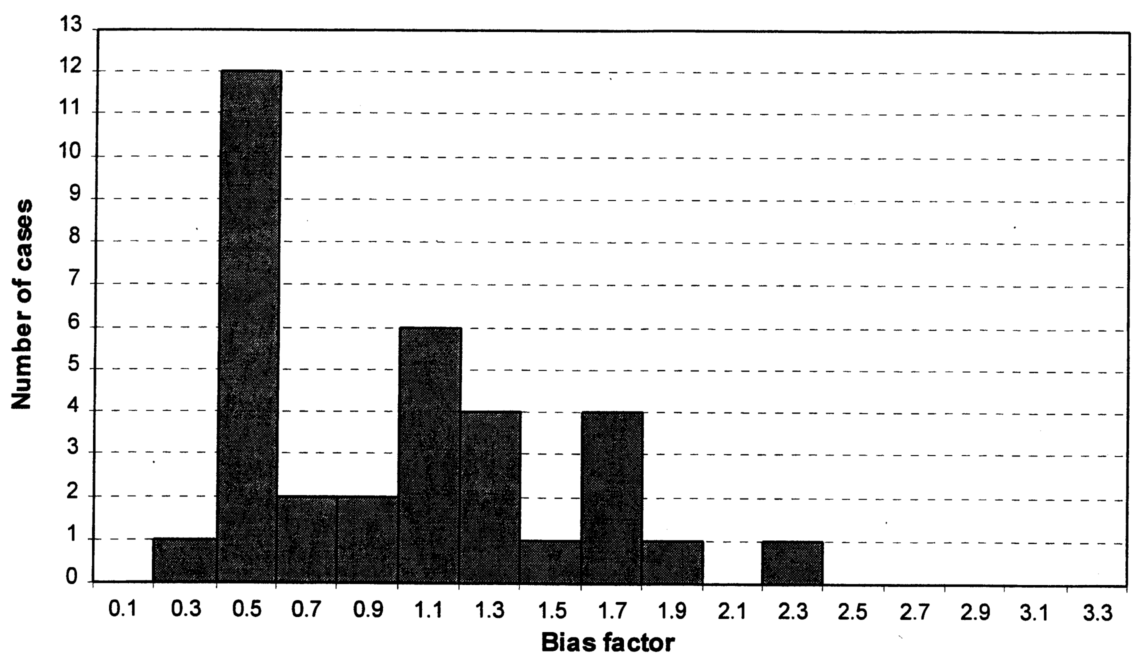

Figure 25: Histogram--α-Tomlinson/ Nordlund method (5h) : $\lambda_R = 1.00$

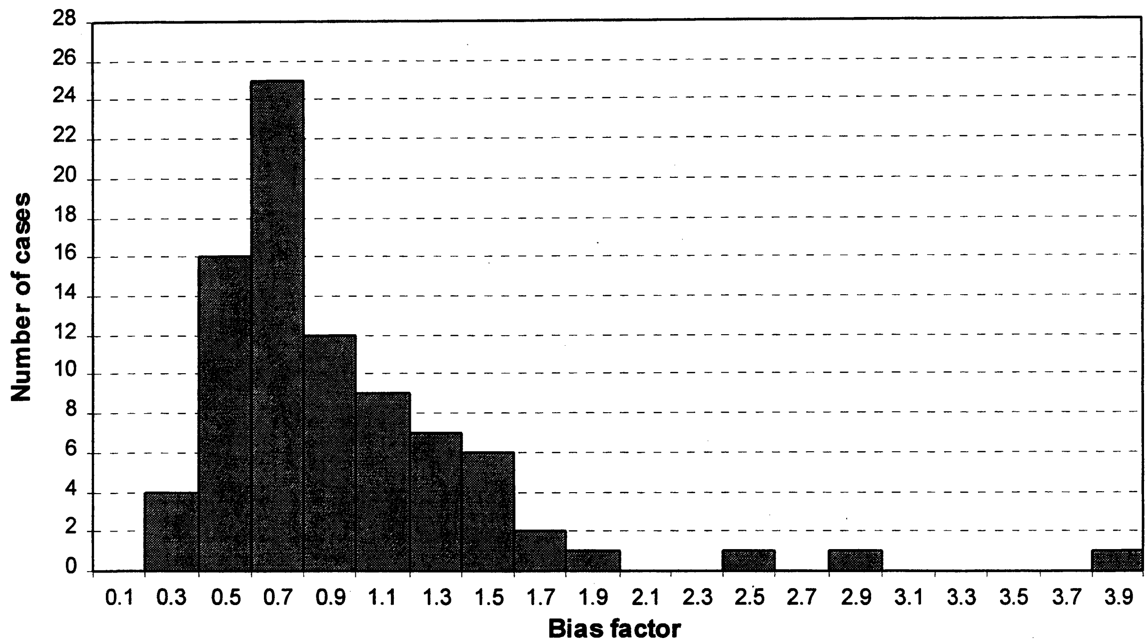

Figure 26: Histogram--β /Thurman method (5h) : $\lambda_R = 0.93$

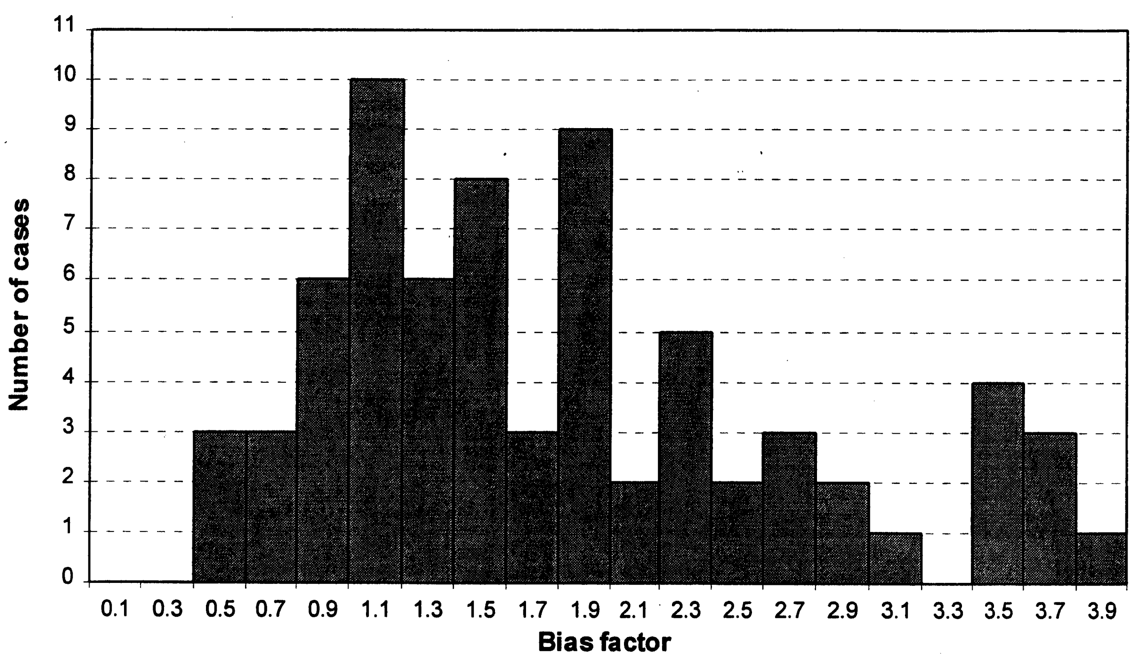

Figure 27: Histogram--Schmertmann SPT mobilized method

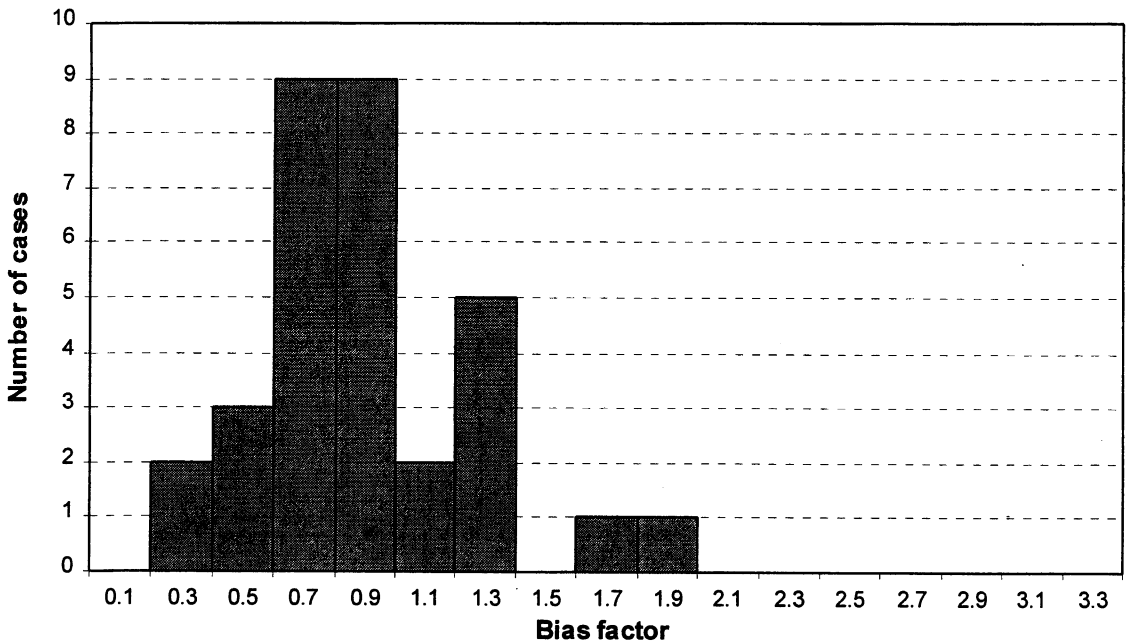

Figure 28: Histogram--Schmertmann CPT method

1.2. Capacity--Results for Pipe Piles

1.2.1. Figures of Capacity Prediction--Pipe Piles in Cohesionless Soils (1)

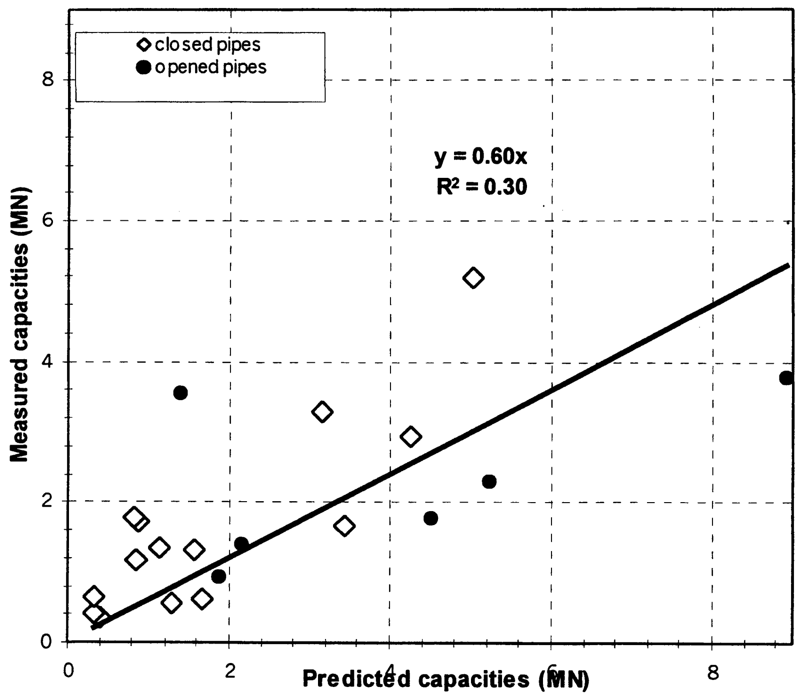

Figure 29: Pipe piles in Cohesionless soil--β method

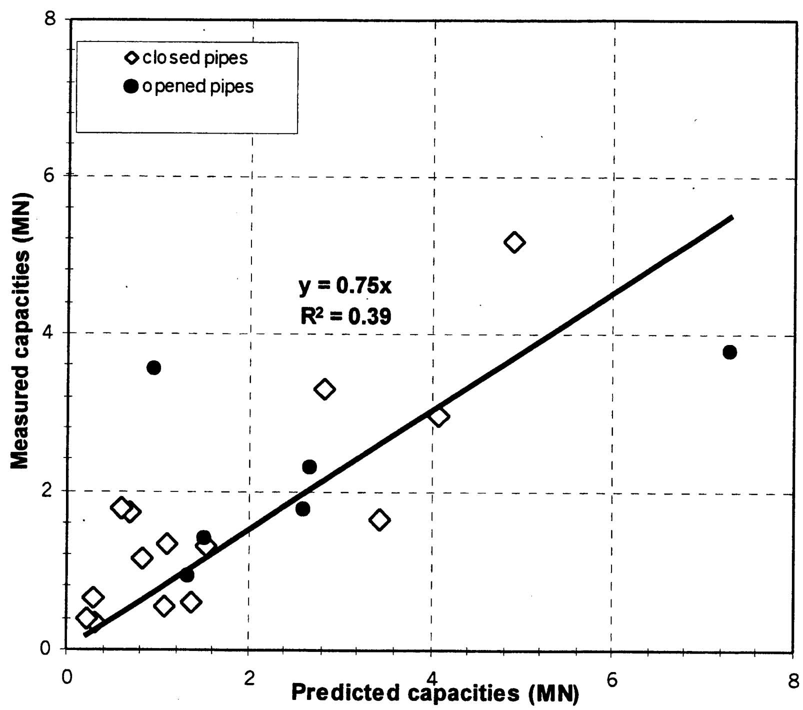

Figure 30: Pipe piles in Cohesionless soil--Nordlund method

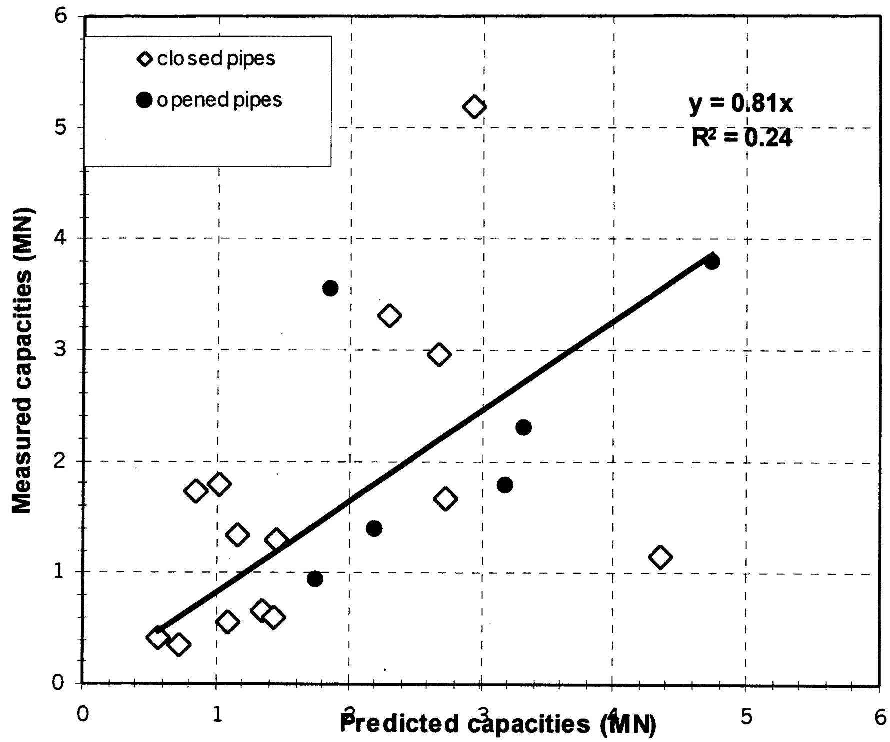

Figure 31: Pipe piles in Cohesionless soil--Meyerhof method

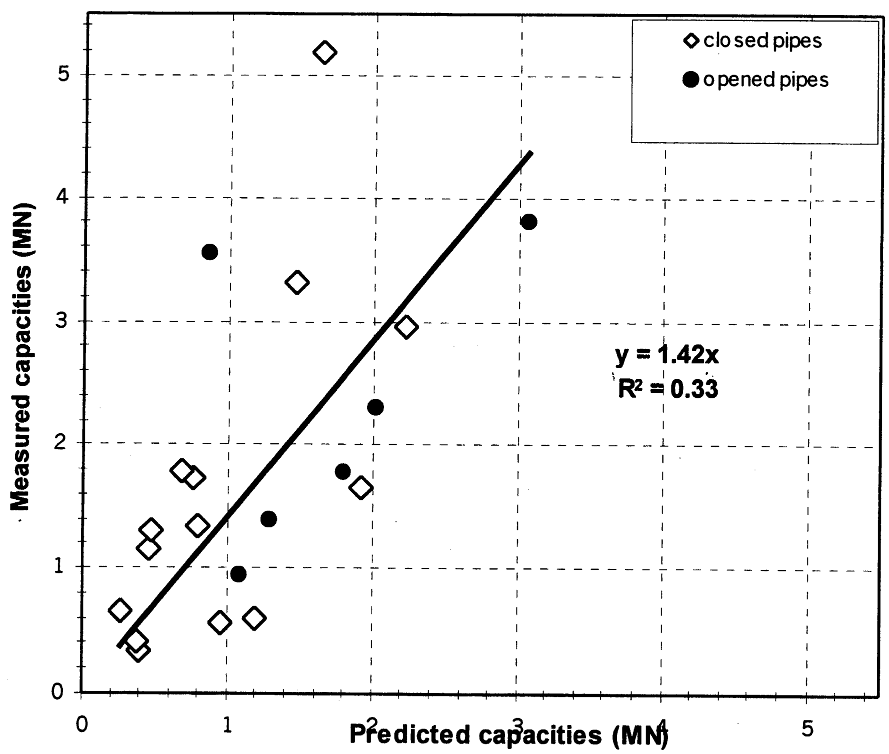

Figure 32: Pipe piles in Cohesionless soil--Schmertmann SPT method

1.2.2. Figures of Capacity Prediction--Pipe Piles in cohesive soil (1)

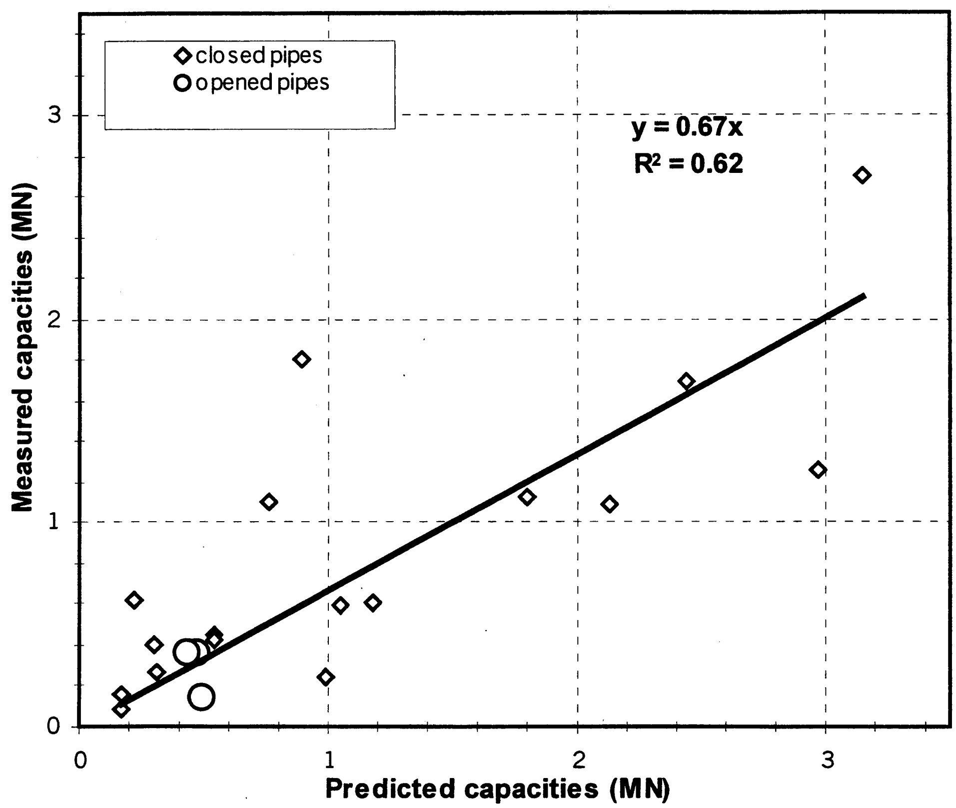

Figure 33: Pipe piles in cohesive soil--α-API method

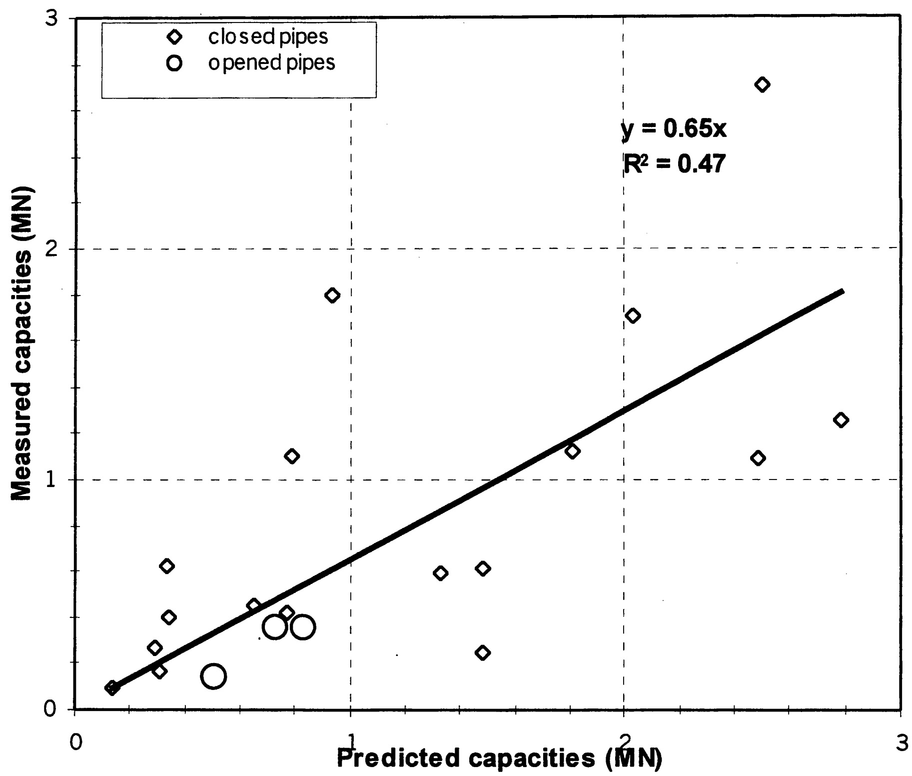

Figure 34: Pipe piles in cohesive soil--α-Tomlinson 1980 method

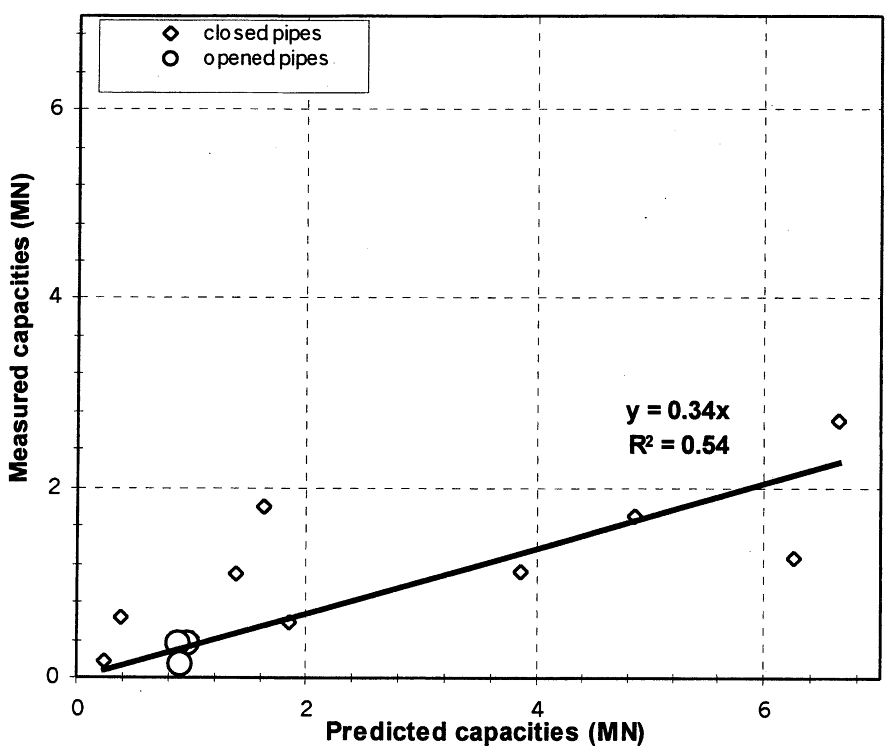

Figure 35: Pipe piles in cohesive soil--β method

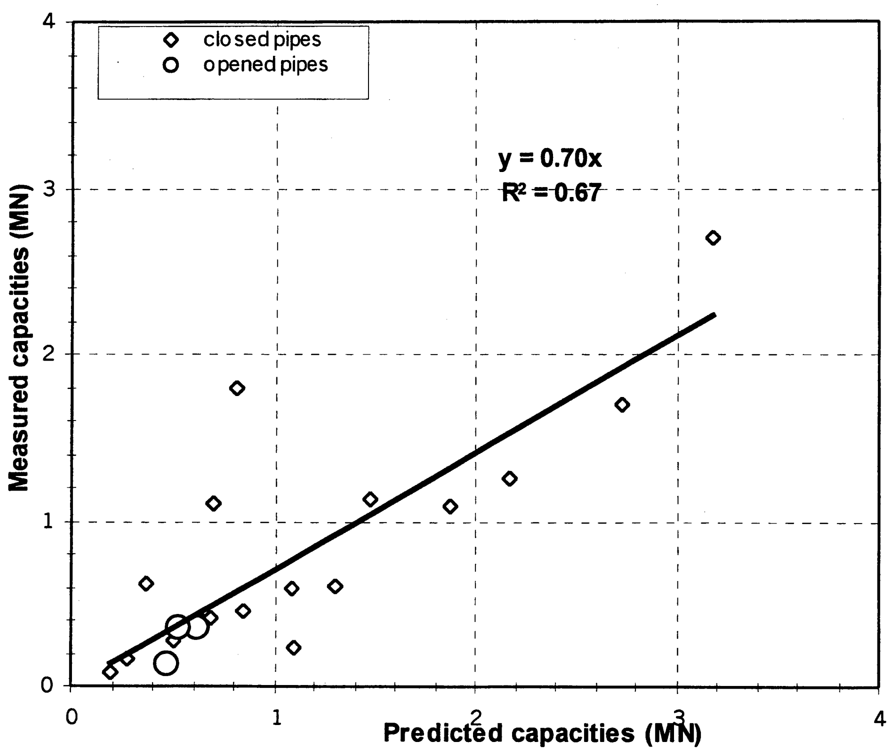

Figure 36: Pipe piles in cohesive soil--λ method

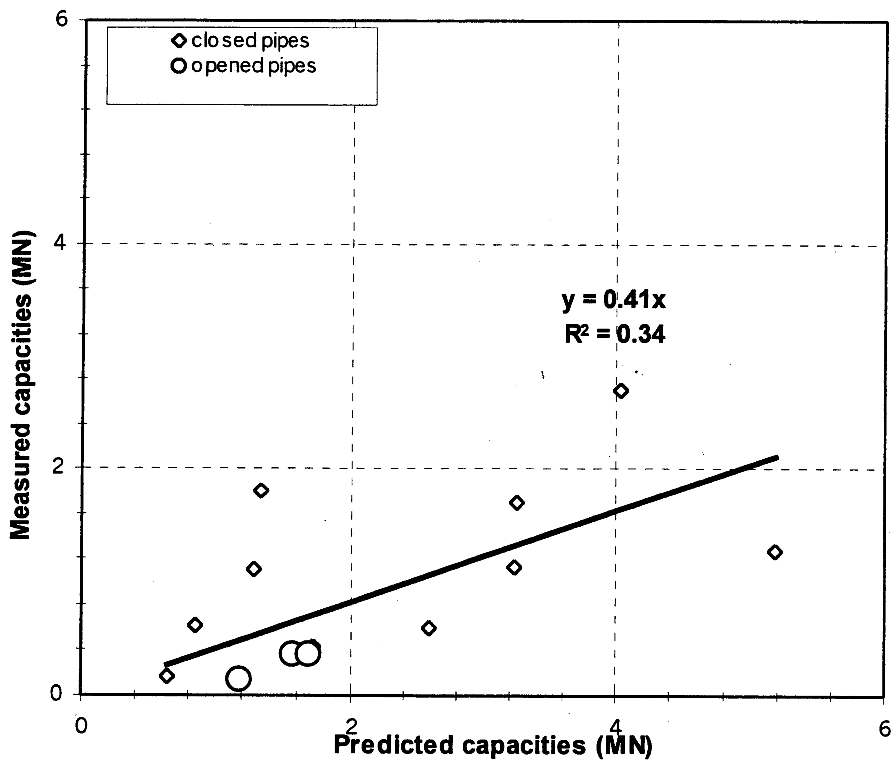

Figure 37: Pipe piles in cohesive soil--Schmertmann SPT method

1.2.3. Figures of Capacity Prediction--Pipe Piles in mixed soils (1):

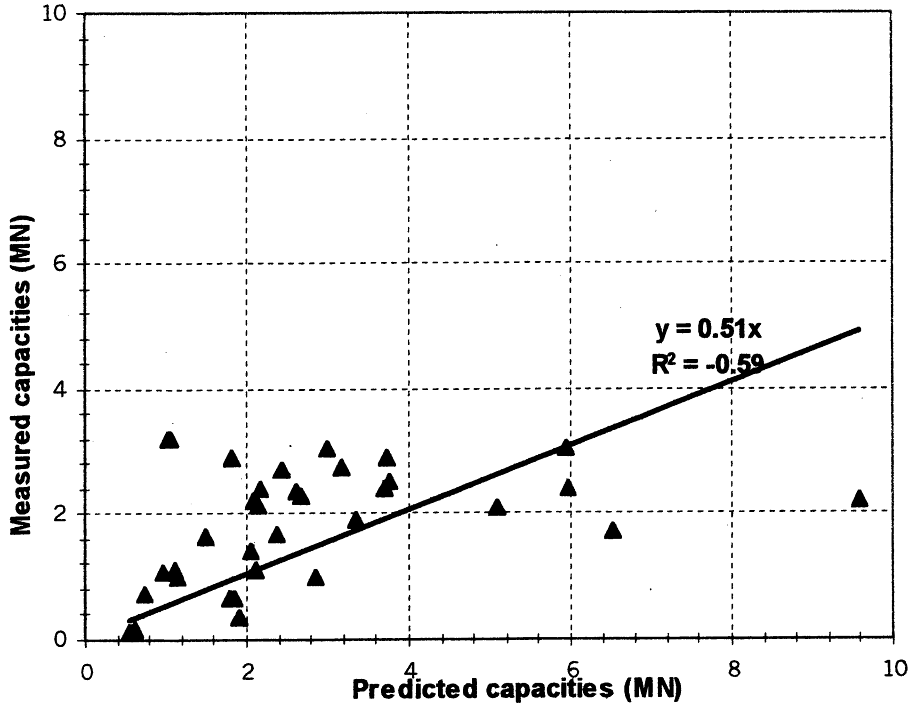

Figure 38: Pipe piles in mixed soils--α-API, Nordlund, Thurman methods

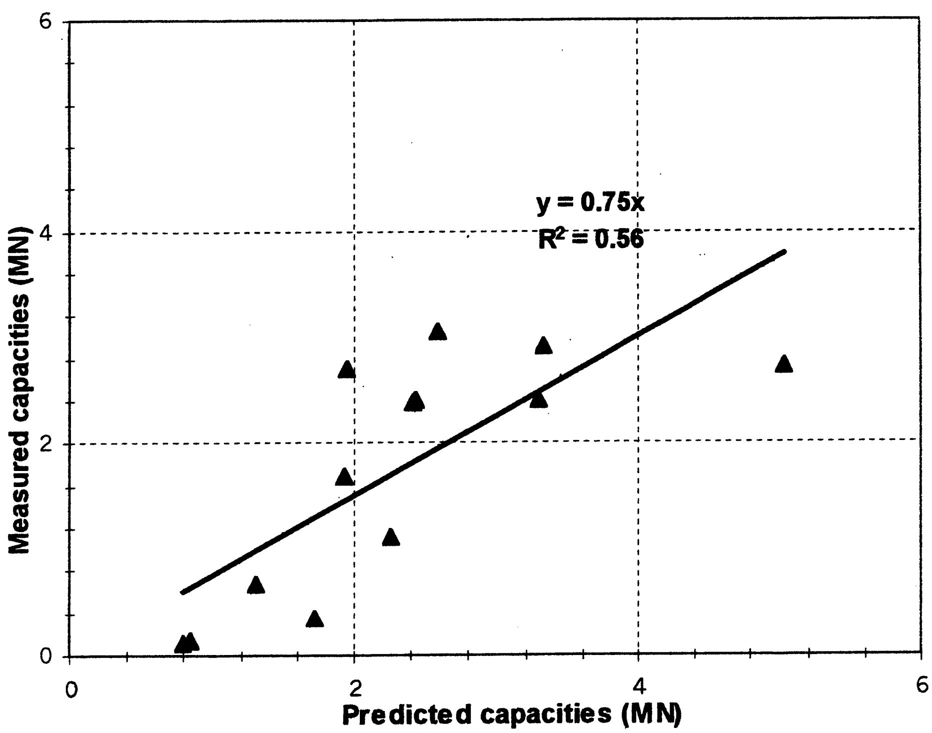

Figure 39: Pipe piles in mixed soils--α-Tomlinson, Nordlund, Thurman methods

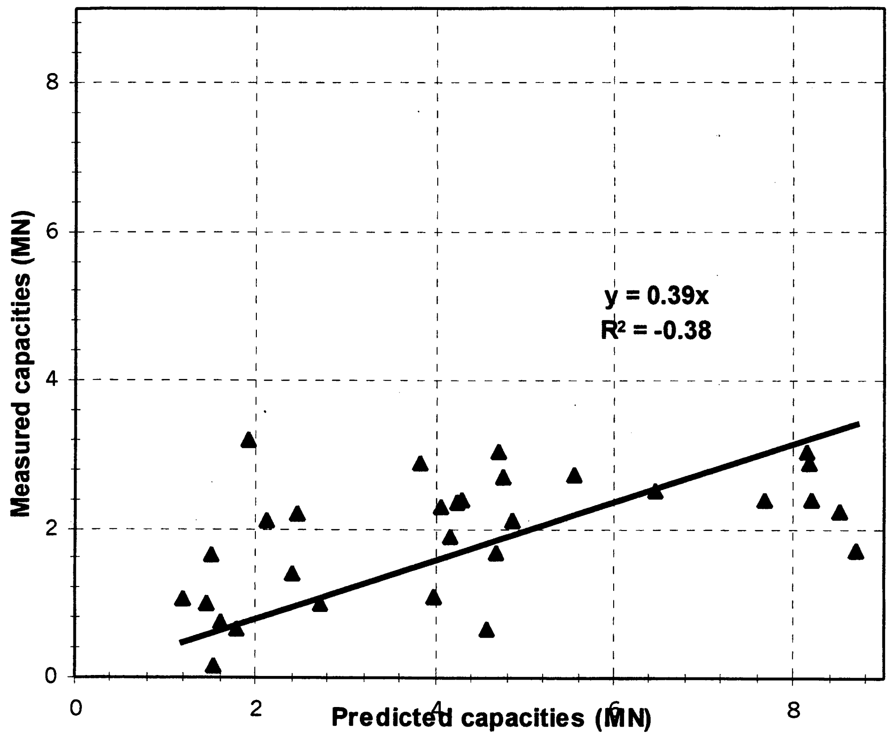

Figure 40: Pipe piles in mixed soils--β methods

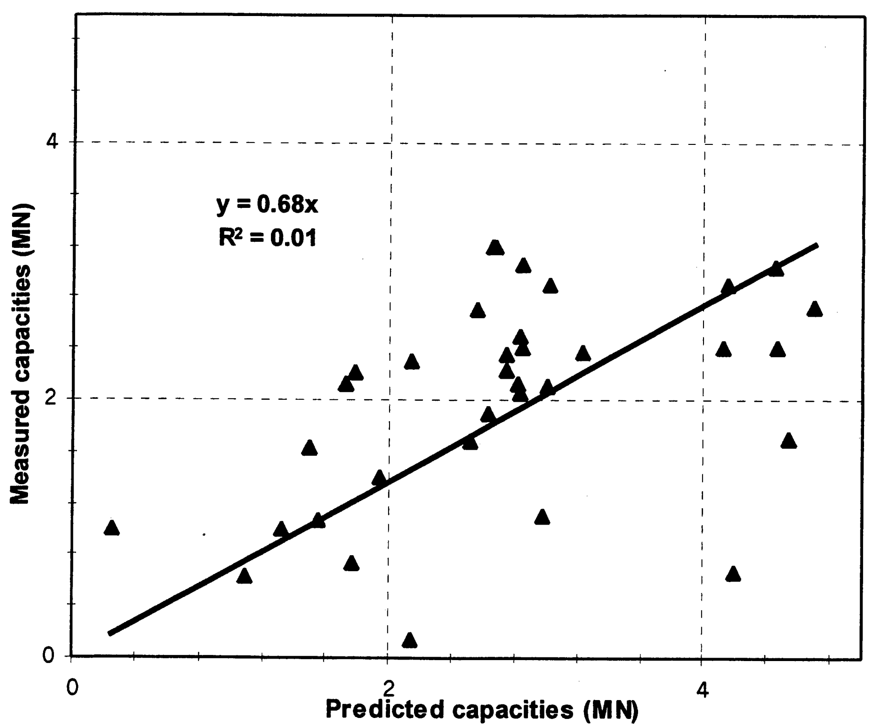

Figure 41: Pipe piles in mixed soils--Schmertmann SPT methods

1.2.4. Histogram--Pipe Piles in Cohesionless Soils (5)

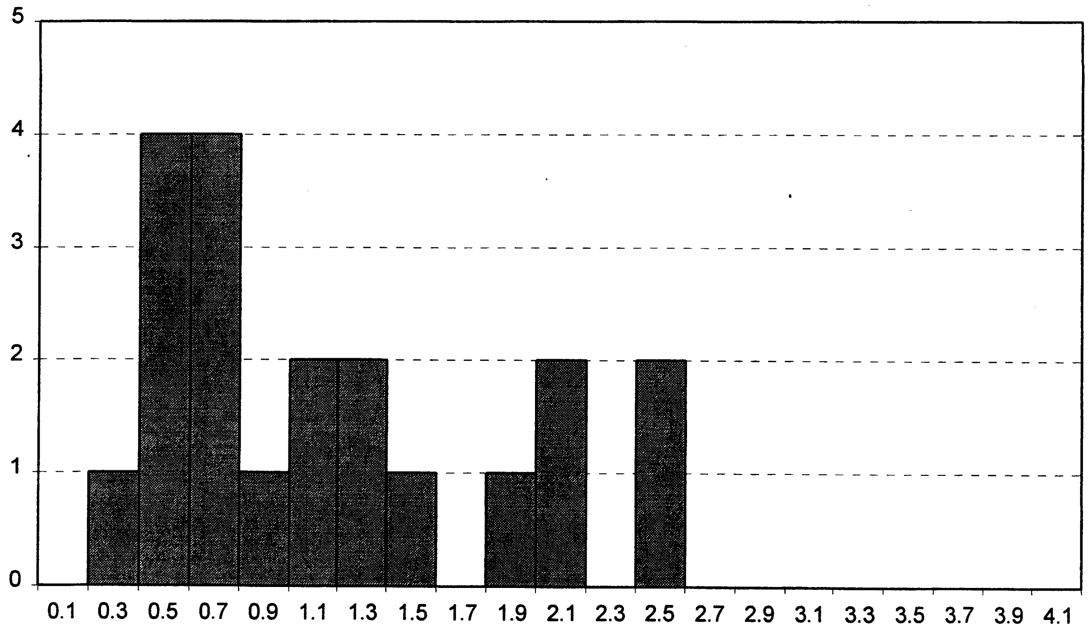

Figure 42: Histogram--β method (5): λ_R =1.18

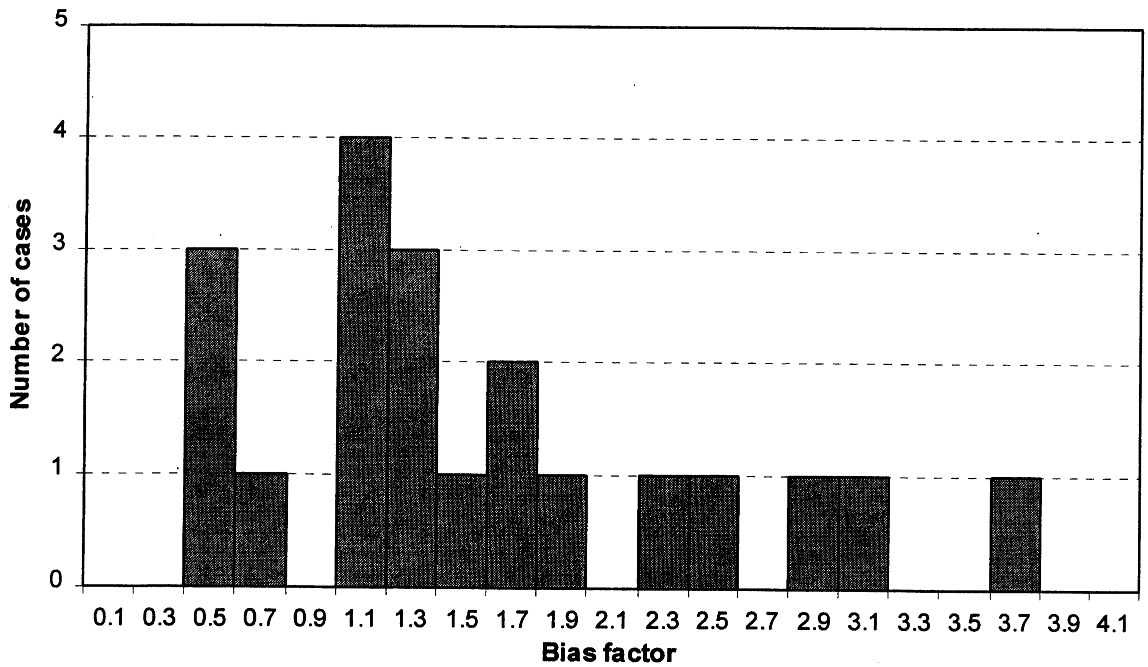

Figure 43: Histogram--Nordlund method (5): λ_R =1.59

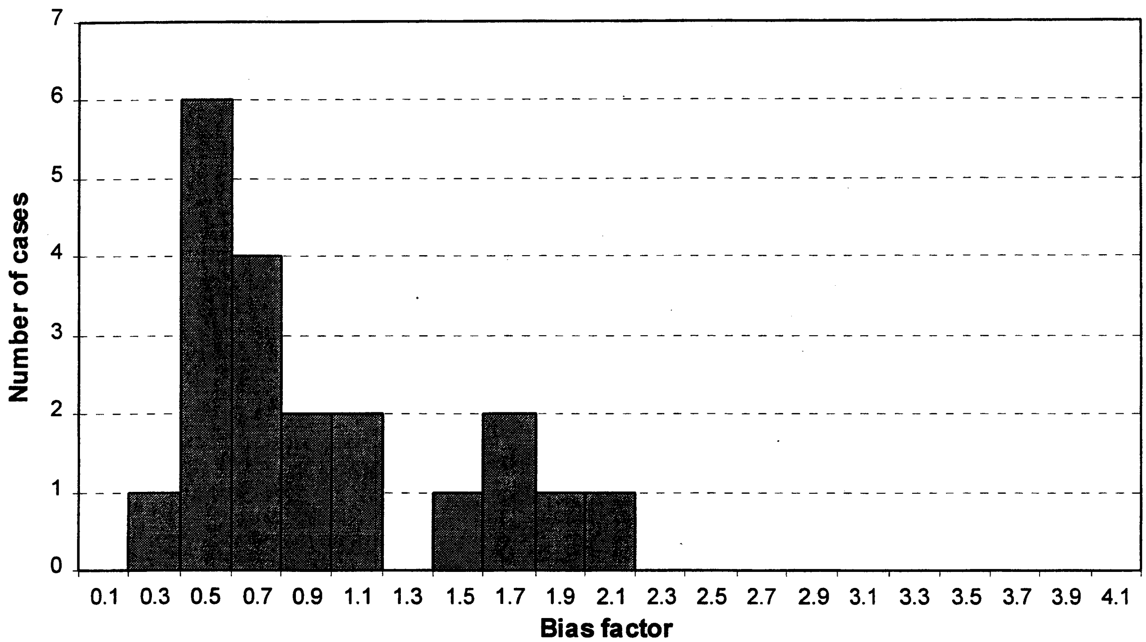

Figure 44: Histogram--Meyerhof method: λ_R =0.94

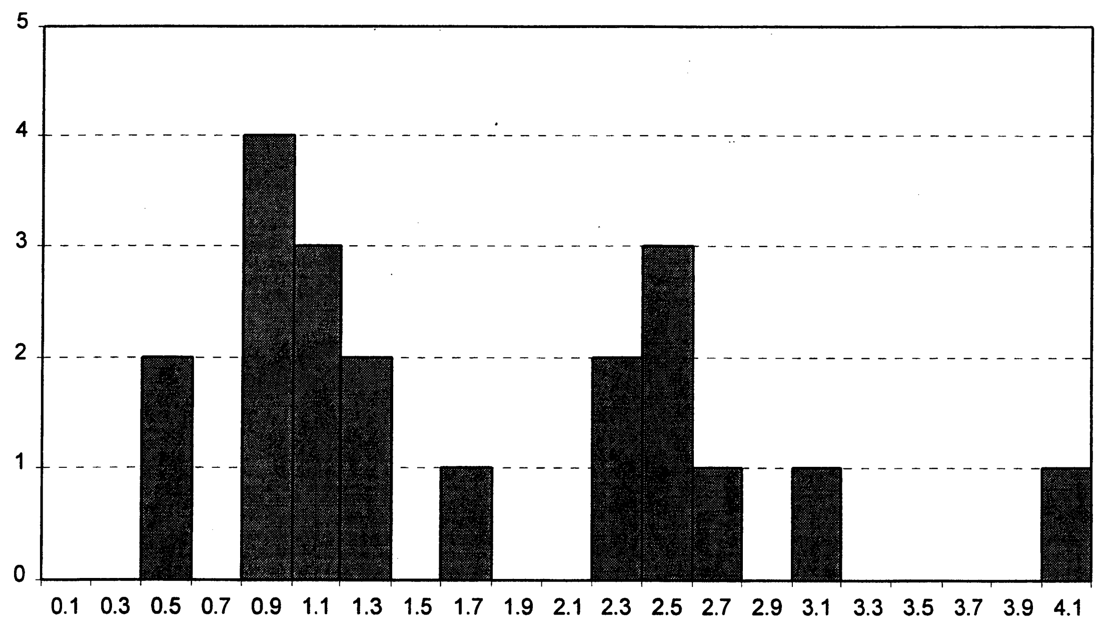

Figure 45: Histogram--Schmertmann SPT mobilized method: λ_R = 1.71

1.2.5. Histogram--Pipe Piles in Cohesive Soils (5h)

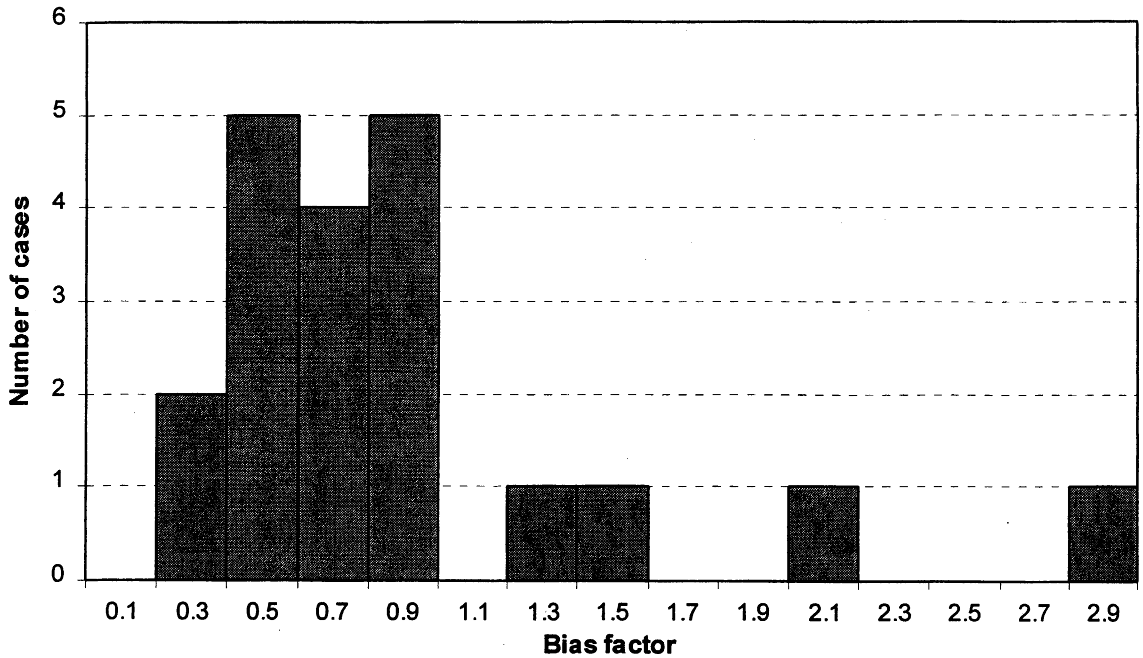

Figure 46: Histogram--α-API method (5h): λ_R =0.90

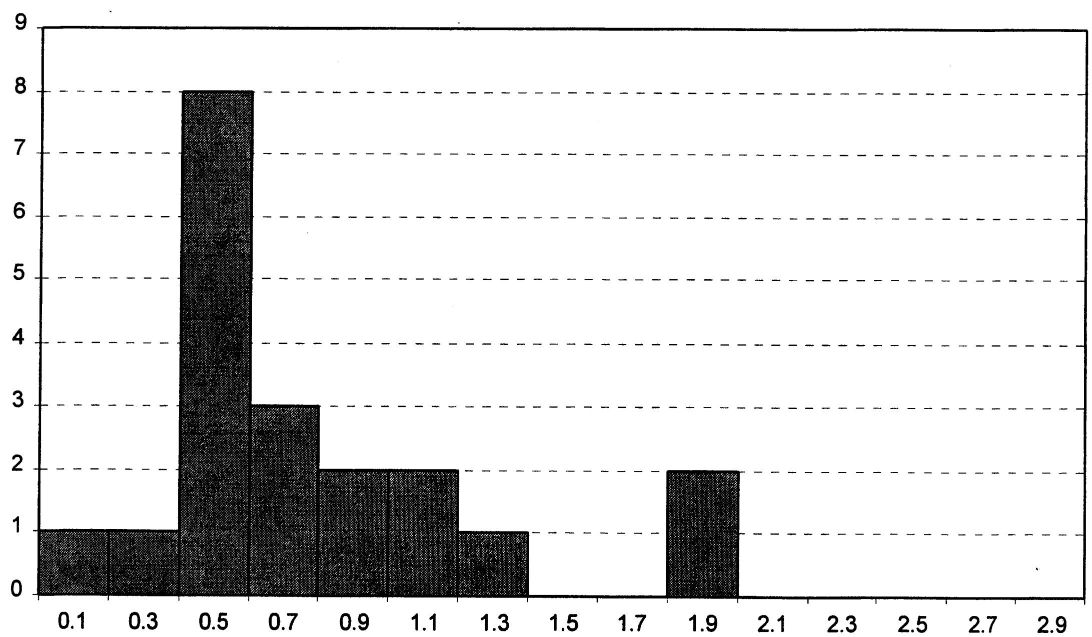

Figure 47: Histogram--α-Tomlinson method (5h) : λ_R =0.77

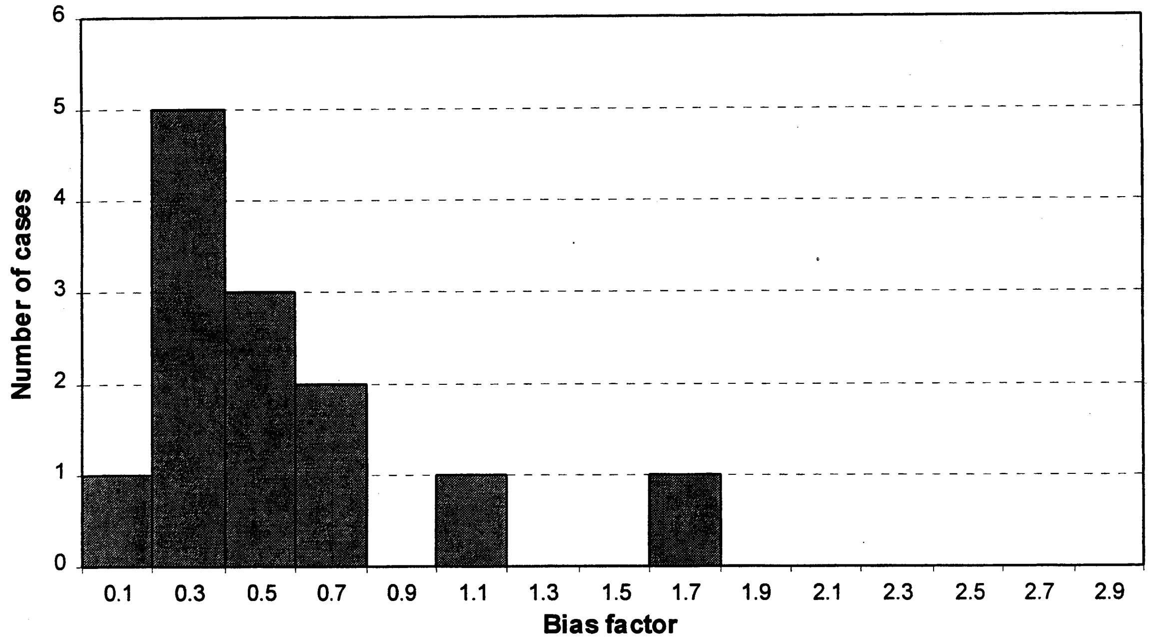

Figure 48: Histogram--β method (5h) : λ_R =0.54

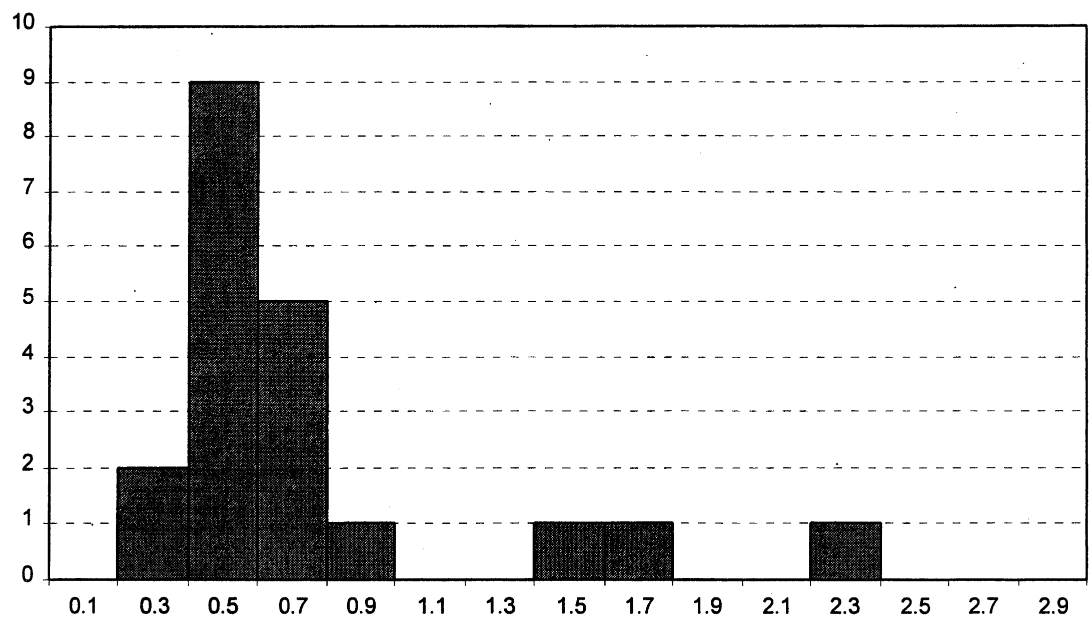

Figure 49: Histogram--λ method (5h) : λ_R = 0.75

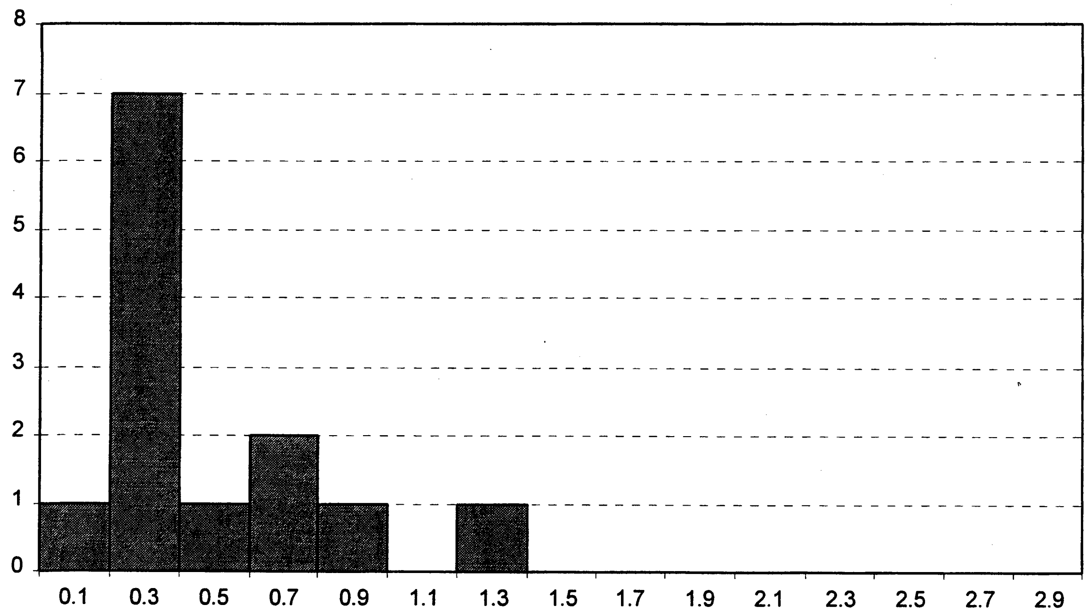

Figure 50: Histogram--Schmertmann SPT mobilized method: $\lambda_R = 0.46$

1.2.6. Histogram--Pipe Piles in Mixed Soils (5)

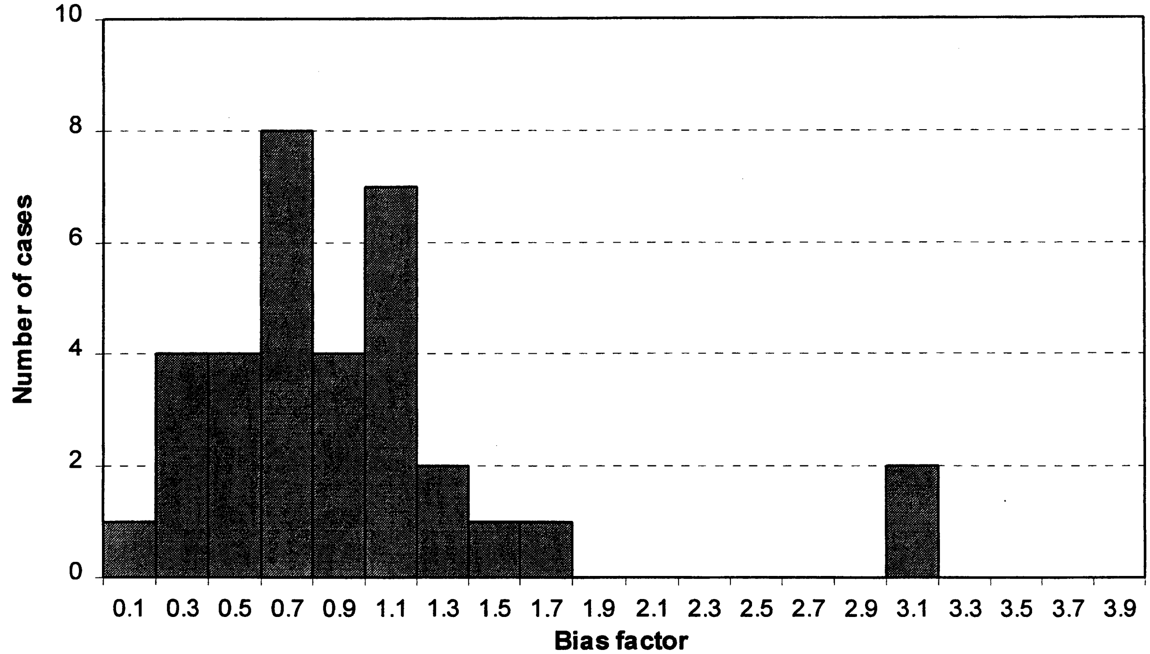

Figure 51: Histogram--α-API/ Nordlund method (5): $\lambda_R = 0.94$

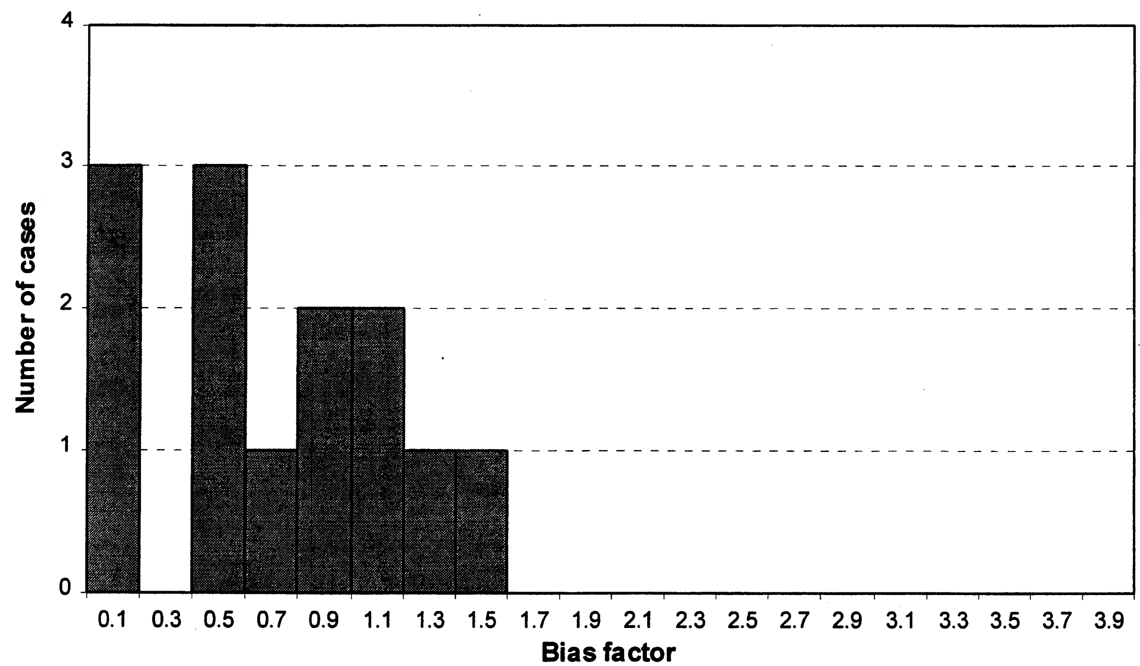

Figure 52: Histogram--α-Tomlinson/ Nordlund method (5): $\lambda_R = 0.74$

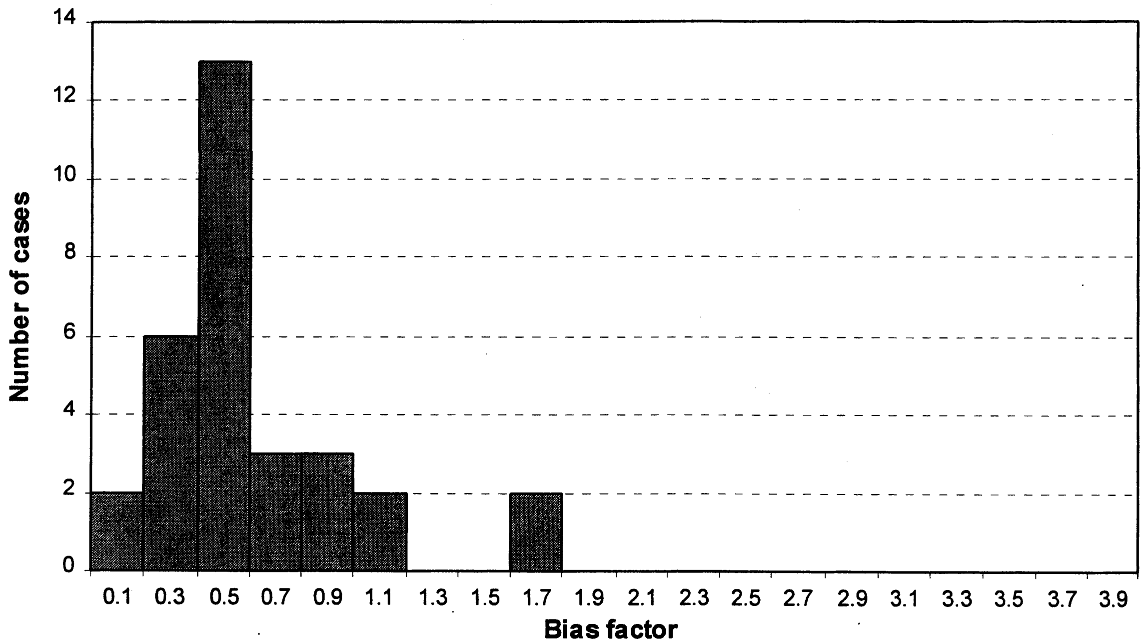

Figure 53: Histogram--β /Thurman method (5) : $\lambda_R = 0.61$

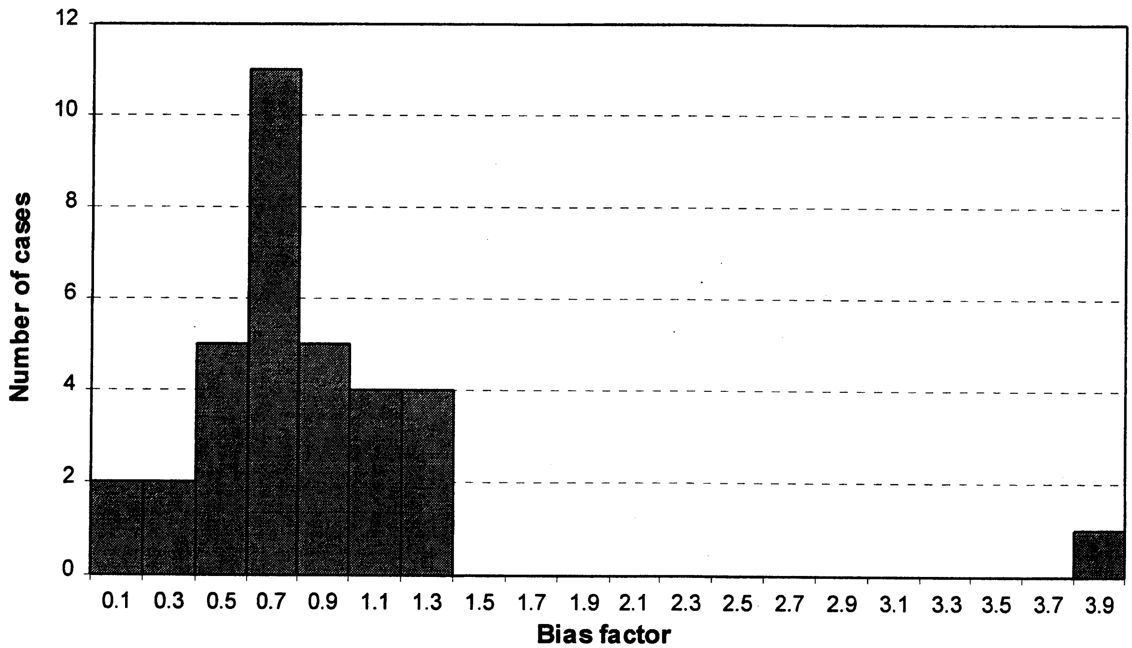

Figure 54: Histogram--Schmertmann SPT mobilized method: $\lambda_R = 0.85$

1.3. Capacity--Results for Plugged H Piles

1.3.1. Figures of Capacity Prediction--Plugged H piles in Cohesionless Soils (5)

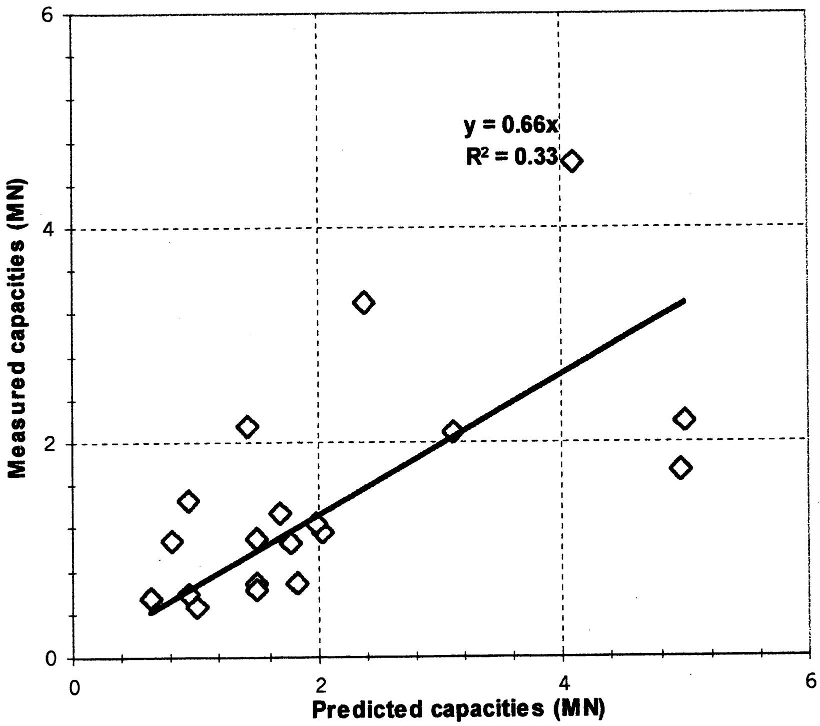

Figure 55: Plugged H piles in Cohesionless soil--β method

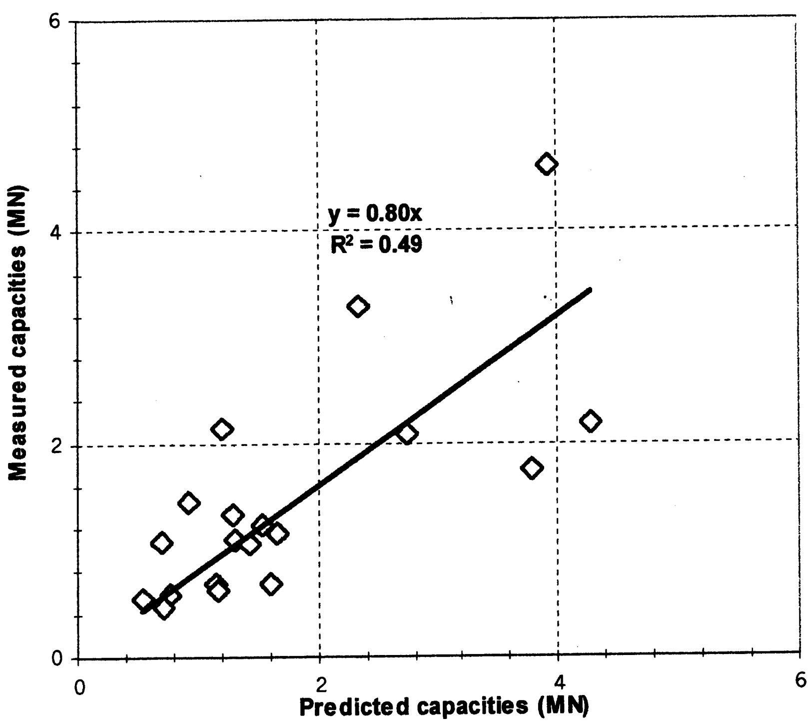

Figure 56: Plugged H piles in Cohesionless soil--Nordlund method

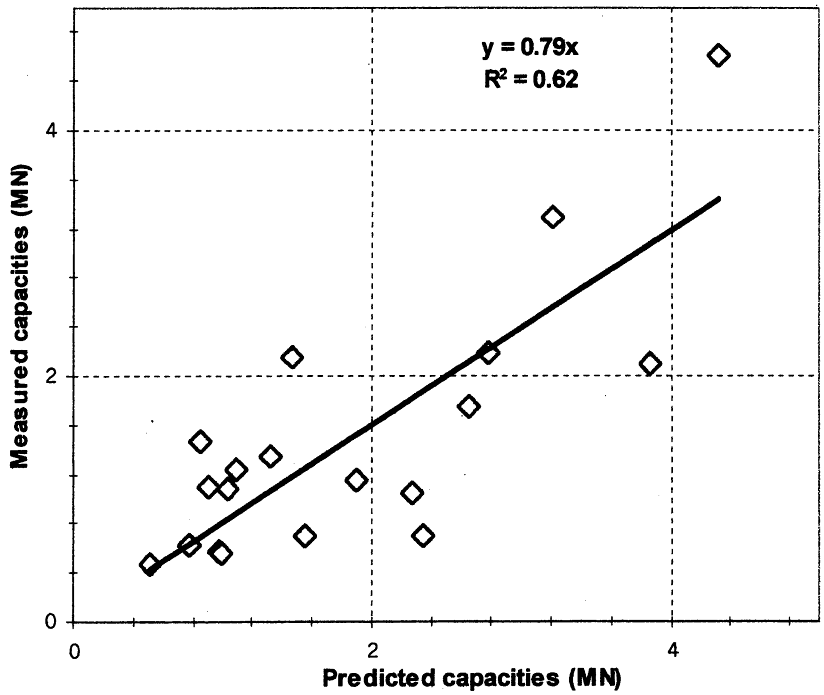

Figure 57: Plugged H piles in Cohesionless soil--Meyerhof method

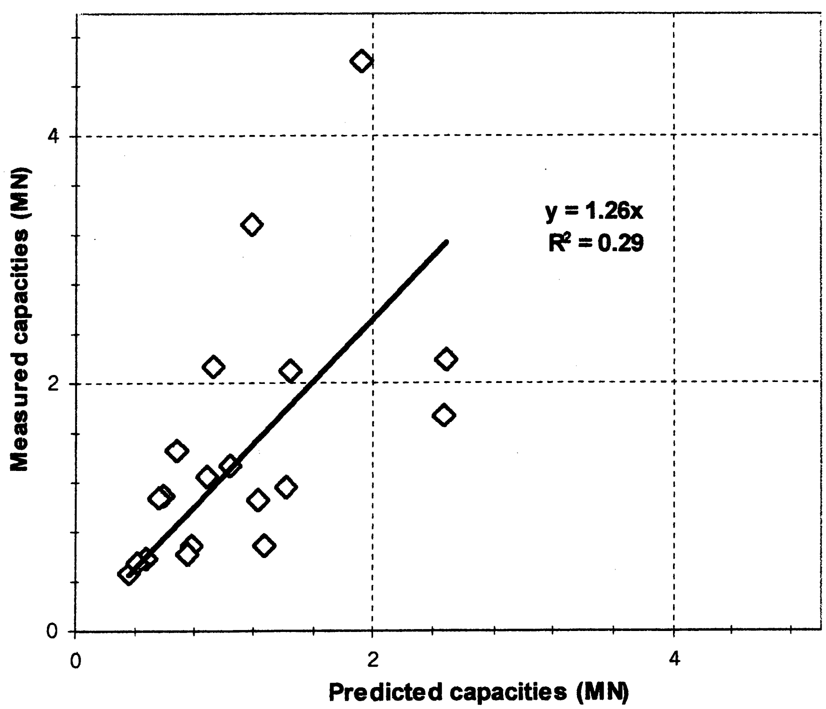

Figure 58: Plugged H piles in Cohesionless soil--Schmertmann SPT method

1.3.2. <u>Figures of Capacity Prediction--Plugged H piles in Cohesive Soils (5)</u>

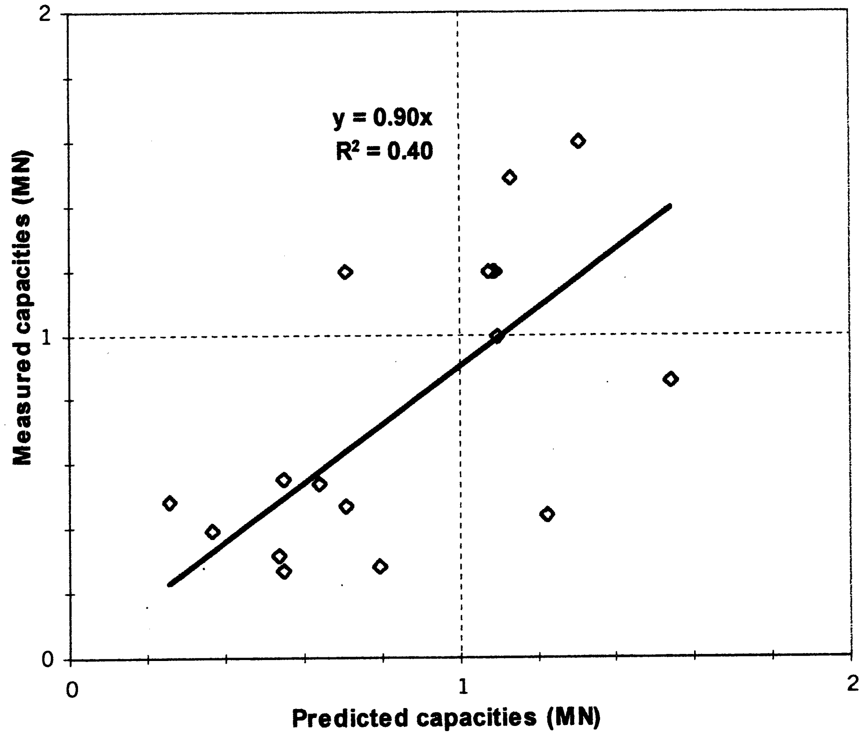

Figure 59: Plugged H piles in cohesive soil--α-API method

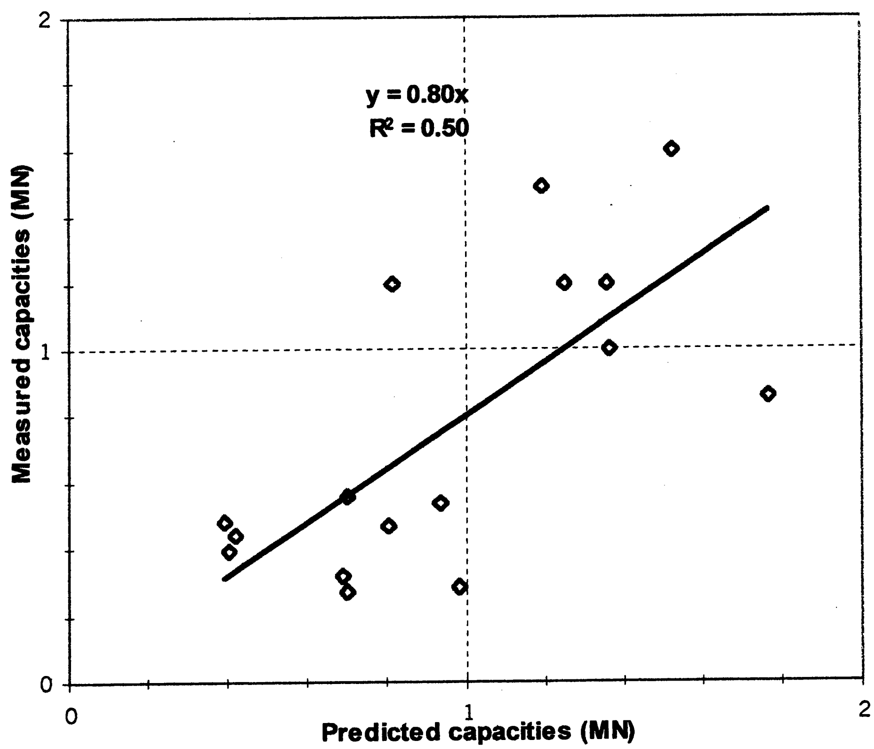

Figure 60: Plugged H piles in cohesive soil--α-Tomlinson 1980 method

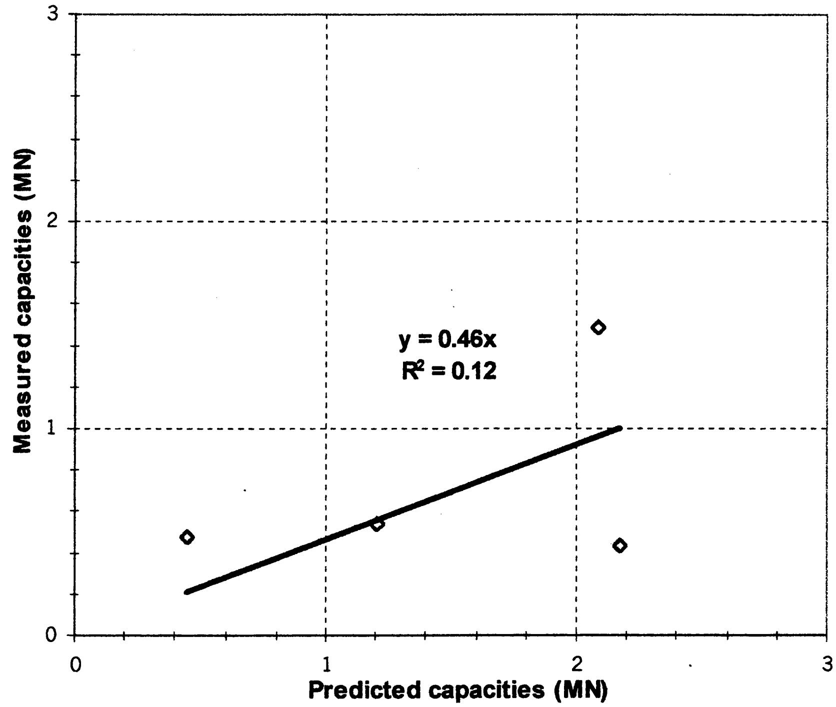

Figure 61: Plugged H piles in cohesive soil--β method

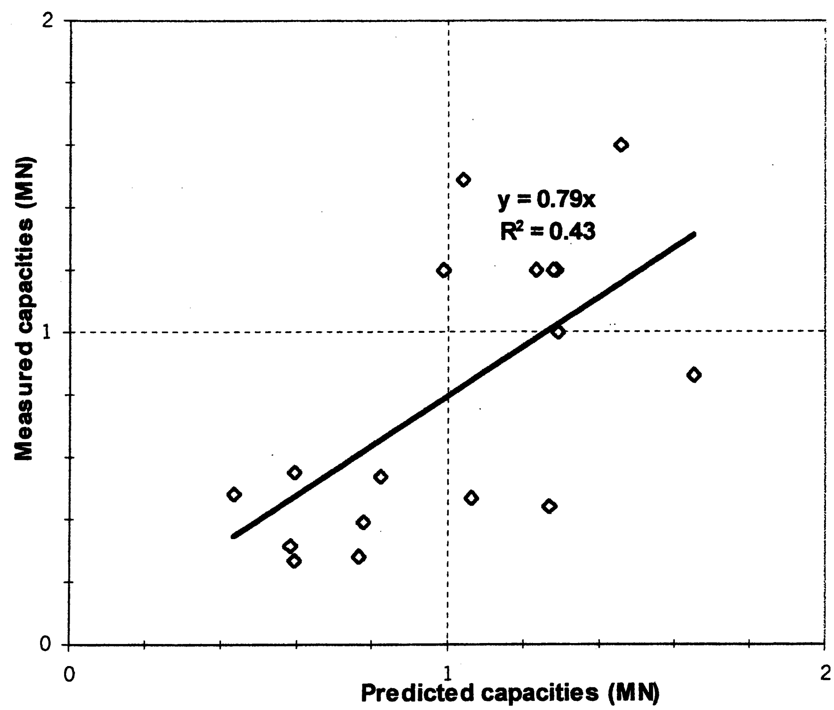

Figure 62: Plugged H piles in cohesive soil--λ method

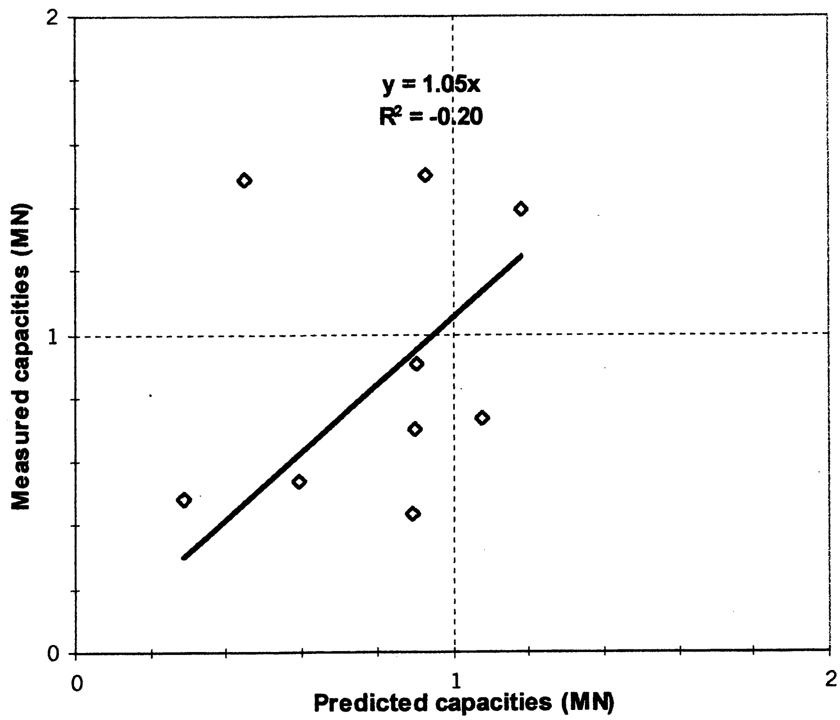

Figure 63: Plugged H piles in cohesive soil--Schmertmann SPT method

1.3.3. Figures of Capacity Prediction--Plugged H piles in Mixed Soils (5)

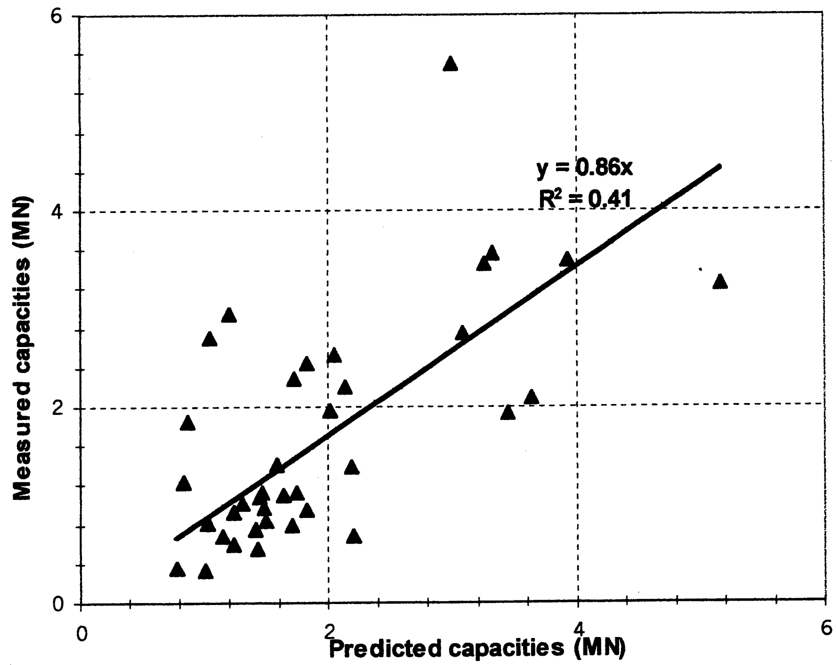

Figure 64: Plugged H piles in mixed soils--α-API, Nordlund, Thurman methods

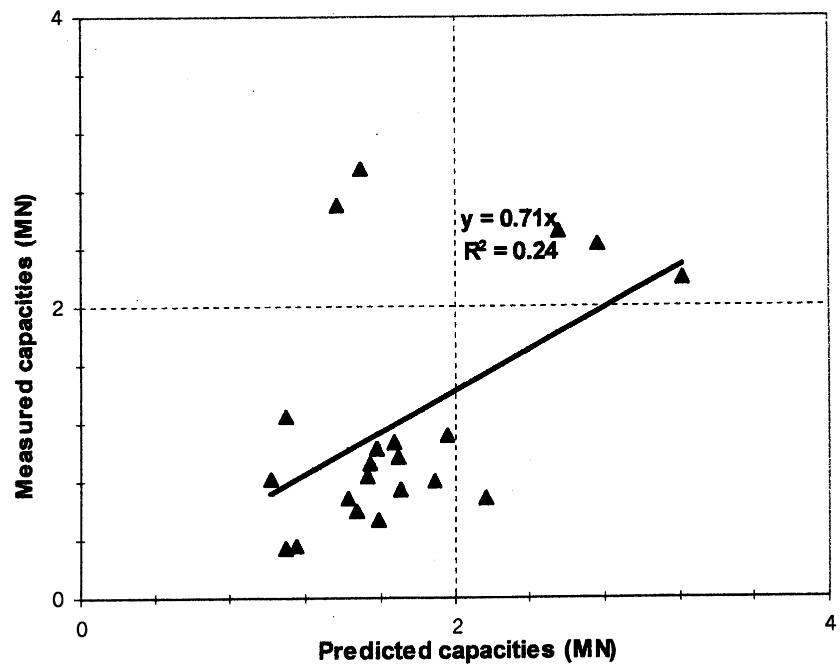

Figure 65: Plugged H piles in mixed soils--α-Tomlinson, Nordlund, Thurman methods

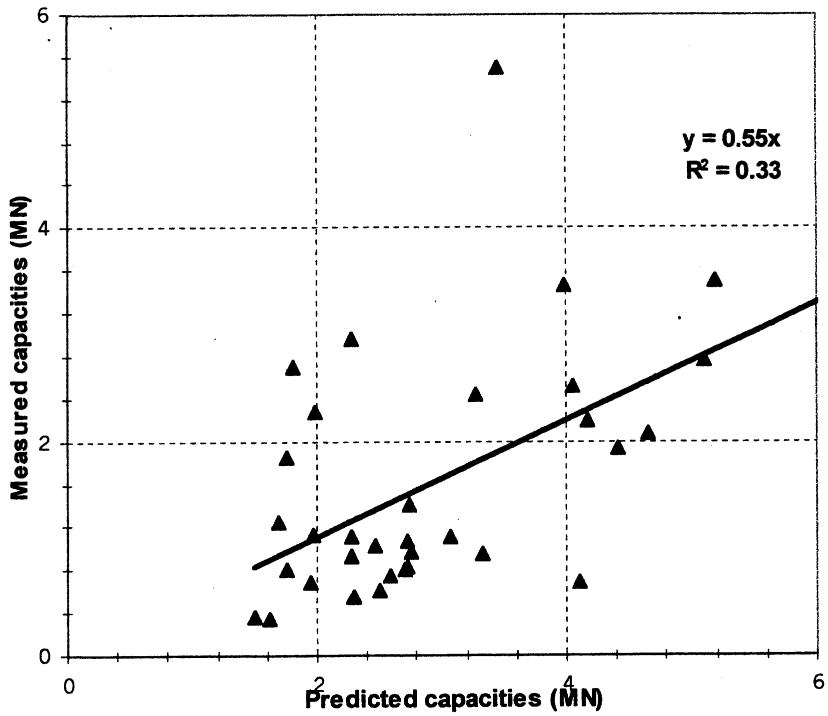

Figure 66: Plugged H piles in mixed soils--β methods

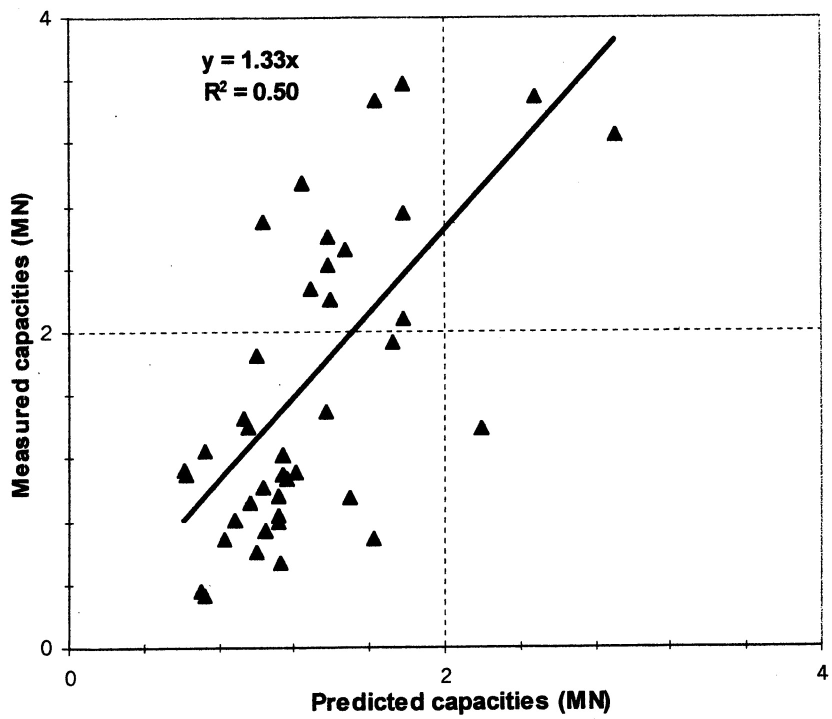

Figure 67: Plugged H piles in mixed soils--Schmertmann SPT methods

1.3.4. Histogram--Plugged H piles in Cohesionless Soils (5)

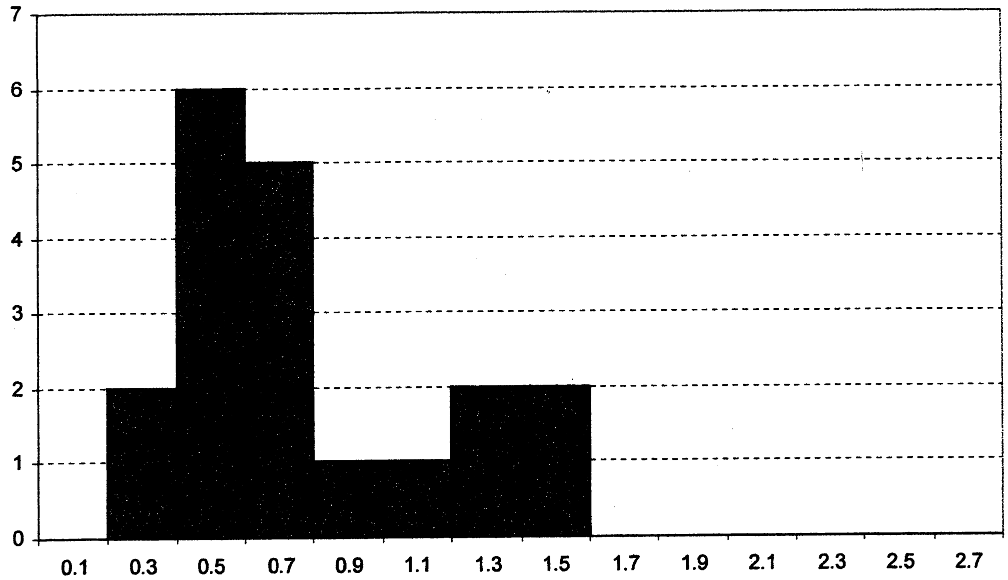

Figure 68: Histogram--β method (5)

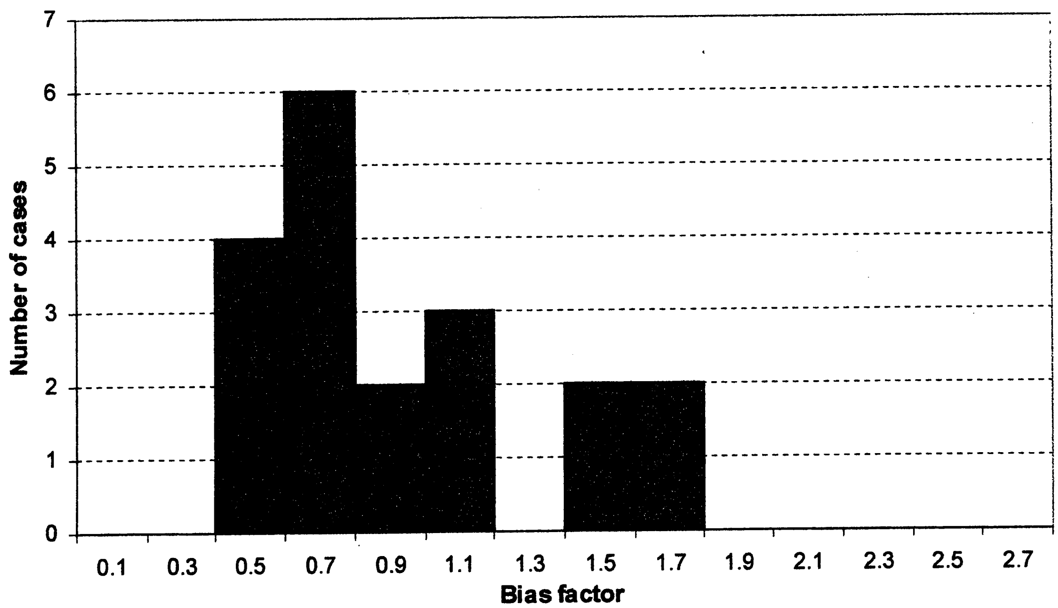

Figure 69: Histogram--Nordlund method (5)

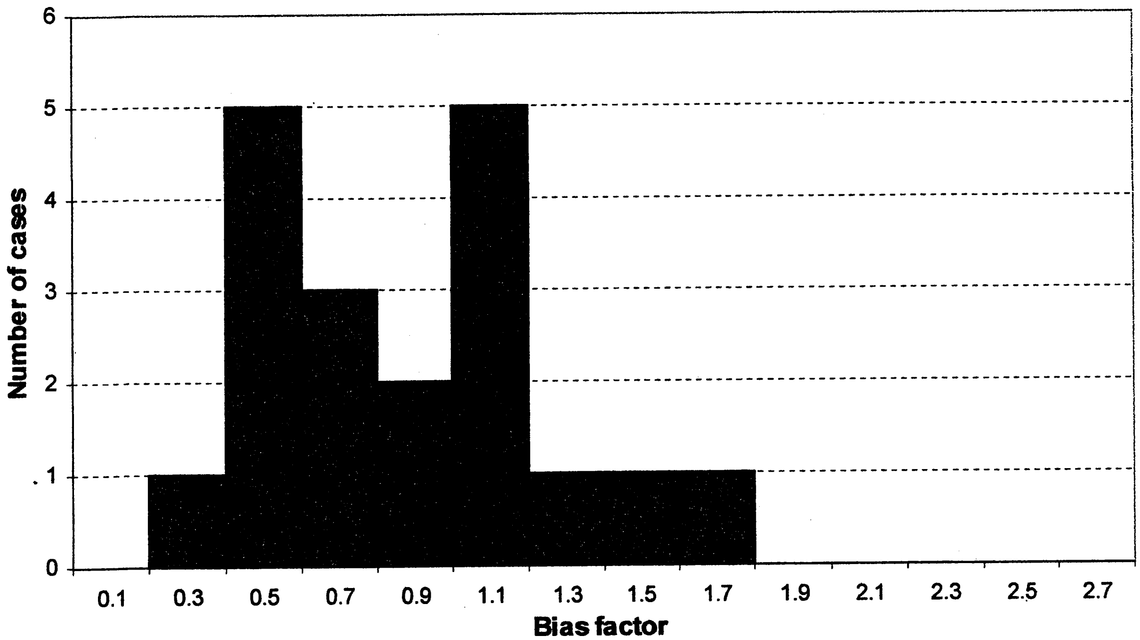

Figure 70: Histogram--Meyerhof method

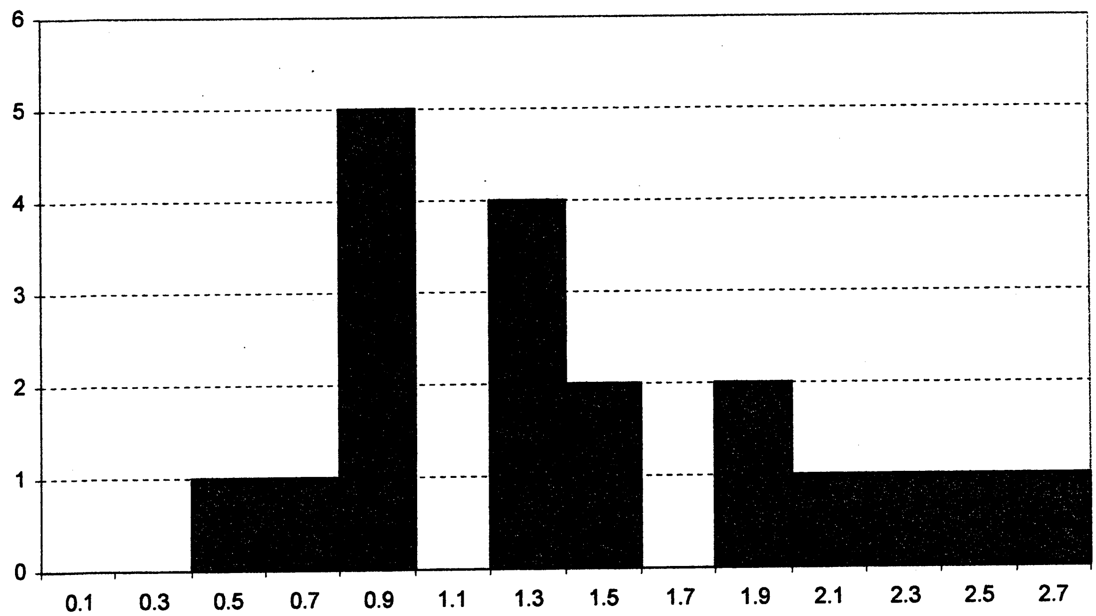

Figure 71: Histogram--Schmertmann SPT mobilized method

1.3.5. Histogram--Plugged H piles in Cohesive Soils (5)

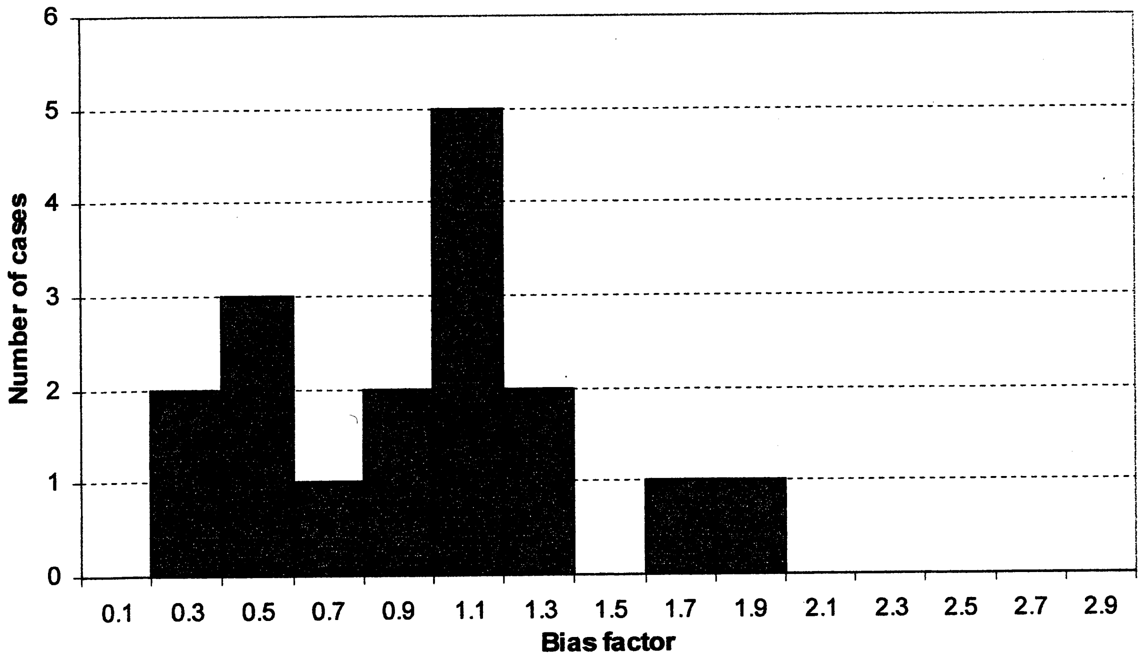

Figure 72: Histogram--α-API method (5)

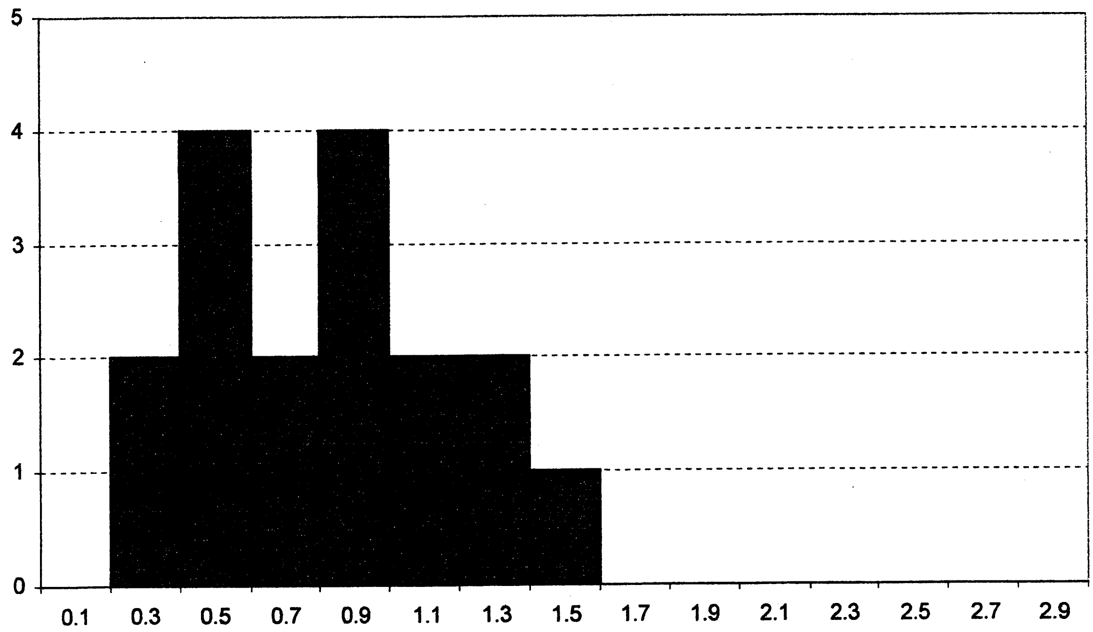

Figure 73: Histogram--α-Tomlinson method (5)

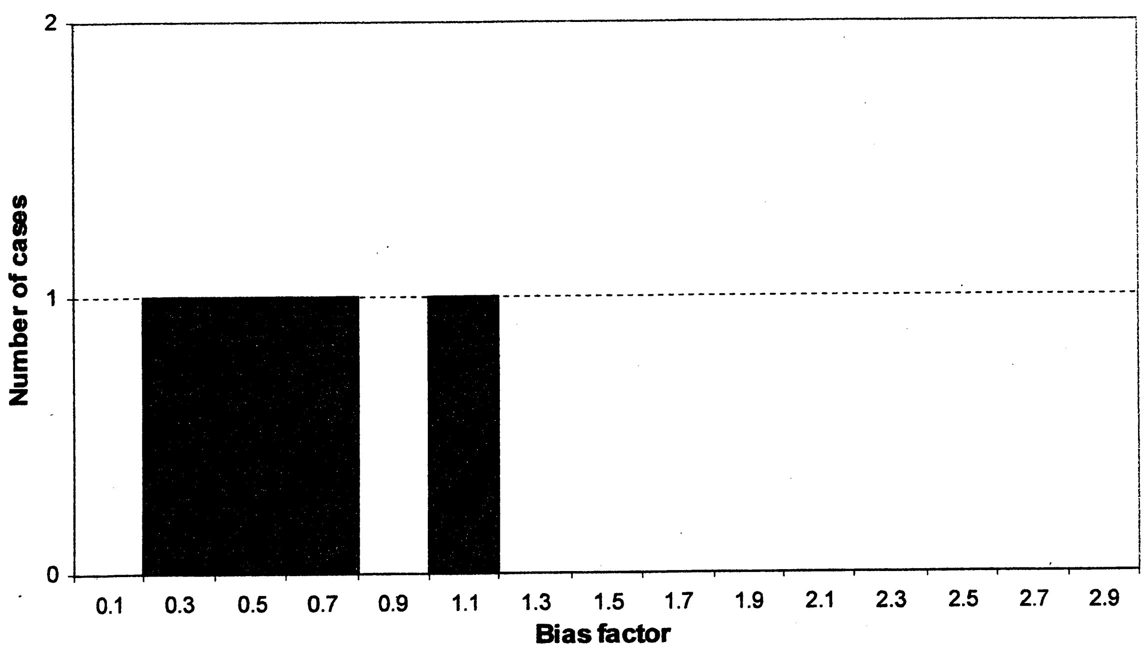

Figure 74: Histogram--β method (5)

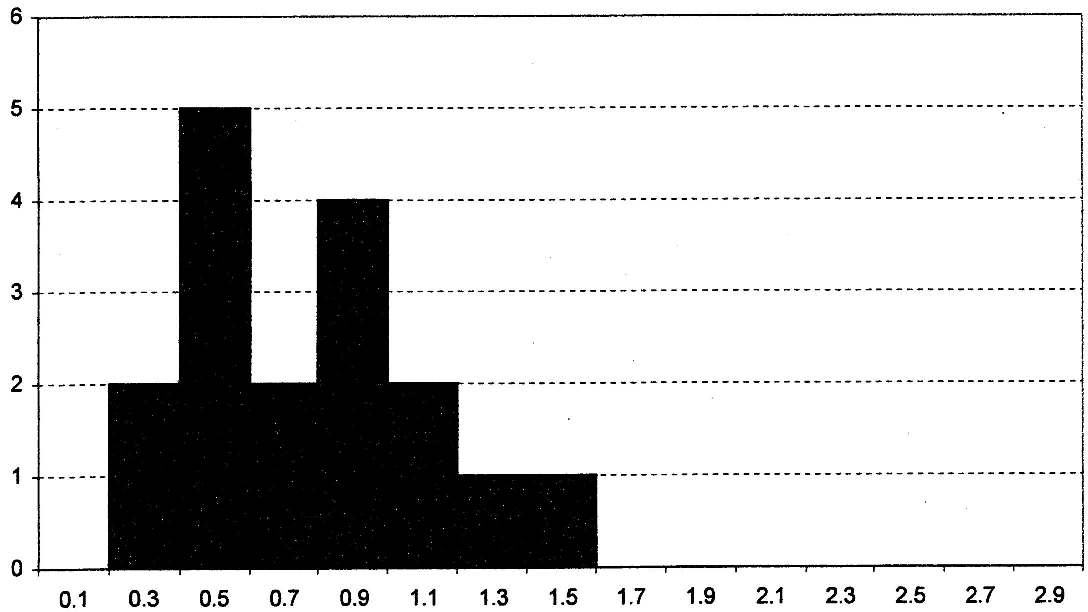

Figure 75: Histogram--λ method (5)

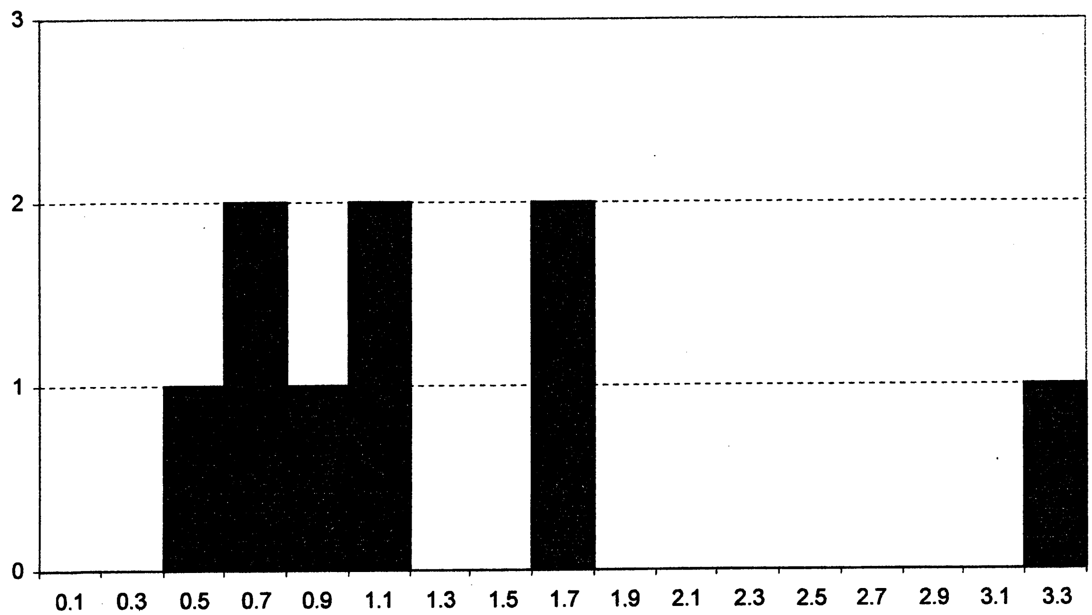

Figure 76: Histogram--Schmertmann SPT mobilized method

1.3.6. Histogram--Plugged H piles in Mixed Soils (5)

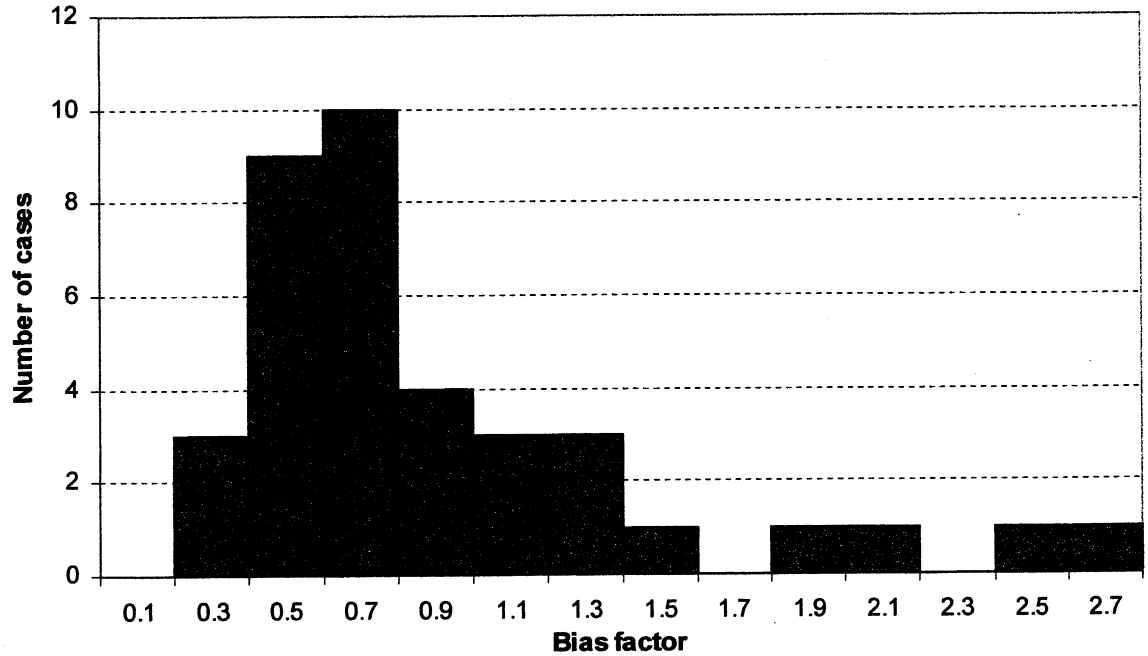

Figure 77: Histogram--α-API/ Nordlund method (5)

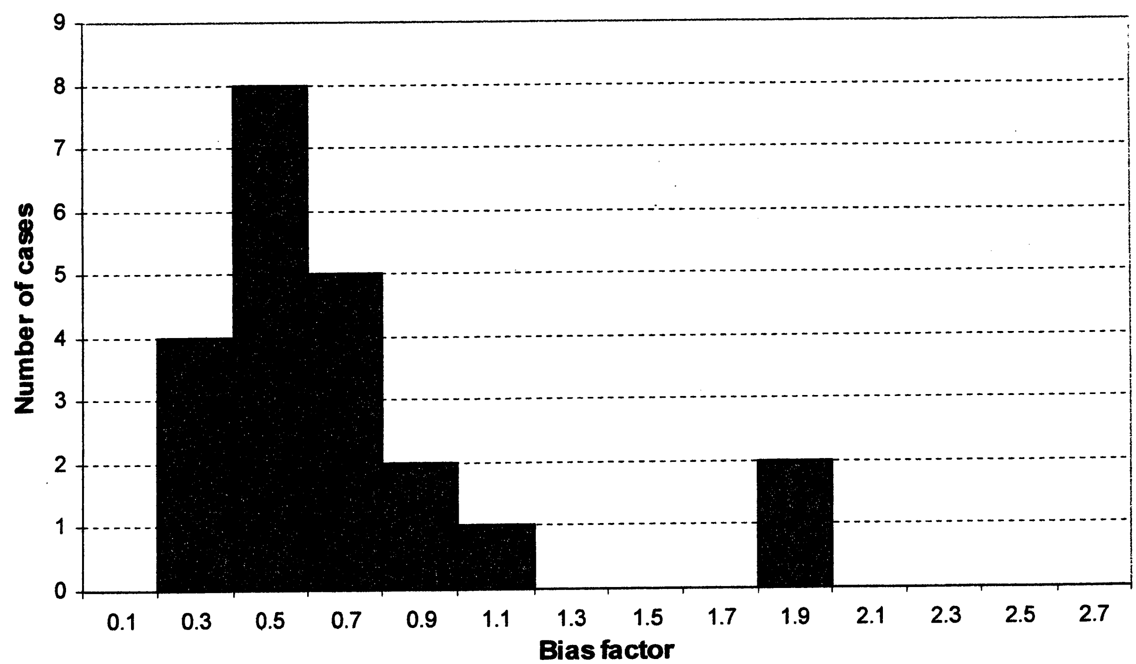

Figure 78: Histogram--α-Tomlinson/ Nordlund method (5)

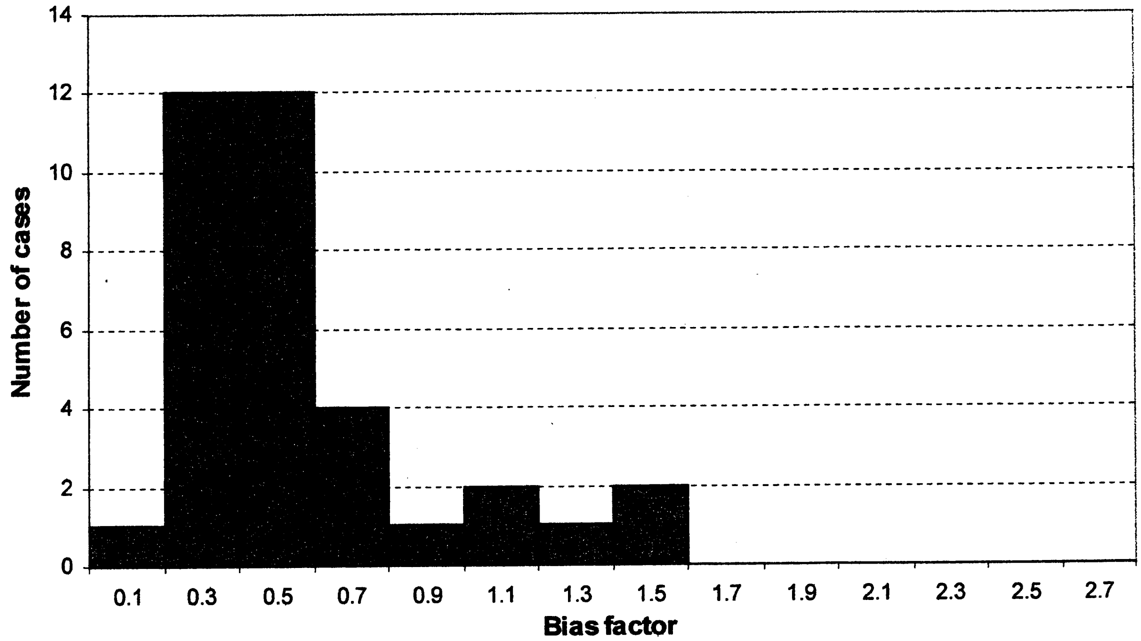

Figure 79: Histogram--β /Thurman method (5)

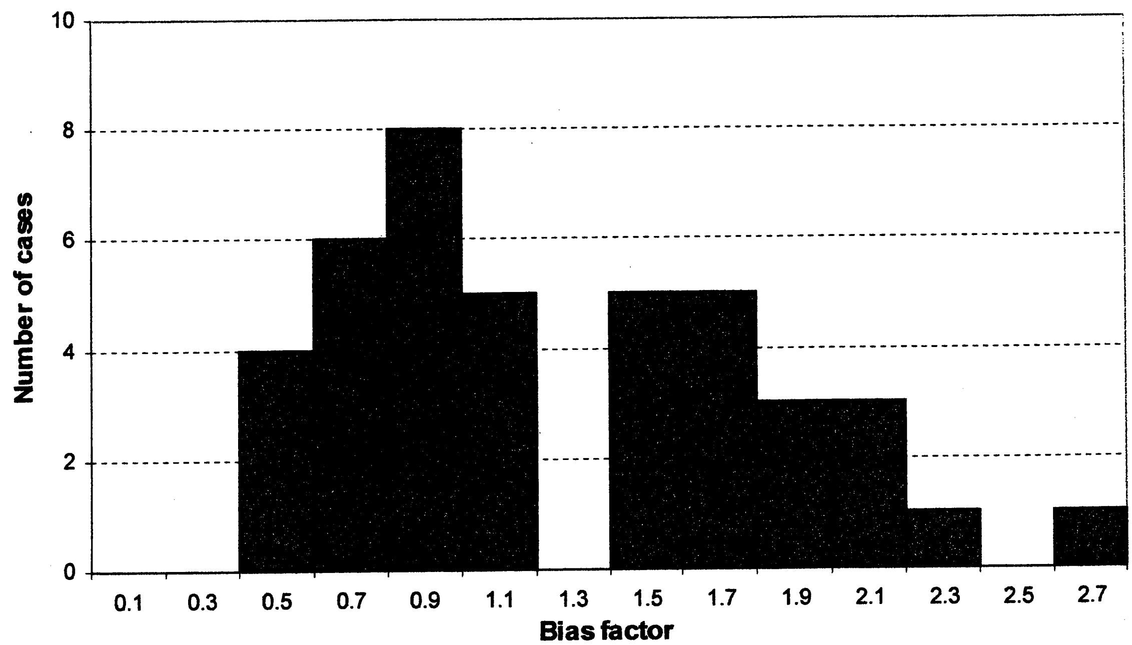

Figure 80: Histogram--Schmertmann SPT mobilized method

1.4. Statistical Results

Table 4 (repeated): Symbols represented the soil parameters used.

Symbol	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
limit ϕ below	40^0				36^0			
contributed zone to tip resistance	2B	11.5B	2B	11.5B	2B	11.5B	2B	11.5B
ϕ, if from SPT, is correlated by	Peck, Hanson and Thornburn		Schmertmann		Peck, Hanson and Thornburn		Schmertmann	
S_u, if from SPT, is correlated by	Terzaghi and Peck							

Symbol	(1h)	(2h)	(3h)	(4h)	(5h)	(6h)	(7h)	(8h)
limit ϕ below	40^0				36^0			
contributed zone to tip resistance	2B	11.5B	2B	11.5B	2B	11.5B	2B	11.5B
ϕ, if from SPT, is correlated by	Peck, Hanson and Thornburn		Schmertmann		Peck, Hanson and Thornburn		Schmertmann	
S_u, if from SPT, is correlated by	Hara							

Table 5. Statistics for Concrete Piles

COHESIONLESS SOILS	Nordlund				β		Meyerhof	Schmertmann	
	DRIVEN	40;2B;P (1)	36;11.5 B;P (6)	36;2B;P (5)	40;2B;P (1)	36;2B;P (5)		SPT mobilized	CPT
N	37	37	37	37	37	37	37	37	2
Bias mean	0.71	0.71	1.06	1.00	0.89	1.17	0.64	1.25	0.61
Standard deviation	0.41	0.41	0.55	0.53	0.51	0.56	0.42	0.62	
COV	0.58	0.58	0.52	0.53	0.58	0.48	0.65	0.49	

COHESIVE SOILS	α Tomlinson			α API revised			β		λ		Schmertmann	
	DRIVEN	(1)	(5h)	APILE	(1)	(5h)	(1)	(5h)	(1)	(5h)	SPT mobilized	CPT
N	19	19	19	19	19	19	8	8	19	19	2	6
Bias mean	1.12	1.08	0.94	1.03	1.08	0.89	0.92	0.81	0.92	0.79	3.99	0.86
Standard deviation	1.55	0.82	0.50	0.76	0.73	0.31	0.60	0.41	0.50	0.25		0.09
COV	1.39	0.76	0.54	0.74	0.68	0.35	0.65	0.50	0.54	0.32		0.11

MIXED SOILS	α Tomlinson, Nordlund and Thurman			α API, Nordlund, Thurman		β; Thurman		Schmertmann	
	DRIVEN	40;2B;P;T &P(1)	36;2B;P; Hara(5h)	40;2B;P;T &P(1)	36;2B;P; Hara(5h)	40;2B;P;T &P(1)	36;2B;P; Hara(5h)	SPT mobilized	CPT
N	34	34	34	85	85	85	85	74	32
Bias mean	1.66	1.48	1.00	1.01	0.88	0.94	0.93	1.97	0.90
Standard deviation	1.34	1.11	0.52	0.62	0.47	0.57	0.55	1.20	0.35
COV	0.81	0.75	0.52	0.62	0.54	0.61	0.59	0.61	0.39

ALL SOILS	α-Tomlinson and/or Nordlund			α-API and/or Nordlund			β		λ		Meyerhof	Schmertmann	
	DRIVEN	(1)	(5h)	APILE	(1)	(5h)	(1)	(5h)	(1)	(5h)		SPT mobilized	CPT
N	90	90	90	19	141	141	130	130	19	19	37	113	40
Bias mean	1.15	1.08	0.99	1.03	0.94	0.91	0.92	0.99	0.92	0.79	0.64	1.77	0.88
Standard deviation	1.18	0.88	0.51	0.76	0.60	0.47	0.55	0.55	0.50	0.25	0.42	1.15	0.33
COV	1.03	0.82	0.52	0.74	0.64	0.51	0.60	0.56	0.54	0.32	0.65	0.65	0.37

Table 6. Statistics for Pipe Piles

COHESIONLESS SOILS	Nordlund and Thurman				β			
	DRIVEN	40;2B;P (1)	40;2B;Sch (3)	36;2B;P (5)	40;2B;P (1)	36;2B;P (5)	Meyerhof	Schmertmann SPT mobilized
N	20	20	20	20	20	20	20	20
Bias mean	1.16	1.29	0.77	1.59	1.02	1.18	0.94	1.71
Standard deviation	0.85	0.91	0.49	0.91	0.66	0.73	0.55	0.98
COV	0.73	0.71	0.64	0.57	0.65	0.61	0.59	0.57

COHESIVE SOILS	α Tomlinson			α API revised			β		λ		Schmert-mann SPT mobilized
	DRIVEN	2B; T&P (5)	2B; Hara (5h)	APILE	2B; T&P (5)	2B; Hara (5h)	2B; T&P (5)	2B; Hara (5h)	2B; T&P (5)	2B; Hara (5h)	
N	20	20	20	20	20	20	13	13	20	20	13
Bias mean	0.78	0.77	0.56	0.88	0.90	0.58	0.54	0.50	0.75	0.47	0.46
Standard deviation	0.62	0.49	0.34	0.61	0.62	0.35	0.42	0.36	0.50	0.26	0.35
COV	0.79	0.64	0.61	0.69	0.69	0.60	0.76	0.73	0.66	0.56	0.77

MIXED SOILS	α Tomlinson, Nordlund and Thurman			α API, Nordlund, Thurman		β; Thurman		Schmert-mann SPT mobilized
	DRIVEN	36;2B;P;T&P(5)	36;2B;P;Hara(5h)	36;2B;P;T&P(5)	36;2B;P;Hara(5h)	36;2B;P;T&P(5)	36;2B;P;Hara(5h)	
N	13	13	13	34	34	31	31	34
Bias mean	0.69	0.74	0.54	0.94	0.71	0.61	0.60	0.85
Standard deviation	0.41	0.44	0.35	0.64	0.41	0.38	0.36	0.62
COV	0.60	0.60	0.66	0.69	0.57	0.61	0.60	0.73

ALL SOILS	α-Tomlinson and/or Nordlund			α-API and/or Nordlund		β		λ		Meyerhof	Schmert-mann SPT mobilized
	DRIVEN	(5)	(5h)	(5)	(5h)	(5)	(5h)	(5)	(5h)		
N	53	53	53	74	74	64	64	20	20	20	67
Bias mean	0.90	1.07	0.94	1.10	0.92	0.78	0.76	0.75	0.47	0.94	1.03
Standard deviation	0.70	0.78	0.80	0.77	0.70	0.58	0.58	0.50	0.26	0.55	0.84
COV	0.77	0.73	0.85	0.70	0.77	0.75	0.76	0.66	0.56	0.59	0.82

Table 7. Statistics for Plugged H piles

COHESION LESS SOILS	Nordlund and Thurman		β		Meyerhof	Schmertmann SPT mobilized
	ϕ≤40; Tip 2B; ϕ by Peck et al. (1)	ϕ≤36; Tip 2B; ϕ by Peck et al. (5)	ϕ≤40; Tip 2B; ϕ by Peck et al. (1)	ϕ≤36; Tip 2B; ϕ by Peck et al. (5)		
N	19	19	19	19	19	19
Bias mean	0.74	0.92	0.66	0.78	0.86	1.42
Standard deviation	0.39	0.41	0.33	0.40	0.37	0.65
COV	0.53	0.45	0.51	0.50	0.43	0.46

COHESIVE SOILS	α Tomlinson		α API revised		β		λ		Schmert-mann SPT mobilized
	2B; T&P (5)	2B; Hara (5h)	2B; T&P (5)	2B; Hara (5h)	2B; T&P (5)	2B; Hara (5h)	2B; T&P (5)	2B; Hara (5h)	
N	17	17	17	17	4	4	17	17	9
Bias mean	0.82	0.75	0.96	0.83	0.61	0.55	0.78	0.68	1.29
Standard deviation	0.33	0.29	0.43	0.37	0.37	0.32	0.33	0.30	0.85
COV	0.40	0.39	0.45	0.44	0.61	0.58	0.42	0.45	0.66

MIXED SOILS	α Tomlinson, Nordlund and Thurman		α API, Nordlund, Thurman		β; Thurman		Schmert-mann SPT mobilized
	ϕ≤36;2B; ϕ by Peck; S_u by T&P(5)	ϕ≤36;2B; ϕ by Peck; S_u by Hara (5h)	ϕ≤36;2B; ϕ by Peck; S_u by T&P(5)	ϕ≤36;2B; ϕ by Peck; S_u by Hara (5h)	ϕ≤36;2B; ϕ by Peck; S_u by T&P(5)	ϕ≤36;2B; ϕ by Peck; S_u by Hara (5h)	
N	22	22	37	37	35	35	41
Bias mean	0.71	0.46	0.92	0.68	0.56	0.55	1.27
Standard deviation	0.46	0.29	0.56	0.37	0.36	0.35	0.58
COV	0.65	0.64	0.61	0.54	0.64	0.64	0.46

ALL SOILS	α-Tomlinson and/or Nordlund		α-API and/or Nordlund		β		λ		Meyerhof	Schmert-mann SPT mobilized
	36;2B;P; T&P(5)	36;2B;P;Hara(5h)	36;2B;P;T&P(5)	36;2B;P;Hara(5h)	36;2B;P;T&P(5)	36;2B;P;Hara(5h)	36;2B;P;T&P(5)	36;2B;P;Hara(5h)		
N	58	58	73	73	58	58	17	17	19	69
Bias mean	0.81	0.69	0.93	0.78	0.64	0.62	0.78	0.68	0.86	1.31
Standard deviation	0.41	0.38	0.49	0.39	0.38	0.38	0.33	0.30	0.37	0.63
COV	0.51	0.55	0.53	0.50	0.59	0.61	0.42	0.45	0.43	0.48

Table 8. Statistics for Unplugged H piles

COHESION LESS SOILS	Nordlund and Thurman		β		Meyerhof	Schmertmann SPT mobilized
	$\phi \leq 40$; Tip 2B; ϕ by Peck et al. (1)	$\phi \leq 36$; Tip 2B; ϕ by Peck et al. (5)	$\phi \leq 40$; Tip 2B; ϕ by Peck et al. (1)	$\phi \leq 36$; Tip 2B; ϕ by Peck et al. (5)		
N	19	19	19	19	19	19
Bias mean	0.85	0.99	0.76	0.80	1.77	1.40
Standard deviation	0.55	0.68	0.55	0.62	1.21	0.73
COV	0.64	0.68	0.72	0.77	0.69	0.52

COHESIVE SOILS	α Tomlinson		α API revised		β		λ		Schmert-mann SPT mobilized
	2B; T&P (5)	2B; Hara (5h)	2B; T&P (5)	2B; Hara (5h)	2B; T&P (5)	2B; Hara (5h)	2B; T&P (5)	2B; Hara (5h)	
N	17	17	17	17	4	4	17	17	9
Bias mean	0.68	0.63	0.78	0.67	0.44	0.44	0.60	0.52	1.22
Standard deviation	0.34	0.33	0.41	0.36	0.27	0.27	0.28	0.25	0.70
COV	0.50	0.52	0.53	0.53	0.62	0.61	0.47	0.48	0.57

MIXED SOILS	α Tomlinson, Nordlund and Thurman		α API, Nordlund, Thurman		β; Thurman		Schmert-mann SPT mobilized
	$\phi \leq 36$;2B; ϕ by Peck; S_u by T&P(5)	$\phi \leq 36$;2B; ϕ by Peck; S_u by Hara (5h)	$\phi \leq 36$;2B; ϕ by Peck; S_u by T&P(5)	$\phi \leq 36$;2B; ϕ by Peck; S_u by Hara (5h)	$\phi \leq 36$;2B; ϕ by Peck; S_u by T&P(5)	$\phi \leq 36$;2B; ϕ by Peck; S_u by Hara (5h)	
N	22	22	37	37	35	35	41
Bias mean	0.53	0.34	0.73	0.55	0.43	0.43	1.03
Standard deviation	0.34	0.22	0.45	0.32	0.30	0.30	0.55
COV	0.64	0.64	0.62	0.58	0.71	0.71	0.53

ALL SOILS	α-Tomlinson and/or Nordlund		α-API and/or Nordlund		β		λ		Meyer hof	Schmert -mann SPT mobilize d
	36;2B;P; T&P(5)	36;2B;P;H ara(5h)	36;2B;P;T &P(5)	36;2B;P;H ara(5h)	36;2B;P;T &P(5)	36;2B;P;H ara(5h)	36;2B;P;T &P(5)	36;2B;P;H ara(5h)		
N	58	58	73	73	58	58	17	17	19	69
Bias mean	0.72	0.64	0.81	0.69	0.55	0.55	0.60	0.52	1.77	1.16
Standard deviation	0.51	0.52	0.51	0.48	0.46	0.46	0.28	0.25	1.21	0.63
COV	0.70	0.81	0.64	0.69	0.83	0.83	0.47	0.48	0.69	0.54

1.5. Analysis and Discussion of Results

Based on the results obtained, the parameters which result in better statistical results, meaning that the COV is smaller and the mean of the bias factor is closer to 1.0, are summarized in the tables below.

Table 9: General observations on the parameters for concrete piles

Soil Type	Generally have better statistical results	Generally have worse statistical results
	ϕ correlated by Peck, Hanson and Thornburn using corrected N-value from SPT.	ϕ correlated by Schmertmann from SPT.
Cohesionless	Tip resistance is calculated based on contributed zone of 8B above tip and 3.5B below tip. $\phi_{avg} = \dfrac{\text{average}\,\phi\ \text{8B above tip} + \text{average}\,\phi\ \text{3.5B below tip}}{2}$	Tip resistance is calculated based on contributed zone of 2B below tip.
	Limit ϕ below 36^0.	Limit ϕ below 40^0.
	S_u correlated by Hara from SPT.	S_u correlated by Terzaghi and Peck from SPT.
Cohesive	Tip resistance is calculated based on contributed zone of 2B below tip.	Tip resistance is calculated based on contributed zone of 8B above tip and 3.5B below tip. $Su_{avg} = \dfrac{\text{average}\,Su\ \text{8B above tip} + \text{average}\,Su\ \text{3.5B below tip}}{2}$

Table 10: General observations on the parameters for pipe piles

Cohesionless--Nordlund method	Limit ϕ below 36 gives smaller COV, but the mean values are significantly higher than 1.0 (conservative--or under-predicted pile capacity).
Cohesive--α (API or Tomlinson), β, λ methods	S_u by Hara gives smaller COV, but the mean values are significantly smaller than 1.0 (unconservative--or over-predicted pile capacity).

Table 11: General observations on the parameters for H piles

Soil Type	Generally have better statistical results	Generally have worse statistical results
Cohesionless	ϕ correlated by Peck, Hanson and Thornburn using corrected N-value from SPT.	ϕ correlated by Schmertmann from SPT.
	Tip resistance is calculated based on contributed zone of 8B above tip and 3.5B below tip.	Tip resistance is calculated based on contributed zone of 2B below tip.
	Limit ϕ below 36^0.	Limit ϕ below 40^0.
	Plugged shape: lower COV, higher Φ	Unplugged shape (even though the bias mean is closer to 1)
Cohesive	S_u by Hara gives smaller COV, but the mean values are significantly smaller than 1.0 (unconservative--or over-predicted pile capacity).	
	Tip resistance is calculated based on contributed zone of 2B below tip.	Tip resistance is calculated based on contributed zone of 8B above tip and 3.5B below tip.
	Plugged shape: closer to 1 and higher bias mean; lower COV; higher Φ	Unplugged shape

Currently, Nordlund, Meyerhof and Schmertmann methods have separate curves or equations for large and small displacement piles. However, the coefficients for α, β, λ and Thurman methods are the same for large and small displacement piles.

Discussion of Results

The following correlations are recommended for the axial capacity prediction:

- Peck, Hanson and Thornburn's equation (cited in Reese et al., 1998) for internal friction angle, ϕ, from corrected blow count, N',
- Limit ϕ below 36^0 for Nordlund method,
- Hara's equation (cited in Kulhawy et al., 1990) for undrained shear strength S_u from uncorrected blow count, N (concrete piles),
- Terzaghi and Peck 's equation (cited in Kulhawy et al., 1990) for undrained shear strength S_u from uncorrected blow count, N (H and pipe piles).

APPENDICES

A. LISTS OF PILES IN THE DATABASE

A.1. Summary of the Driven Pile Database

SOIL TYPE		NUMBER OF CASES		
TIP	SIDE	H-PILES	CONCRETE	PIPE
ROCK	CLAY		0	0
	SAND		0	0
	MIX		15	3
	TOTAL		**15**	**3**
SAND	CLAY		0	0
	SAND		37	20
	MIX		50	19
	TOTAL		**87**	**39**
CLAY	CLAY		19	20
	SAND		1	0
	MIX		34	15
	TOTAL		**54**	**35**
UNKNOWN OR NOT ENOUGH DATA			7	1
ALL CASES			**163**	**78**

A.2. Concrete Piles

			English (kip; ft; in)					SI (KN,m, cm)		
Case #	Data ID	Project Name	Width	Void	Length	Max Load	Max Settl	Width	Void	Length
1	16	SURFRIDER CONDOMINIUM	12.0	-	29.0	200	0.7	30	-	8.8
2	28	KARIDAS CONDOMINIUM #2	12.0	-	8.5	44	0.7	30	-	2.6
3	310	BEACHES OF LONGBOAT	12.0	-	14.0	200	0.3	30	-	4.3
4	411	VIENTA CONDOMINIUM	12.0	-	13.0	180	0.9	30	-	4.0
5	512	APP. BAY BRIDGE BENT 101	24.0	12	62.1	822	0.8	61	30	18.9
6	613	APP. BAY BRIDGE BENT 133	24.0	12	104.9	866	1.4	61	30	32.0
7	714	ARVIDA HOTEL	12.0	-	35.0	240	0.7	30	-	10.7
8	815	VERANDA HOTEL, SARASOTA	12.0	-	18.0	160	0.5	30	-	5.5
9	916	LONGBOAT COVE, SARASOTA	12.0	-	16.0	160	0.8	30	-	4.9
10	117	I-95 WEST PALM BEACH #1	18.0	-	26.5	127	1.0	46	-	8.1
11	118	I-95 WEST PALM BEACH #2	18.0	-	37.2	156	0.6	46	-	11.3
12	119	BLOUNT ISLAND SITE 215	10.0	-	68.0	180	1.6	25	-	20.7
13	120	BLOUNT ISLAND SITE 316	14.0	-	52.0	320	1.1	36	-	15.8
14	122	I-275 34th ST. PINELLAS	18.0	-	69.0	448	1.7	46	-	21.0
15	124	APP. BAY BRIDGE BENT 145	24.0	12	103.0	983	1.4	61	30	31.4
16	126	SIESTA KEY SARASOTA	12.0	-	16.3	200	0.4	30	-	5.0
17	127	DeSOTA CONDOMINIUM MS.	16.0	-	23.8	340	1.5	41	-	7.3
18	128	WASHINGTON CONDOMINIUM	14.0	-	52.5	300	1.3	36	-	16.0
19	130	SUNSHINE SKYWAY SITE 1 A	24.0	-	49.2	1,200	1.8	61	-	15.0
20	231	SUNSHINE SKYWAY SITE 1 B	20.0	-	47.3	600	0.6	51	-	14.4
21	232	SUNSHINE SKYWAY SITE 3	24.0	-	48.0	1,050	0.7	61	-	14.6
22	233	SUNSHINE SKYWAY SITE 10	24.0	-	27.9	1,200	0.7	61	-	8.5
23	234	SUNSHINE SKYWAY SITE 13 A	20.0	-	20.6	600	0.4	51	-	6.3
24	236	ST. JOHN'S RIVER (ASCE)-3B	20.0	-	46.0	596	0.7	51	-	14.0
25	237	ST. JOHN'S RIVER (ASCE) 3C	14.0	-	60.0	374	1.0	36	-	18.3
26	238	ST. AUGUSTINE (ASCE) 4A	12.0	-	28.0	150	0.5	30	-	8.5
27	244	BLOUNT ISLAND TERM. B-20	20.0	-	46.2	600	0.7	51	-	14.1
28	245	FLORENCE/MARION 3 ASD	18.0	-	25.0	500	0.8	46	-	7.6
29	246	FLORENCE / MARION 3 BSD	18.0	-	40.0	272	1.2	46	-	12.2
30	347	FLORENCE / MARION 3 CSD	18.0	-	38.0	296	1.0	46	-	11.6
31	348	NORTHEAST VILLA MIRADA - 6	14.0	-	8.8	100	0.5	36	-	2.7

Case #	Data ID	Project Name	English (kip; ft; in) Width	Void	Length	Max Load	Max Settl	SI (KN,m, cm) Width	Void	Length
32	49	SARASOTA MEM. HOSPITAL	12.0	-	25.0	188	0.7	30	-	7.6
33	51	PORT ORANGE BENT 2 PILE 6	18.0	-	30.1	369	2.2	46	-	9.2
34	52	SEAWAY HOTELS, SAND KEY	14.0	-	29.8	400	0.5	36	-	9.1
35	53	BLOUNT ISLAND TERM. B-21	20.0	-	36.4	496	1.4	51	-	11.1
36	58	HOWARD FRANKLAND / LS3	30.0	-	39.6	2,000	1.2	76	-	12.1
37	63	CHOCTAWHATCHEE P-5	30.0	18	53.9	1,485	1.8	76	46	16.4
38	64	CHOCTAWHATCHEE P-11	30.0	18	85.5	1,508	2.0	76	46	26.1
39	65	CHOCTAWHATCHEE P-17	30.0	18	77.8	1,620	1.8	76	46	23.7
40	66	CHOCTAWHATCHEE P-23	30.0	18	82.5	810	1.7	76	46	25.1
41	67	CHOCTAWHATCHEE P-29	30.0	18	84.4	990	1.7	76	46	25.7
42	68	CHOCTAWHATCHEE P-35	30.0	18	79.0	1,484	2.2	76	46	24.1
43	69	HOWARD FRANK. / LS4 SHORT	30.0	-	24.6	2,000	0.9	76	-	7.5
44	70	CHOCTAWHATCHEE P-41	30.0	18	65.2	1,440	1.3	76	46	19.9
45	72	CHOCTAWHATCHEE FSB-26	24.0	-	87.2	960	1.9	61	-	26.6
46	74	CAPE CANAVERAL T-6	18.0	-	53.1	340	0.8	46	-	16.2
47	75	CAPE CANAVERAL T-7	14.0	-	76.2	410	0.9	36	-	23.2
48	76	CAPE CANAVERAL T-14	14.0	-	69.5	391	2.2	36	-	21.2
49	79	WHITE CITY BRIDGE TP3	24.0	-	37.2	700	0.7	61	-	11.3
50	80	HOWARD FRANK. / LS4 LONG	30.0	-	73.5	1,750	6.8	76	-	22.4
51	83	WHITE CITY BRIDGE TP6	24.0	-	28.5	600	0.9	61	-	8.7
52	86	ACOSTA BRIDGE PEIR F6	24.0	-	58.5	915	2.8	61	-	17.8
53	87	ACOSTA BRIDGE PEIR G13	24.0	-	46.1	1,315	1.3	61	-	14.1
54	88	ACOSTA BRIDGE PEIR H2	24.0	-	35.9	718	1.9	61	-	10.9
55	89	WEST BAY BRIDGE TP9	30.0	18	128.4	955	1.6	76	46	39.1
56	90	WEST BAY BRIDGE TP15	30.0	18	103.6	855	1.4	76	46	31.6
57	92	ESCAMBIA RIVER BENT5	24.0	-	85.7	934	2.1	61	-	26.1
58	94	ROOSEVELT BRIDGE A-	30.0	-	53.4	1,220	1.4	76	-	16.3
59	95	ROOSEVELT BRIDGE B-30-W	30.0	-	43.8	1,040	1.5	76	-	13.4
60	96	BUCKMAN BRIDGE TS-13	30.0	14	94.5	1,180	1.9	76	36	28.8
61	97	BUCKMAN BRIDGE TS-19	30.0	14	89.3	1,481	2.0	76	36	27.2
62	98	BUCKMAN BRIDGE TS-24	30.0	14	80.8	1,168	2.1	76	36	24.6
63	99	BUCKMAN BRIDGE TS-29	30.0	14	80.0	1,400	2.1	76	36	24.4
64	102	APPALACHICOLA RIVER PIER14	30.0	18	58.8	953	0.7	76	46	17.9
65	113	APPALACHICOLA RIVER PIER25	24.0	12	55.5	836	1.2	61	30	16.9

			English (kip; ft; in)					SI (KN,m, cm)		
Case #	Data ID	**Project Name**	Width	Void	Length	Max Load	Max Settl	Width	Void	Length
66	131	MARCO ISLAND TP2	14.0	-	33.0	200	0.4	36	-	10.1
67	134	MARINA BAY CLUB TP7	14.0	-	83.0	279	0.6	36	-	25.3
68	135	APPALACHICOLA BAY BENT 41	24.0	12	52.3	548	1.0	61	30	15.9
69	136	ST. MARISSA CONDO. TP8 & Pile 20	14.0	-	50.0	284	1.0	36	-	15.2
70	140	GEORGIA/FLORIDA BOUNDARY	10.0	-	43.0	150	1.1	25	-	13.1
71	141	JACKSONVILLE SITE B	14.0	-	33.0	368	0.9	36	-	10.1
72	142	JACKSONVILLE SITE D	14.0	-	62.0	486	0.9	36	-	18.9
73	143	SAINT JOHN RIVER SITE F	18.0	-	35.0	300	0.6	46	-	10.7
74	145	LONGBOAT KEY - SARASOTA	12.0	-	49.1	200	0.5	30	-	15.0
75	146	SUNSHINE SKYWAY SITE 13 B	24.0	-	26.9	1,200	1.6	61	-	8.2
76	497	116 GRL Piles-164/Cimaron Rvr Br, OK	19.9	-	64.3	800	2.7	51		19.6
77	498	116 GRL Piles-164/Cimaron Rvr Br, OK	24.0	-	63.3	1,778	2.7	61		19.3
78	502	BRIDGE SITE 3046A, Hinds, MS	16.0	-	27.0	144	1.1	41		8.2
79	503	BRIDGE SITE 3046B, Hinds, MS	16.0	-	37.0	264	1.1	41		11.3
80	514	Dist. 08 P 455-03-04, 455-05-03 TP33,	16.0	-	80.0	194	0.9	41		24.4
81	515	Dist. 61 P 50-05-15 TP1, Ascension, LA	16.0	-	87.0	228	0.7	41		26.5
82	516	Dist. 61 P 742-01-39, East Baton, LA	14.0	-	70.0	170	0.7	36		21.3
83	517	118 GRL Piles Bailey Fork, Bailey, TN	14.0	-	45.0	300	0.6	36		13.7
84	518	119 GRL Piles-White City Bridge, FL	24.0	-	40.0	600	0.9	61		12.2
85	519	119 GRL Piles-White City Bridge, FL	24.0	-	40.0	994	1.1	61		12.2
86	520	123 GRL Piles-Dawhoo River Bridge, SC	16.0	-	80.0	1,400	6.4	41		24.4
87	521	123 GRL Piles-Dawhoo River Bridge, SC	24.0	4	90.0	698	7.4	61	10	27.4
88	522	124 GRL Piles-Socastee W. Way Br, SC	21.8	-	85.0	1,520	1.4	55		25.9
89	523	125 GRL Piles-Doughty St Prk Gar, SC	12.0	-	91.0	367	1.1	31		27.7
90	524	126 GRL Piles-Battery Creek, SC	24.0	-	81.5	1,200	1.1	61		24.8
91	525	126 GRL Piles-Battery Creek, SC	24.0	-	66.5	570	1.1	61		20.3
92	526	128 GRL Piles-Howard Franklin Br, FL	24.0	-	85.6	1,001	1.0	61		26.1
93	529	204 GRL Piles-C&D Canal, Pier 17, DE	24.0	-	75.0	1,200	1.9	61		22.9
94	531	99 GRL Piles I-165/Water St Int, AL	18.0	-	77.0	900	1.1	46		23.5
95	532	99 GRL Piles I-165/Water St Int, AL	36.0	-	74.0	600	1.1	92		22.6
96	533	99 GRL Piles I-165/Water St Int, AL	18.0	-	67.0	1,200	1.2	46		20.4
97	534	99 GRL Piles I-165/Water St Int, AL	24.0	-	77.0	430	1.1	61		23.5
98	535	99 GRL Piles I-165/Water St Int, AL	24.0	-	67.0	700	1.2	61		20.4
99	536	Axial Pile-Mission Avenue, Viaduct, CA	14.0	-	17.4	261	0.9	36		5.3

Case #	Data ID	Project Name	English (kip; ft; in) Width	Void	Length	Max Load	Max Settl	SI (KN,m, cm) Width	Void	Length
100	537	Axial Pile-Mission Avenue, Viaduct, CA	14.0	-	34.0	288	1.0	36		10.4
101	538	Axial Pile-Mission Avenue, Viaduct, CA	14.0	-	24.0	235	1.0	36		7.3
102	539	Doheny Park Rd U.C. Sta 451+85.5, CA	12.0	-	56.3	280	1.0	31		17.1
103	540	LOAD TRANSFER #35-3, OK	24.0	-	57.0	1,770	2.7	61		17.4
104	542	West Seattle Fwy Harbor Island, WA	19.9	-	86.0	900	1.6	51		26.2
105	571	BRIDGE SITE 1067, HINDS, MS	18.0	-	30.5	300	1.0	46		9.3
106	572	BRIDGE SITE 1068, HINDS, MS	14.0	-	30.0	180	1.2	36		9.1
107	573	BRIDGE SITE 1069, HINDS, MS	14.0	-	39.6	330	1.2	36		12.1
108	575	BRIDGE SITE 1072A, HINDS, MS	18.0	-	28.5	240	1.0	46		8.7
109	577	BRIDGE SITE 3024, HINDS, MS	14.0	-	36.5	318	1.5	36		11.1
110	580	Dist. 02 P 7-03-40 TP5, St. Charles LA	24.0	-	110.0	644	0.9	61		33.5
111	583	Dist. 02 P 855-14-7 and P 855-14-5 , LA	14.0	-	110.0	250	0.9	36		33.5
112	587	Dist. 03 P 455-02-04 TP2, St. Landry LA	24.0	-	65.0	440	0.9	61		19.8
113	1001	Luling Bridge; TP2	54.0	44	81.1	717	0.8	137	112	24.7
114	1002	Luling Bridge; TP3; Circular void	24.0	12	82.6	419	0.7	61	30	25.2
115	1003	Luling Bridge; TP4; Circular void	30.0	19	82.5	525	0.8	76	47	25.2
116	1004	Luling Bridge; TP5	30.0	19	83.0	555	0.6	76	47	25.3
117	1005	Luling Bridge; TP6	36.0	26	82.5	547	1.0	91	66	25.2
118	1006	Luling Bridge; TP7	36.0	26	81.8	541	0.8	91	66	24.9
119	1007	Orlando International Airport; D22	14.0	-	90.0	842	1.2	36	-	27.4
120	1162	Site 33, Pile 3, Reinforced Concrete	12.0	-	114.3	298	0.5	31		34.8
121	1163	Site 33, Pile 4, Reinforced Concrete	12.0	-	54.5	298	0.3	31		16.6
122	1175	Site 35, Pile 10, Reinforced Concrete	12.0	-	48.0	511	3.2	31		14.6
123	1260	Jacksonville - Industrial zone # 1	20.0	-	46.0	581	0.6	51	-	14.0
124	1261	Jacksonville - Industrial # 2	20.0	-	36.0	495	1.3	51	-	11.0
125	1262	Ft Myers	14.0	-	67.0	280	1.5	36	-	20.4
126	1263	Apalachicola River Bridge - Pier 3	24.0	12	90.6	95	0.0	61	30	27.6
127	1264	Apalachicola Bay Bridge - Bent 22	18.0	-	64.0	422	0.6	46	-	19.5
128	1265	Apalachicola Bay Bridge - Bent 16	18.0	-	61.0	349	0.9	46	-	18.6
129	1267	Port Orange - Bent 19	18.0	-	30.9	296	1.9	46	-	9.4
130	2344	Choctahatchee Bay, FL3	24.0	-	77.7	458	1.7	61	-	23.7
131	2345	Choctahatchee Bay, FL26	24.0	-	64.8	314	1.0	61	-	19.8
132	3162	065-90-0024_and_855-04-0046_ Tp1	14.0	-	80.0	270	1.2	36	-	24.4

Case #	Data ID	Project Name	English (kip; ft; in) Width	Voi d	Length	Max Load	Max Settl	SI (KN,m, cm) Width	Void	Length
133	3163	065-90-0024_and_855-04-0046_ Tp2	14.0	-	70.0	110	2.1	36		21.3
134	3164	065-90-0024_and_855-04-0046_ TP3	14.0	-	80.0	250	1.4	36	-	24.4
135	3165	065-90-0024_and_855-04-0046_ TP4	14.0	-	81.0	240	1.1	36	-	24.7
136	3166	065-90-0024_and_855-04-0046_ TP5	16.0	-	71.5	220	2.4	41	-	21.8
137	3170	260-05-0020_Tickfaw_River_; TP1	30.0	-	59.3	984	4.8	76	-	18.1
138	3173	262-06-09_Tickfaw_River_ #1; TP1	24.0	-	84.9	480	1.0	61	-	25.9
139	3174	262-06-09_Tickfaw_River_ #1; TP2	24.0	-	105.0	540	0.7	61	-	32.0
140	3176	283-09-52_New_Orleans	18.0	-	125.0	362	0.3	46		38.1
141	3178	424-05-0078_Bayou_Boeuf_Main_; TP1	14.0	-	70.0	330	0.6	36	-	21.3
142	3179	424-05-0078_Bayou_Boeuf_Main_; TP2	14.0	-	70.0	232	2.0	36	-	21.3
143	3180	424-05-0078_Bayou_Boeuf_Main_; TP5	14.0	-	80.0	250	0.9	36	-	24.4
144	3181	424-05-0081_Bayou_Boeuf_West_; TP1	14.0	-	89.5	230	0.5	36		27.3
145	3182	424-05-0081_Bayou_Boeuf_West_; TP2	30.0	-	110.0	640	1.3	76	-	33.5
146	3183	424-05-0081_Bayou_Boeuf_West_; TP3	14.0	-	63.5	330	0.6	36	-	19.4
147	3184	424-05-0081_Bayou_Boeuf_West_; TP4	16.0	-	70.0	200	0.5	41	-	21.3
148	3185	424-05-0087_Bayou_Ramos_ TP1	16.0	-	78.0	280	2.2	41	-	23.8
149	3186	424-05-0087_Bayou_Ramos_ TP2	30.0	-	88.0	1,050	1.8	76	-	26.8
150	3187	424-05-0087_Bayou_Ramos_ TP3	30.0	-	104.0	1,000	3.1	76	-	31.7
151	3188	424-05-0087_Bayou_Ramos_ TP4	30.0	-	99.3	1,200	3.0	76	-	30.3
152	3189	424-05-0087_Bayou_Ramos_ TP5	30.0	-	113.0	1,150	1.9	76	-	34.4
153	3191	424-05-0087_Bayou_Ramos_ TP7	16.0	-	77.0	230	1.6	41	-	23.5
154	3192	424-06-0005_Bayou_Boeuf_East_ F1;	14.0	-	68.0	210	1.6	36	-	20.7
155	3193	424-06-0005_Bayou_Boeuf_East_ F2;	14.0	-	71.0	195	1.1	36	-	21.6
156	3194	424-06-0005_Bayou_Boeuf_East_ F3	14.0	-	77.5	200	2.0	36	-	23.6
157	3195	424-06-0005_Bayou_Boeuf_East_ F4	14.0	-	79.0	240	1.1	36	-	24.1
158	3196	424-06-0005_Bayou_Boeuf_East_ F5	14.0	-	79.0	180	0.8	36	-	24.1
159	3197	424-06-0005_Bayou_Boeuf_East_ F6	30.0	-	110.0	640	2.6	76	-	33.5
160	3201	424-07-0009_Gibson_Raceland_ TP1	30.0	-	116.0	1,344	4.4	76	-	35.4
161	3203	424-07-0009_Gibson_Raceland_ TP4	30.0	-	124.0	1,312	3.1	76	-	37.8
162	3208	450-366-02_Luling_Bridge	30.0	-	112.0	1,000	2.0	76	-	34.1
163	3215	855-14-13_Houma	18.0	-	105.0	434	1.0	46	-	32.0

A.3. Pipe Piles

				English (kip; ft; in)					SI (KN,m, cm)		
Case #	ID in Databa	**Project Name**	End Con.	Dia	Thick	Length	Max Load	Max Settl	Dia	Thick	Length
1	197	HOUSTON, TEXAS	Plugged	10.8	0.19	51.3	168	0.4	27.3	0.48	15.6
2	198	HUNTER'S POINT	Closed End	10.8	0.19	30.0	98	1.0	27.3	0.48	9.14
3	199	ST. LOUIS, MISSOURI	Plugged	14.0	0.22	47.2	250	1.2	35.6	0.56	14.4
4	200	ST. LOUIS, MISSOURI	Plugged	16.0	0.19	47.8	400	2.9	40.6	0.48	14.6
5	204	DELAWARE L.R. 795-B3	Plugged	18.0	0.22	89.0	530	1.6	45.7	0.56	27.1
6	205	NW CONN. OC-RETROFIT	Plugged	16.0	0.19	66.5	520	1.0	40.6	0.48	20.3
7	206	NW CONN. OC RETROFIT #2	Plugged	16.0	0.19	56.5	402	1.5	40.6	0.48	17.2
8	207	SOUTHERN FREEWAY, CA	Plugged	16.0	0.19	41.1	800	1.0	40.6	0.48	12.5
9	208	HAMILTON BAYFRONT,CAN	Closed End	12.7	0.37	110.0	1,191	2.4	32.4	0.95	33.5
10	209	HAMILTON BAYFRONT #2	Closed End	12.7	0.37	83.0	899	1.6	32.4	0.95	25.3
11	210	HAMILTON BAYFRONT #3	Closed End	12.7	0.37	60.0	674	2.8	32.4	0.95	18.3
12	478	101 GRL Piles-Fore, ME	Closed End	18.0	0.50	71.5	600	1.6	45.7	1.27	21.8
13	479	101 GRL Piles-Fore Portland, ME	Closed End	18.0	0.50	59.7	400	1.1	45.7	1.27	18.2
14	480	101 GRL Piles-Fore Portland, ME	Closed End	18.0	0.50	120.0	540	1.3	45.7	1.27	36.6
15	481	110 GRL Piles-Peosta, IA	Closed End	14.0	0.50	80.0	420	1.3	35.6	1.27	24.4
16	482	110 GRL Piles Peosta, IA	Closed End	14.0	0.50	100.0	666	0.9	35.6	1.27	30.5
17	483	116 GRL Piles-164/Cimaron, OK	Closed End	26.0	0.75	63.3	800	3.1	66	1.91	19.3
18	484	LOAD TRANSFER #35-1, CA	Closed End	10.7	0.37	30.0	109	1.6	27.3	0.93	9.14
19	485	LOAD TRANSFER #35-2,	Closed End	10.7	0.37	18.5	116	2.2	27.3	0.93	5.64
20	486	117 GRL Piles-St Rte 115 MO	Closed End	14.0	0.37	62.0	260	1.0	35.6	0.95	18.9
21	487	117 GRL Piles-St Rte 115 MO	Closed End	14.0	0.37	86.5	340	2.2	35.6	0.95	26.4
22	488	124 GRL Piles-Socastee W., SC	Closed End	24.0	0.50	85.0	701	2.4	61	1.27	25.9
23	489	130 GRL Piles-Jones Island, WI	Closed End	9.6	0.54	166.2	580	2.2	24.5	1.38	50.7
24	490	130 GRL Piles-Jones Island, WI	Closed End	12.8	0.31	161.3	540	2.0	32.4	0.79	49.2
25	491	130 GRL Piles-Jones Island, WI	Closed End	12.8	0.31	161.0	694	2.8	32.4	0.79	49.1
26	492	130 GRL Piles-Jones Island, WI	Closed End	12.8	0.31	140.4	691	2.7	32.4	0.79	42.8
27	493	130 GRL Piles-Jones Island, WI	Closed End	9.6	0.54	154.6	380	1.6	24.5	1.38	47.1
28	494	130 GRL Piles-Jones Island, WI	Closed End	12.8	0.31	165.9	415	3.6	32.4	0.79	50.6

				English (kip; ft; in)					SI (KN,m, cm)		
Case #	ID in Databa	**Project Name**	End Con.	Dia	Thick	Length	Max Load	Max Settl	Dia	Thick	Length
29	495	130 GRL Piles-Jones Island, WI	Closed End	9.6	0.54	145.0	657	2.1	24.5	1.38	44.2
30	496	130 GRL Piles-Jones Island, WI	Closed End	9.6	0.54	155.4	657	2.4	24.5	1.38	47.4
31	563	Bayshore Fwy Viaduct Site F, CA	Unplugged	24.0	0.75	73.0	900	1.2	61	1.91	22.3
32	566	Ventura Underpass, CA	Unplugged	12.0	0.13	39.1	170	1.6	30.5	0.34	11.9
33	1008	Orlando International Airport; D23	Closed End	12.7	0.25	83.3	514	1.1	32.4	0.63	25.4
34	1009	Belleville; LPT 1	Unplugged	12.0	0.25	44.4	210	14.5	30.5	0.63	13.5
35	1010	Belleville; LPT4; pre excavate	Closed End	12.0	0.25	66.5	790	1.3	30.5	0.63	20.3
36	1011	Belleville; LPT5; pre excavate	Closed End	12.0	0.18	66.7	800	1.9	30.5	0.45	20.3
37	1013	Detroit; LPT1	Unplugged	12.0	0.18	69.5	64	10.8	30.5	0.45	21.2
38	1014	Detroit; LPT2	Closed End	12.0	0.18	78.6	490	5.3	30.5	0.45	24
39	1016	Detroit LTP10	Closed End	12.0	0.23	81.0	550	6.3	30.5	0.58	24.7
40	1037	Site 2, Pile 4; ARMCO	Closed End	12.0	0.14	24.5	281	0.4	30.5	0.36	7.47
41	1098	Site 2, Pile 5, Armco Steel Tube	Closed End	12.0	0.17	19.0	214	2.0	30.5	0.44	5.79
42	1099	Site 4, Pile 2, Armco Steel Tube	Closed End	12.8	0.19	118.0	160	3.1	32.4	0.48	36
43	1113	Site 9, Pile 5, Steel Tube	Closed End	12.8	0.25	70.0	399	0.4	32.4	0.63	21.3
44	1117	Site 13, Pile 19, Steel Tube	Closed End	12.8	0.25	64.8	326	0.7	32.4	0.63	19.7
45	1119	Site 14, Pile 2, Steel Tube	Closed End	12.8	0.20	60.0	67	1.1	32.4	0.5	18.3
46	1120	Site 14, Pile 3, Steel Tube	Closed End	12.8	0.25	95.0	298	0.9	32.4	0.63	29
47	1129	Site 22, Pile 3, Steel Tube	Closed End	12.8	0.20	50.2	62	2.0	32.4	0.52	15.3
48	1130	Site 22, Pile 4, Steel Tube	Closed End	12.8	0.25	98.9	274	3.7	32.4	0.63	30.1
49	1131	Site 22, Pile 5, Steel Tube	Closed End	12.8	0.20	50.1	73	3.7	32.4	0.52	15.3
50	1132	Site 23, Pile 2, Steel Tube	Closed End	12.8	0.25	9.9	121	1.4	32.4	0.63	3.02
51	1134	Site 24, Pile 2, Steel Tube	Closed End	12.8	0.20	50.5	250	8.1	32.4	0.52	15.4
52	1135	Site 24, Pile 3, Steel Tube	Closed End	12.8	0.20	73.5	253	9.6	32.4	0.52	22.4
53	1138	Site 25, Pile 1, Steel Tube	Closed End	12.8	0.25	18.5	79	1.9	32.4	0.63	5.64
54	1140	Site 25, Pile 5, Steel Tube	Closed End	12.8	0.25	60.2	174	1.6	32.4	0.63	18.4
55	1141	Site 25, Pile 6, Steel Tube	Closed End	12.8	0.25	30.4	114	1.1	32.4	0.63	9.27
56	1143	Site 26, Pile 1, Steel Tube	Closed End	12.8	0.25	40.0	29	1.9	32.4	0.63	12.2
57	1144	Site 26, Pile 4, Steel Tube	Closed End	12.8	0.25	100.0	279	1.9	32.4	0.63	30.5
58	1145	Site 26, Pile 5, Steel Tube	Closed End	12.8	0.25	140.0	298	2.0	32.4	0.63	42.7

				English (kip; ft; in)					SI (KN,m, cm)		
Case #	ID in Databa	**Project Name**	End Con.	Dia	Thick	Length	Max Load	Max Settl	Dia	Thick	Length
59	1155	Site 28, Pile 7, Steel Tube	Closed End	12.8	0.25	20.0	160	1.9	32.4	0.63	6.1
60	1156	Site 28, Pile 8, Steel Tube	Closed End	12.8	0.25	60.0	171	1.9	32.4	0.63	18.3
61	1157	Site 28, Pile 9, Steel Tube	Closed End	12.8	0.25	39.5	146	1.3	32.4	0.63	12
62	1158	Site 30, Pile 1, Steel Tube	Closed End	12.8	0.25	131.5	466	1.2	32.4	0.63	40.1
63	1159	Site 30, Pile 2, Steel Tube	Closed End	12.8	0.25	130.7	478	1.2	32.4	0.63	39.8
64	1161	Site 33, Pile 2, Steel Tube	Closed End	12.8	0.25	107.2	585	3.1	32.4	0.63	32.7
65	1172	Site 35, Pile 4, Steel Tube	Closed End	12.8	0.25	48.2	377	2.9	32.4	0.63	14.7
66	1174	Site 35, Pile 6, Steel Tube	Closed End	12.8	0.25	90.0	596	2.7	32.4	0.63	27.4
67	1187	Site 38, Pile 4, Steel Tube	Closed End	12.8	0.37	39.0	247	1.8	32.4	0.95	11.9
68	1188	Site 38, Pile 5, Steel Tube	Closed End	12.8	0.37	52.8	585	3.7	32.4	0.95	16.1
69	1190	Site 39, Pile 3, Steel Tube	Closed End	12.8	0.37	83.3	337	1.9	32.40	0.95	25.4
70	1192	Site 40, Pile 3, Steel Tube	Closed End	12.8	0.37	56.4	268	1.6	32.40	0.95	17.2
71	1195	Site 41, Pile 3, Steel Tube	Closed End	12.8	0.37	52.5	402	0.5	32.40	0.95	16.0
72	2164	Deer Island Pier Facilities; LT-1	Closed End	14.0	0.50	87.0	540	1.0	35.56	1.27	26.5
73	2165	Deer Island Pier Facilities; LT-2	Closed End	14.0	0.50	54.0	540	0.7	35.56	1.27	16.5
74	2166	Deer Island Pier Facilities; LT-3	Closed End	14.0	0.50	89.0	540	1.1	35.56	1.27	27.1
75	2167	Deer Island Pier; L1-2; LT-4	Closed End	14.0	0.50	83.0	648	0.9	35.56	1.27	25.3
76	2168	Deer Island Pier; P-14; LT-5	Closed End	14.0	0.50	88.0	528	2.1	35.56	1.27	26.8
77	2170	Deer Island Pier; P-11; LT-7	Closed End	14.0	0.50	89.0	528	2.6	35.56	1.27	27.1
78	2171	Deer Island Pier; P-12; LT-8	Closed End	14.0	0.50	96.4	720	1.1	35.56	1.27	29.4

B. SUB-CATEGORY OF CONCRETE PILES

Table 12. Total Cases of Static Load Tests on Concrete Piles in Soils

Soil Type	S[1]	ST[2]	Total Number of Cases	
Cohesionless soils	4	33	**37**	
Cohesive soils	14	5	**19**	
Mixed soils	40	45	**85**	
Sub Total for Soils			**141**	
Soils and Rocks	0	15	**15**	
Total	**58**	**98**	**156**	
NA [3]			**7**	**163**

[1] S: Predominant side resistance (Friction piles)
[2] ST: Side and tip resistance
[3] NA: Soil type is unknown/ or lack information to predict capacities

Table 13. The Data for Side Resistance (S) Concrete Piles

Soil Type	SPT	CPT	Lab	Overlap SPT, CPT & Lab	**Total**	OCR and Dr correlated from i situ
cohesionless soils	4	0	0	0	**4**	4
cohesive soils	0	6	9	1	**14**	6
mixed soils	17	25	0	2	**40**	40
Mixed soils and Rocks	0	0	0	0	**0**	0
Total	**21**	**31**	**9**	**3**	**58**	**50**

Table 14. The Data for Side and Tip Resistance (ST) Concrete Piles

Soil Type	SPT	CPT	Lab	Overlap SPT, CPT & Lab	**Total**	OCR and Dr correlated from i situ
cohesionless soils	33	2	0	2	**33**	33
cohesive soils	2	0	3	0	**5**	2
mixed soils	42	7	0	4	**45**	45
Mixed soils and Rocks	**15**	**0**	**0**	**0**	**15**	0
Total	**92**	**9**	**3**	**6**	**98**	**80**

Table 15. The Data for All Concrete Piles

Soil Type	SPT	CPT	Lab	Overlap SPT, CPT & Lab	**Total**	OCR and Dr correlated from i situ
cohesionless soils	37	2	0	2	**37**	37
cohesive soils	2	6	12	1	**19**	8
mixed soils	59	32	0	6	**85**	85
Sub total of Soils	**74**	**40**	**12**	**9**	**141**	**130**
Mixed soils and Rocks	15	-	-	-	**15**	-
Total	**113**				**156**	

B.1. Side Resistance Only

TABLE 16. COHESIVE SOILS: Summary of Populations by Analysis Method used to Obtain *Side Resistance Only for Concrete Piles*

	Analysis Method				
	αAPI	α Tomlinson	λ	β	Schmertmann CPT
Number of cases	14	14	14	6	6

TABLE 17. MIXED SOILS: Summary of Populations by Analysis Method used to Obtain *Side Resistance Only for Concrete Piles*

Soil Type	Analysis Method				
cohesionless	Nordlund	Nordlund	β	Schmertmann SPT	Schmertmann CPT
cohesive	αAPI	α Tomlinson	β	Schmertmann SPT	Schmertmann CPT
Number of cases	40	22	40	17	25

Note 1: COHESIONLESS SOILS: Population is 4 for Nordlund, β and Schmertmann SPT methods (Not enough for statistical analysis)

TABLE 18. ALL SOILS: Analysis methods for Concrete Piles with *Side Resistance Only*

Soil Type	Analysis Method					
cohesionless	Nordlund	Nordlund	β	Schmertmann SPT	Schmertmann CPT	
cohesive	αAPI	α Tomlinson	β	Schmertmann SPT	Schmertmann CPT	λ
Number of cases	58	40	50	21	31	14

Note 2: Numbers are referred from Table 22, 23 and the constraints in Table 4.

B.2. Total Capacity

TABLE 19. COHESIONLESS SOILS: Summary of Populations by Analysis Method used to Obtain *Total Capacity for Concrete Piles*

Capacity	Analysis Method				
Side	Nordlund	β	Meyerhof	Schmertmann SPT	Schmertmann CPT
Tip	Thurman	Thurman	Meyerhof	Schmertmann SPT	Schmertmann CPT
Number of cases	37	37	37	37	2

TABLE 20. COHESIVE SOILS: Summary of Populations by Analysis Method used to Obtain *Total Capacity for Concrete Piles*

Capacity	Analysis Method					
Side	αAPI	α Tomlinson	λ	β	Schmertmann SPT	Schmertmann CPT
Tip	9 Su	9 Su	9 Su	9 Su	Schmertmann SPT	Schmertmann CPT
Number of cases	19	19	19	8	2	6

TABLE 21. MIXED SOILS: Summary of Populations by Analysis Method used to Obtain *Total Capacity for Concrete Piles*

<table>
<tr><th>Capacity</th><th>Soil Type</th><th colspan="7">Analysis Method</th></tr>
<tr><td rowspan="2">Side</td><td>cohesionless</td><td colspan="2">Nordlund</td><td colspan="2">Nordlund</td><td colspan="2">β</td><td rowspan="4">Schmertmann CPT</td></tr>
<tr><td>cohesive</td><td colspan="2">αAPI</td><td colspan="2">α Tomlinson</td><td colspan="2">β</td></tr>
<tr><td rowspan="2">Tip</td><td>cohesionless</td><td>Thurman</td><td></td><td>Thurman</td><td></td><td>Thurman</td><td></td></tr>
<tr><td>cohesive</td><td></td><td>9 Su</td><td></td><td>9 Su</td><td></td><td>9 Su</td></tr>
<tr><td colspan="2" rowspan="2">Number of cases</td><td>50</td><td>35</td><td>0</td><td>34</td><td>50</td><td>35</td><td rowspan="2">32</td></tr>
<tr><td colspan="2">85</td><td colspan="2">34</td><td colspan="2">85</td></tr>
</table>

TABLE 22. MIXED SOILS and ROCKS: Summary of Populations by Analysis Method used to Obtain *Total Capacity for Concrete Piles*

Capacity	Soil Type	Analysis Method	
Side	cohesionless	Schmertmann SPT	
	cohesive	Schmertmann SPT	
Tip	cohesionless	Schmertmann SPT	
	cohesive	Schmertmann SPT	
Number of cases		rock exists	no rock
		15	59
		74	

TABLE 23. ALL SOILS: Summary of Populations by Analysis Method used to Obtain *Total Capacity for Concrete Piles*

<table>
<tr><th>Capacity</th><th>Soil Type</th><th colspan="7">Analysis Method</th></tr>
<tr><td rowspan="2">Side</td><td>cohesionless</td><td colspan="2">Nordlund</td><td colspan="2">Nordlund</td><td colspan="2">β</td><td rowspan="4">Schmertmann CPT</td></tr>
<tr><td>cohesive</td><td colspan="2">αAPI</td><td colspan="2">α Tomlinson</td><td colspan="2">β</td></tr>
<tr><td rowspan="2">Tip</td><td>cohesionless</td><td>Thurman</td><td></td><td>Thurman</td><td></td><td>Thurman</td><td></td></tr>
<tr><td>cohesive</td><td></td><td>9 Su</td><td></td><td>9 Su</td><td></td><td>9 Su</td></tr>
<tr><td colspan="2" rowspan="2">Number of cases</td><td>87</td><td>54</td><td>37</td><td>53</td><td>87</td><td>43</td><td rowspan="2">40</td></tr>
<tr><td colspan="2">141</td><td colspan="2">90</td><td colspan="2">130</td></tr>
</table>

TABLE 24. ALL SOILS and ROCKS: Summary of Populations by Analysis Method used to Obtain *Total Capacity for Concrete Piles*

Capacity	Soil Type	Analysis Method	
Side	cohesionless	Schmertmann SPT	
	cohesive	Schmertmann SPT	
Tip	cohesionless	Schmertmann SPT	
	cohesive	Schmertmann SPT	
Number of cases		rock exists	no rock
		15	98
		113	

Note: Numbers are referred from Tables 24; 29 to 33 and the constraints in Table 4.

C. AXIAL CAPACITIES--CONCRETE PILES

Table 25: Axial capacities (KN): Concrete Piles in Cohesionless soils

				Nordlund								
Case #	situ ID#	lab ID#	Divisson	DRIVEN	40;2B;P (1)	40;11.5B; P (2)	40;2B;Sc h (3)	40;11.5B; Sch (4)	36;2B;P (5)	36;11.5B; P (6)	36;2B;Sc h (7)	36;11.5B; Sch (8)
1	1		805	1,001	1,007	1,308	1,974	1,714	753	861	1,000	953
2	1		140	399	399	208	477	465	218	178	224	224
3	1		890	757	757	604	810	810	374	350	394	394
4	1		525	663	663	663	669	669	320	320	320	320
7	1		925	1,924	1,924	1,506	2,039	2,015	1,030	996	1,047	1,047
8	1		682	782	782	711	803	803	391	391	391	391
9	1		525	932	932	932	951	951	460	460	462	462
10	1		500	451	445	451	505	611	445	451	430	537
11	1		618	574	574	574	871	763	574	574	773	666
12	1		692	737	737	710	1,643	1,463	712	685	1,105	1,068
13	1		1,310	787	787	787	1,136	867	781	781	933	664
16	1		870	889	889	889	895	895	433	433	434	434
24	1		2,570	7,741	7,741	7,248	8,742	8,742	4,202	4,202	4,427	4,427
25	1		1,540	1,589	1,589	1,640	3,148	2,924	1,415	1,456	1,977	1,846
26	1		600	528	528	374	800	396	422	307	373	238
27	1		2,600	8,080	8,080	7,460	9,075	8,958	4,303	4,303	4,529	4,529
32	1		670	1,137	1,137	736	1,325	1,207	612	523	679	679
33	1		1,235	1,636	1,636	1,787	3,878	3,875	1,545	1,649	1,935	1,935
35	1		1,650	3,016	3,016	2,981	5,220	4,698	2,573	2,276	2,612	2,538
46	1		1,405	5,188	5,188	2,336	6,382	3,969	3,094	2,025	3,489	2,477
56	1		3,730	8,827	8,827	8,827	10,995	10,995	8,827	8,827	10,039	10,039
57	1		3,800	7,763	7,763	7,856	10,105	10,501	7,763	7,856	9,118	9,490
58	2		4,010	11,329	11,329	6,191	16,359	10,264	7,635	5,240	8,163	6,452
59	1		3,450	11,739	11,739	5,817	12,132	5,929	6,200	3,766	6,093	3,110
61	1		4,820	11,302	11,302	11,479	21,170	18,591	10,147	10,102	15,329	13,602
66	1		890	658	658	585	1,899	1,383	631	558	1,087	1,017
83	1		1,200	2,781	2,781	2,781	2,952	2,952	1,459	1,459	1,548	1,548
94	3		3,820	8,469	8,469	6,964	10,555	9,988	5,021	4,846	5,869	5,869
95	1		2,490	15,026	14,860	15,939	31,616	29,634	13,455	12,712	15,869	15,869
96	2		5,100	7,951	7,951	6,946	9,094	8,528	4,239	4,223	4,917	4,917
97	1		1,670	12,901	12,901	9,354	17,249	16,292	7,757	7,346	9,179	9,179
98	1		2,620	5,548	5,548	5,767	14,102	13,453	5,428	5,555	7,467	7,467
101	1		970	494	494	503	1,661	1,644	494	503	942	942
103	1		7,250	10,430	10,430	7,295	12,114	10,959	5,363	4,504	6,429	6,429
122	1		1,800	1,973	1,973	1,634	2,827	2,574	1,274	1,270	1,325	1,325
123	1		2,510	4,950	4,810	4,953	7,242	7,242	3,553	3,553	3,685	3,685
123	3		2,510									
124	1		1,640	2,235	2,235	2,547	5,153	4,312	1,885	1,942	2,584	2,560
124	2		1,640									

(cont.): Axial capacities (KN): Concrete Piles in Cohesionless soils

Case #	situ ID#	lab ID#	β 40;2B;P (1)	β 36;2B;P (5)	Meyerhof	Schmertmann SPT mobilized	Schmertmann CPT
1	1		724	724	837	1,074	
2	1		386	211	960	290	
3	1		710	367	2,233	577	
4	1		602	312	2,795	774	
7	1		1,668	974	2,903	1,157	
8	1		701	372	2,915	754	
9	1		838	446	2,827	902	
10	1		345	345	570	214	
11	1		431	431	790	301	
12	1		725	725	987	547	
13	1		512	512	917	637	
16	1		794	423	3,261	964	
24	1		5,918	3,245	9,872	3,334	
25	1		1,098	1,098	2,372	1,136	
26	1		430	357	639	355	
27	1		6,138	3,314	13,333	3,028	
32	1		1,088	599	490	723	
33	1		1,135	1,135	2,343	1,142	
35	1		2,048	2,037	3,314	1,977	
46	1		4,553	2,569	4,833	2,016	
56	1		6,252	6,252	3,191	2,618	
57	1		5,521	5,521	3,255	2,263	
58	2		9,416	6,144	7,105	2,631	
59	1		10,370	5,160	4,221	1,822	
61	1		6,859	6,859	9,594	4,162	
66	1		589	589	1,510	584	
83	1		2,329	1,322	5,262	1,542	
94	3		6,487	3,793	7,388	3,502	
95	1		11,127	10,011	18,615	6,792	
96	2		6,033	3,339	4,557	2,098	
97	1		10,453	5,663	12,526	4,718	
98	1		3,968	3,968	4,418	2,741	
101	1		459	459	930	595	
103	1		8,879	4,489	7,384	2,603	
122	1		1,778	1,199	987	775	
123	1		3,436	2,736	5,265	2,839	
123	3						3,211
124	1		1,374	1,374	2,669	1,736	
124	2						3,703

Table 26. Axial capacities (KN): Concrete Piles in Cohesive soils

Case #	situ ID#	lab ID#	Davisson	α API revised					α Tomlinson				
				APILE	2B; T&P (1)	11.5B; T&P (2)	2B; Hara (5h)	11.5B; Hara (6h)	DRIVEN	2B; T&P (1)	11.5B; T&P (2)	2B; Hara (5h)	11.5B; Hara (6h)
54	1		2,550	864	833	825	1,620	1,535	367	683	675	1,170	1,085
78	1	1	560	687	711	631	723	643	372	415	334	415	334
80	1	1	860	1,511	1,319	1,442	1,641	1,883	3,578	1,898	2,022	2,157	2,399
81		1	950	1,100	1,109	1,103	1,109	1,103	3,336	1,239	1,233	1,239	1,233
82	1	1	735	865	878	840	1,145	1,069	2,929	1,008	971	1,157	1,081
99	2		1,060	321	344	318	669	624	440	441	415	642	597
105		1	1,230	1,350	1,315	1,196	1,315	1,196	1,259	1,342	1,222	1,342	1,222
106		1	722	828	831	791	831	791	304	441	401	441	401
108		1	965	1,160	1,143	1,120	1,143	1,120	1,081	1,142	1,119	1,142	1,119
109		1	1,040	1,990	1,896	1,906	1,896	1,906	2,002	1,751	1,761	1,751	1,761
110		1	2,850	3,859	3,881	3,888	3,881	3,888	3,949	3,201	3,208	3,201	3,208
111		1	1,060	903	843	875	843	875	1,460	720	753	720	753
112		1	1,850	2,429	2,304	2,319	2,304	2,319	4,146	3,532	3,547	3,532	3,547
136	1		960	1,220	792	794	792	794	1,485	886	888	886	888
144	1		1,000										
144	2	1	1,000	1,137	1,092	1,032	1,325	1,197	1,598	938	878	1,169	1,042
154	1		880	1,055	954	953	954	953	1,834	1,548	1,547	1,548	1,547
155	1		865	1,064	916	917	916	917	1,592	1,380	1,381	1,380	1,381
158	1		750	1,463	1,230	1,230	1,230	1,230	2,783	1,970	1,970	1,970	1,970
159	1		2,800	5,310	4,924	4,946	4,924	4,946	8,810	7,076	7,098	7,076	7,098

(cont.). Axial capacities (KN): Concrete Piles in Cohesive soils

Case #	situ ID#	lab ID#	β				λ				Schmertmann	
			2B; T&P (1)	11.5B; T&P (2)	2B; Hara (5h)	11.5B; Hara (6h)	2B; T&P (1)	11.5B; T&P (2)	2B; Hara (5h)	(6h)	SPT mobilized	CPT
54	1		1,391	1,383	1,847	1,762	1,059	1,051	1,890	1,805	466	
78	1	1					980	900	1,018	938		
80	1	1					1,246	1,370	1,559	1,801		
81		1					1,149	1,143	1,149	1,143		
82	1	1					971	934	1,240	1,164		
99	2		563	537	708	663	534	508	1,095	1,050	423	
105		1					2,075	1,955	2,075	1,955		
106		1					1,185	1,145	1,185	1,145		
108		1					1,647	1,625	1,647	1,625		
109		1					2,917	2,927	2,917	2,927		
110		1					3,002	3,009	3,002	3,009		
111		1					1,050	1,082	1,050	1,082		
112		1					2,532	2,547	2,532	2,547		
136	1		1,144	1,147	1,144	1,147	776	778	776	778		1,054
144	1		1,586	1,585	1,586	1,585						1,237
144	2	1					1,139	1,079	1,368	1,240		
154	1		1,327	1,326	1,327	1,326	931	929	931	929		899
155	1		1,300	1,300	1,300	1,300	877	878	877	878		919
158	1		1,752	1,752	1,752	1,752	1,120	1,120	1,120	1,120		982
159	1		6,811	6,832	6,811	6,832	3,750	3,772	3,750	3,772		3,659

Table 27: Axial capacities (KN): Concrete Piles in Mixed soils

Case	situ	lab	Davisson	α Tomlinson, Nordlund and Thurman					α API, Nordlund, Thurman			
#	#	#		DRIVEN	40;2B;P; T&P(1)	40;2B;P; Hara(1h)	36;11.5B; Sch;T&P(8)	36;2B;P; Hara(5h)	40;2B;P; T&P(1)	40;2B;P; Hara(1h)	36;11.5B; Sch;T&P(8)	36;2B;P; Hara(5h)
5	1		3,300	1,509	1,926	3,388	1,849	3,388	1,695	2,965	1,618	2,965
6	1		3,400	2,166	2,162	3,031	1,901	3,031	3,110	4,716	2,848	4,716
14	1		1,690	3,221	3,151	3,669	3,437	3,358	3,050	3,358	3,336	3,046
17	1		1,120						1,441	1,635	767	960
18	1		1,030						4,965	5,020	2,579	2,635
19	1		3,350									
20	1		2,450									
21	1		4,300									
22	1		4,270	1,450	2,368	3,651	2,220	3,651	3,025	4,517	2,877	4,517
23	1		2,669									
28	1		1,920						1,482	1,531	1,064	1,229
29	1		1,075	789	1,315	2,343	1,285	2,343	978	1,694	948	1,694
30	1		1,160	1,828	1,684	2,556	1,733	2,556	1,286	1,824	1,335	1,824
31	1		383						299	316	333	313
34	1		1,779									
36	1		6,420	8,318	7,824	12,202	7,799	12,202	4,541	6,509	4,517	6,509
37	1		5,420						7,373	7,557	3,922	6,394
38	1		6,260						5,867	6,217	10,305	6,217
39	1		6,600						5,839	6,138	9,074	6,138
40	1		2,800						6,419	6,631	9,870	6,570
41	1		4,005						6,651	6,894	11,408	6,894
42	1		6,400						6,751	7,060	11,524	6,952
43	1		5,650						1,611	1,696	3,737	1,696
44	1		6,100	2,796	2,876	4,524	3,460	4,524	2,922	4,632	3,506	4,632
45	1		3,320						8,687	8,768	12,736	8,765
47	1		1,800						3,032	3,282	3,512	2,884
48	1		1,635						1,819	1,935	1,944	1,819
49	1		2,870						951	1,120	2,267	1,120
50	1		3,200									
51	1		2,120						1,315	1,381	1,709	1,381
52	1		3,400						3,788	4,066	5,492	3,871
53	1		4,800									
60	1		4,360						10,984	11,879	9,416	10,451
62	1		4,900	11,644	11,703	12,479	10,424	9,677	11,695	12,317	10,415	9,514
63	1		4,850						15,603	15,718	11,795	10,294
64	1		4,200						5,326	5,503	7,000	5,503
65	1		3,070						2,576	2,745	3,657	2,658
67	1		1,239									
68	1		2,200						1,033	1,453	2,526	1,453
69	1		1,225									
70	1		515									
71	1		1,605									
72	1		2,090									
73	1		1,050									
74	1		890	1,305	1,290	1,739	1,490	1,737	963	1,191	1,163	1,189
75	1		2,640									
76	2		3,350	4,950	4,379	7,193	4,644	7,148	3,026	4,436	3,291	4,390
77	1		7,200	5,311	5,770	9,600	6,041	9,539	3,630	5,376	3,901	5,315
84	1		2,150						1,354	1,629	2,627	1,629
85	2		4,150						5,639	5,656	3,104	3,289
86	1	1	3,600	1,455	1,788	2,807	1,762	2,807	1,817	2,845	1,791	2,845
87	1		2,170	2,737	3,579	6,194	3,576	6,194	3,655	6,144	3,652	6,144
88	1		4,700						5,644	6,276	7,351	5,519
89	1		1,600	967	975	1,489	971	1,489	1,137	1,901	1,133	1,901
90	1		4,780						5,927	6,221	8,895	5,818
91	1		2,050									
93	1		5,030						8,120	8,954	8,309	7,868
100	1		1,140	637	635	840	843	840	589	777	796.	777
102	1		1,040						3,265	3,890	1,977	2,601
113	2		3,180	945	1,587	1,915	5,058	1,915	2,914	4,871	6,384	4,871
114	1		1,840						1,239	2,334	1,097	2,334

Case	situ	lab	Davisson	α Tomlinson, Nordlund and Thurman					α API, Nordlund, Thurman			
#	#	#		DRIVEN	40;2B;P; T&P(1)	40;2B;P; Hara(1h)	36;11.5B; Sch;T&P(8)	36;2B;P; Hara(5h)	40;2B;P; T&P(1)	40;2B;P; Hara(1h)	36;11.5B; Sch;T&P(8)	36;2B;P; Hara(5h)
115	2		2,280	546	604	1,328	554	1,328	1,565	3,452	1,516	3,452
116	2		2,470	553	615	1,346	563	1,346	1,586	3,494	1,534	3,494
117	2		2,400	548	616	1,377	558	1,377	1,526	3,397	1,468	3,397
118	2		2,400	535	594	1,336	538	1,336	1,489	3,330	1,434	3,330
119	1		3,750						2,575	2,691	3,613	2,620
120	1		2,360						6,331	6,592	5,054	4,980
121	1		2,370						2,602	2,864	1,606	1,814
125	1		1,245						1,107	1,107	1,363	1,107
125	2		1,245									
126	1		4,250						5,371	5,616	6,460	5,616
126	2		4,250									
127	1		1,890	535	706	1,220	642	1,220	998	2,012	933	2,012
127	3		1,890									
128	3		1,400						2,661	2,661	2,015	2,480
129	1		890						902	1,083	873	1,019
129	2		890									
130	1		2,200						2,395	2,659	2,410	2,659
130	3		2,200									
131	1		4,250	1,585	1,574	1,867	1,964	1,867	1,769	2,226	2,158	2,226
131	2		4,250									
132	1		1,080	2,089	1,925	1,925	1,585	1,584	1,984	1,984	1,644	1,643
133	1		475	1,019	1,102	1,102	1,055	1,064	1,020	1,020	973	983
134	1		1,070	1,595	1,417	1,417	1,165	1,162	1,581	1,581	1,329	1,327
135	1		1,040	1,613	1,141	1,141	1,023	1,022	1,386	1,386	1,267	1,266
137	1		4,100	3,927	3,973	3,973	3,693	3,772	3,701	3,701	3,422	3,500
138	1		1,920						9,087	9,087	7,887	7,873
139	1		2,400						12,581	12,581	11,918	11,472
140	1		2,900						3,543	3,543	3,527	3,527
142	1		1,000						1,352	1,352	1,613	1,351
143	1		1,085	1,611	1,025	1,025	1,016	1,024	766	766	757	765
145	1		2,800						5,693	5,693	4,997	5,689
146	1		1,468						2,943	2,943	1,659	1,797
147	1		800	1,721	1,502	1,502	1,487	1,480	1,142	1,142	1,127	1,120
148	1		1,200						1,607	1,607	1,800	1,607
149	1		4,475						10,972	10,972	9,479	9,479
150	1		4,205						10,383	10,383	8,733	9,716
151	1		4,650						9,263	9,263	10,133	8,155
152	1		4,875						17,075	17,075	8,904	9,877
153	1		990	753	812	812	670	774	1,484	1,484	1,343	1,447
156	1		885	1,910	1,771	1,771	1,769	1,771	1,171	1,171	1,169	1,171
157	1		1,065	2,139	2,034	2,034	2,014	2,026	1,490	1,490	1,471	1,482
160	1		5,400						6,263	6,263	6,334	6,160
161	1		5,200						10,620	10,620	8,569	8,569
162	1		4,190						10,832	10,832	8,940	8,940
163	1		1,931	3,926	3,471	3,471	3,388	3,398	2,627	2,627	2,545	2,554

(cont.): Axial capacities (KN): Concrete Piles in Mixed soils

Case #	situ #	lab #	β; Thurman				Schmertmann	
			40;2B;P;T&P (1)	40;2B;P;Hara(1h)	36;11.5B;Sch;T&P(8)	36;2B;P;Hara(5h)	SPT mobilized	CPT
5	1		2,682	3,209	2,545	3,209	923	
6	1		4,880	5,449	4,838	5,449	1,640	
14	1		2,156	2,365	2,175	2,365	1,915	
17	1		1,530	1,530	918	918	969	
18	1		3,662	3,662	2,257	2,257	2,101	
19	1						3,204	
20	1						2,481	
21	1						2,845	
22	1		4,631	5,276	4,478	5,276	2,274	
23	1						1,137	
28	1		1,138	1,138	906	1,022	1,121	
29	1		1,480	1,782	1,445	1,782	895	
30	1		1,402	1,750	1,332	1,750	1,159	
31	1		309	309	332	309	223	
34	1						663	
36	1		6,543	7,551	6,363	7,551	2,876	
37	1		6,549	6,549	3,291	5,476	1,967	
38	1		4,478	4,478	6,763	4,478	2,509	
39	1		4,641	4,641	6,707	4,641	2,108	
40	1		4,468	4,468	6,072	4,468	3,087	
41	1		4,663	4,663	7,817	4,663	2,911	
42	1		4,479	4,479	7,539	4,479	2,952	
43	1		1,460	1,460	3,178	1,460	1,085	
44	1		3,809	4,556	3,759	4,556	1,731	
45	1		6,088	6,088	7,450	6,088	3,010	
47	1		2,695	2,695	3,102	2,695	1,876	
48	1		1,523	1,523	1,495	1,523	1,222	
49	1		1,018	1,018	1,947	1,018	784	
50	1						6,350	
51	1		1,253	1,253	1,453	1,253	1,144	
52	1		3,374	3,374	4,600	3,374	2,288	
53	1						2,617	
60	1		10,584	10,584	9,289	9,289	3,663	
62	1		5,579	6,110	5,579	6,110	4,567	
63	1		12,543	12,543	7,554	7,966	4,179	
64	1		4,146	4,146	5,096	4,146	2,228	
65	1		2,128	2,128	3,051	2,128	1,092	
67	1						668	

Case #	situ #	lab #	β; Thurman 40;2B;P;T&P (1)	40;2B;P;Hara(1h)	36;11.5B;Sch;T&P(8)	36;2B;P;Hara(5h)	Schmertmann SPT mobilized	CPT
68	1		1,331	1,331	2,475	1,331	552	
69	1						807	
70	1						677	
71	1						878	
72	1						1,489	
73	1						962	
74	1		1,127	1,270	1,167	1,270	1,012	
75	1						1,921	
76	2		5,426	5,806	5,467	5,806	3,324	
77	1		6,209	6,773	6,253	6,773	3,961	
84	1		1,544	1,544	2,522	1,544	857	
85	2		4,739	4,739	2,517	2,712	1,792	
86	1	1	3,911	3,903	3,903	3,903	2,262	
87	1		7,069	7,486	7,058	7,486	3,424	
88	1		5,096	5,096	6,742	5,096	3,343	
89	1		2,011	2,105	2,013	2,105	857	
90	1		4,303	4,303	6,330	4,303	2,765	
91	1						2,359	
93	1		6,390	6,390	7,417	6,390	3,768	
100	1		630	753	623	753	810	
102	1		3,819	3,819	2,672	2,672	2,044	
113	2		4,928	4,928	8,399	4,928	1,089	
114	1		2,264	2,264	2,240	2,264	297	
115	2		3,047	3,513	2,998	3,513	667	
116	2		3,092	3,563	3,040	3,563	685	
117	2		2,929	3,456	2,871	3,456	669	
118	2		2,860	3,377	2,805	3,377	638	
119	1		2,436	2,436	2,785	2,436	1,625	
120	1		5,865	5,865	4,938	4,938	3,516	
121	1		2,913	2,913	2,003	2,003	1,608	
125	1		984	984	984	984	1,004	
125	2							1,014
126	1		4,516	4,516	4,763	4,516	2,329	
126	2							4,929
127	1		2,039	2,319	1,975	2,319	820	
127	3							1,568
128	3		1,973	1,973	1,507	1,972		1,428
129	1		512	692	503	692	879	
129	2							3,138
130	1		1,855	1,855	1,855	1,855	789	

Case #	situ #	lab #	β; Thurman				Schmertmann	
			40;2B;P;T&P (1)	40;2B;P;Hara(1h)	36;11.5B;Sch;T&P(8)	36;2B;P;Hara(5h)	SPT mobilized	CPT
130	3							2,658
131	1		1,542	1,762	1,633	1,762	682	
131	2							2,522
132	1		1,765	1,765	1,767	1,765		1,576
133	1		1,213	1,213	1,204	1,213		909
134	1		1,481	1,481	1,484	1,481		1,237
135	1		1,585	1,585	1,586	1,585		1,106
137	1		4,343	4,343	4,265	4,343		3,372
138	1		4,523	4,523	4,538	4,523		3,495
139	1		6,853	6,853	7,298	6,853		6,612
140	1		3,765	3,765	3,765	3,765		2,304
142	1		1,447	1,447	1,709	1,447		982
143	1		974	974	965	974		544
145	1		6,627	6,627	5,935	6,627		4,758
146	1		3,102	3,102	1,896	2,033		1,165
147	1		1,527	1,527	1,534	1,527		1,238
148	1		1,716	1,716	1,909	1,716		1,651
149	1		7,027	7,027	6,548	6,548		5,150
150	1		7,695	7,695	6,712	7,695		5,439
151	1		5,385	5,385	7,363	5,385		6,084
152	1		16,852	16,852	9,382	10,355		7,515
153	1		1,704	1,704	1,600	1,704		1,259
156	1		1,548	1,548	1,546	1,548		936
157	1		2,108	2,108	2,097	2,108		1,111
160	1		6,972	6,972	7,146	6,972		7,514
161	1		10,616	10,616	9,224	9,224		5,941
162	1		10,008	10,008	8,688	8,688		6,423
163	1		3,344	3,344	3,334	3,344		1,638

D. BIAS FACTORS--CONCRETE PILES

Table 28: Bias factor: Concrete Piles in Cohesionless soils

			Nordlund								
Case #	situ ID#	lab ID#	DRIVEN	40;2B;P (1)	40;11.5B; P (2)	40;2B;Sch (3)	40;11.5B; Sch (4)	36;2B;P (5)	36;11.5B; P (6)	36;2B;Sch (7)	36;11.5B; Sch (8)
1	1		0.80	0.80	0.62	0.41	0.47	1.07	0.93	0.81	0.84
2	1		0.35	0.35	0.67	0.29	0.30	0.64	0.79	0.62	0.62
3	1		1.18	1.18	1.47	1.10	1.10	2.38	2.54	2.26	2.26
4	1		0.79	0.79	0.79	0.79	0.79	1.64	1.64	1.64	1.64
7	1		0.48	0.48	0.61	0.45	0.46	0.90	0.93	0.88	0.88
8	1		0.87	0.87	0.96	0.85	0.85	1.75	1.75	1.75	1.75
9	1		0.56	0.56	0.56	0.55	0.55	1.14	1.14	1.14	1.14
10	1		1.11	1.12	1.11	0.99	0.82	1.12	1.11	1.16	0.93
11	1		1.08	1.08	1.08	0.71	0.81	1.08	1.08	0.80	0.93
12	1		0.94	0.94	0.97	0.42	0.47	0.97	1.01	0.63	0.65
13	1		1.66	1.66	1.66	1.15	1.51	1.68	1.68	1.40	1.97
16	1		0.98	0.98	0.98	0.97	0.97	2.01	2.01	2.00	2.00
24	1		0.33	0.33	0.35	0.29	0.29	0.61	0.61	0.58	0.58
25	1		0.97	0.97	0.94	0.49	0.53	1.09	1.06	0.78	0.83
26	1		1.14	1.14	1.61	0.75	1.51	1.42	1.95	1.61	2.52
27	1		0.32	0.32	0.35	0.29	0.29	0.60	0.60	0.57	0.57
32	1		0.59	0.59	0.91	0.51	0.55	1.09	1.28	0.99	0.99
33	1		0.75	0.75	0.69	0.32	0.32	0.80	0.75	0.64	0.64
35	1		0.55	0.55	0.55	0.32	0.35	0.64	0.72	0.63	0.65
46	1		0.27	0.27	0.60	0.22	0.35	0.45	0.69	0.40	0.57
56	1		0.42	0.42	0.42	0.34	0.34	0.42	0.42	0.37	0.37
57	1		0.49	0.49	0.48	0.38	0.36	0.49	0.48	0.42	0.40

Case #	situ ID#	lab ID#	Nordlund DRIVEN	40;2B;P (1)	40;11.5B; P (2)	40;2B;Sch (3)	40;11.5B; Sch (4)	36;2B;P (5)	36;11.5B; P (6)	36;2B;Sch (7)	36;11.5B; Sch (8)
58	2		0.35	0.35	0.65	0.25	0.39	0.53	0.77	0.49	0.62
59	1		0.29	0.29	0.59	0.28	0.58	0.56	0.92	0.57	1.11
61	1		0.43	0.43	0.42	0.23	0.26	0.48	0.48	0.31	0.35
66	1		1.35	1.35	1.52	0.47	0.64	1.41	1.59	0.82	0.87
83	1		0.43	0.43	0.43	0.41	0.41	0.82	0.82	0.78	0.78
94	3		0.45	0.45	0.55	0.36	0.38	0.76	0.79	0.65	0.65
95	1		0.17	0.17	0.16	0.08	0.08	0.19	0.20	0.16	0.16
96	2		0.64	0.64	0.73	0.56	0.60	1.20	1.21	1.04	1.04
97	1		0.13	0.13	0.18	0.10	0.10	0.22	0.23	0.18	0.18
98	1		0.47	0.47	0.45	0.19	0.19	0.48	0.47	0.35	0.35
101	1		1.96	1.96	1.93	0.58	0.59	1.96	1.93	1.03	1.03
103	1		0.70	0.70	0.99	0.60	0.66	1.35	1.61	1.13	1.13
122	1		0.91	0.91	1.10	0.64	0.70	1.41	1.42	1.36	1.36
123	1		0.51	0.52	0.51	0.35	0.35	0.71	0.71	0.68	0.68
123	3										
124	1		0.73	0.73	0.64	0.32	0.38	0.87	0.84	0.63	0.64
124	2										
N			**37**	**37**	**37**	**37**	**37**	**37**	**37**	**37**	**37**
Bias mean			**0.71**	**0.71**	**0.79**	**0.49**	**0.55**	**1.00**	**1.06**	**0.87**	**0.94**
STD deviation			**0.41**	**0.41**	**0.42**	**0.27**	**0.33**	**0.53**	**0.55**	**0.50**	**0.57**
COV			**0.58**	**0.58**	**0.53**	**0.56**	**0.60**	**0.53**	**0.52**	**0.57**	**0.60**

(cont.): Bias factor: Concrete Piles in Cohesionless soils

Case #	situ ID#	lab ID#	β 40;2B;P (1)	β 36;2B;P (5)	Meyerhof	Schmertmann SPT mobilized	Schmertmann CPT	Predominant side resistance
1	1		1.11	1.11	0.96	0.75		
2	1		0.36	0.66	0.15	0.48		
3	1		1.25	2.42	0.40	1.54		
4	1		0.87	1.68	0.19	0.68		
7	1		0.55	0.95	0.32	0.80		
8	1		0.97	1.83	0.23	0.90		
9	1		0.63	1.18	0.19	0.58		
10	1		1.45	1.45	0.88	2.34		Y
11	1		1.43	1.43	0.78	2.05		
12	1		0.95	0.95	0.70	1.27		Y
13	1		2.56	2.56	1.43	2.06		Y
16	1		1.10	2.06	0.27	0.90		
24	1		0.43	0.79	0.26	0.77		
25	1		1.40	1.40	0.65	1.36		
26	1		1.39	1.68	0.94	1.69		
27	1		0.42	0.78	0.20	0.86		
32	1		0.62	1.12	1.37	0.93		
33	1		1.09	1.09	0.53	1.08		
35	1		0.81	0.81	0.50	0.83		
46	1		0.31	0.55	0.29	0.70		
56	1		0.60	0.60	1.17	1.42		Y
57	1		0.69	0.69	1.17	1.68		
58	2		0.43	0.65	0.56	1.52		
59	1		0.33	0.67	0.82	1.89		
61	1		0.70	0.70	0.50	1.16		
66	1		1.51	1.51	0.59	1.52		
83	1		0.52	0.91	0.23	0.78		
94	3		0.59	1.01	0.52	1.09		

Case #	situ ID#	lab ID#	β 40;2B;P (1)	β 36;2B;P (5)	Meyerhof	Schmertmann SPT mobilized	Schmertmann CPT	Predominant side resistance
95	1		0.22	0.25	0.13	0.37		
96	2		0.85	1.53	1.12	2.43		
97	1		0.16	0.29	0.13	0.35		
98	1		0.66	0.66	0.59	0.96		
101	1		2.11	2.11	1.04	1.63		
103	1		0.82	1.62	0.98	2.79		
122	1		1.01	1.50	1.82	2.32		
123	1		0.73	0.92	0.48	0.88		
123	3						0.78	
124	1		1.19	1.19	0.61	0.94		
124	2						0.44	
N			**37**	**37**	**37**	**37**	**2**	
Bias mean			**0.89**	**1.17**	**0.64**	**1.25**	**0.61**	
STD deviation			**0.51**	**0.56**	**0.42**	**0.62**		
COV			**0.58**	**0.48**	**0.65**	**0.49**		

Table 29. Bias factor: Concrete Piles in Cohesive soils

			α API revised					α Tomlinson				
Case #	situ ID#	lab ID#	APILE	2B; T&P (1)	11.5B; T&P (2)	2B; Hara (5h)	11.5B; Hara (6h)	DRIVEN	2B; T&P (1)	11.5B; T&P (2)	2B; Hara (5h)	11.5B; Hara (6h)
54	1		2.95	3.06	3.09	1.57	1.66	6.95	3.73	3.78	2.18	2.35
78	1	1	0.82	0.79	0.89	0.77	0.87	1.51	1.35	1.67	1.35	1.67
80	1	1	0.57	0.65	0.60	0.52	0.46	0.24	0.45	0.43	0.40	0.36
81		1	0.86	0.86	0.86	0.86	0.86	0.28	0.77	0.77	0.77	0.77
82	1	1	0.85	0.84	0.87	0.64	0.69	0.25	0.73	0.76	0.64	0.68
99	2		3.3	3.08	3.33	1.58	1.70	2.41	2.40	2.56	1.65	1.77
105		1	0.91	0.94	1.03	0.94	1.03	0.98	0.92	1.01	0.92	1.01
106		1	0.87	0.87	0.91	0.87	0.91	2.38	1.64	1.80	1.64	1.80
108		1	0.83	0.84	0.86	0.84	0.86	0.89	0.85	0.86	0.85	0.86
109		1	0.52	0.55	0.55	0.55	0.55	0.52	0.59	0.59	0.59	0.59
110		1	0.74	0.73	0.73	0.73	0.73	0.72	0.89	0.89	0.89	0.89
111		1	1.17	1.26	1.21	1.26	1.21	0.73	1.47	1.41	1.47	1.41
112		1	0.76	0.80	0.80	0.80	0.80	0.45	0.52	0.52	0.52	0.52
136	1		0.79	1.21	1.21	1.21	1.21	0.65	1.08	1.08	1.08	1.08
144	1											
144	2	1	0.88	0.92	0.97	0.75	0.84	0.63	1.07	1.14	0.86	0.96
154	1		0.83	0.92	0.92	0.92	0.92	0.48	0.57	0.57	0.57	0.57
155	1		0.81	0.94	0.94	0.94	0.94	0.54	0.63	0.63	0.63	0.63
158	1		0.51	0.61	0.61	0.61	0.61	0.27	0.38	0.38	0.38	0.38
159	1		0.53	0.57	0.57	0.57	0.57	0.32	0.40	0.39	0.40	0.39
N			**19**	**19**	**19**	**19**	**19**	**19**	**19**	**19**	**19**	**19**
Bias mean			**1.03**	**1.08**	**1.10**	**0.89**	**0.92**	**1.12**	**1.08**	**1.12**	**0.94**	**0.98**
Standard deviation			**0.76**	**0.73**	**0.77**	**0.31**	**0.34**	**1.55**	**0.82**	**0.85**	**0.50**	**0.57**
COV			**0.74**	**0.68**	**0.70**	**0.35**	**0.37**	**1.39**	**0.76**	**0.76**	**0.54**	**0.57**

(cont.). Bias factor: Concrete Piles in Cohesive soils

Case #	situ ID#	lab ID#	β 2B; T&P (1)	β 11.5B; T&P (2)	β 2B; Hara (5h)	β 11.5B; Hara (6h)	λ 2B; T&P (1)	λ 11.5B; T&P (2)	λ 2B; Hara (5h)	λ (6h)	Schmertmann SPT mobilized	Schmertmann CPT
54	1		1.83	1.84	1.38	1.45	2.41	2.43	1.35	1.41	5.47	
78	1	1					0.57	0.62	0.55	0.60		
80	1	1					0.69	0.63	0.55	0.48		
81		1					0.83	0.83	0.83	0.83		
82	1	1					0.76	0.79	0.59	0.63		
99	2		1.88	1.97	1.50	1.60	1.99	2.09	0.97	1.01	2.51	
105		1					0.59	0.63	0.59	0.63		
106		1					0.61	0.63	0.61	0.63		
108		1					0.59	0.59	0.59	0.59		
109		1					0.36	0.36	0.36	0.36		
110		1					0.95	0.95	0.95	0.95		
111		1					1.01	0.98	1.01	0.98		
112		1					0.73	0.73	0.73	0.73		
136	1		0.84	0.84	0.84	0.84	1.24	1.23	1.24	1.23		0.91
144	1		0.63	0.63	0.63	0.63						0.81
144	2	1					0.88	0.93	0.73	0.81		
154	1		0.66	0.66	0.66	0.66	0.95	0.95	0.95	0.95		0.98
155	1		0.67	0.67	0.67	0.67	0.99	0.99	0.99	0.99		0.94
158	1		0.43	0.43	0.43	0.43	0.67	0.67	0.67	0.67		0.76
159	1		0.41	0.41	0.41	0.41	0.75	0.74	0.75	0.74		0.77
N			**8**	**8**	**8**	**8**	**19**	**19**	**19**	**19**	**2**	**6**
Bias mean			**0.92**	**0.93**	**0.81**	**0.84**	**0.92**	**0.93**	**0.79**	**0.80**	**3.99**	**0.86**
Standard deviation			**0.60**	**0.62**	**0.41**	**0.45**	**0.50**	**0.51**	**0.25**	**0.26**		**0.09**
COV			**0.65**	**0.66**	**0.50**	**0.54**	**0.54**	**0.54**	**0.32**	**0.32**		**0.11**

Table 30: Bias factor: Concrete Piles in Mixed soils

Case	situ	lab	α Tomlinson, Nordlund and Thurman					α API, Nordlund, Thurman			
#	#	#	DRIVE N	40;2B;P; T&P(1)	40;2B;P; Hara(1h)	36;11.5B;Sch;T&P(8)	36;2B;P; Hara(5h)	40;2B;P; T&P(1)	40;2B;P; Hara(1h)	36;11.5B;Sch;T&P(8)	36;2B;P; Hara(5h)
5	1		2.19	1.71	0.97	1.79	0.97	1.95	1.11	2.04	1.11
6	1		1.57	1.57	1.12	1.79	1.12	1.09	0.72	1.19	0.72
14	1		0.52	0.54	0.46	0.49	0.50	0.55	0.50	0.51	0.55
17	1							0.78	0.69	1.46	1.17
18	1							0.21	0.21	0.40	0.39
19	1										
20	1										
21	1										
22	1		2.94	1.80	1.17	1.92	1.17	1.41	0.95	1.48	0.95
23	1										
28	1							1.30	1.25	1.80	1.56
29	1		1.36	0.82	0.46	0.84	0.46	1.10	0.63	1.13	0.63
30	1		0.63	0.69	0.45	0.67	0.45	0.90	0.64	0.87	0.64
31	1							1.28	1.21	1.15	1.22
34	1										
36	1		0.77	0.82	0.53	0.82	0.53	1.41	0.99	1.42	0.99
37	1							0.74	0.72	1.38	0.85
38	1							1.07	1.01	0.61	1.01
39	1							1.13	1.08	0.73	1.08
40	1							0.44	0.42	0.28	0.43
41	1							0.60	0.58	0.35	0.58
42	1							0.95	0.91	0.56	0.92
43	1							3.51	3.33	1.51	3.33
44	1		2.18	2.12	1.35	1.76	1.35	2.09	1.32	1.74	1.32
45	1							0.38	0.38	0.26	0.38
47	1							0.59	0.55	0.51	0.62
48	1							0.90	0.84	0.84	0.90
49	1							3.02	2.56	1.27	2.56
50	1										
51	1							1.61	1.53	1.24	1.53
52	1							0.90	0.84	0.62	0.88
53	1										
60	1							0.40	0.37	0.46	0.42
62	1		0.42	0.42	0.39	0.47	0.51	0.42	0.40	0.47	0.52
63	1							0.31	0.31	0.41	0.47
64	1							0.79	0.76	0.60	0.76
65	1							1.19	1.12	0.84	1.15
67	1.										

Case	situ	lab	α Tomlinson, Nordlund and Thurman					α API, Nordlund, Thurman			
#	#	#	DRIVEN	40;2B;P; T&P(1)	40;2B;P; Hara(1h)	36;11.5B;Sch;T&P(8)	36;2B;P; Hara(5h)	40;2B;P; T&P(1)	40;2B;P; Hara(1h)	36;11.5B;Sch;T&P(8)	36;2B;P; Hara(5h)
68	1							2.13	1.51	0.87	1.51
69	1										
70	1										
71	1										
72	1										
73	1										
74	1		0.68	0.69	0.51	0.60	0.51	0.92	0.75	0.77	0.75
75	1										
76	2		0.68	0.77	0.47	0.72	0.47	1.11	0.76	1.02	0.76
77	1		1.36	1.25	0.75	1.19	0.75	1.98	1.34	1.85	1.35
84	1							1.59	1.32	0.82	1.32
85	2							0.74	0.73	1.34	1.26
86	1	1	2.47	2.01	1.28	2.04	1.28	1.98	1.27	2.01	1.27
87	1		0.79	0.61	0.35	0.61	0.35	0.59	0.35	0.59	0.35
88	1							0.83	0.75	0.64	0.85
89	1		1.65	1.64	1.07	1.65	1.07	1.41	0.84	1.41	0.84
90	1							0.81	0.77	0.54	0.82
91	1										
93	1							0.62	0.56	0.61	0.64
100	1		1.79	1.79	1.36	1.35	1.36	1.94	1.47	1.43	1.47
102	1							0.32	0.27	0.53	0.40
113	2		3.37	2.00	1.66	0.63	1.66	1.09	0.65	0.50	0.65
114	1							1.49	0.79	1.68	0.79
115	2		4.18	3.78	1.72	4.11	1.72	1.46	0.66	1.50	0.66
116	2		4.47	4.02	1.84	4.39	1.84	1.56	0.71	1.61	0.71
117	2		4.38	3.89	1.74	4.30	1.74	1.57	0.71	1.63	0.71
118	2		4.49	4.04	1.80	4.46	1.80	1.61	0.72	1.67	0.72
119	1							1.46	1.39	1.04	1.43
120	1							0.37	0.36	0.47	0.47
121	1							0.91	0.83	1.48	1.31
125	1							1.12	1.12	0.91	1.12
125	2										
126	1							0.79	0.76	0.66	0.76
126	2										
127	1		3.53	2.68	1.55	2.95	1.55	1.89	0.94	2.03	0.94
127	3										
128	3							0.53	0.53	0.69	0.56
129	1							0.99	0.82	1.02	0.87
129	2										
130	1							0.92	0.83	0.91	0.83

Case	situ	lab	α Tomlinson, Nordlund and Thurman					α API, Nordlund, Thurman			
#	#	#	DRIVE N	40;2B;P; T&P(1)	40;2B;P; Hara(1h)	36;11.5B;Sch;T&P(8)	36;2B;P; Hara(5h)	40;2B;P; T&P(1)	40;2B;P; Hara(1h)	36;11.5B;Sch;T&P(8)	36;2B;P; Hara(5h)
130	3										
131	1		2.68	2.70	2.28	2.16	2.28	2.40	1.91	1.97	1.91
131	2										
132	1		0.52	0.56	0.56	0.68	0.68	0.54	0.54	0.66	0.66
133	1		0.47	0.43	0.43	0.45	0.45	0.47	0.47	0.49	0.48
134	1		0.67	0.76	0.76	0.92	0.92	0.68	0.68	0.81	0.81
135	1		0.64	0.91	0.91	1.02	1.02	0.75	0.75	0.82	0.82
137	1		1.04	1.03	1.03	1.11	1.09	1.11	1.11	1.20	1.17
138	1							0.21	0.21	0.24	0.24
139	1							0.19	0.19	0.20	0.21
140	1							0.82	0.82	0.82	0.82
142	1							0.74	0.74	0.62	0.74
143	1		0.67	1.06	1.06	1.07	1.06	1.42	1.42	1.43	1.42
145	1							0.49	0.49	0.56	0.49
146	1							0.50	0.50	0.88	0.82
147	1		0.46	0.53	0.53	0.54	0.54	0.70	0.70	0.71	0.71
148	1							0.75	0.75	0.67	0.75
149	1							0.41	0.41	0.47	0.47
150	1							0.41	0.41	0.48	0.43
151	1							0.50	0.50	0.46	0.57
152	1							0.29	0.29	0.55	0.49
153	1		1.31	1.22	1.22	1.48	1.28	0.67	0.67	0.74	0.68
156	1		0.46	0.50	0.50	0.50	0.50	0.76	0.76	0.76	0.76
157	1		0.50	0.52	0.52	0.53	0.53	0.71	0.71	0.72	0.72
160	1							0.86	0.86	0.85	0.88
161	1							0.49	0.49	0.61	0.61
162	1							0.39	0.39	0.47	0.47
163	1		0.49	0.56	0.56	0.57	0.57	0.74	0.74	0.76	0.76
N			**34**	**34**	**34**	**34**	**34**	**85**	**85**	**85**	**85**
Bias mean			**1.66**	**1.48**	**0.98**	**1.49**	**1.00**	**1.01**	**0.82**	**0.93**	**0.88**
Standard deviation			**1.34**	**1.11**	**0.53**	**1.21**	**0.52**	**0.62**	**0.48**	**0.48**	**0.47**
COV			**0.81**	**0.75**	**0.54**	**0.81**	**0.52**	**0.62**	**0.59**	**0.52**	**0.54**

(cont.): Bias factor: Concrete Piles in Mixed soils

Case #	situ #	lab #	β; Thurman				Schmertmann		Predominant side resistance
			40;2B;P;T&P (1)	40;2B;P;Hara(1h)	36;11.5B;Sch;T&P(8)	36;2B;P;Hara(5h)	SPT mobilized	CPT	
5	1		1.23	1.03	1.30	1.03	3.58		
6	1		0.70	0.62	0.70	0.62	2.07		
14	1		0.78	0.71	0.78	0.71	0.88		Y
17	1		0.73	0.73	1.22	1.22	1.16		
18	1		0.28	0.28	0.46	0.46	0.49		
19	1						1.05		
20	1						0.99		
21	1						1.51		
22	1		0.92	0.81	0.95	0.81	1.88		
23	1						2.35		
28	1		1.69	1.69	2.12	1.88	1.71		
29	1		0.73	0.60	0.74	0.60	1.20		Y
30	1		0.83	0.66	0.87	0.66	1.00		Y
31	1		1.24	1.24	1.15	1.24	1.71		
34	1						2.68		
36	1		0.98	0.85	1.01	0.85	2.23		
37	1		0.83	0.83	1.65	0.99	2.76		
38	1		1.40	1.40	0.93	1.40	2.49		Y
39	1		1.42	1.42	0.98	1.42	3.13		
40	1		0.63	0.63	0.46	0.63	0.91		
41	1		0.86	0.86	0.51	0.86	1.38		
42	1		1.43	1.43	0.85	1.43	2.17		
43	1		3.87	3.87	1.78	3.87	5.21		
44	1		1.60	1.34	1.62	1.34	3.52		
45	1		0.55	0.55	0.45	0.55	1.10		
47	1		0.67	0.67	0.58	0.67	0.96		
48	1		1.07	1.07	1.09	1.07	1.34		Y
49	1		2.82	2.82	1.47	2.82	3.66		
50	1						0.50		
51	1		1.69	1.69	1.46	1.69	1.85		
52	1		1.01	1.01	0.74	1.01	1.49		Y
53	1						1.83		
60	1		0.41	0.41	0.47	0.47	1.19		
62	1		0.88	0.80	0.88	0.80	1.07		Y
63	1		0.39	0.39	0.64	0.61	1.16		
64	1		1.01	1.01	0.82	1.01	1.89		
65	1		1.44	1.44	1.01	1.44	2.81		
67	1						1.85		

Case #	situ #	lab #	β; Thurman				Schmertmann		Predominant side resistance
			40;2B;P;T&P (1)	40;2B;P;Hara(1h)	36;11.5B;Sch;T&P(8)	36;2B;P;Hara(5h)	SPT mobilized	CPT	
68	1		1.65	1.65	0.89	1.65	3.99		
69	1						1.52		
70	1						0.76		
71	1						1.83		
72	1						1.40		
73	1						1.09		
74	1		0.79	0.70	0.76	0.70	0.88		
75	1						1.37		
76	2		0.62	0.58	0.61	0.58	1.01		Y
77	1		1.16	1.06	1.15	1.06	1.82		Y
84	1		1.39	1.39	0.85	1.39	2.51		
85	2		0.88	0.88	1.65	1.53	2.32		
86	1	1	0.92	0.92	0.92	0.92	1.59		Y
87	1		0.31	0.29	0.31	0.29	0.63		Y
88	1		0.92	0.92	0.70	0.92	1.41		
89	1		0.80	0.76	0.79	0.76	1.87		Y
90	1		1.11	1.11	0.76	1.11	1.73		
91	1						0.87		
93	1		0.79	0.79	0.68	0.79	1.33		Y
100	1		1.81	1.51	1.83	1.51	1.41		Y
102	1		0.27	0.27	0.39	0.39	0.51		
113	2		0.65	0.65	0.38	0.65	2.92		
114	1		0.81	0.81	0.82	0.81	6.20		
115	2		0.75	0.65	0.76	0.65	3.42		
116	2		0.80	0.69	0.81	0.69	3.61		
117	2		0.82	0.69	0.84	0.69	3.59		
118	2		0.84	0.71	0.86	0.71	3.76		Y
119	1		1.54	1.54	1.35	1.54	2.31		
120	1		0.40	0.40	0.48	0.48	0.67		
121	1		0.81	0.81	1.18	1.18	1.47		
125	1		1.27	1.27	1.27	1.27	1.24		Y
125	2							1.23	Y
126	1		0.94	0.94	0.89	0.94	1.83		
126	2							0.86	
127	1		0.93	0.81	0.96	0.81	2.30		
127	3							1.21	
128	3		0.71	0.71	0.93	0.71		0.98	
129	1		1.74	1.29	1.77	1.29	1.01		
129	2							0.28	
130	1		1.19	1.19	1.19	1.19	2.79		Y

Case #	situ #	lab #	β; Thurman				Schmertmann		Predominant side resistance
			40;2B;P;T&P (1)	40;2B;P;Hara(1h)	36;11.5B;Sc h;T&P(8)	36;2B;P;Ha ra(5h)	SPT mobilized	CPT	
130	3							0.83	Y
131	1		2.76	2.41	2.60	2.41	6.24		
131	2							1.69	
132	1		0.61	0.61	0.61	0.61		0.69	Y
133	1		0.39	0.39	0.39	0.39		0.52	Y
134	1		0.72	0.72	0.72	0.72		0.86	Y
135	1		0.66	0.66	0.66	0.66		0.94	Y
137	1		0.94	0.94	0.96	0.94		1.22	Y
138	1		0.42	0.42	0.42	0.42		0.55	Y
139	1		0.35	0.35	0.33	0.35		0.36	Y
140	1		0.77	0.77	0.77	0.77		1.26	Y
142	1		0.69	0.69	0.59	0.69		1.02	Y
143	1		1.11	1.11	1.12	1.11		2.00	Y
145	1		0.42	0.42	0.47	0.42		0.59	Y
146	1		0.47	0.47	0.77	0.72		1.26	
147	1		0.52	0.52	0.52	0.52		0.65	Y
148	1		0.70	0.70	0.63	0.70		0.73	Y
149	1		0.64	0.64	0.68	0.68		0.87	Y
150	1		0.55	0.55	0.63	0.55		0.77	Y
151	1		0.86	0.86	0.63	0.86		0.76	
152	1		0.29	0.29	0.52	0.47		0.65	Y
153	1		0.58	0.58	0.62	0.58		0.79	Y
156	1		0.57	0.57	0.57	0.57		0.95	Y
157	1		0.51	0.51	0.51	0.51		0.96	Y
160	1		0.77	0.77	0.76	0.77		0.72	Y
161	1		0.49	0.49	0.56	0.56		0.88	Y
162	1		0.42	0.42	0.48	0.48		0.65	Y
163	1		0.58	0.58	0.58	0.58		1.18	Y
N			**85**	**85**	**85**	**85**	**74**	**32**	
Bias mean			**0.94**	**0.90**	**0.88**	**0.93**	**1.97**	**0.90**	
Standard deviation			**0.57**	**0.55**	**0.43**	**0.55**	**1.20**	**0.35**	
COV			**0.61**	**0.61**	**0.48**	**0.59**	**0.61**	**0.39**	

E. SUB-CATEGORY OF PIPE PILES

Table 31. Total Cases of Static Load Tests on Pipe Piles in Soils

	S[1]	ST[2]	Total Number of Cases	
Cohesionless soils	1	19	20	
Cohesive soils	12	8	20	
Mixed soils	12	22	34	
Sub total for Soils			**74**	
Mixed soils and Rocks	**0**	**3**	**3**	
TOTAL	**25**	**52**	**77**	
NA [3]			**1**	**78**

[1] S: Predominant side resistance (Friction piles)
[2] ST: Side and tip resistance
[3] NA: Soil type is unknown/ or lack information to predict capacities

Table 32. The Data for Side Resistance (S) Pipe Piles

Soil Type	SPT	CPT	Lab	Overlap SPT, CPT & Lab	**Total**	OCR and Dr correlated from in-situ
cohesionless soils	1	0	0	0	**1**	1
cohesive soils	9	0	3	0	**12**	9
mixed soils	12	0	0	0	**12**	12
Mixed soils and Rocks	0	0	0	0	**0**	0
Total	**22**	**0**	**3**	**0**	**25**	22

Table 33. The Data for Side and Tip Resistance (ST) Pipe Piles

Soil Type	SPT	CPT	Lab	Overlap SPT, CPT & Lab	Total	OCR and Dr correlated from in-situ
cohesionless soils	19	0	0	0	**19**	19
cohesive soils	4	0	4	0	**8**	4
mixed soils	19	0	3	0	**22**	19
Mixed soils and Rocks	**3**	**0**	**0**	**0**	**3**	0
Total	**45**	**0**	**7**	**0**	**52**	42

Table 34. The Data for All Pipe Piles

Soil Type	SPT	CPT	Lab	Overlap SPT, CPT & Lab	Total	OCR and Dr correlated from in-situ
cohesionless soils	20	0	0	0	**20**	20
cohesive soils	13	0	7	0	**20**	13
mixed soils	31	0	3	0	**34**	31
Sub total of Soils	**64**	**0**	**10**	**0**	**64**	**64**
Mixed soils and Rocks	3	-	-	-	**3**	-
Total	**67**				**77**	

E.1. Side Resistance Only

TABLE 35. COHESIVE SOILS: Summary of Populations by Analysis Method used to Obtain *Side Resistance Only for Pipe Piles*

	Analysis Method				
	αAPI	α Tomlinson	λ	Schmertmann SPT	β
Number of cases	12	12	12	9	9

TABLE 36. MIXED SOILS: Summary of Populations by Analysis Method used to Obtain *Side Resistance Only for Pipe Piles*

Soil Type	Analysis Method			
cohesionless	Nordlund	Nordlund	β	Schmertmann SPT
cohesive	αAPI	α Tomlinson	β	Schmertmann SPT
Number of cases	12	5	12	12

NOTE 1: COHESIONLESS SOILS: Population is 1 for Nordlund, β and Schmertmann SPT methods (Not enough for statistical analysis)

TABLE 37. ALL SOILS: Summary of Populations by Analysis Method used to Obtain *Side Resistance Only for Pipe Piles*

Soil Type	Analysis Method			
cohesionless	Nordlund	Nordlund	β	Schmertmann SPT
cohesive	αAPI	α Tomlinson	β	Schmertmann SPT
Number of cases	25	18	22	22

Note 2: Numbers are referred from Tables 38, 39 and the constraints in Table 4.

E.2. Total Capacity

TABLE 38. COHESIONLESS SOILS: Summary of Populations by Analysis Method used to Obtain *Total Capacity for Pipe Piles*

Capacity	Analysis Method			
Side	Nordlund	β	Meyerhof	Schmertmann SPT
Tip	Thurman	Thurman	Meyerhof	Schmertmann SPT
Number of cases	20	20	20	20

TABLE 39. COHESIVE SOILS: Summary of Populations by Analysis Method used to Obtain *Total Capacity for Pipe Piles*

Capacity	**Analysis Method**				
Side	αAPI	α Tomlinson	λ	β	Schmertmann SPT
Tip	9 Su	9 Su	9 Su	9 Su	Schmertmann SPT
Number of cases	20	20	20	13	13

TABLE 40. MIXED SOILS: Summary of Populations by Analysis Method used to Obtain *Total Capacity for Pipe Piles*

Capacity	**Soil type**	**Analysis Method**					
Side	cohesionless	Nordlund		Nordlund		β	
	cohesive	αAPI		α Tomlinson		β	
Tip	cohesionless	Thurman		Thurman		Thurman	
	cohesive		9 Su		9 Su		9 Su
Number of cases		19	15	0	13	19	12
		34		13		31	

TABLE 41. MIXED SOILS and ROCKS: Summary of Populations by Analysis Method used to Obtain *Total Capacity for Pipe Piles*

Capacity	Soil Type	Analysis Method	
Side	cohesionless	Schmertmann SPT	
	cohesive	Schmertmann SPT	
Tip	cohesionless	Schmertmann SPT	
	cohesive	Schmertmann SPT	
Number of cases		rock exists	no rock
		3	31
		34	

TABLE 42. ALL SOILS: Summary of Populations by Analysis Method used to Obtain *Total Capacity for Pipe Piles*

Capacity	**Soil type**	**Analysis Method**							
Side	cohesionless	Nordlund		Nordlund		β		—	Meyerhof
	cohesive	αAPI		α Tomlinson		β		λ	--
Tip	cohesionless	Thurman		Thurman		Thurman		..	Meyerhof
	cohesive		9 Su		9 Su		9 Su	9 Su	--
Number of cases		39	35	20	33	39	25		
		74		53		64		20	20

TABLE 43. ALL SOILS and ROCKS: Summary of Populations by Analysis Method used to Obtain *Total Capacity for Concrete Piles*

Capacity	Soil Type	Analysis Method	
Side	cohesionless	Schmertmann SPT	
	cohesive	Schmertmann SPT	
Tip	cohesionless	Schmertmann SPT	
	cohesive	Schmertmann SPT	
Number of cases		rock exists	no rock
		3	64
		67	

Note: Numbers are referred from Tables 40; 45 to 49 and the constraints in Table 4.

F. AXIAL CAPACITIES--PIPE PILES

Table 44: Axial capacities (KN): Pipe Piles in Cohesionless soils

Case	situ	lab	Davisson	Nordlund								
#	ID#	ID#		DRIVEN	40;2B;P (1)	40;11.5B; P (2)	40;2B;Sch (3)	40;11.5B; Sch (4)	36;2B;P (5)	36;11.5B; P (6)	36;2B;Sch (7)	36;11.5B; Sch (8)
2	1		340	319	319	298	855	850	313	291	423	423
9	1		5,180	4,890	4,890	4,667	5,159	4,935	3,056	3,056	3,171	3,171
10	1		3,300	2,830	2,830	1,800	3,110	2,113	1,721	1,377	1,834	1,834
11	1		1,720	692	692	681	941	778	678	668	867	867
15	1		1,660	3,446	3,446	3,475	6,280	5,996	3,249	3,215	3,648	3,648
16	1		2,950	4,087	4,087	4,149	6,528	6,807	3,890	3,951	4,800	4,800
18	1		405	232	232	219	832	771	232	219	439	439
40	1		1,160	820	820	657	879	879	403	385	426	426
41	1		650	307	307	255	622	533	278	213	299	299
44	1		1,330	1,090	1,090	1,108	2,120	1,906	929	922	1,425	1,425
51	1		555	1,080	1,077	996	2,800	1,933	1,041	951	1,547	1,547
52	1		600	1,367	1,367	1,323	2,182	1,865	1,331	1,287	1,759	1,759
65	1		1,300	1,523	1,523	1,218	2,197	1,973	945	940	973	973
71	1		1,780	590	587	614	1,232	1,359	587	614	1,147	1,147
3	1		940	1,950	1,329	1,220	2,001	2,001	911	888	918	918
4	1		1,400	2,280	1,508	1,541	2,639	2,605	1,166	1,144	1,176	1,176
6	1		2,300	6,350	2,676	2,506	3,857	3,506	1,862	1,862	1,865	1,865
7	1		1,790	5,140	2,606	2,395	3,516	3,172	1,608	1,608	1,611	1,611
8	1		3,550	1,253	938	815	2,079	1,933	938	815	965	965
31	2		3,800	10,700	7,283	4,961	7,595	6,364	3,288	3,029	3,349	3,349

(cont.): Axial capacities (KN): Pipe Piles in Cohesionless soils

Case	situ	lab	β		Meyerhof	Schmertmann SPT mobilized
#	ID#	ID#	40;2B;P (1)	36;2B;P (5)		
2	1		418	418	727	400
9	1		5,013	3,953	2,943	1,642
10	1		3,166	2,118	2,292	1,468
11	1		873	873	851	770
15	1		3,443	3,443	2,737	1,919
16	1		4,261	4,261	2,680	2,224
18	1		344	344	577	382
40	1		850	459	4,360	461
41	1		335	307	1,354	267
44	1		1,139	1,139	1,160	802
51	1		1,283	1,283	1,083	957
52	1		1,674	1,674	1,441	1,190
65	1		1,556	1,042	1,446	479
71	1		832	832	1,021	690
3	1		1,881	1,496	1,747	1,073
4	1		2,167	1,862	2,199	1,282
6	1		5,219	4,631	3,326	2,025
7	1		4,508	3,692	3,188	1,803
8	1		1,381	1,381	1,858	860
31	2		8,918	5,156	4,745	3,071

Table 45: Axial capacities (KN): Pipe Piles in Cohesive soils

				α Tomlinson					α API revised				
Case #	situ ID#	lab ID#	Davisson	DRIVEN	2B; T&P (1)	11.5B; T&P (2)	2B; Hara (5h)	11.5B; Hara (6h)	APILE	2B; T&P (1)	11.5B; T&P (2)	2B; Hara (5h)	11.5B; Hara (6h)
19	1		160	396	309	309	315	314	180	170	170	339	339
38	1		1,100	744	787	770	1,769	1,742	730	755	738	1,339	1,311
39	1		1,800	768	938	897	1,979	1,926	884	892	851	1,529	1,476
45	1	1	240	1,690	1,486	1,452	1,713	1,626	1,079	994	960	1,251	1,164
46	1	1	1,090	2,621	2,487	2,485	2,988	2,985	112	2,129	2,126	2,689	2,686
50	1	1	400	253	340	348	375	422	285	305	313	359	406
53		2	270	387	295	270	295	270	380	310	286	310	286
54		1	610	1,520	1,484	1,506	1,484	1,506	1,200	1,181	1,203	1,181	1,203
55		1	450	696	648	664	648	664	555	542	558	542	558
56	1	0	89	150	141	117	258	209	154	174	150	330	281
57	1	2	1,125	1,830	1,807	1,808	3,119	3,122	1,795	1,803	1,805	2,724	2,727
58	1	2	1,260	2,806	2,785	2,787	4,840	4,843	2,838	2,974	2,976	4,474	4,476
59	1		622	300	330	331	410	412	228	217	218	430	431
60	1		595	1,355	1,325	1,263	1,614	1,528	996	1,047	985	1,731	1,644
61	1		420	896	773	776	923	929	567	540	543	953	958
76	1		1,700	2,126	2,035	2,035	3,207	3,207	2,420	2,442	2,442	3,530	3,530
78	1		2,700	2,532	2,507	2,507	3,931	3,931	3,106	3,148	3,148	4,505	4,505
32	2		360	944	726	738	768	787	520	474	487	870	890
34	2		360	1,017	832	832	948	950	430	426	427	835	836
37	1		140	592	506	502	1,286	1,279	492	491	487	924	917

(cont.) : Axial capacities (KN): Pipe Piles in Cohesive soils

Case #	situ ID#	lab ID#	β 2B; T&P (1)	β 11.5B; T&P (2)	β 2B; Hara (5h)	β 11.5B; Hara (6h)	λ 2B; T&P (1)	λ 11.5B; T&P (2)	λ 2B; Hara (5h)	λ 11.5B; Hara (6h)	Schmertmann SPT mobilized
19	1		250	249	312	311	280	280	610	610	643
38	1		1,396	1,379	1,504	1,476	694	676	1,268	1,241	1,286
39	1		1,638	1,597	1,760	1,707	816	776	1,436	1,383	1,339
45	1	1					1,101	1,067	1,437	1,351	
46	1	1					1,875	1,873	2,434	2,432	
50	1	1					642	651	873	920	
53		2					501	476	501	476	
54		1					1,297	1,320	1,297	1,320	
55		1					849	866	849	866	
56	1	0					197	173	377	328	
57	1	2	3,866	3,868	3,949	3,952	1,480	1,482	2,288	2,291	3,239
58	1	2	6,257	6,259	6,342	6,345	2,171	2,173	3,275	3,277	5,183
59	1		381	381	438	439	367	368	753	755	844
60	1		1,847	1,784	1,971	1,885	1,087	1,025	1,844	1,758	2,601
61	1		1,005	1,008	1,074	1,080	690	694	1,273	1,279	1,733
76	1		4,862	4,862	5,034	5,034	2,732	2,732	4,088	4,088	3,243
78	1		6,633	6,633	6,805	6,805	3,178	3,178	4,685	4,685	4,027
32	2		971	983	1,040	1,059	622	635	1,216	1,236	1,569
34	2		879	880	943	944	526	527	1,091	1,093	1,695
37	1		915	911	979	971	468	464	920	912	1,179

Table 46: Axial capacities (KN): Pipe Piles in Mixed soils

Case #	situ ID#	lab ID#	Davisson	α API, Nordlund, Thurman 40;2B;P;T&P(1)	36;2B;P (5)	36;11.5B;Sch;T&P(8)	36;2B;P;Hara(5h)	β; Thurman 40;2B;P;T&P(1)	36;2B;P (5)	36;11.5B;Sch;T&P(8)	36;2B;P;Hara(5h)
12	2		2,120	2,145	1,976	3,101	2,271	2,131	2,131	2,972	2,131
13	1		1,640	1,509	1,338	2,034	1,633	1,500	1,500	2,143	1,500
14	1		2,100	5,093	4,924	6,191	5,219	4,854	4,854	4,854	4,854
17	2		2,720	3,163	3,123	3,365	4,665	5,566	5,566	5,566	6,087
20	1		1,070	971	955	1,663	1,052	1,198	1,198	1,662	1,198
21	1		1,400	2,068	2,050	2,640	2,147	2,413	2,413	2,539	2,413
22	3		2,220	9,578	5,049	5,508	5,333	8,499	4,737	4,737	4,737
23	1		2,500	3,755	2,798	2,860	3,536	6,458	5,853	5,853	5,853
24	2		2,400	5,985	4,295	4,328	5,535	8,186	7,125	7,125	7,125
25	2		3,030	5,955	4,275	4,309	5,515	8,159	7,098	7,098	7,098
26	1		3,050	3,013	2,830	2,993	3,792	4,719	4,719	4,719	4,860
27	1		1,680	2,391	2,182	2,244	2,961	4,677	4,677	4,677	4,758
28	2		1,700	6,539	4,654	4,688	5,894	8,679	7,617	7,617	7,617
29	2		2,900	1,827	1,779	1,793	2,704	3,819	3,819	3,820	3,819
30	1		2,700	2,429	2,216	2,279	3,005	4,757	4,757	4,757	4,838
33	1		2,205	2,083	1,559	1,619	1,718	2,447	2,004	2,004	2,004
35	3		3,200	1,036	1,036	1,060	1,682	1,902	1,902	1,902	2,029
36	3		3,200	1,045	1,045	1,069	1,694	1,920	1,920	1,920	2,047
42	1		340	1,907	1,907	2,154	2,612				
47	1	1	120	569	569	575	806				
48	1	1	660	1,806	1,806	1,890	2,453	4,575	4,575	4,575	4,705
49	1	1	140	606	606	612	916	1,526	1,526	1,526	1,630
62	1		2,050								
63	1		2,120								
64	1		1,900	3,366	2,666	2,832	2,883	4,169	3,656	3,656	3,656
66	1	0	2,300	2,680	2,482	2,571	2,946	4,048	3,997	3,997	3,997
67	1		640	1,856	952	953	1,002	1,788	1,075	1,075	1,075
68	1		1,000	2,841	1,389	1,391	1,439	2,710	1,648	1,648	1,648
69	1	1	1,120	1,105	1,105	1,735	1,281				
70	1		1,000	1,160	920	927	1,157	1,455	1,216	1,216	1,216
72	2		2,400	3,711	3,545	3,564	4,821	7,689	7,689	7,689	7,862
73	1		2,400	2,187	2,135	2,136	3,042	4,290	4,290	4,290	4,462
74	2		2,370	2,629	2,443	2,506	3,335	4,243	4,243	4,243	4,415
75	2		2,900	3,744	3,637	3,637	5,022	8,160	8,160	8,160	8,333
77	2		1,100	2,111	2,082	2,089	3,013	3,991	3,991	3,991	4,164
1	1		740	738	738	1,036	1,101	1,616	1,616	1,896	1,616
5	1		2,350								

(cont.): Axial capacities (KN): Pipe Piles in Mixed soils

Case #	situ ID#	lab ID#	α Tomlinson, Nordlund and Thurman					Schmertmann SPT mobilized
			DRIVEN	40;2B;P; T&P(1)	36;2B;P (5)	36;11.5B;S ch;T&P(8)	36;2B;P;H ara(5h)	
12	2							1,721
13	1							1,491
14	1							3,008
17	2		5,799	5,039	4,998	5,241	8,370	4,694
20	1							1,559
21	1							1,936
22	3							2,747
23	1							2,827
24	2							4,464
25	2							4,460
26	1		2,538	2,583	2,400	2,563	2,948	2,850
27	1		1,848	1,931	1,722	1,784	2,162	2,516
28	2							4,538
29	2							3,015
30	1		1,857	1,957	1,744	1,806	2,192	2,553
33	1							1,786
35	3							2,656
36	3							2,666
42	1		1,756	1,721	1,721	1,968	2,137	
47	1	1	899	794	794	800	1,001	
48	1	1	1,361	1,307	1,307	1,391	2,026	4,195
49	1	1	898	846	846	852	1,112	2,146
62	1							2,831
63	1							2,810
64	1							2,628
66	1	0						2,139
67	1							1,098
68	1							1,324
69	1	1						
70	1							253
72	2		3,315	3,296	3,130	3,149	4,632	4,120
73	1		2,588	2,431	2,379	2,380	3,743	2,849
74	2		2,442	2,419	2,234	2,297	2,978	3,236
75	2		3,255	3,335	3,228	3,228	4,887	4,150
77	2		2,308	2,260	2,232	2,239	3,577	2,978
1	1							1,770
5	1							2,744

G. BIAS FACTORS--PIPE PILES

Table 47: Bias factor: Pipe Piles in Cohesionless soils

Case	situ	lab	Nordlund								
#	ID#	ID#	DRIVEN	40;2B;P (1)	40;11.5B; P (2)	40;2B;Sch (3)	40;11.5B; Sch (4)	36;2B;P (5)	36;11.5B; P (6)	36;2B;Sch (7)	36;11.5B; Sch (8)
2	1		1.06	1.06	1.14	0.40	0.40	1.09	1.17	0.80	0.80
9	1		1.06	1.06	1.11	1.00	1.05	1.69	1.69	1.63	1.63
10	1		1.17	1.17	1.83	1.06	1.56	1.92	2.40	1.80	1.80
11	1		2.49	2.49	2.52	1.83	2.21	2.54	2.57	1.98	1.98
15	1		0.48	0.48	0.48	0.26	0.28	0.51	0.52	0.45	0.45
16	1		0.72	0.72	0.71	0.45	0.43	0.76	0.75	0.61	0.61
18	1		1.74	1.74	1.85	0.49	0.53	1.74	1.85	0.92	0.92
40	1		1.41	1.41	1.76	1.32	1.32	2.88	3.01	2.72	2.72
41	1		2.12	2.12	2.55	1.04	1.22	2.34	3.05	2.17	2.17
44	1		1.22	1.22	1.20	0.63	0.70	1.43	1.44	0.93	0.93
51	1		0.51	0.52	0.56	0.20	0.29	0.53	0.58	0.36	0.36
52	1		0.44	0.44	0.45	0.27	0.32	0.45	0.47	0.34	0.34
65	1		0.85	0.85	1.07	0.59	0.66	1.38	1.38	1.34	1.34
71	1		3.02	3.03	2.90	1.44	1.31	3.03	2.90	1.55	1.55
3	1		0.48	0.71	0.77	0.47	0.47	1.03	1.06	1.02	1.02
4	1		0.61	0.93	0.91	0.53	0.54	1.20	1.22	1.19	1.19
6	1		0.36	0.86	0.92	0.60	0.66	1.24	1.24	1.23	1.23
7	1		0.35	0.69	0.75	0.51	0.56	1.11	1.11	1.11	1.11
8	1		2.83	3.79	4.36	1.71	1.84	3.79	4.36	3.68	3.68
31	2		0.36	0.52	0.77	0.50	0.60	1.16	1.25	1.13	1.13
N			**20**	**20**	**20**	**20**	**20**	**20**	**20**	**20**	**20**
Bias mean			**1.16**	**1.29**	**1.43**	**0.77**	**0.85**	**1.59**	**1.70**	**1.35**	**1.35**
Standard deviation			**0.85**	**0.91**	**1.00**	**0.49**	**0.55**	**0.91**	**1.03**	**0.83**	**0.83**
COV			**0.73**	**0.71**	**0.70**	**0.64**	**0.65**	**0.57**	**0.61**	**0.61**	**0.61**

(cont.): Bias factor: Pipe Piles in Cohesionless soils

Case #	situ ID#	lab ID#	β 40;2B;P (1)	β 36;2B;P (5)	Meyerhof	Schmertmann SPT mobilized	Predominant side resistance
2	1		0.81	0.81	0.47	0.85	
9	1		1.03	1.31	1.76	3.15	
10	1		1.04	1.56	1.44	2.25	
11	1		1.97	1.97	2.02	2.23	Y
15	1		0.48	0.48	0.61	0.87	
16	1		0.69	0.69	1.10	1.33	
18	1		1.18	1.18	0.70	1.06	
40	1		1.36	2.53	0.27	2.51	
41	1		1.94	2.11	0.48	2.44	
44	1		1.17	1.17	1.15	1.66	
51	1		0.43	0.43	0.51	0.58	
52	1		0.36	0.36	0.42	0.50	
65	1		0.84	1.25	0.90	2.72	
71	1		2.14	2.14	1.74	2.58	
3	1		0.50	0.63	0.54	0.88	
4	1		0.65	0.75	0.64	1.09	
6	1		0.44	0.50	0.69	1.14	
7	1		0.40	0.48	0.56	0.99	
8	1		2.57	2.57	1.91	4.13	
31	2		0.43	0.74	0.80	1.24	
N			**20**	**20**	**20**	**20**	
Bias mean			**1.02**	**1.18**	**0.94**	**1.71**	
Standard deviation			**0.66**	**0.73**	**0.55**	**0.98**	
COV			**0.65**	**0.61**	**0.59**	**0.57**	

Table 48: Bias factor: Pipe Piles in Cohesive soils

Case #	situ ID#	lab ID#	α Tomlinson DRIVE N	α Tomlinson 2B; T&P (1)	α Tomlinson 11.5B; T&P (2)	α Tomlinson 2B; Hara (5h)	α Tomlinson 11.5B; Hara (6h)	α API revised APILE	α API revised 2B; T&P (1)	α API revised 11.5B; T&P (2)	α API revised 2B; Hara (5h)	α API revised 11.5B; Hara (6h)
19	1		0.40	0.52	0.52	0.51	0.51	0.89	0.94	0.94	0.47	0.47
38	1		1.48	1.40	1.43	0.62	0.63	1.51	1.46	1.49	0.82	0.84
39	1		2.34	1.92	2.01	0.91	0.93	2.04	2.02	2.11	1.18	1.22
45	1	1	0.14	0.16	0.17	0.14	0.15	0.22	0.24	0.25	0.19	0.21
46	1	1	0.42	0.44	0.44	0.36	0.37	0.49	0.51	0.51	0.41	0.41
50	1	1	1.58	1.18	1.15	1.07	0.95	1.40	1.31	1.28	1.12	0.99
53		2	0.70	0.92	1.00	0.92	1.00	0.71	0.87	0.94	0.87	0.94
54		1	0.40	0.41	0.40	0.41	0.40	0.51	0.52	0.51	0.52	0.51
55		1	0.65	0.69	0.68	0.69	0.68	0.81	0.83	0.81	0.83	0.81
56	1	0	0.59	0.63	0.76	0.34	0.43	0.58	0.51	0.59	0.27	0.32
57	1	2	0.61	0.62	0.62	0.36	0.36	0.63	0.62	0.62	0.41	0.41
58	1	2	0.45	0.45	0.45	0.26	0.26	0.44	0.42	0.42	0.28	0.28
59	1		2.07	1.88	1.88	1.52	1.51	2.73	2.87	2.86	1.45	1.44
60	1		0.44	0.45	0.47	0.37	0.39	0.60	0.57	0.60	0.34	0.36
61	1		0.47	0.54	0.54	0.45	0.45	0.74	0.78	0.77	0.44	0.44
76	1		0.80	0.84	0.84	0.53	0.53	0.70	0.70	0.70	0.48	0.48
78	1		1.07	1.08	1.08	0.69	0.69	0.87	0.86	0.86	0.60	0.60
32	2		0.38	0.50	0.49	0.47	0.46	0.69	0.76	0.74	0.41	0.40
34	2		0.35	0.43	0.43	0.38	0.38	0.84	0.85	0.84	0.43	0.43
37	1		0.24	0.28	0.28	0.11	0.11	0.28	0.29	0.29	0.15	0.15
N			**20**	**20**	**20**	**20**	**20**	**20**	**20**	**20**	**20**	**20**
Bias mean			**0.78**	**0.77**	**0.78**	**0.56**	**0.56**	**0.88**	**0.90**	**0.91**	**0.58**	**0.59**
Standard deviation			**0.62**	**0.49**	**0.51**	**0.34**	**0.33**	**0.61**	**0.62**	**0.63**	**0.35**	**0.34**
COV			**0.79**	**0.64**	**0.65**	**0.61**	**0.59**	**0.69**	**0.69**	**0.69**	**0.60**	**0.59**

(cont.) : Bias factor: Pipe Piles in Cohesive soils

			β				λ					
Case #	situ ID#	lab ID#	2B; T&P (1)	11.5B; T&P (2)	2B; Hara (5h)	11.5B; Hara (6h)	2B; T&P (1)	11.5B; T&P (2)	2B; Hara (5h)	11.5B; Hara (6h)	Schmertmann SPT mobilized	Predominant side resistance
19	1		0.64	0.64	0.51	0.51	0.57	0.57	0.26	0.26	0.25	
38	1		0.79	0.80	0.73	0.75	1.59	1.63	0.87	0.89	0.86	Y
39	1		1.10	1.13	1.02	1.05	2.20	2.32	1.25	1.30	1.34	Y
45	1	1					0.22	0.22	0.17	0.18		Y
46	1	1					0.58	0.58	0.45	0.45		
50	1	1					0.62	0.61	0.46	0.43		
53		2					0.54	0.57	0.54	0.57		
54		1					0.47	0.46	0.47	0.46		Y
55		1					0.53	0.52	0.53	0.52		Y
56	1	0					0.45	0.51	0.24	0.27		
57	1	2	0.29	0.29	0.28	0.28	0.76	0.76	0.49	0.49	0.35	Y
58	1	2	0.20	0.20	0.20	0.20	0.58	0.58	0.38	0.38	0.24	Y
59	1		1.63	1.63	1.42	1.42	1.69	1.69	0.83	0.82	0.74	Y
60	1		0.32	0.33	0.30	0.32	0.55	0.58	0.32	0.34	0.23	Y
61	1		0.42	0.42	0.39	0.39	0.61	0.61	0.33	0.33	0.24	
76	1		0.35	0.35	0.34	0.34	0.62	0.62	0.42	0.42	0.52	
78	1		0.41	0.41	0.40	0.40	0.85	0.85	0.58	0.58	0.67	Y
32	2		0.37	0.37	0.35	0.34	0.58	0.57	0.30	0.29	0.23	
34	2		0.41	0.41	0.38	0.38	0.68	0.68	0.33	0.33	0.21	Y
37	1		0.15	0.15	0.14	0.14	0.30	0.30	0.15	0.15	0.12	Y
N			**13**	**13**	**13**	**13**	**20**	**20**	**20**	**20**	**13**	
Bias mean			**0.54**	**0.55**	**0.50**	**0.50**	**0.75**	**0.76**	**0.47**	**0.47**	**0.46**	
Standard deviation			**0.42**	**0.42**	**0.36**	**0.36**	**0.50**	**0.52**	**0.26**	**0.27**	**0.35**	
COV			**0.76**	**0.76**	**0.73**	**0.73**	**0.66**	**0.68**	**0.56**	**0.57**	**0.77**	

Table 49: Bias factor: Pipe Piles in Mixed soils

Case #	situ ID#	lab ID#	α API, Nordlund, Thurman 40;2B;P;T&P(1)	36;2B;P (5)	36;11.5B;Sch;T&P(8)	36;2B;P;Hara(5h)	β; Thurman 40;2B;P;T&P(1)	36;2B;P (5)	36;11.5B;Sch;T&P(8)	36;2B;P;Hara(5h)
12	2	1.07	0.99	1.09	0.68	0.93	1.00	1.00	0.71	1.00
13	1	1.23	1.09	0.99	0.81	1.00	1.09	1.09	0.77	1.09
14	1	0.43	0.41	0.45	0.34	0.40	0.43	0.43	0.43	0.43
17	2	0.87	0.86	0.76	0.81	0.58	0.49	0.49	0.49	0.45
20	1	1.12	1.10	1.08	0.64	1.02	0.89	0.89	0.64	0.89
21	1	0.68	0.68	0.70	0.53	0.65	0.58	0.58	0.55	0.58
22	3	0.44	0.23	0.32	0.40	0.42	0.26	0.47	0.47	0.47
23	1	0.89	0.67	0.57	0.87	0.71	0.39	0.43	0.43	0.43
24	2	0.56	0.40	0.33	0.55	0.43	0.29	0.34	0.34	0.34
25	2	0.71	0.51	0.43	0.70	0.55	0.37	0.43	0.43	0.43
26	1	1.08	1.01	0.77	1.02	0.80	0.65	0.65	0.65	0.63
27	1	0.77	0.70	0.53	0.75	0.57	0.36	0.36	0.36	0.35
28	2	0.37	0.26	0.24	0.36	0.29	0.20	0.22	0.22	0.22
29	2	1.63	1.59	1.06	1.62	1.07	0.76	0.76	0.76	0.76
30	1	1.22	1.11	0.91	1.18	0.90	0.57	0.57	0.57	0.56
33	1	1.41	1.06	1.02	1.36	1.28	0.90	1.10	1.10	1.10
35	3	3.09	3.09	2.09	3.02	1.90	1.68	1.68	1.68	1.58
36	3	3.06	3.06	2.10	2.99	1.89	1.67	1.67	1.67	1.56
42	1	0.18	0.18	0.27	0.16	0.13				
47	1	0.21	0.21	0.34	0.21	0.15				
48	1	0.37	0.37	0.50	0.35	0.27	0.14	0.14	0.14	0.14
49	1	0.23	0.23	0.35	0.23	0.15	0.09	0.09	0.09	0.09
62	1									
63	1									
64	1	0.71	0.56	0.73	0.67	0.66	0.46	0.52	0.52	0.52
66	1	0.93	0.86	0.84	0.89	0.78	0.57	0.58	0.58	0.58
67	1	0.67	0.34	0.58	0.67	0.64	0.36	0.60	0.60	0.60
68	1	0.72	0.35	0.90	0.72	0.69	0.37	0.61	0.61	0.61
69	1	1.01	1.01	1.17	0.65	0.87				
70	1	1.09	0.86	0.85	1.08	0.86	0.69	0.82	0.82	0.82
72	2	0.68	0.65	0.48	0.67	0.50	0.31	0.31	0.31	0.31
73	1	1.12	1.10	0.78	1.12	0.79	0.56	0.56	0.56	0.54
74	2	0.97	0.90	0.68	0.95	0.71	0.56	0.56	0.56	0.54
75	2	0.80	0.77	0.57	0.80	0.58	0.36	0.36	0.36	0.35
77	2	0.53	0.52	0.77	0.53	0.37	0.28	0.28	0.28	0.26
1	1	1.00	1.00	0.68	0.71	0.67	0.46	0.46	0.39	0.46
5	1									
N		**34**	**34**	**34**	**34**	**34**	**31**	**31**	**31**	**31**
Bias mean			**0.85**	**0.94**	**0.85**	**0.71**	**0.57**	**0.61**	**0.58**	**0.60**
Standard deviation			**0.66**	**0.64**	**0.63**	**0.41**	**0.38**	**0.38**	**0.36**	**0.36**
COV			**0.78**	**0.69**	**0.74**	**0.57**	**0.66**	**0.61**	**0.61**	**0.60**

(cont.): Bias factor: Pipe Piles in Mixed soils

Case #	situ ID#	lab ID#	α Tomlinson, Nordlund and Thurman DRIVEN	40;2B;P; T&P(1)	36;2B;P (5)	36;11.5B;Sch;T&P(8)	36;2B;P;Hara(5h)	Schmertmann SPT mobilized	Predominant side resistance
12	2							1.23	
13	1							1.10	
14	1							0.70	
17	2		0.47	0.54	0.54	0.52	0.32	0.58	Y
20	1							0.69	
21	1							0.72	
22	3							0.81	
23	1							0.88	Y
24	2							0.54	Y
25	2							0.68	Y
26	1		1.20	1.18	1.27	1.19	1.03	1.07	Y
27	1		0.91	0.87	0.98	0.94	0.78	0.67	Y
28	2							0.37	Y
29	2							0.96	Y
30	1		1.45	1.38	1.55	1.49	1.23	1.06	Y
33	1							1.23	
35	3							1.20	
36	3							1.20	
42	1		0.19	0.20	0.20	0.17	0.16		
47	1	1	0.13	0.15	0.15	0.15	0.12		
48	1	1	0.48	0.50	0.50	0.47	0.33	0.16	
49	1	1	0.16	0.17	0.17	0.16	0.13	0.07	
62	1							0.72	
63	1							0.75	
64	1							0.72	
66	1	0						1.08	Y
67	1							0.58	
68	1							0.76	
69	1	1							
70	1							3.95	
72	2		0.72	0.73	0.77	0.76	0.52	0.58	
73	1		0.93	0.99	1.01	1.01	0.64	0.84	Y
74	2		0.97	0.98	1.06	1.03	0.80	0.73	
75	2		0.89	0.87	0.90	0.90	0.59	0.70	
77	2		0.48	0.49	0.49	0.49	0.31	0.37	
1	1							0.42	Y
5	1							0.86	
N			**13**	**13**	**13**	**13**	**13**	**34**	
Bias mean			**0.69**	**0.70**	**0.74**	**0.72**	**0.54**	**0.85**	
Standard deviation			**0.41**	**0.39**	**0.44**	**0.43**	**0.35**	**0.62**	
COV			**0.60**	**0.57**	**0.60**	**0.59**	**0.66**	**0.73**	

H. AXIAL CAPACITIES--PLUGGED H PILES

Table 50: Axial capacities (KN): Plugged H piles in Cohesionless soils

ID #	situ ID#	Davisson	Nordund; φ correlated by Peck et al.			β (w/ Tip influence zone: 2B)		Meyerhof	Schmertmann SPT mobilized
			limit φ ≤40°; Tip influence zone: 2B (1)	limit φ ≤36°; Tip influence zone: 2B (5)	limit φ ≤36°; Tip influence zone: 11.5B (6)	limit φ ≤40° (1)	limit φ ≤36°; (5)		
166	1	1,160	2,344	1,649	1,649	2,359	2,048	1,902	1,411
167	1	695	2,044	1,157	1,157	2,233	1,500	1,550	777
168	2	1,100	1,315	1,315	1,020	1,498	1,498	906	586
170	3	585	783	766	696	959	959	984	477
171	1	475	723	723	722	1,029	1,029	525	356
189	2	2,100	5,080	2,739	2,457	4,792	3,104	3,857	1,447
545	1	2,150	1,205	1,205	1,425	1,436	1,436	1,471	929
561	1	3,300	4,395	2,344	2,344	4,149	2,393	3,209	1,187
616	1	555	535	535	486	634	634	989	419
1136	1	1,250	1,569	1,538	1,489	1,983	1,983	1,100	883
1137	1	630	1,200	1,169	1,069	1,506	1,506	781	757
1171	1	1,470	1,459	918	918	1,431	955	848	673
1180	1	1,060	2,018	1,434	1,434	2,095	1,773	2,275	1,232
1182	1	1,750	4,618	3,780	3,673	5,419	4,965	2,657	2,469
1183	1	700	2,282	1,606	1,606	2,163	1,826	2,355	1,268
1185	1	2,200	5,394	4,277	4,131	5,636	4,998	2,775	2,491
3222	1	4,620	5,909	3,930	3,812	5,862	4,106	4,328	1,919
3230	1	1,080	700	698	678	811	811	1,038	565
3244	1	1,350	2,084	1,284	1,284	2,330	1,700	1,328	1,046

Table 51: Axial capacities (KN): Plugged H piles in Cohesive soils

ID #	situ ID#	lab ID#	Davisson	α Tomlinson		α API		β		λ		Schmert-mann SPT mobilized
				S_u correlated by Terzaghi & Peck (5)	S_u correlated by Hara (5h)	S_u correlated by Terzaghi & Peck (5)	S_u correlated by Hara (5h)	S_u correlated by Terzaghi & Peck (5)	S_u correlated by Hara (5h)	S_u correlated by Terzaghi & Peck (5)	S_u correlated by Hara (5h)	
193	1		700									900
194	1		910	rock	rock	rock	rock	rock	rock	rock	rock	907
196	1		740									1,077
625		1	1,200	820	820	708	708			990	990	
626		1	1,200	1,362	1,362	1,093	1,093			1,285	1,285	
627		1	1,200	1,357	1,357	1,086	1,086			1,280	1,280	
628		1	1,000	1,367	1,367	1,099	1,099			1,291	1,291	
629		1	1,200	1,253	1,253	1,074	1,074			1,236	1,236	
631		1	1,600	1,527	1,527	1,308	1,308			1,459	1,459	
1015	1		1,490	1,193	2,511	1,134	1,940	2,083	2,236	1,037	1,821	452
1102		1	285	983	983	792	792			764	764	
1103		1	318	689	689	538	538			582	582	
1104		1	555	705	705	551	551			592	592	
1105		1	270	705	705	551	551			592	592	
1133	1	1	395	407	452	364	429			777	1,056	
1139		1	860	1,768	1,768	1,540	1,540			1,650	1,650	
1142		1	470	806	806	709	709			1,060	1,060	
1149	1		482	393	487	256	504	450	513	434	889	285
1150	1		440	424	530	1,221	2,021	2,171	2,307	1,268	2,156	892
1151	1		540	936	1,119	642	1,129	1,206	1,278	820	1,507	593
3233	1		1,390	rock	rock	rock	rock	rock	rock	rock	rock	1,184
3234	1		1,500									928

Table 52: Axial capacities (KN): Plugged H piles in Mixed soils

ID #	situ ID#	lab ID#	Davisson	α Tomlinson, Nordlund, Thurman		α API, Nordlund, Thurman		β; Thurman		Schmertmann SPT mobilized
				φ correlated by Peck et al. and limit below 36^0						
				S_u correlated by Terzaghi & Peck (5)	S_u correlated by Hara (5h)	S_u correlated by Terzaghi & Peck (5)	S_u correlated by Hara (5h)	S_u correlated by Terzaghi & Peck (5)	S_u correlated by Hara (5h)	
150	1		1,110	1,962	2,988	1,738	2,337	3,065	3,218	1,205
151	1		1,020	1,579	2,586	1,299	1,852	2,459	2,590	1,040
152	1		920	1,541	2,505	1,229	1,734	2,269	2,406	964
153	1		1,070	1,675	2,708	1,448	2,087	2,728	2,874	1,164
154	1		960	1,700	2,738	1,477	2,112	2,751	2,897	1,117
155	1		740	1,704	2,735	1,398	1,965	2,588	2,735	1,042
156	1		940			1,819	2,412	3,333	3,333	1,499
157	1		800	1,893	2,712	1,699	2,131	2,714	2,858	1,109
158	1		540	1,592	2,337	1,425	1,835	2,292	2,418	1,121
159	1		335	1,098	1,575	1,000	1,251	1,619	1,696	728
160	1		830	1,534	2,239	1,493	1,974	2,730	2,828	1,113
161	1		600	1,479	2,531	1,229	1,828	2,502	2,618	999
162	1		680	2,166	3,317	2,210	3,008	4,104	4,263	1,619
163	1		355	1,152	2,018	779	1,192	1,503	1,620	705
164	1		680	1,428	2,235	1,140	1,537	1,940	2,067	826
165	1		1,240	1,093	1,965	838	1,368	1,685	1,807	725
177	1		2,950	1,495	2,538	1,200	1,898	2,279	2,446	1,239
179	1		2,700	1,374	2,082	1,037	1,640	1,803	1,945	1,029
182	1		2,200	3,221	4,979	2,141	3,005	4,169	4,397	1,387
184	1		2,430	2,770	4,386	1,817	2,592	3,271	3,499	1,374
190	1		1,500							1,366
191	1		1,100	rock	rock	rock	rock	rock	rock	617
192	1		1,220							1,143
195	1		1,450							936
546	1		2,520	2,562	3,308	2,047	3,404	4,057	4,198	1,471
1012	3		1,850			867	1,450	1,751	1,751	999
1108	1	1	1,950			2,016	2,522			
1111	1		810	1,015	1,554	1,020	1,556	1,756	1,849	887
1122	1		1,930			3,451	3,899	4,417	4,417	1,724
1123	1		2,080			3,642	4,090	4,648	4,648	1,779
1127	1	1	1,380			2,192	2,192			2,199
1160	1		3,500			3,936	4,350	5,195	5,195	2,482
1173	1		2,750			3,087	3,791	5,107	5,107	1,786
1186	1		2,280			1,721	1,781	1,987	1,987	1,291
1189	1	1	1,100			1,632	1,774	2,269	2,269	1,139
1191	1	1	1,120			1,457	1,711	1,972	1,993	615
3227	1		3,250			5,172	5,677	6,791	6,791	2,903
3228	1		3,570			3,327	4,399	6,421	6,421	1,779
3232	1		2,605	rock	rock	rock	rock	rock	rock	1,378
3241	1		5,500			3,008	3,483	3,444	3,444	2,555
3249	1		1,400	1,860	2,997	1,577	2,300	2,745	2,925	959
3252	1		3,460			3,262	3,547	3,990	3,990	1,632

I. BIAS FACTORS--PLUGGED H PILES

Table 53: Bias factor: Plugged H piles in Cohesionless soils

ID #	situ ID#	lab ID#	Nordund; φ correlated by Peck et al.			β (w/ Tip influence zone: 2B)		Meyerhof	Schmertmann SPT mobilized
			limit φ ≤40⁰; Tip influence zone: 2B (1)	limit φ ≤36⁰; Tip influence zone: 2B (5)	limit φ ≤36⁰; Tip influence zone: 11.5B (6)	limit φ ≤40⁰ (1)	limit φ ≤36⁰; (5)		
166	1		0.49	0.70	0.70	0.49	0.57	0.61	0.82
167	1		0.34	0.60	0.60	0.31	0.46	0.45	0.89
168	2		0.84	0.84	1.08	0.73	0.73	1.21	1.88
170	3		0.75	0.76	0.84	0.61	0.61	0.59	1.23
171	1		0.66	0.66	0.66	0.46	0.46	0.90	1.34
189	2		0.41	0.77	0.85	0.44	0.68	0.54	1.45
545	1		1.78	1.78	1.51	1.50	1.50	1.46	2.31
561	1		0.75	1.41	1.41	0.80	1.38	1.03	2.78
616	1		1.04	1.04	1.14	0.88	0.88	0.56	1.32
1136	1		0.80	0.81	0.84	0.63	0.63	1.14	1.42
1137	1		0.53	0.54	0.59	0.42	0.42	0.81	0.83
1171	1		1.01	1.60	1.60	1.03	1.54	1.73	2.18
1180	1		0.53	0.74	0.74	0.51	0.60	0.47	0.86
1182	1		0.38	0.46	0.48	0.32	0.35	0.66	0.71
1183	1		0.31	0.44	0.44	0.32	0.38	0.30	0.55
1185	1		0.41	0.51	0.53	0.39	0.44	0.79	0.88
3222	1		0.78	1.18	1.21	0.79	1.13	1.07	2.41
3230	1		1.54	1.55	1.59	1.33	1.33	1.04	1.91
3244	1		0.65	1.05	1.05	0.58	0.79	1.02	1.29
		N	19	19	19	19	19	19	19
Bias mean			**0.74**	**0.92**	**0.94**	**0.66**	**0.78**	**0.86**	**1.42**
Standard deviation			0.39	0.41	0.38	0.33	0.40	0.37	0.65
		COV	**0.53**	**0.45**	**0.41**	**0.51**	**0.50**	**0.43**	**0.46**

Table 54: Bias factor: Plugged H piles in Cohesive soils

ID #	situ ID#	lab ID#	α Tomlinson		α API		β		λ		Schmert-mann SPT mobilized
			S_u correlated by Terzaghi & Peck (5)	S_u correlated by Hara (5h)	S_u correlated by Terzaghi & Peck (5)	S_u correlated by Hara (5h)	S_u correlated by Terzaghi & Peck (5)	S_u correlated by Hara (5h)	S_u correlated by Terzaghi & Peck (5)	S_u correlated by Hara (5h)	
193	1		rock	rock	rock	rock	rock	rock	rock	rock	0.78
194	1										1.00
196	1										0.69
625		1	1.46	1.46	1.69	1.69			1.21	1.21	
626		1	0.88	0.88	1.10	1.10			0.93	0.93	
627		1	0.88	0.88	1.10	1.10			0.94	0.94	
628		1	0.73	0.73	0.91	0.91			0.77	0.77	
629		1	0.96	0.96	1.12	1.12			0.97	0.97	
631		1	1.05	1.05	1.22	1.22			1.10	1.10	
1015	1		1.25	0.59	1.31	0.77	0.72	0.67	1.44	0.82	3.30
1102		1	0.29	0.29	0.36	0.36			0.37	0.37	
1103		1	0.46	0.46	0.59	0.59			0.55	0.55	
1104		1	0.79	0.79	1.01	1.01			0.94	0.94	
1105		1	0.38	0.38	0.49	0.49			0.46	0.46	
1133	1	1	0.97	0.87	1.08	0.92			0.51	0.37	
1139		1	0.49	0.49	0.56	0.56			0.52	0.52	
1142		1	0.58	0.58	0.66	0.66			0.44	0.44	
1149	1		1.23	0.99	1.89	0.96	1.07	0.94	1.11	0.54	1.69
1150	1		1.04	0.83	0.36	0.22	0.20	0.19	0.35	0.20	0.49
1151	1		0.58	0.48	0.84	0.48	0.45	0.42	0.66	0.36	0.91
3233	1		rock	rock	rock	rock	rock	rock	rock	rock	1.17
3234	1										1.62
N			17	17	17	17	4	4	17	17	9
Bias mean			**0.82**	**0.75**	**0.96**	**0.83**	**0.61**	**0.55**	**0.78**	**0.68**	**1.29**
Standard deviation			0.33	0.29	0.43	0.37	0.37	0.32	0.33	0.30	0.85
COV			**0.40**	**0.39**	**0.45**	**0.44**	**0.61**	**0.58**	**0.42**	**0.45**	**0.66**

Table 55: Bias factor: Plugged H piles in Mixed soils

ID #	situ ID#	lab ID#	α Tomlinson, Nordlund, Thurman		α API, Nordlund, Thurman		β; Thurman		Schmertmann SPT mobilized
			φ correlated by Peck et al. and limit below 36^0						
			S_u correlated by Terzaghi & Peck (5)	S_u correlated by Hara (5h)	S_u correlated by Terzaghi & Peck (5)	S_u correlated by Hara (5h)	S_u correlated by Terzaghi & Peck (5)	S_u correlated by Hara (5h)	
150	1		0.57	0.37	0.64	0.48	0.36	0.34	0.92
151	1		0.65	0.39	0.79	0.55	0.41	0.39	0.98
152	1		0.60	0.37	0.75	0.53	0.41	0.38	0.95
153	1		0.64	0.40	0.74	0.51	0.39	0.37	0.92
154	1		0.56	0.35	0.65	0.45	0.35	0.33	0.86
155	1		0.43	0.27	0.53	0.38	0.29	0.27	0.71
156	1				0.52	0.39	0.28	0.28	0.63
157	1		0.42	0.29	0.47	0.38	0.29	0.28	0.72
158	1		0.34	0.23	0.38	0.29	0.24	0.22	0.48
159	1		0.31	0.21	0.34	0.27	0.21	0.20	0.46
160	1		0.54	0.37	0.56	0.42	0.30	0.29	0.75
161	1		0.41	0.24	0.49	0.33	0.24	0.23	0.60
162	1		0.31	0.21	0.31	0.23	0.17	0.16	0.42
163	1		0.31	0.18	0.46	0.30	0.24	0.22	0.50
164	1		0.48	0.30	0.60	0.44	0.35	0.33	0.82
165	1		1.13	0.63	1.48	0.91	0.74	0.69	1.71
177	1		1.97	1.16	2.46	1.55	1.29	1.21	2.38
179	1		1.97	1.30	2.60	1.65	1.50	1.39	2.62
182	1		0.68	0.44	1.03	0.73	0.53	0.50	1.59
184	1		0.88	0.55	1.34	0.94	0.74	0.69	1.77
190	1								1.10
191	1		rock	rock	rock	rock	rock	rock	1.78
192	1								1.07
195	1								1.55
546	1		0.98	0.76	1.23	0.74	0.62	0.60	1.71
1012	3				2.13	1.28	1.06	1.06	1.85
1108	1	1			0.97	0.77			
1111	1		0.80	0.52	0.79	0.52	0.46	0.44	0.91
1122	1				0.56	0.50	0.44	0.44	1.12
1123	1				0.57	0.51	0.45	0.45	1.17
1127	1	1			0.63	0.63			0.63
1160	1				0.89	0.80	0.67	0.67	1.41
1173	1				0.89	0.73	0.54	0.54	1.54
1186	1				1.33	1.28	1.15	1.15	1.77
1189	1	1			0.67	0.62	0.48	0.48	0.97
1191	1	1			0.77	0.65	0.57	0.56	1.82
3227	1				0.63	0.57	0.48	0.48	1.12
3228	1				1.07	0.81	0.56	0.56	2.01
3232	1		rock	rock	rock	rock	rock	rock	1.89
3241	1				1.83	1.58	1.60	1.60	2.15
3249	1		0.75	0.47	0.89	0.61	0.51	0.48	1.46
3252	1				1.06	0.98	0.87	0.87	2.12
		N	22	22	37	37	35	35	41
Bias mean			**0.71**	**0.46**	**0.92**	**0.68**	**0.56**	**0.55**	**1.27**
Standard deviation			0.46	0.29	0.56	0.37	0.36	0.35	0.58
		COV	**0.65**	**0.64**	**0.61**	**0.54**	**0.64**	**0.64**	**0.46**

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DRIVEN PILES - STATIC ANALYSIS

DESCRIPTION OF STATIC CAPACITY EVALUATION METHODS

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LIST OF SYMBOLS

α: The α factor in α-Tomlinson or α-API method.

β: The β factor in β-Burland (in cohesive soil) or β-Bushan method (in cohesionless soil).

λ: The λ factor in λ method.

σ_v (or p): Total stress at the depth of interest.

σ_v' (or p'): Effective stress at the depth of interest.

S_u (or C_u): Undrained shear strength.

L: Embedment depth of the pile.

D_B (or D, or D_A): Bearing embedment depth of the pile.

B: The width or diameter of the pile section.

ϕ: Internal friction angle.

Φ: Resistance factor.

RESEARCH METHODOLOGY

A driven pile is one type of deep foundation in which the structural loads are transferred to the ground through the side and tip resistance as well as the stiffness of the pile itself. Lateral loads or batter piles are not within the scope of this project; only the axial loading capacity of vertical piles is investigated. In design, the axial capacity can be predicted by various static capacity prediction methods as presented in Section 1.1. Alternatively, the static capacity can be interpreted from the static load test as in Section 1.2.

1.1. Axial Pile Capacity Prediction by Static Methods

Axial pile capacities can be predicted by various static capacity prediction methods. These methods use certain properties of the soil, such as the effective friction angle, ϕ', or the CPT cone bearing q_c. However, the computed capacities may not reflect the actual capacities exactly, so designs based on static methods must be conservative, which is reflected in the resistance factor, Φ. This section will discuss specific ways to predict the axial pile capacities.

1.1.1. Axial Loading Capacity of a Pile

The ultimate resistance of a pile, R_{ult} (or R_n--Norminal resistance), is given below:

$$R_{ult} = R_P + R_s \qquad \text{Eq. 1}$$

where: pile tip resistance $R_p = q_p A_p$,

pile side resistance $R_s = \sum q_{si} \Delta z_i U$,

q_p = unit tip resistance. Predictions of q_p are in Sections 1.1.2.2, 1.1.2.4 and 1.1.3.

q_s = unit side resistance, which is regarded as constant along segment Δz_i of the pile. Predictions of q_s can be found in Sections 1.1.2.1, 1.1.2.3 and 1.1.3,

U = perimeter of the pile's shaft, and

A_p = area of the tip of the pile.

In this project, the eight methods listed below are considered in predicting the axial capacities:

1. α-method, which include α-Tomlinson (Tomlinson, 1980/1995) and α-API (Reese et al., 1998) for cohesive soil

2. λ-method (Vijayvergiya and Focht, cited in US Army Corps of Engineers, No.7, 1992) for cohesive soil,

3. β-method (Bushan--cited in Bowles, 1996) for cohesionless soil,

4. β-method (Burland, Esrig and Kirby, 1979) for cohesive soil,

5. Nordlund method (Nordlund, 1963) for cohesionless soil,

(Methods number 4 to 5 above are cited in Hannigan et al., 1995).

6. Nottingham and Schmertmann CPT method (McVay and Townsend, 1989) for all soil types,

7. Meyerhof SPT (Meyerhof, 1976/1981) for cohesionless soil,

8. Schmertmann SPT method (Lai and Graham, 1995) for all soil types and rock.

The first five methods, as discussed in Section 1.1.2, are semi-empirical methods. These methods are based on the empirical relationship between soil properties and the stress states (both effective and total stress analyses are used). The last three methods, as discussed in Section 1.1.3, predict the pile capacities based on the data from in-situ tests (SPT and CPT).

1.1.2. Semi-Empirical Methods

Semi-empirical methods are used to relate the adhesion between the pile and the surrounding soil to the internal friction angle, ϕ, or the undrained shear strength, S_u. This section presents five different semi-empirical methods, will be divided into 2 sub-sections: Section 1.1.2.1 for cohesive soil and 1.1.2.2 for cohesionless soil.

1.1.2.1. Side resistance in cohesive soil

In cohesive soil, the side resistance of a pile is usually predicted using the undrained shear strength, S_u, or the over-consolidation ratio, OCR. This section reviews different methods predicting the side resistance in cohesive soil.

1.1.2.1.1. α-Tomlinson method

The α-Tomlinson method (Tomlinson, 1980/1995), based on total stress analysis, is used to relate the adhesion between the pile and a clay to the undrained shear strength of the clay, S_u. The ultimate unit side resistance may be taken as:

$$q_{si} = \alpha S_{ui} \qquad \text{Eq. 2}$$

where: α = adhesion factor (Figure 1), which depends on the bearing embedment in stiff clay and the width of the pile,

S_u (or C_u) = average undrained shear strength of the soil in the segment of interest.

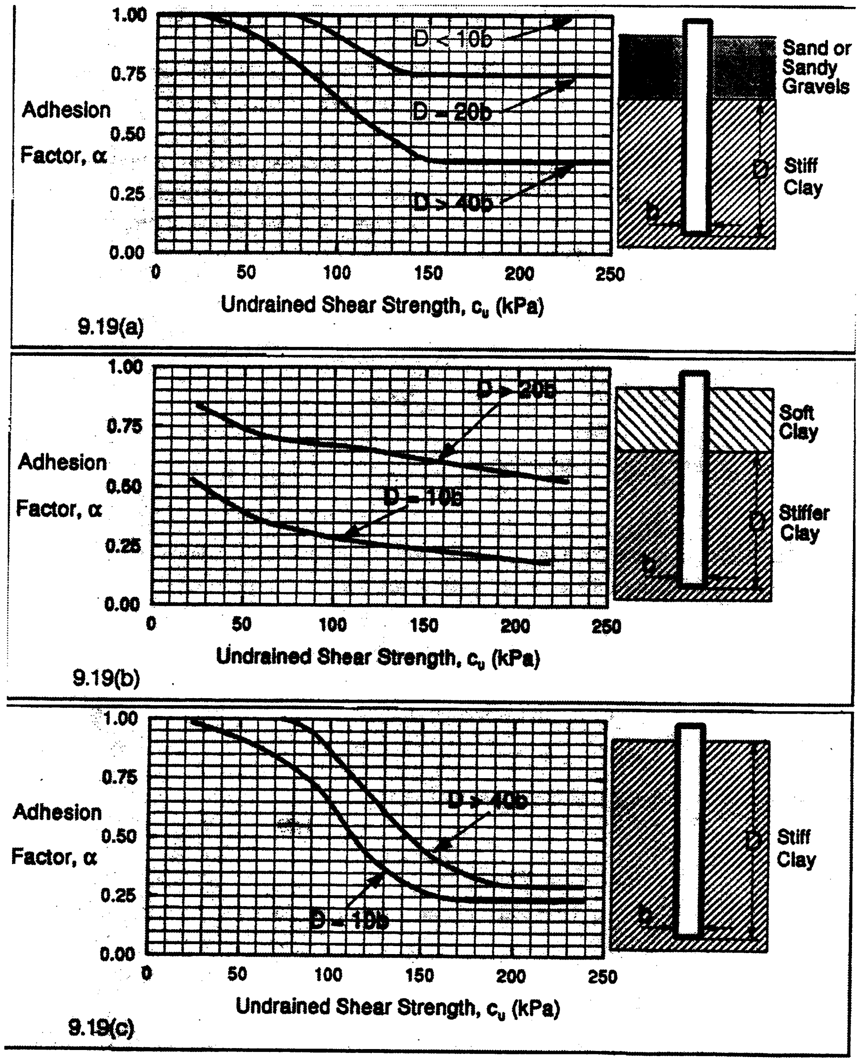


Figure 1: α factors; Tomlinson method (Tomlinson, 1995)

Following is the discussion on the α-Tomlinson method:

The α method is simple to use and it has been used over many years. However, it is a total stress analysis and does not depend on the ground water level; therefore, the resistance based on the α method may not be close to the measured capacity, which depends on the in-situ stress state and the ground water level.

Tomlinson originally developed the α-method for large displacement piles. Therefore, for small-displacement piles (H and pipe piles), the α method may not be suitable. Similarly, the Tomlinson method is only suitable for pile embedded in stiff clay. For cohesive layers that lie above the bearing layer, other methods should be used.

The assumptions behind the α Tomlinson method include the following:

- If there is a soft clay layer above the bearing stiff clay, then the soft clay will be dragged down to the stiff layer and will lower the α factor (see Figure 1),
- Similarly, the cohesionless soil in Figure1 will be dragged down and therefore, the α factor will be increased,
- Other intermediate cases, such as the layer right above the stiff layer is silt, or is only a very thin lens, etc. will decrease the accuracy of the prediction.

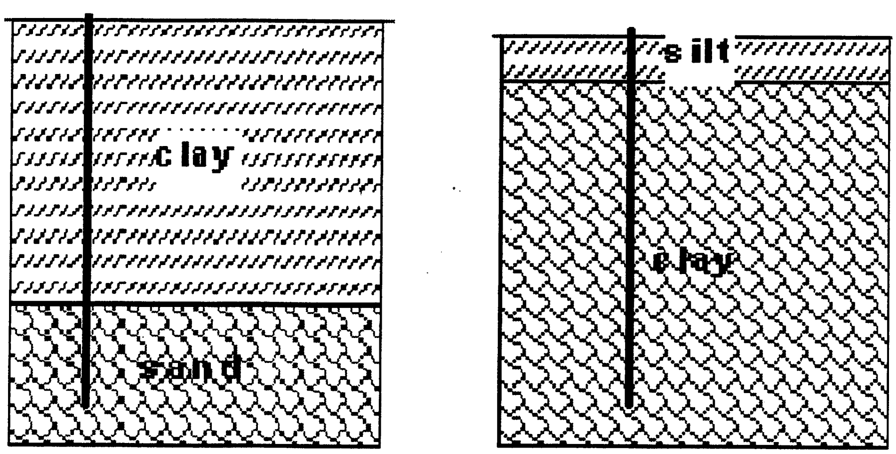


Figure 2: Examples when α-Tomlinson is not applicable.

1.1.2.1.2. α-API revised method (1987)

The α-API method (cited in Reese et al., 1998) is similar to the α-Tomlinson method, and the ultimate unit side resistance, in the same unit as S_u, is taken as:

$$q_{si} = \alpha S_{ui} \qquad \text{Eq. 2}$$

where: $\alpha = 0.5\,\psi^{-0.5}$ if $\psi \leq 1.0$,

$\alpha = 0.5\,\psi^{-0.25}$ if $\psi > 1.0$, and max $\alpha = 1.0$,

$\psi = S_u / \sigma_v'$, and

σ_v' (or p_v') = the vertical effective overburden pressure at the depth of interest.

From Eq. 2, the adhesion factor, α, with different effective stresses is generated in Figure 3.

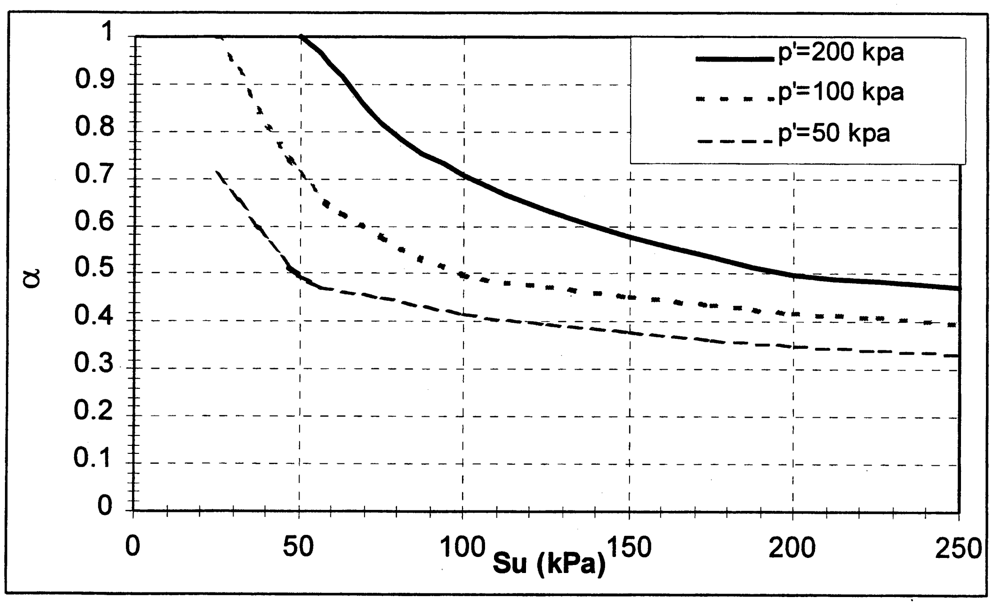


Figure 3: α factors; Revised API 1987 method (generated from Eq. 2)

The α-API method is a mixed method between total stress analysis (S_u) and effective stress analysis (σ_v'). It is much easier to use than the Tomlinson method. For example, the user will have no issue with considering other layers that lie above the bearing layer. Finally, the α-API method has simple equations; thus it is easily automated.

Following is the discussion on the unit side resistance of both α methods:

As shown in Figure 4, the unit side resistance $q_s = \alpha\ S_u$ for the α-API method follows a logical trend: the stiffer the soil (higher S_u), the more the side resistance.

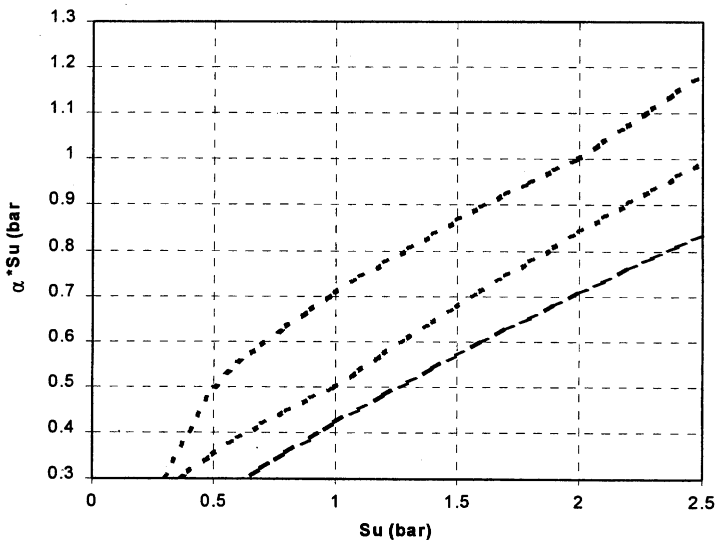


Figure 4: The unit side resistance of α-API method (generated from Eq. 2)

However, as indicated in Figure 5, for the α-Tomlinson 1980 method, when S_u is in the range of about 0.8 to 1.7 bar (80 to 170 kPa), the unit side resistance decreases as the soil becomes stronger, which does not follow traditional wisdom on pile behavior. (In that range, there is very sharp decrease of the α factor).

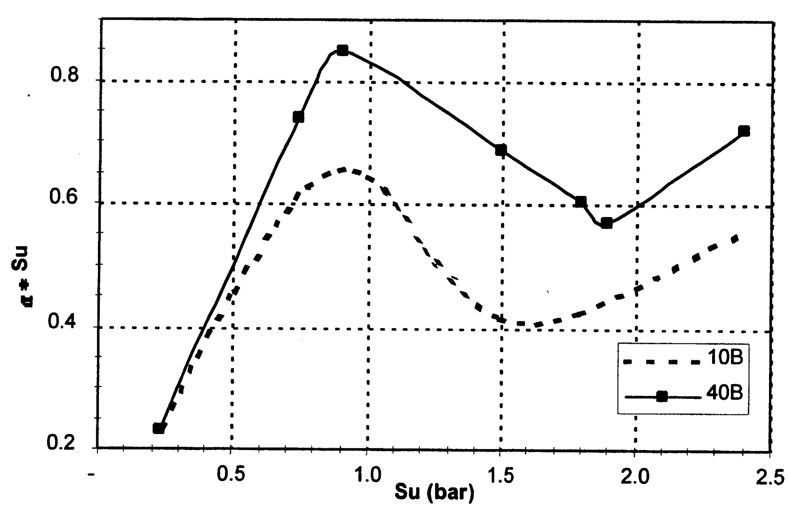


Figure 5: The unit side resistance of α-Tomlinson 1980 case #3 (generated from Figure 1)

1.1.2.1.3. β-Burland method (1973)

From the theory of soil mechanics, based on effective stress analysis, we have $q_s = K (\tan \delta)\, \sigma_v'$,

where: K = horizontal stress ratio,

δ = adhesion angle between soil and piles,

σ_v' = vertical effective stress.

The equation can be rewritten as:

$$q_s = \beta \sigma_v' \qquad \text{Eq. 3}$$

where: σ_v' = vertical effective stress,

β = factor depended on the over-consolidation ratio OCR (Figure 6).

Following is the discussion on the β-Burland method:

Esrig and Kirby (1979) (cited in Hannigan et al., 1995) suggested that for heavily over-consolidated clays, the value of β should not exceed two.

Practically, the OCR ratio is not usually measured in the laboratory. Therefore, this method is difficult to implement. In this project, all OCR ratios are obtained from in-situ tests through correlations (Section 2.2).

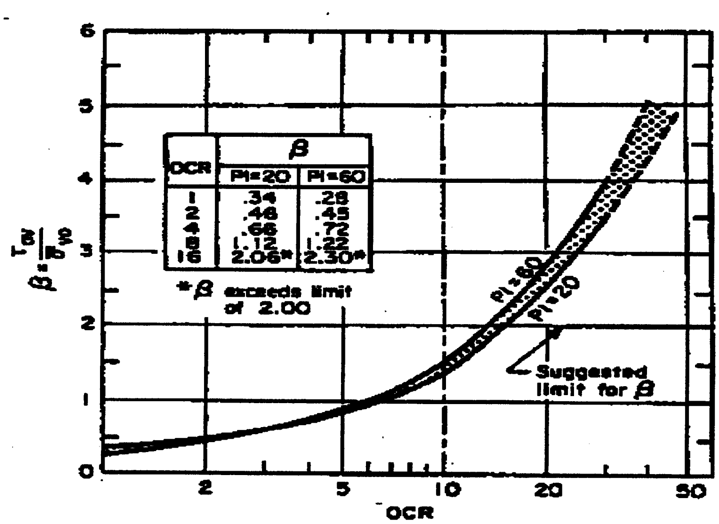


Figure 6: β factors (AASHTO 1996/2000)

1.1.2.1.4. λ-Method

The λ-method (cited in US Army Corps of Engineers, 1992), based on effective and total stress analysis, may be used to relate the unit side resistance to the passive earth pressure as:

$$q_s = \lambda(\sigma' + 2S_u) \qquad \text{Eq. 4}$$

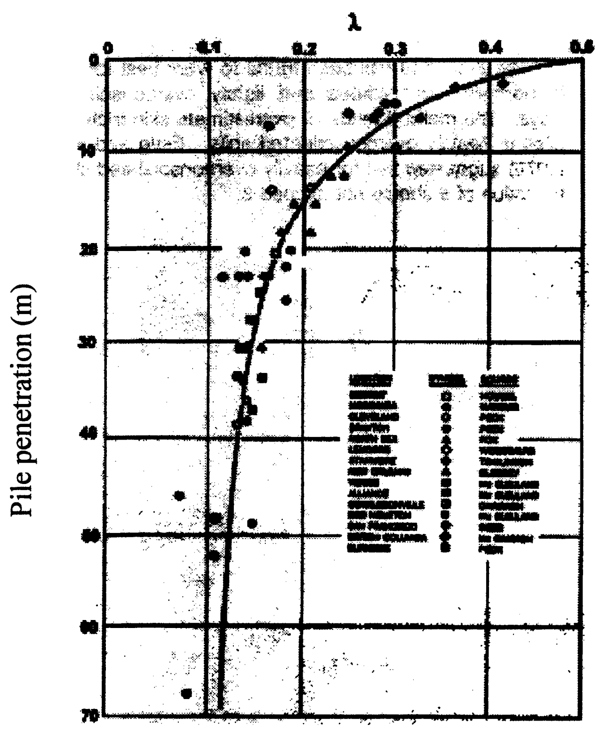


Figure 7: λ factors (US Army Corps of Engineers, 1992)

where: λ = an empirical coefficient, which depends on the pile embedment as shown in Figure 7.

The λ factor was empirically suggested by examining pipe piles in only 15 locations. The main drawback of this method is that it assumes a single value of λ for the whole pile.

1.1.2.2. Tip resistance in cohesive soil

The ultimate unit tip resistance of piles in saturated clay (Reese at al., 1998) may be taken as:

$$q_p = 9\ S_u \qquad \text{Eq. 5}$$

where: S_u = average undrained shear strength in the range from 2B to 3.5B below the tip, and B is the diameter of the pile.

With unsaturated clay, Eq. 5 is still used to predict tip resistance, which may reduce the accuracy of the answer.

1.1.2.3. Side resistance in cohesionless soil

In cohesionless soil, the side resistance of a pile is usually predicted using the adhesion angle, δ, or the relative density, D_r. The adhesion angle, δ, is related to the internal friction angle of the soil, φ, through the volume displacement, the material, the shape of the pile and the roughness of the pile. This section reviews different methods predicting the side resistance in cohesionless soil.

1.1.2.3.1. β-Bushan method (1982)

Similar to the β method in cohesive soil, the unit side resistance is related to effective stress as (cited in Hannigan et al., 1995):

$$q_s = \beta\sigma_v' \qquad \text{Eq. 3}$$

where: $\beta = 0.18 + 0.65\ D_r$, and

D_r = Relative density in decimals. Therefore, max $\beta = 0.83$ when $D_r = 1$ (much lower than max β in cohesive soil--Section 1.1.2.1.3.)

1.1.2.3.2. Nordlund method

Nordlund (cited in Hannigan et al., 1995) developed the following equation for the unit resistance:

$$q_s = K_\delta C_F \sigma_v' \frac{\sin(\delta + \varpi)}{\cos\varpi} \qquad \text{Eq. 6}$$

where: K_δ = Coefficient of lateral earth pressure at the depth of interest. (Figures 8 to 11),

δ = friction angle between pile and soil. For non-taper piles: $\delta \le \phi$. (Figure 12),

C_F = Correction factor for K_δ when $\delta \neq \phi$. $C_F \approx 0.6$ to 1.0. (Figure 13),

σ_v' = effective over-burden pressure at the center of the layer of interest, and

ω = angle of the pile taper from vertical.

For a uniform cross section pile ($\omega = 0$), the Nordlund equation becomes

$$q_s = K_\delta C_F \sigma_v' \sin\delta \qquad \text{Eq. 7}$$

The equation of this semi-empirical method is somewhat similar to the theory of soil mechanics, in which $q_s = K_\delta \sigma' \tan \delta$ Eq. 8

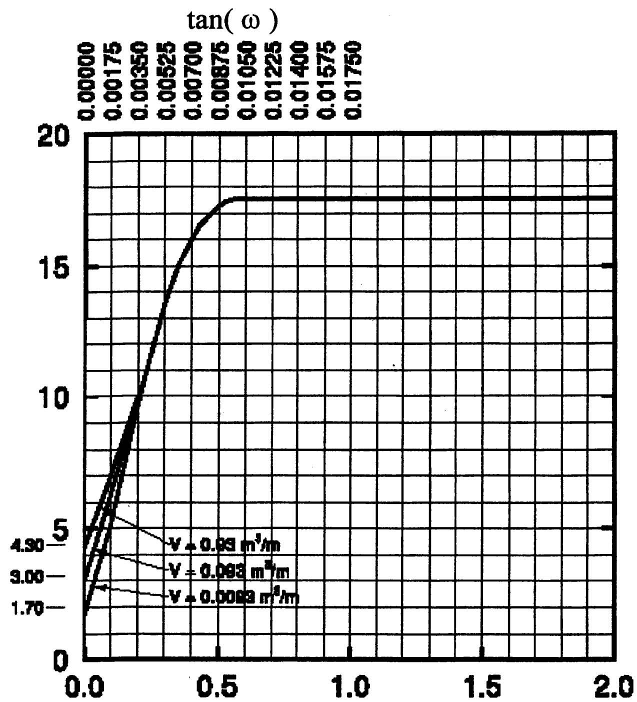


Figure 8: Design curve for evaluating K_δ when $\phi = 40$ (Norlund 1963--cited in Hannigan et al., 1995)

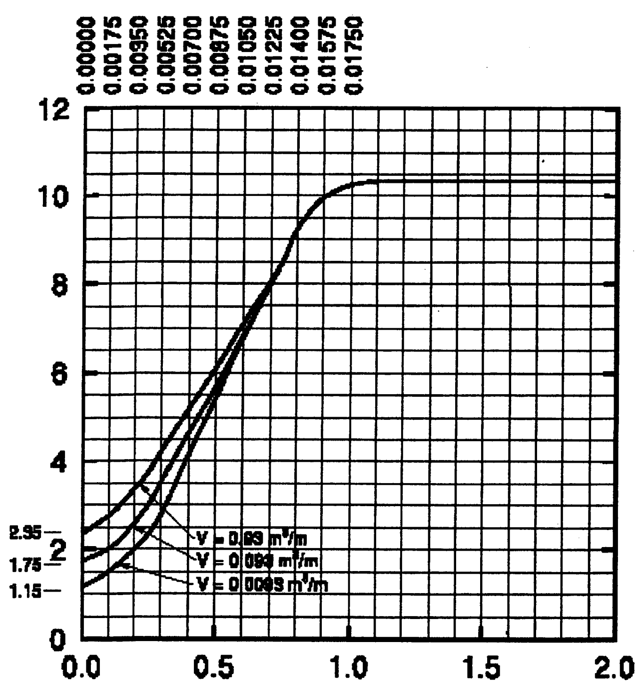


Figure 9: Design curve for evaluating K_δ when $\phi = 35$ (Norlund 1963--cited in Hannigan et al., 1995)

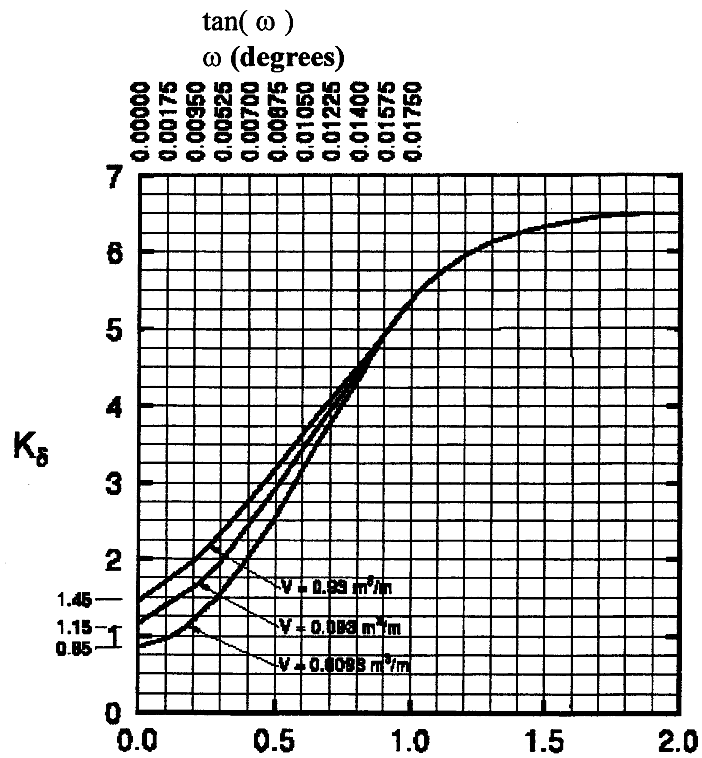


Figure 10: Design curve for evaluating K_δ when $\phi = 30$ (Norlund 1963--cited in Hannigan et al., 1995)

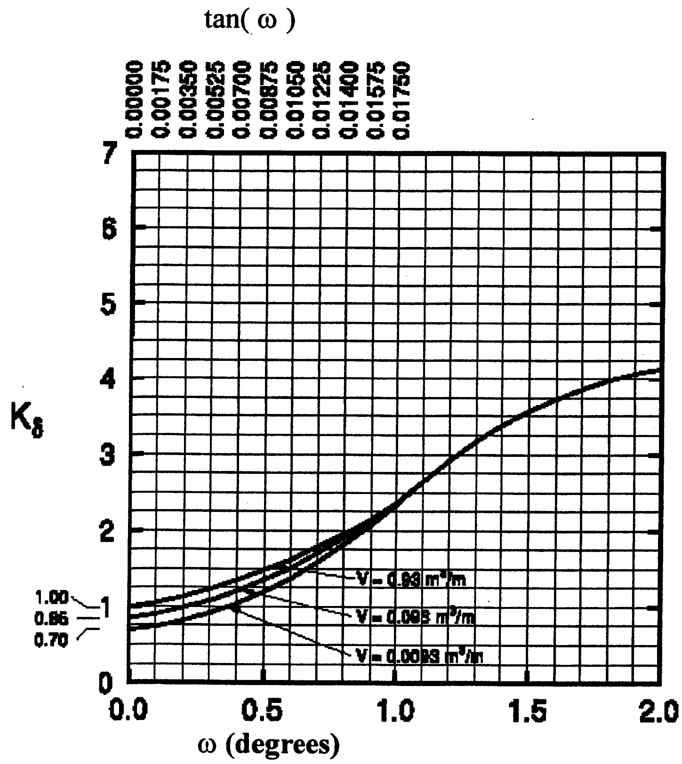
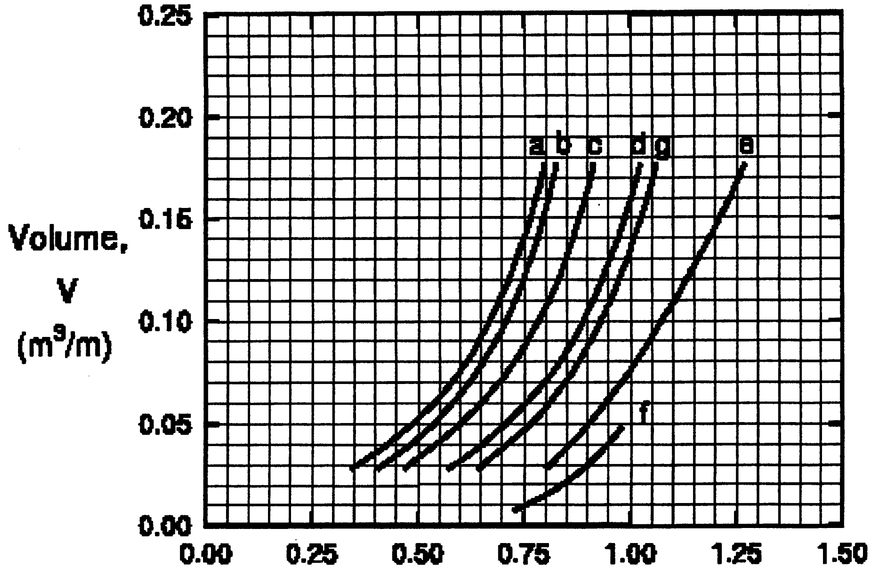
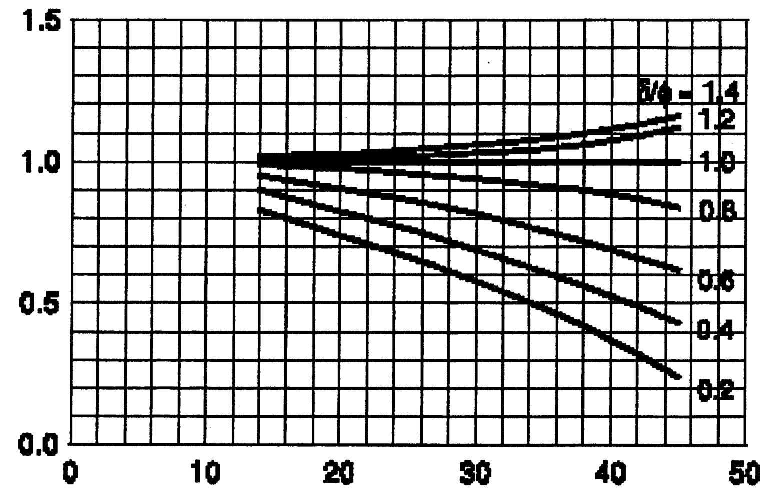


Figure 11: Design curve for evaluating K_δ when $\phi = 25$ (Norlund 1963--cited in Hannigan et al., 1995)



a. Pipe piles
b. Timber piles
c. Concrete piles
d. Raymond step taper piles
e. Raymond uniform taper piles
f. H piles
g. Tapered portion of monotube piles

Figure 12: Relation δ/ϕ and pile displacement
(Norlund 1963--cited in Hannigan et al., 1995)



φ (degrees)

Figure 13:Correction factor (C_F) for K_δ
(Norlund 1963--cited in Hannigan et al., 1995)

1.1.2.4. Tip resistance in cohesionless soil--Thurman method

From bearing capacity theory, Thurman (cited in Hannigan et al., 1995) related the unit tip resistance in sand with effective stress as:

$$q_p = \alpha_t N'_q \sigma_v' \qquad \text{Eq. 9}$$

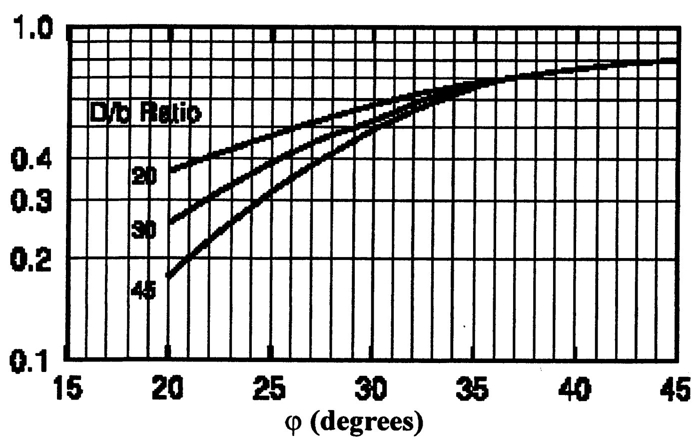
where: α_t = dimensionless factor (Figure 14),

N'_q = Bearing capacity factor (Figure 15),

σ_v' = effective overburden pressure at the pile tip. σ_v' is limited to 150 kPa (tip resistance reaches a limiting value at some distance below the ground),

q_p also has a limit as shown in Figure 16.

N'_q is very high at high internal friction angles (N'_q>250 when ϕ>42°). Therefore, some software, e.g. DRIVEN (FHWA, 1998) recommends the limit of only 36° for ϕ.



D: Embedded pile length
b: Pile diameter or width

Figure 14: α_T coefficient (FHWA--DRIVEN, 1998)

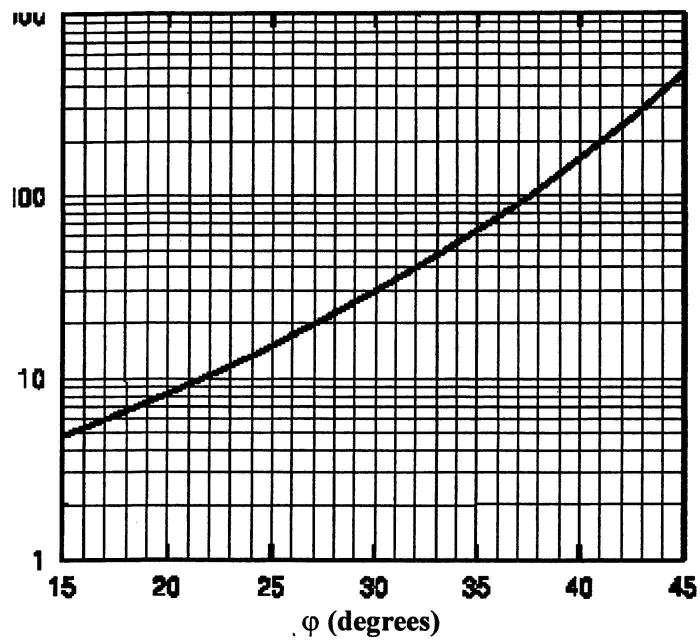


Figure 15: Bearing capacity factor Nq'(FHWA--DRIVEN, 1998)

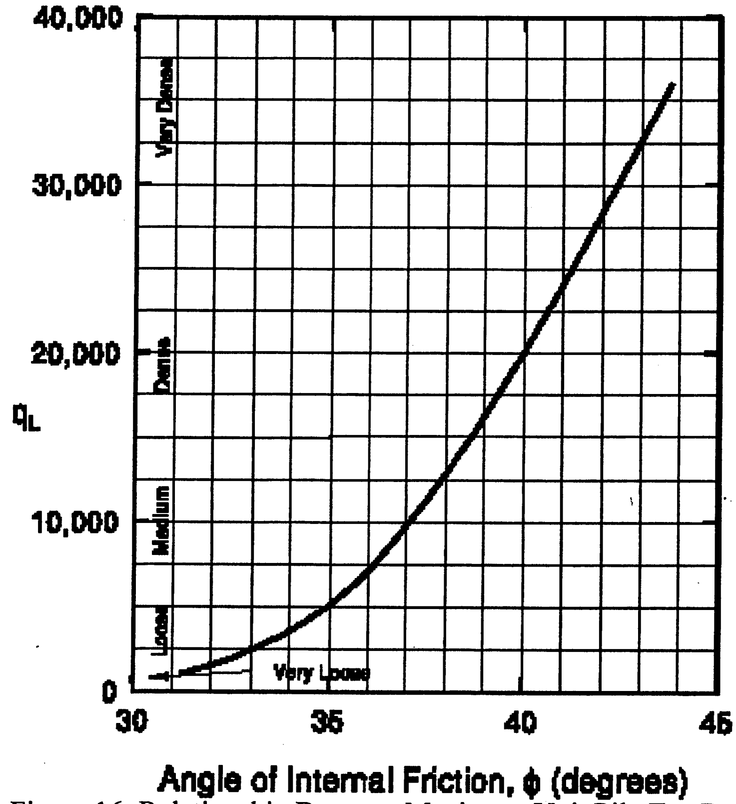


Figure 16: Relationship Between Maximum Unit Pile Toe Resistance q_L (kPa) and Friction Angle for Cohesionless Soils (Meyerhof, 1976/1981).

1.1.3. Empirical Methods

This section presents different empirical methods, which directly relate the resistances with the in-situ test results, i.e. the N-value and the CPT readings.

1.1.3.1. Meyerhof method for piles in cohesionless soil

There are more than four different variations of Meyerhof 1976 method. Different agencies have different guidelines, e.g. EM 1110-1-1905 (US Army Corps of Engineers, 1992), leading to different results, even though the method is still referred to as Meyerhof 1976 method. The primary variations of the side resistance of the method are listed below:

1) AASHTO provision (AASHTO 1996/2000): $q_s = k' N$ (kPa).

2) Meyerhof original paper (Meyerhof, 1976/1981): $q_s = k\ N$ (kPa).

3) Bowles (Bowles, 1996): $q_s = k\ N'$ (kPa).

4) U.S. CORPS of Engineers: No guidelines.

where: N = Uncorrected blow count,

N' = Corrected blow count,

k = 2 for displacement piles, and k = 1 for small displacement piles.

AASHTO (AASHTO 1996/2000) uses strict unit conversion, therefore k' = 1.9 for displacement piles and 0.96 for small displacement piles.

The side resistance should be limited as given below:

$$q_s \leq 100 \text{ kPa}$$

The tip resistance for Meyerhof method is similarly obtained as:

1) AASHTO provision (AASHTO 1996/2000): $q_p = 0.38\ N_t'\ D/B$ (bar).

where: N_t' = Corrected blow count near the pile tip,

D = The embedment of the pile in cohesionless soil, and

B = The diameter or width of the pile cross-section.

2) Original paper by Meyerhof (Meyerhof, 1976/1981): $q_p = 0.4\ N_t\ D/B$ (bar).

where: N_t = Uncorrected blow count near the pile tip.

It should be noted that, in 1976, the concept of correcting the N-value due to the overburden pressure was very new. Therefore, Meyerhof did not correct N. Similarly, in Meyerhof's paper, he used approximate conversion: 1 tsf ≈ 100 kPa (1 bar).

3) Bowles (Bowles, 1996): $q_p = 0.4\ N_{8+3B}'\ D/B$ (bar).

where: N_{8+3B}' = Corrected blow count in the depth of 8B above tip and 3B below tip.

4) EM 1110-1-1905 (U.S. Army CORPS of Engineers, 1992)

$$q_p = 0.38\ N_{8+3B}\ D/B \quad \text{(bar)} = 0.8\ N_{8+3B}\ D/B \quad \text{(ksf)}.$$

where: N_{8+3B} = Uncorrected N in the depth of 8B above tip and 3B below tip.

Similar to the side resistance calculations, the value of the maximum tip resistance is also limited as shown below:

1) AASHTO provision (AASHTO 1996/2000)

q_L = 4 N_t' in sand,

= 3 N_t' in silt.

2) Meyerhof original paper (Meyerhof, 1976/1981)

q_L = 4 N_t in sand,

= 3 N_t in silt.

3) Bowles (Bowles, 1996)

q_L = 3.8 N_{8+3B}'.

4) EM 1110-1-1905 (U.S. Army CORPS of Engineers, 1992)

q_L = 3.8 N_{8+3B} (bar) = 8 N_{8+3B} (ksf).

1.1.3.2. Schmertmann method for SPT

The procedure of the Schmertmann method for SPT (Lai and Graham, 1995) is described below. First of all, the SPT blow count N is adjusted as shown below:

If $N < 5$ then $N = 0$ (ignores side resistance in weak soil), and

If $N \geq 60$ then $N = 60$ (limit on side resistance)

Discussion:

It is conservative to ignore side resistance when N is in the range of 4 to 5. For example, in Luling bridge, ID 1001, the soil is mostly clay with $N < 5$. The truncation of $N < 5$ significantly lowers the predicted capacity compared to the measured capacity.

The ultimate side resistance for different types of piles and soil types is presented in Table 1.

Table 1: Side resistance--Schmertmann method for SPT

Ty-Pe	Description	Ultimate unit side resistance q_s(tsf)		
		Concrete	Steel H piles	pipe piles
1	Plastic clay	2.0N(110-N) /4006.6	2N(110-N)/5335.94	0.949+0.238lnN
2	Clay-silt-sand mixtures Very silty sand, silts	2.0N(110-N) /4583.3	$-0.0227+0.033N-4.57610^{-4}*N^2+2.465E-6*N^3$	0.243+0.147lnN
3	Clean sands	0.019N	0.0116N	0.058+0.152lnN
4	Soft limestone, very shelly sand	0.01N	0.0076N	0.018+0.134lnN

At any point A, the unit tip resistance is

$$q_{p@A} = \frac{\text{weighted average of } q_p \text{ 8B above A} + \text{weighted average of } q_p \text{ 3.5B below A}}{2}$$

The weighted average of q_p is based on values calculated from Table 2.

Table 2: Tip resistance--Schmertmann method for SPT

Type	**Description**	**Ultimate** unit end bearing q_p (tsf)	
		Concrete and H piles	Pipe piles
1	Plastic clay	0.7 N	0.48 N
2	Clay-silt-sand mixtures Very silty sand, silts	1.6 N	0.96 N
3	Clean sands	3.2 N	1.312 N
4	Soft limestone, very shelly sand	3.6 N	1.92 N

For concrete and H piles, the mobilized tip resistance is expected to be one third (1/3) of the ultimate tip resistance. For pipe piles, the mobilized tip resistance is expected to be one half (1/2) of the ultimate tip resistance.

The ultimate resistance is only fully mobilized when the bearing embedment is sufficient, i.e. $D_A = D_C$

where: D_A = Actual bearing embedment, and D_C = Critical bearing embedment, which is shown in Table 3.

Table 3: Critical depth ratio--Schmertmann method for SPT

Soil Type	Description	Critical depth ratio (D_C/B)
1	Plastic clay	2
2	Clay-silt-sand mixtures Very silty sand, silts	4
3	Clean sands N = 12 or less N = 30 or less N greater than 30	6 9 12
4	Soft limestone, very shelly sand	6

If $D_A < D_C$ and the bearing layer is stronger than the overlying layer, then:

$$q_p = q_{LC} + \frac{D_A}{D_C}(q_T - q_{LC}) \qquad \text{Eq. 10}$$

where: q_p = Reduced tip resistance,

q_{LC} = Unit tip resistance at layer change, and

q_T = Uncorrected unit tip resistance at pile tip.

$$CSFBL = \frac{SFBL}{q_T}\left[q_{LC} + \frac{D_A}{2D_C}(q_T - q_{LC})\right] \qquad \text{Eq. 11}$$

where: CSFBL = Reduced side resistance in the bearing layer, and

SFBL = Uncorrected side resistance in the bearing layer.

If $D_A > D_C$ and the bearing layer is stronger than the overlying layer, then:

$$CSFACD = \frac{USFACD}{q_{CD}}\left[q_{LC} + 0.5(q_{CD} - q_{LC})\right] \qquad \text{Eq. 12}$$

where:

CSFACD = Corrected side resistance between the top of the bearing layer and the critical depth,

USFACD = uncorrected side resistance in the bearing layer from the top of the bearing layer to the critical depth, and

q_{CD} = unit tip resistance at critical depth.

1.1.3.3. Nottingham and Schmertmann method for CPT

The procedure of the Nottingham and Schmertmann method for CPT (McVay and Townsend, 1989) is described below.

The ultimate side resistance of piles may be taken as:

From ground level down to the depth of 8B: $q_s = K\ f_s/2$ Eq. 13

From 8B down to the tip: $q_s = K\ f_s$ Eq. 14

where: f_s = sleeve friction from CPT data,

In cohesionless soil, the ratio K (or Ks) is a function of the ratio between penetration depth(Z) and pile width (D)--see Figure 17.

In cohesive soils: K is written as α (or Kc), which is the α factor, similar to the α-method--see Figure 17.

Similarly, the ultimate tip resistance of piles may be taken as:

$$q_p = \frac{q_{c1} + q_{c2}}{2}$$ Eq. 15

where: q_c = The tip resistance in CPT data (if q_c>100 tsf, limit q_c to 100 tsf),

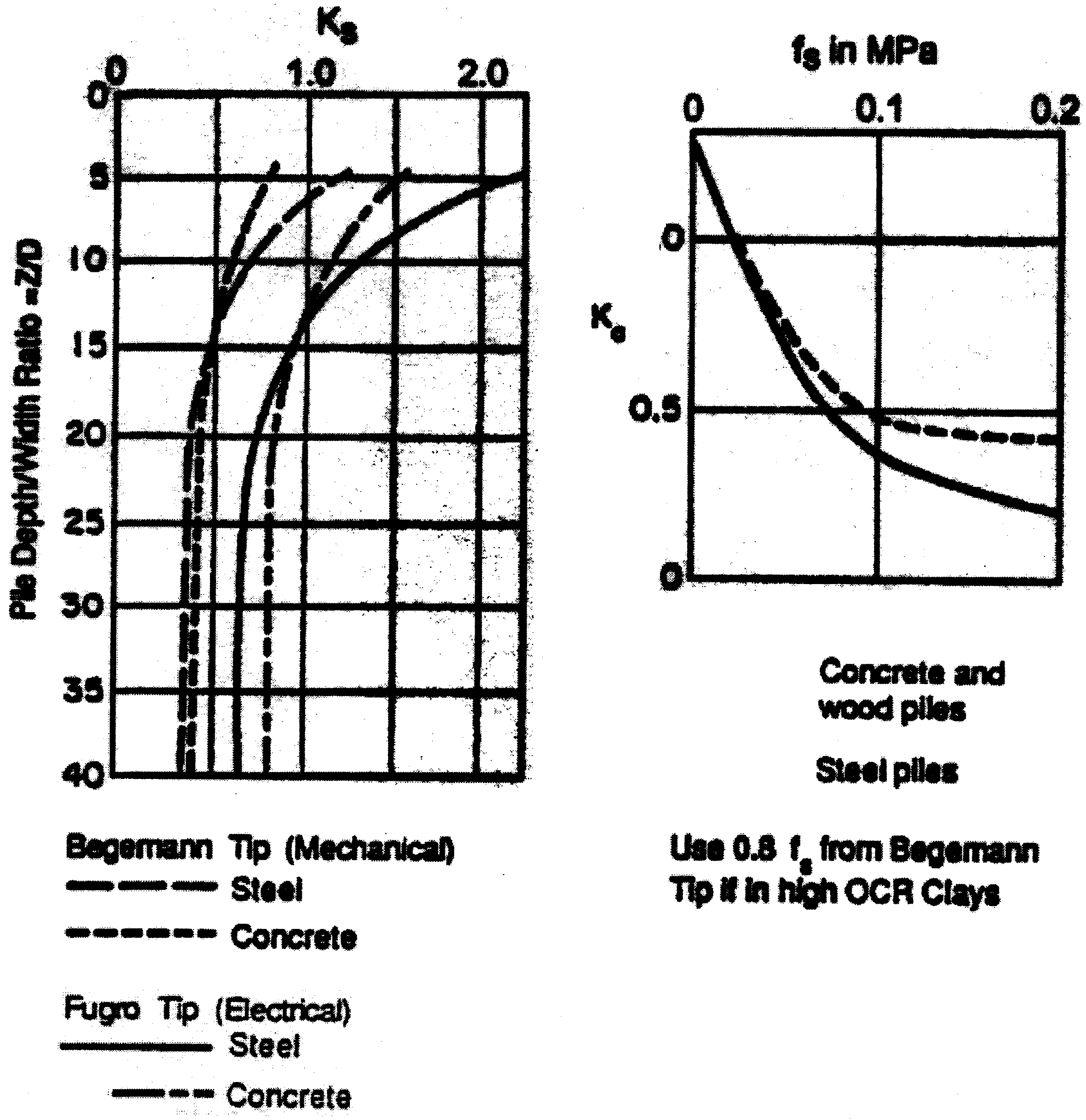
q_{c1} = The average q_c over a distance of xB below the tip, which is shown below:

Sum of the minimum values (upward)/ xB + Sum of the actual values (downward)/ xB

Use the minimum q_{c1} with different xB ranging from 0.7B to 3.75B,

q_{c2} = The average q_c over a distance of 8B above the tip using the minimum q_c values (minimum path). Figure 18 depicts the procedure for tip resistance prediction.

For mechanical cone in cohesive soils, due to the effects of the base of the friction sleeve on q_c, the q_p value should be reduced by about 40%.



Side friction

Figure 17: Ks and Kc ratio in cohesionless and cohesive soil, respectively (cited in McVay and Townsend, 1989)

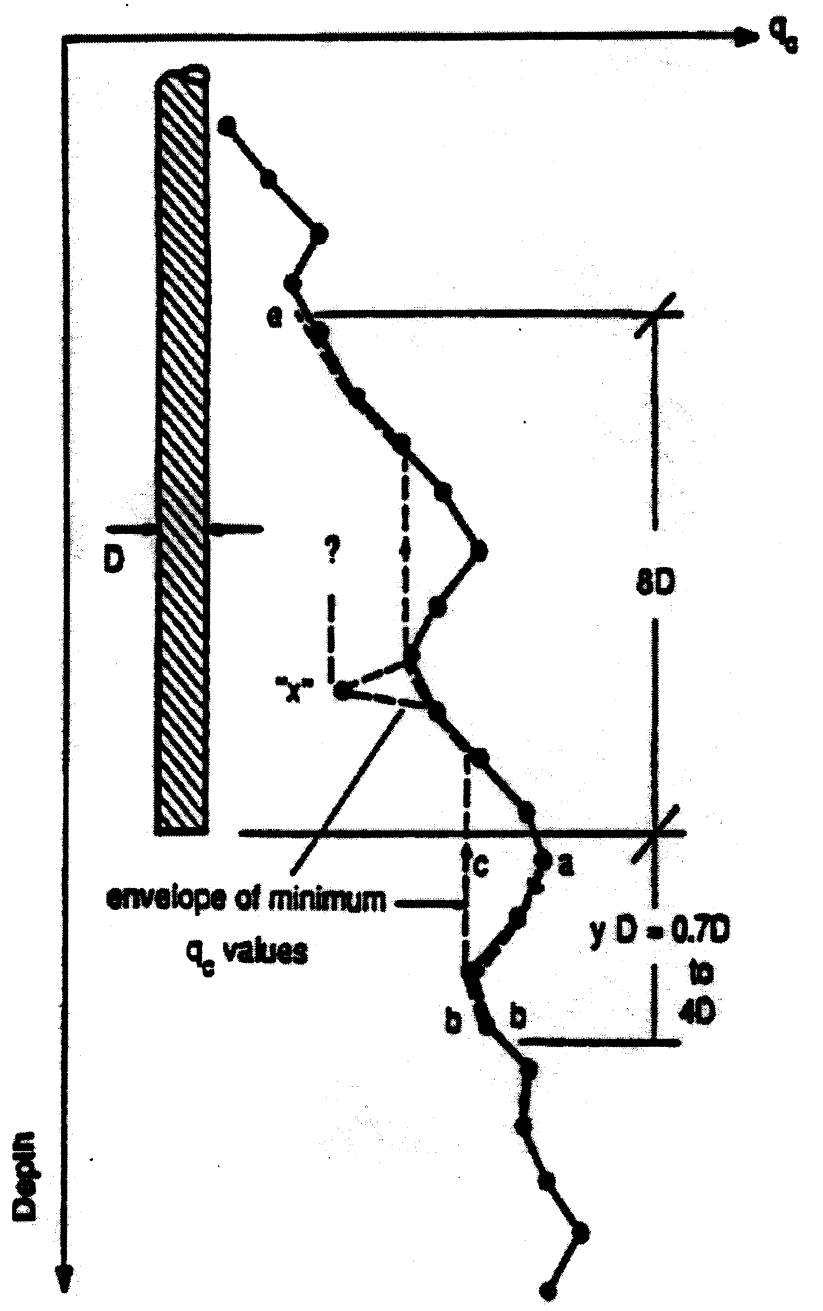


Figure 18: Tip resistance computation procedure--Nottingham 1975.
(cited in McVay and Townsend, 1989)

A summary of all the methods is presented in Table 4 below.

Table 4: Summary of the methods

Methods	Side resistance	Tip resistance	Parameters required	4. Constraints
α-Tomlinson (Tomlinson, 1980/1995)	$q_s = \alpha S_u$	$q_p = 9\ S_u$	S_u; D_b (bearing embedment)	+Bearing layer must be stiff cohesive + Number of soil layers ≤ 2
α-API (Reese et al., 1998)			S_u	
β in cohesive (AASHTO, 1996/2000)	$q_s = \beta\sigma'$		OCR	
λ (US Army Corps of Engineers, 1992)	$q_s = \lambda(\sigma' + 2S_u)$		S_u	Only for cohesive soils
β in cohesionless (Bowles, 1996)	$\beta\sigma'$		D_r	
Nordlund and Thurman (Hannigan et al., 1995)	$q_s = K_\delta C_F \sigma' \dfrac{\sin(\delta + \varpi)}{\cos\varpi}$	$q_p = \alpha_t N'_q \sigma'$	φ	
Meyerhof SPT (Meyerhof, 1976/1981)	$q_s = k\ N$	$q_p = 0.4D/BN'$	N	+ For cohesionless soils + SPT data
Schmertmann SPT (Lai and Graham, 1995)	q_s = function(N)	q_p = fn(N)	N	SPTdata
Schmertmann CPT (McVay and Townsend, 1989)	q_s = function(f_s)	q_p = fn(q_c)	q_c, f_s	CPT data

1.1.4. Evaluation of Axial Capacity Prediction Software

Four common programs are used in predicting the pile axial capacity. This section briefly evaluates the programs, such as the methods they use, and the results they get.

1.1.4.1. APILE 3.0 (EnSoft)

APILE 3.0 is a commercial Windows-based software developed by EnSoft, Inc (Reese et al., 1998).

For cohesive soil, the following static methods are used: The α-Tomlinson method, the α-API method and the λ method. For cohesionless soil, the following static methods are used: The Nordlund method and the API method. The combination of the α-Tomlinson and Nordlund methods in mixed soils is called the FHWA procedure.

The following problems were detected in using this program:

- For piles with non-circular sections, when the unit is changed from SI to English, the predicted axial capacity as well as the shapes of the capacity graph change,
- Outputs of EnSoft's examples 2 and 4 are different from the User Manual's printout,
- In FHWA procedure, the program does not correctly interpret the data when there are more than two layers: The bearing embedment depth will be calculated by the program as the distance from the tip to the nearest interface, which may be less than the actual bearing embedment,
- There is no box for inputting ground-water level, the user has to use buoyant unit weight whenever applicable.

1.1.4.2. DRIVEN 1.1 (FHWA)

DRIVEN is a freeware Windows-based software provided by the Federal Highway Administration (FHWA, 1998). Driven is the Windows version of the old SPILE MSDOS-based program. The α-Tomlinson method is used in cohesive soils, while the Nordlund and Thurman methods are used in cohesionless soils.

For concrete pile, DRIVEN does not support the circular section.

The α-Tomlinson method is only suitable for pile embedded in stiff clay. However, DRIVEN allows the user to specify the α-Tomlinson method in cohesive layer(s) that lies above the bearing layer.

1.1.4.3. PL-AID (University of Florida)

PL-AID is a MSDOS-based program developed at the University of Florida predicting capacities of piles using CPT data, i.e. Nottingham and Schmertmann method (McVay and Townsend, 1989).

PL-AID works correctly under the following conditions:

- CPT data (raw truck format or interpreted format) are in SI units,
- The increments in data input are equal (e.g. 5 or 10 cm).

PL-AID was not originally developed for H piles and unplugged open-ended pipe piles.

1.1.4.4. SPT-97 (FDOT and University of Florida)

SPT-97 is a Windows-based program predicting capacities using SPT data, i.e. Schmertmann method (Lai and Graham, 1995). SPT97 works normally well, except two following cases:

- When $D_c < D_a$ (the critical depth is smaller than the actual depth), there is a problem in correcting the resistance.
- The capacity is erroneously computed for pipe piles.

A summary of the issues related to the above software can be found in Table 5.

Table 5. Issues with using Driven Pile Axial Capacities Analysis Software

APILE 3.0 Plus	DRIVEN 1.1	SPT 97	PL-AID
All results using SI unit are erroneous (this problem was corrected on September 18, 00)	The computed results for H piles, SI units are erroneous.	When $D_c < D_a$ (critical depth is smaller than actual depth), there is a problem in correcting the resistance.	Increments must be equal. This is always true if the data are originally collected from the tests. However, some of the data are actually digitized from graphs. In this case, interpolation must be made to get equal increments.
Error in critical depth (this problem was corrected on November 27, 00)	Concrete piles: There is no option to input circular pile.		
Wrong handling of volume displacement for H and pipe piles (this problem was corrected on January 04, 01)			
The λ method is only suitable for pile in clay. In sand, APILE still has λ capacities for piles by converting ϕ to S_u (but in the APILE manual, it said that it took capacities from API)	The Nordlund method: Open ended pipe piles: Curve f (for H piles) is used for open ended pipe piles, which leads to higher capacities of plugged open ended pipe than those of close ended pipe with the same dimensions		
The α-Tomlinson (1980) method: The layer system must be converted to 1 or 2 layer(s). This causes an approximation and lowers the reliability, especially when the soil system is complex.	The α-Tomlinson 1980 method: In order to get correct results, the layer system must be converted to 1 or 2 layer(s). This causes an approximation and lowers the reliability, especially when the soil system is complex.	For pipe piles, the capacity from SPT97 is erroneously computed	
Error in user specified q_L (this problem was corrected on January 24, 01)			
Different capacities with different increments. (This problem was unsuccessfully corrected on February 14, 01)			

1.2. Interpretation of the Pile-Load Tests

Static load tests to failure determine the load capacities directly, therefore they are more precise than the prediction methods presented in Section 1.1. However, from load-settlement curves, a determination of the ultimate capacity or of the mobilized capacity is required. Many different methods of interpretations have been proposed and some of most common methods are presented in this section.

1.2.1. Debeer Method

The DeBeer capacity (Bowles, 1996) is determined as follow. The load test data is plotted in a log-log scale. The intersection between the two straight portions of the graph will correspond to the DeBeer capacity.

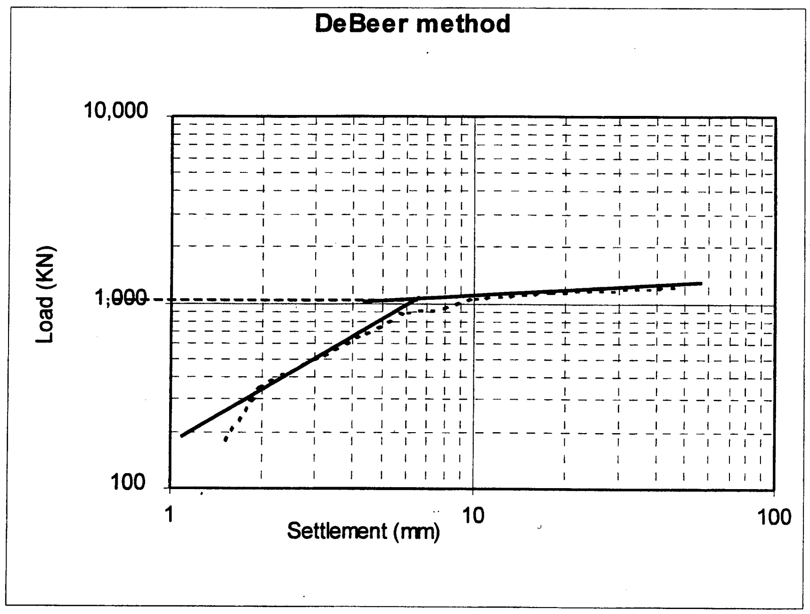


Figure 19: Example of the Debeer method

One of the most common problems with this method is that, in some cases, the two straight portions in the graph are not clearly defined.

1.2.2. Davisson Method

The Davisson method (Coduto, 2001) is one of the most popular methods and it is based on the elastic compression of the pile. The procedure to define the the Davisson capacity is as described below:

1. In the load test graph, plot the base line with the slope of: AE/L

   where: A = Cross sectional of the material,
   E = Modulus of elasticity of the material, and
   L = Embedment length of the pile

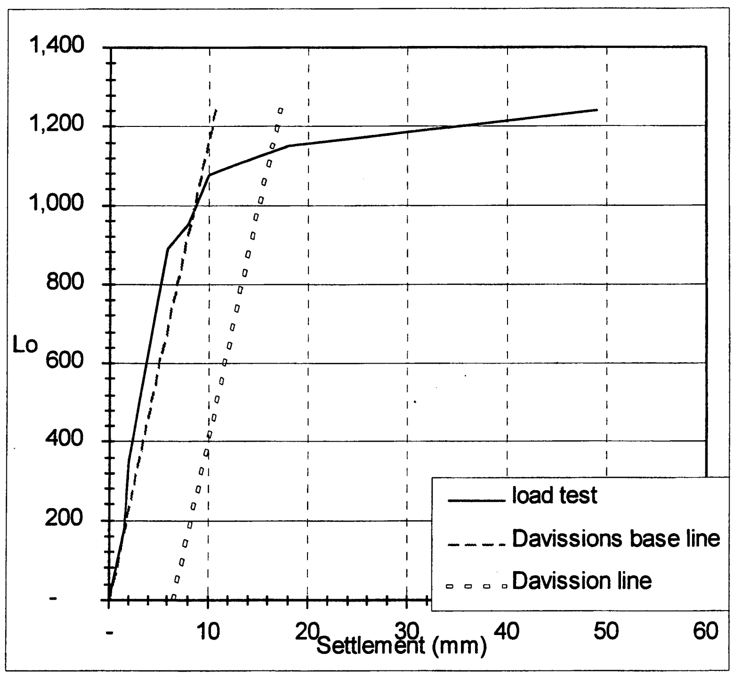


Figure 20: Example of the Davisson method

2. Plot the Davisson line parallel to the base line. The distance between the two lines is $0.15+\frac{B}{120}$ (in)
3. The intersection of that line and the load test graph represents the Davisson capacity.

Davisson method is most suitable for quick load tests on friction piles. For end bearing piles, as the load test curve does not flatten, the Davisson method is usually not applicable.

1.2.3. Other Methods

The pile capacity can be at a point where the settlement of the pile is 5% of its diameter. (or 2%, 3%, or even 10% depending on the pile shape and load test procedure). These methods are very simple. However, they do not account for the length and the material of the pile.

1.3. Soil Properties Correlated from Insitu Tests

About 95% of the soil data from the database are from SPT and CPT tests. Moreover, the α, β, λ and the Nordlund method all use the angle friction ϕ and/or the undrained shear strength S_u or the over-consolidation ratio OCR. Therefore, correlations are required to obtain these properties from the SPT and CPT tests.

There are usually two or more correlations for the same property. Therefore, in the following, the capacities are predicted based on the various different correlations available.

The correlations used in this study to get engineering soil properties from SPT are presented in Table 6. Similarly, Table 7 shows the correlations to get engineering soil properties from CPT.

Table 6: Correlations of soil properties from SPT

Properties	**From SPT**	**Figure**	**Reference**	
ϕ	Peck, Hanson and Thornburn: $\approx 54 - 27.6034 \exp(-0.014N')$	2`	Figure 4.12	Kulhawy and Mayne, 1990
	Schmertmannφ' $\approx \tan^{-1} [N / (12.2 + 20.3\, \sigma')]^{0.34}$	22	Figure 4.13 and Equation 4.11	
S_u (bar)	Terzaghi and Peck (1967): $0.06\,N$		Equation 4.59	
	Hara 1974: $0.29\,N^{0.72}$		Equation 4.60	
OCR for clay	Mayne and Kemper $\approx 0.5\,N / \sigma'_o$ (σ'_o in bar)	23	Figures 3.9 and 3.18	
Dr	Gibbs and Holtz's Figures	24	Figures 2.13 and 2.14	

where: N is the uncorrected blow counts, and

N' is the corrected blow counts

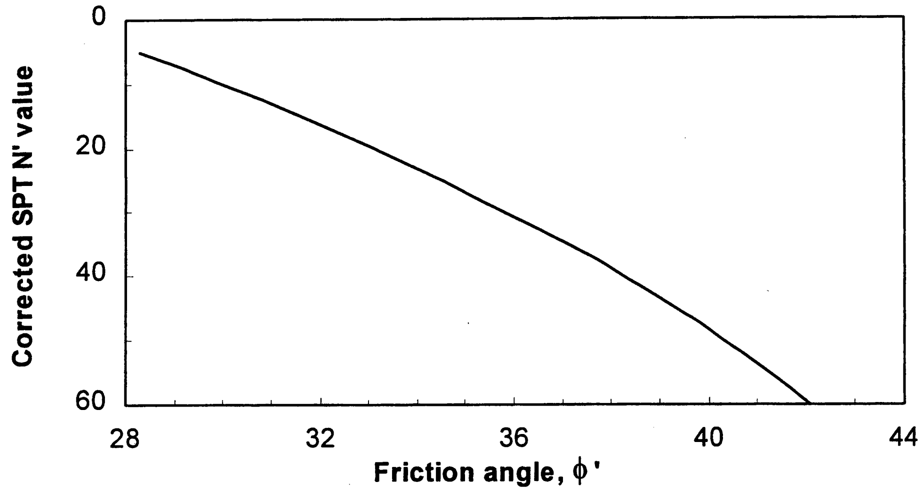


Figure 21: ϕ' by Peck, Hanson and Thornburn (Kulhawy and Mayne, 1990)

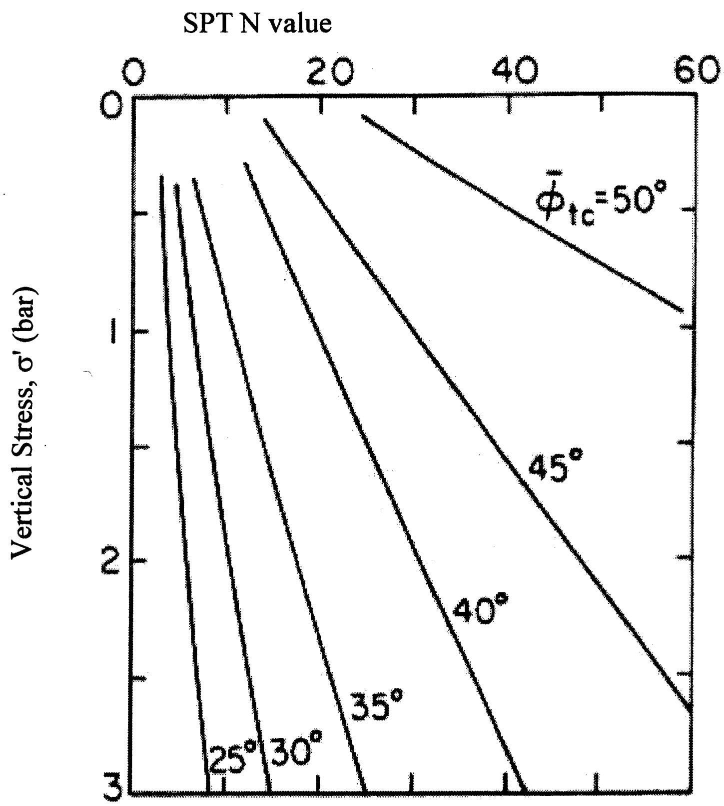


Figure 21: ϕ' by Schmertmann (Kulhawy and Mayne, 1990)

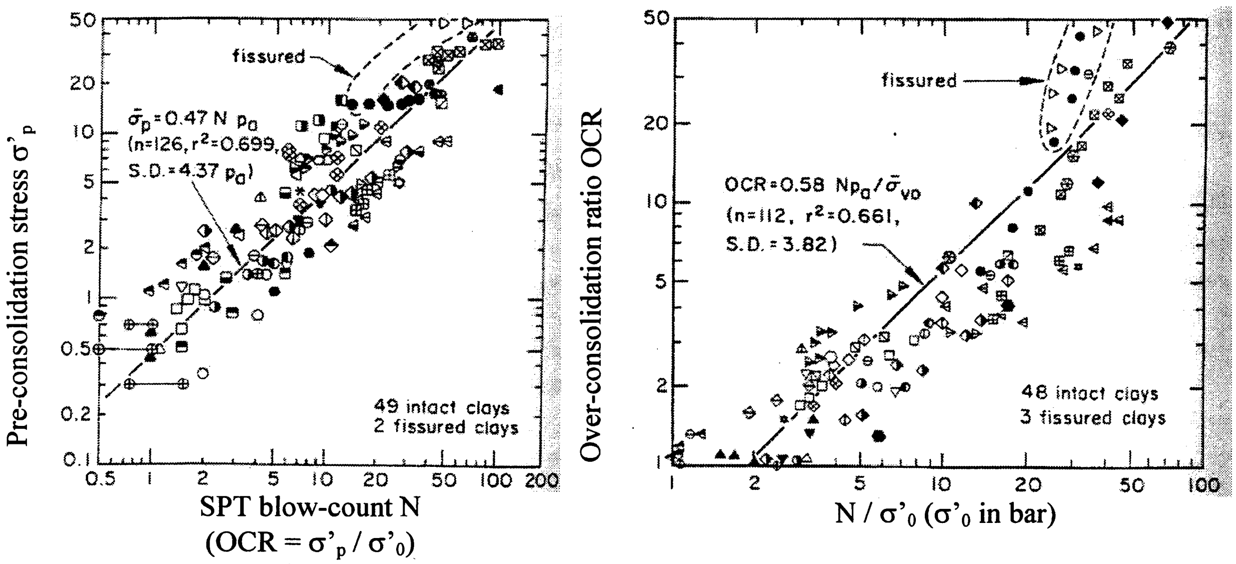


Figure 22: OCR--N Relationship (Kulhawy and Mayne, 1990)

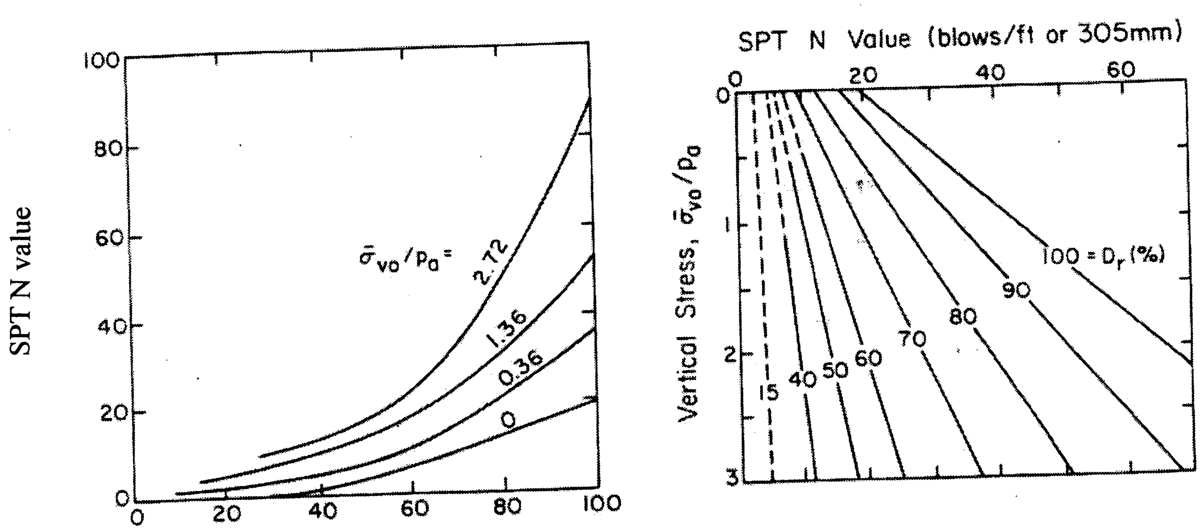


Figure 23: Relative Density--N--Stress Relationship (Kulhawy and Mayne, 1990)

Table 7. Correlations of soil properties from CPT

Properties	From CPT	Figure	Reference	
ϕ	Robertson and Campanella: $\mathrm{atan}(0.1+0.38*\mathrm{Log}(q_c/\sigma'))$	25	Figure 4.14 and Equation 4.12	Kulhawy and Mayne, 1990
S_u (bar)	Theoretical: $(q_c - \sigma_o) / Nk$ q_c and σ_o in bars.		Equation 4.61	
OCR for clay	Mayne: $0.29\, q_c / \sigma'_o$ q_c and σ_o in bars.	27	Figure 3.10	
Dr	Jamiolkowski: $68 \log(q_{cn}) - 68$ $q_{cn} = \frac{q'_c}{\sqrt{P_a \sigma'_0}}$ (dimensionless) $q'_c = q_c / K_q$ $K_q = 0.9 + Dr/300$ q_c and σ'_o in bars.	26	Figure 2.24 and Equation 2.20	

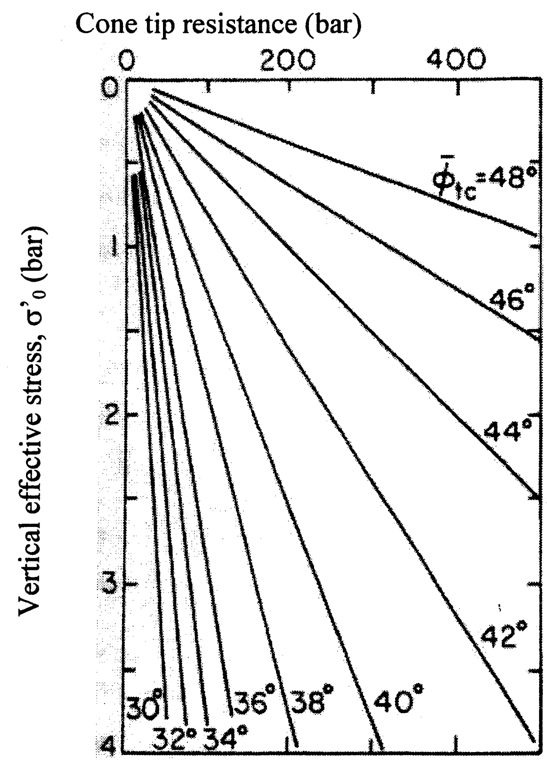


Figure 24: ϕ'_{tc} correlated from q_c for NC, uncemented quartz sands (Kulhawy and Mayne, 1990)

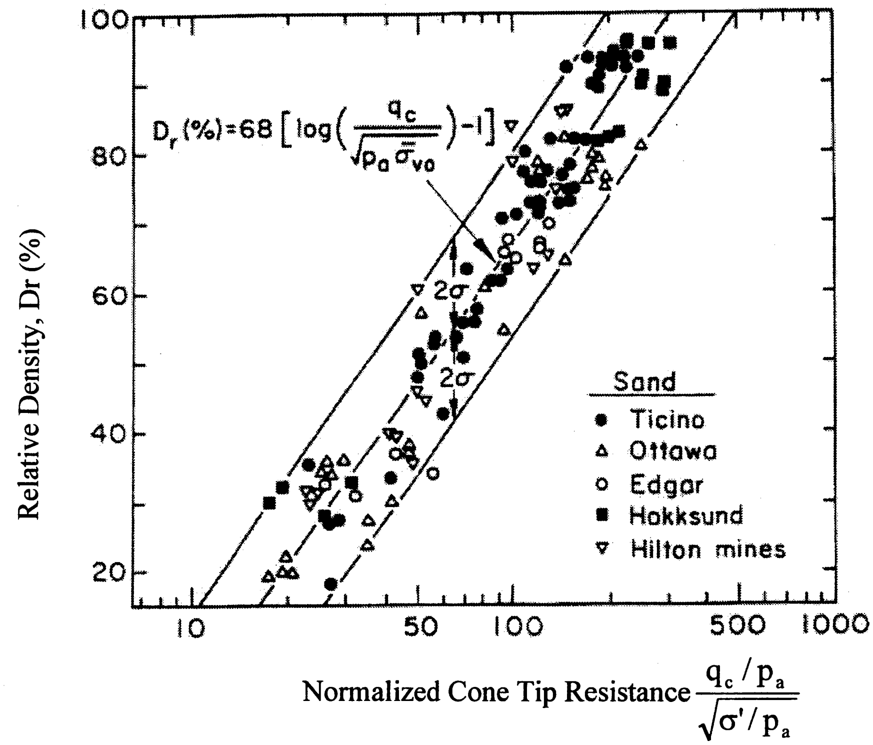


Figure 25: Correlation between Dr and q_c (uncorrected for boundary effect) (Kulhawy and Mayne, 1990)

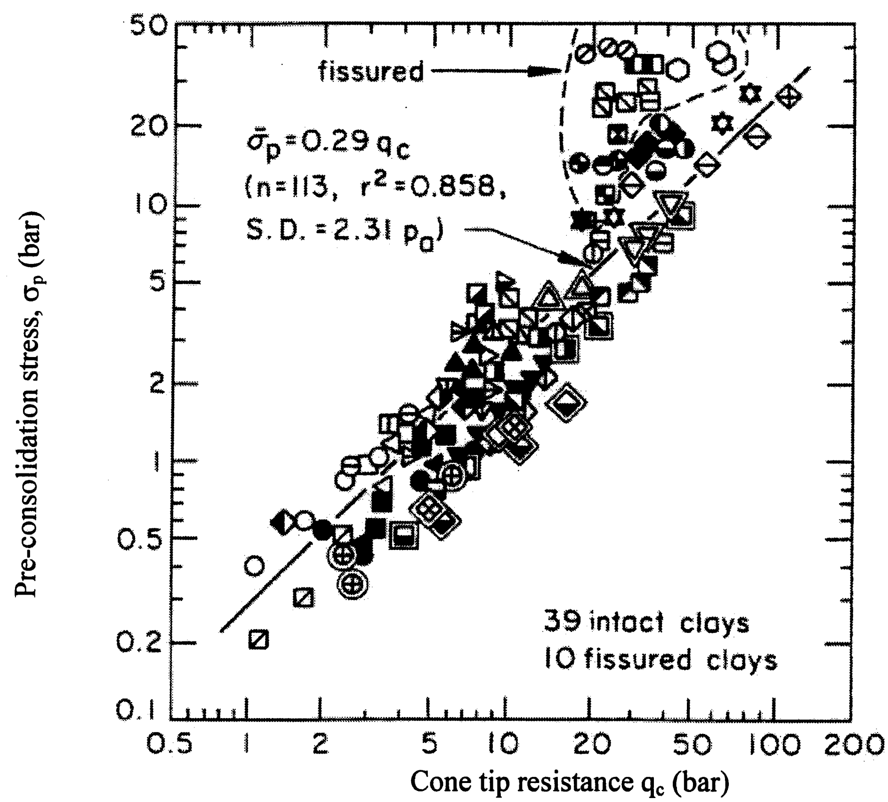


Figure 26: σ_p correlated with q_c (OCR = σ_p / σ'_0) (Kulhawy and Mayne, 1990)

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DRILLED SHAFTS – STATIC ANALYSIS

SUMMARY OF RESISTANCE FACTORS

Resistance Factor, ϕ of Drilled Shaft for $Q_D/Q_L = 2.0$

	Soil Type	Design Method	Construction Method	No. of Data	Bias Factor λ_R	COV_R	Resistance Factors					
							$\beta_T = 2.0$		$\beta_T = 2.5$		$\beta_T = 3.0$	
							ϕ	ϕ/λ	ϕ	ϕ/λ	ϕ	ϕ/λ
Skin Friction + End Bearing	Sand	FHWA	Mixed	34	1.93	0.69	0.49	0.25	0.34	0.18	0.24	0.12
			Casing	14	2.47	0.5	0.91	0.37	0.71	0.29	0.55	0.22
			Slurry	14	2.04	0.86	0.37	0.18	0.24	0.12	0.16	0.08
		R&W	Mixed	34	1.42	0.81	0.28	0.20	0.19	0.13	0.13	0.09
			Casing	14	1.93	0.72	0.46	0.24	0.32	0.17	0.22	0.11
			Slurry	14	1.32	1.18	0.12	0.09	0.07	0.05	0.04	0.03
	Clay	FHWA	Mixed	54	0.95	0.54	0.32	0.34	0.25	0.26	0.19	0.20
			Casing	14	0.99	0.71	0.24	0.24	0.17	0.17	0.12	0.12
			Dry	40	0.88	0.48	0.34	0.39	0.27	0.31	0.21	0.24
	Sand + Clay	FHWA	Mixed	48	1.33	0.45	0.54	0.41	0.43	0.32	0.34	0.26
			Casing	23	1.21	0.53	0.42	0.35	0.32	0.26	0.25	0.21
			Dry	13	1.47	0.44	0.61	0.41	0.49	0.33	0.39	0.27
			Slurry	12	1.43	0.39	0.66	0.46	0.54	0.38	0.44	0.31
		R&W	Mixed	48	1.19	0.45	0.48	0.40	0.39	0.33	0.31	0.26
			Casing	23	1.07	0.48	0.41	0.38	0.32	0.30	0.25	0.23
			Dry	13	1.36	0.47	0.53	0.39	0.42	0.31	0.33	0.24
			Slurry	12	1.28	0.37	0.61	0.48	0.51	0.40	0.42	0.33
	All Soils	FHWA	Mixed	136	1.32	0.69	0.33	0.25	0.24	0.18	0.17	0.13
		R&W	Mixed	136	1.28	0.66	0.34	0.27	0.25	0.20	0.18	0.14
	Rock	C&K	Mixed	49	1.37	0.55	0.46	0.34	0.35	0.26	0.26	0.19
			Dry	32	1.50	0.56	0.49	0.33	0.37	0.25	0.28	0.19
		IGM	Mixed	49	1.42	0.46	0.57	0.40	0.45	0.32	0.36	0.25
			Dry	32	1.53	0.45	0.62	0.41	0.50	0.33	0.40	0.26

	Soil Type	Design Method	Construction Method	No. of Data	Bias Factor λ_R	COV_R	Resistance Factors					
							$\beta_T = 2.0$		$\beta_T = 2.5$		$\beta_T = 3.0$	
							ϕ	ϕ/λ	ϕ	ϕ/λ	ϕ	ϕ/λ
Skin	Sand	FHWA	Mixed	11	1.09	0.51	0.39	0.36	0.30	0.28	0.24	0.22
		R&W	Mixed	11	0.83	0.54	0.28	0.34	0.22	0.27	0.16	0.19
	Clay	FHWA	Mixed	16	0.87	0.37	0.42	0.48	0.34	0.39	0.29	0.33
	Sand & Clay	FHWA	Mixed	13	1.49	0.49	0.56	0.38	0.44	0.30	0.34	0.23
		R&W	Mixed	16	1.35	0.49	0.51	0.38	0.40	0.30	0.31	0.23
	All Soils	FHWA	Mixed	40	1.16	0.53	0.40	0.34	0.31	0.27	0.24	0.21
		R&W	Mixed	27	1.14	0.55	0.38	0.33	0.29	0.25	0.22	0.19
	Rock	C&K	Mixed	17	1.33	0.62	0.38	0.29	0.28	0.21	0.21	0.16
		IGM	Mixed	17	1.39	0.53	0.48	0.35	0.37	0.27	0.28	0.20

Methodology and Correlations:

1.) FHWA Method, i.e., Reese, L. C. and M. W. O'Neill (1988) "Drilled Shaft: Construction Procedures and Design Methods", FHWA-HI-88-042 for Sands and Clays.
For Sands, β method was used.
For Clays, α method was used. For Su, the SPT correlation given by Terzaghi and Peck (1967) was used.

2.) R &W Method, i.e., Reese, L. C. and S. J. Wright (1977) "Construction Procedures and Design for Axial Loading.", Drilled Shaft Manual HDV-22, for Sand.
In Sand-Clay Mix Deposit, α method was used for Clays

3.) C & K Method, i.e., Carter, J. P. and F. H. Kulhawy (1988) "Analysis and Design of Foundations Socketed into Rock," Report Number EL-5918 for Rock.

4.) IGM Method (Intermediate Geomaterials), i.e., O'Neill, et al (1996) "Load Transfer for Drilled Shafts in Intermediate Geomaterials" FHWA-RD-95-172, and O'Neill, M. W. and L. C. Reese (1999) 'Drilled Shaft: Construction Procedures and Design Methods", FHWA-IF-99-025.
The design assumed smooth rock socket for skin friction and closed joints for end bearing.

DRILLED SHAFTS – STATIC ANALYSIS

RESULTS AND STATISTICS

- Summary Table of the Cases analyzed, their statistical parameters and related figure number
- Detailed Table of Shafts (Total Resistance) in Soils and related figures (1-19)
- Detailed Table of Shafts (Skin Friction) in Soils and related figures (20-26)
- Detailed Table of Shafts (Total Resistance) in Rock and related figures (27-30)
- Detailed Table of Shafts (Skin Friction) in Rock and related figures (31-32)

FIGURE	SOIL TYPE	NO. OF DATA	CONSTRUCTION METHOD	DESIGN METHOD	LOAD TRANSFER	MEAN	STDV	VAR
1	Sand	34	Mixed	FHWA	Total	1.93	1.34	0.69
2	Clay	54	Mixed	FHWA	Total	0.95	0.51	0.54
3	Sand & Clay	48	Mixed	FHWA	Total	1.33	0.60	0.45
4	Total Soil	136	Mixed	FHWA	Total	1.32	0.91	0.69
5	Sand	34	Mixed	R & W	Total	1.42	1.14	0.81
6	Sand & Clay	48	Mixed	R & W	Total	1.19	0.53	0.45
7	Total Soil	136	Mixed	R & W	Total	1.28	0.84	0.66
8	Sand & Clay	23	Casing	FHWA	Total	1.21	0.64	0.53
9	Sand & Clay	23	Casing	R & W	Total	1.07	0.51	0.48
10	Sand & Clay	13	Dry	FHWA	Total	1.47	0.64	0.44
11	Sand & Clay	13	Dry	R & W	Total	1.36	0.64	0.47
12	Sand & Clay	12	Slurry	FHWA	Total	1.43	0.55	0.39
13	Sand & Clay	12	Slurry	R & W	Total	1.28	0.47	0.37
14	Sand	14	Casing	FHWA	Total	2.47	1.24	0.50
15	Sand	14	Casing	R & W	Total	1.93	1.39	0.72

16	Sand	14	Slurry	FHWA	Total	2.04	1.75	0.86
17	Sand	14	Slurry	R & W	Total	1.32	1.55	1.18
18	Clay	14	Casing	FHWA	Total	0.99	0.70	0.71
19	Clay	40	Dry	FHWA	Total	0.88	0.42	0.48
20	Sand	11	Mixed	FHWA	Skin	1.09	0.56	0.51
21	Clay	13	Mixed	FHWA	Skin	0.87	0.32	0.37
22	Sand & Clay	16	Mixed	FHWA	Skin	1.49	0.72	0.49
23	Total Soil	40	Mixed	FHWA	Skin	1.16	0.62	0.53
24	Sand	11	Mixed	R & W	Skin	0.83	0.45	0.54
25	Sand & Clay	16	Mixed	R & W	Skin	1.35	0.66	0.49
26	Total Soil	27	Mixed	R & W	Skin	1.14	0.63	0.55
27	Rock	49	Mixed	C & K	Total	1.37	0.75	0.55
28	Rock	49	Mixed	IGM	Total	1.42	0.64	0.46
29	Rock	32	Dry	C & K	Total	1.50	0.83	0.56
30	Rock	32	Dry	IGM	Total	1.53	0.69	0.45
31	Rock	17	Mixed	C & K	Skin	1.33	0.82	0.62
32	Rock	17	Mixed	IGM	Skin	1.39	0.74	0.53

NOTE:

1. SOIL TYPE – SAND - SHAFT PRIMARILY IN SAND DEPOSIT
   CLAY - SHAFT PRIMARILY IN CLAY DEPOSIT
   SAND & CLAY – SHAFT IN SAND & CLAY MIXED DEPOSIT
   TOTAL – ALL SHAFTS IN SOIL DEPOSIT
   ROCK - SHAFT PRIMARILY IN ROCK DEPOSIT

2. CONSTRUCTION METHOD - MIXED- INCLUDE ALL CONSTRUCTION METHODS
   CASING – SHAFT USED CASING
   DRY – SHAFT USED DRY METHOD
   SLURRY – SHAFT USED SLURRY

3. LOAD TRANSFER - TOTAL – ALL SHAFTS
   SKIN – SHAFT CAPACITY PRIMINARILY IS SIDE FRICTION

4. DETAIL DATA SEE EXCEL FILES:

   SHAFT-SOIL.XLS – ALL SHAFTS IN SOIL DEPOSIT, **(FIGURES 1 TO 19)**
   SOIL.SKIN.XLS – SHAFT CAPACITY PRIMARILY IS SIDE FRICTION OF SAND DEPOSIT, **(FIGURES 20 TO 26.)**
   SHAFT-ROCK.XLS – ALL SHAFT IN ROCK DEPOSIT,(**FIGURES 27 TO 30)**
   ROCK-SKIN.XLS – SHAFT CAPACITY PRIMARY IS SIDE FRICTION OF ROCK DEPOSIT**. (FIGURES 31 TO 32)**

Table		Failure		FHWA	FHWA	W&R	W&R
SHAFT #	Construction	Load (kips)	Soil Type	Predict(kips)	Ratio	Predict(kips)	Ratio
1	Casing	400	A	594	0.67	676.00	0.59
4	Casing	2000	A	1964	1.02	2058.00	0.97
5	Casing	180	A	126.82	1.42	175.56	1.03
6	Casing	322	A	370	0.87	426.00	0.76
7	Casing	324	A	412	0.79	662.00	0.49
10	Casing	240	A	304.66	0.79	336.48	0.71
12	Casing	10.8	A	8.62	1.25	7.60	1.42
13	Casing	18	A	12	1.50	12.06	1.49
14	Casing	140.	A	154.3	0.91	163.90	0.85
15	Casing	140	A	126.46	1.11	145.62	0.96
16	Casing	750	A	610	1.23	637.66	1.18
17	Casing	560	A	375.3	1.49	402.10	1.39
18	Casing	1100	A	1250	0.88	1282.00	0.86
19	Casing	1160	A	725.66	1.60	727.08	1.60
20	Casing	1080	A	923.24	1.17	949.72	1.14
21	Casing	400	A	477.34	0.84	507.78	0.79
22	Casing	1700	A	548	3.10	679.24	2.50
23	Casing	1100	A	379.48	2.90	502.28	2.19
24	Casing	670	A	625.08	1.07	677.94	0.99
25	Casing	360	A	616.12	0.58	827.00	0.44
31	Casing	980	A	837.04	1.17	963.20	1.02
33	Casing	600	A	644.3	0.93	749.10	0.80
36	Casing	760	A	1444	0.53	1582.00	0.48
37	Dry	2000	A	1136	1.76	1606.00	1.25
38	Dry	1474	A	1293.28	1.14	1635.78	0.90
41	Dry	1418	A	1408	1.01	1819.30	0.78
53	Dry	1900	A	2056	0.92	1886.00	1.01
54	Dry	2200	A	1620	1.36	1302.00	1.69
56	Dry	860	A	872.46	0.99	1038.04	0.83
57	Dry	750	A	990	0.76	1060.00	0.71
85	Dry	184	A	118.58	1.55	141.40	1.30
95	Dry	2400	A	758	3.17	798.00	3.01
98	Dry	2000	A	1138	1.76	1218.00	1.64
153	Dry	1200	A	662	1.81	691.30	1.74
154	Dry	1080	A	732	1.48	762.94	1.42
155	Dry & Casing	1100	A	658.26	1.67	686.92	1.60

TOTAL SHAFT in SOIL

FHWA Method				
	Soil Type			
	A	B	C	ALL
Mean	1.33	1.93	0.95	1.32
Stdv	0.60	1.34	0.51	0.91
Var	0.45	0.69	0.54	0.69

R & W Method				
	Soil Type			
	A	B		ALL
Mean	1.19	1.42		1.28
Stdv	0.53	1.14		0.84
Var	0.45	0.81		0.66

NOTE:
SOIL TYPE A = SAND & CLAY
SOIL TYPE B = SAND
SOIL TYPE C = CLAY

TYPE **A** SOIL in **CASING**		
	FHWA	R&W
MEAN	1.21	1.07
STDV	0.64	0.51
VAR	0.53	0.48

TYPE **A** SOIL in **DRY Method**		
	FHWA	R&W
MEAN	1.47	1.36
STDV	0.64	0.64
VAR	0.44	0.47

229	Slurry	2146	A	1742	1.23	1902.00	1.13
230	Slurry	2194	A	1390	1.58	1550.00	1.42
231	Slurry	2644	A	1768	1.50	1930.00	1.37
232	Slurry	1708	A	958	1.78	1118.00	1.53
233	Slurry	2644	A	984	2.69	1118.00	2.36
234	Slurry	1596	A	958	1.67	1118.00	1.43
235	Slurry	1596	A	1418	1.13	1578.00	1.01
236	Slurry	1596	A	1768	0.90	1930.00	0.83
237	Slurry	1590	A	1742	0.91	1902.00	0.84
156	Slurry	820	A	920.74	0.89	955.24	0.86
227	Slurry & Casing	452	A	378	1.20	632.00	0.72
228	Slurry & Casing	900	A	574	1.57	914.00	0.98
9	Casing	108	B	95.78	1.13	131.20	0.82
11	Casing	380	B	232.9	1.63	483.30	0.79
27	Casing	1080	B	300.72	3.59	277.82	3.89
28	Casing	1500	B	304.14	4.93	281.52	5.33
29	Casing	390	B	185.5	2.10	347.34	1.12
30	Casing	900	B	373.12	2.41	608.94	1.48
32	Casing	356	B	397.2	0.90	735.96	0.48
34	Casing	940	B	904	1.04	924.00	1.02
35	Casing	2000	B	497.24	4.02	834.62	2.40
43	Casing	2400	B	648.96	3.70	823.90	2.91
45	Casing	1980	B	844	2.35	1306.74	1.52
52	Casing	1340	B	609.94	2.20	781.40	1.71
58	Casing	1620	B	746.6	2.17	977.94	1.66
59	Casing & Slurry	1880	B	1311.16	1.43	2071.96	0.91
60	Dry	1900	B	1114	1.71	1204.00	1.58
64	Dry	1300	B	736	1.77	1130.00	1.15
65	Dry	990	B	994	1.00	1188.00	0.83
66	Dry	930	B	688	1.35	956.00	0.97
67	Dry	1380	B	814	1.70	1466.00	0.94
82	Dry & Casing	1880	B	3762	0.50	5900.00	0.32
83	Slurry	2464	B	992	2.48	1414.00	1.74
89	Slurry	131.2	B	72.78	1.80	138.98	0.94
112	Slurry	1000	B	246	4.07	398.00	2.51

TYPE **A** SOIL in **SLURRY**		
	FHWA	R&W
MEAN	1.43	1.28
STDV	0.55	0.47
VAR	0.39	0.37

TYPE **B** SOIL in **CASING**		
	FHWA	R&W
MEAN	2.47	1.93
STDV	1.24	1.39
VAR	0.50	0.72

TYPE **B** SOIL in **SLURRY**		
	FHWA	R&W
MEAN	2.04	1.32
STDV	1.75	1.55
VAR	0.86	1.18

TYPE **C** SOIL in **CASING**		
	FHWA	R&W
MEAN	0.99	n/a
STDV	0.70	n/a
VAR	0.71	n/a

TYPE **C** SOIL in **DRY METHOD**		
	FHWA	R&W
MEAN	0.88	n/a
STDV	0.42	n/a
VAR	0.48	n/a

113	Slurry	1800	B	306	5.88	468.00	3.85
123	Slurry	70	B	52.3	1.34	103.48	0.68
124	Slurry	84	B	52.3	1.61	103.48	0.81
163	Slurry	420	B	214	1.96	266.00	1.58
164	Slurry	76	B	326	0.23	400.00	0.19
165	Slurry	118	B	234	0.50	400.00	0.30
175	Slurry	650	B	1202	0.54	1176.00	0.55
176	Slurry & Casing	800	B	860	0.93	836.00	0.96
177	Slurry & Casing	760	B	572	1.33	538.00	1.41
251	Slurry&Casing	116	B	632	0.18	930.00	0.12
252	Slurry&Casing	50	B	42	1.19	68.00	0.74
61	Casing	300	C	362	0.83		
62	Casing	710	C	362	1.96		
90	Casing	200	C	222	0.90		
91	Casing	200	C	208	0.96		
107	Casing	510	C	1552	0.33		
110	Casing	440	C	388	1.13		
131	Casing	4720	C	8418	0.56		
132	Casing	400	C	610	0.66		
133	Casing	400	C	133.28	3.00		
134	Casing	800	C	894.48	0.89		
148	Casing	200	C	290.9	0.69		
149	Casing	190	C	197.44	0.96		
151	Casing	1680	C	2552	0.66		
152	Casing	1512	C	4674	0.32		
162	Dry	1528	C	1618	0.94		
180	Dry	480	C	596	0.81		
181	Dry	720	C	552	1.30		
182	Dry	90	C	144	0.63		
183	Dry	220	C	820	0.27		
184	Dry	712	C	1138	0.63		
185	Dry	108	C	332	0.33		
186	Dry	200	C	360	0.56		
187	Dry	800	C	408	1.96		
188	Dry	1560	C	734	2.13		

189	Dry	1500	C	1390	1.08		
190	Dry	1700	C	1380	1.23		
191	Dry	1940	C	2432	0.80		
192	Dry	2000	C	2288	0.87		
193	Dry	1940	C	4024	0.48		
194	Dry	1940	C	3900	0.50		
195	Dry	760	C	698	1.09		
196	Dry	580	C	328	1.77		
197	Dry	1100	C	1360	0.81		
198	Dry	1180	C	942	1.25		
199	Dry	1100	C	1250	0.88		
200	Dry	400	C	408	0.98		
201	Dry	440	C	316	1.39		
202	Dry	360	C	422	0.85		
203	Dry	1800	C	1276	1.41		
204	Dry	1060	C	1118	0.95		
205	Dry	1060	C	1118	0.95		
206	Dry	800	C	1270	0.63		
207	Dry	800	C	1426	0.56		
208	Dry	980	C	1688	0.58		
209	Dry	1380	C	1368	1.01		
210	Dry	780	C	946	0.82		
211	Dry	950	C	1230	0.77		
212	Dry	800	C	1032	0.78		
213	Dry	988	C	1838	0.54		
238	Dry	7000	C	8304	0.84		
239	Dry	4400	C	11478	0.38		
240	Dry	4200	C	9738	0.43		
241	Dry	3700	C	11594	0.32		
242	Dry	4400	C	5178	0.85		

DRILLED SHAFT IN SAND

CONSTRUCTION METHOD= MIXED
LOAD TRANSFER= SKIN FRICTION + END BEARING
N= 34
MEAN=1.93
STANARD DEVIATION = 1.34
COEFFICIENT OF VARIATION =0.69

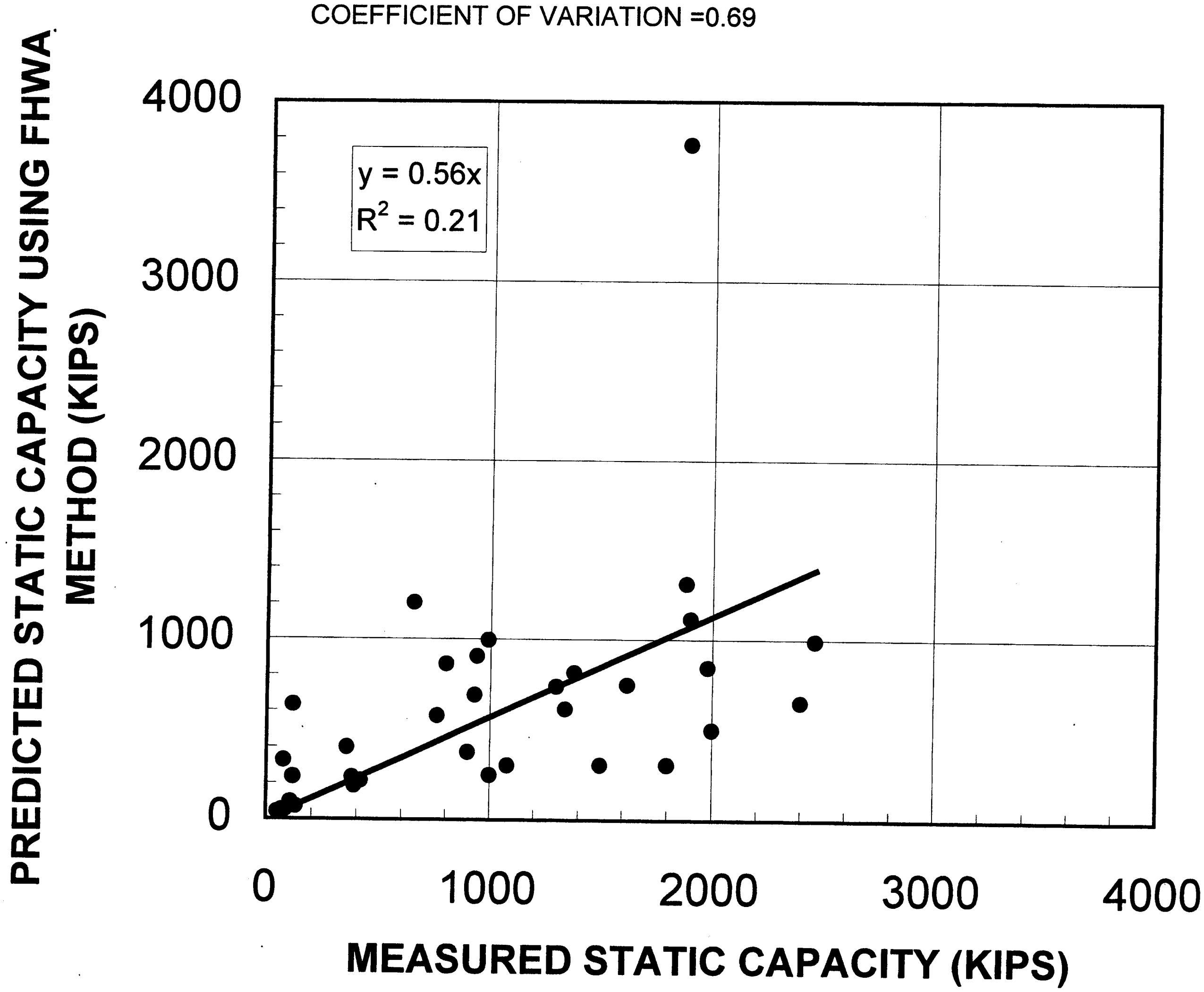

Figure 1

DRILLED SHAFT IN CLAY

CONSTRUCTION METHOD= MIXED
LOAD TRANSFER= SKIN FRICTION + END BEARING
N= 54
MEAN=0.95
STANARD DEVIATION = 0.51
COEFFICIENT OF VARIATION =0.54

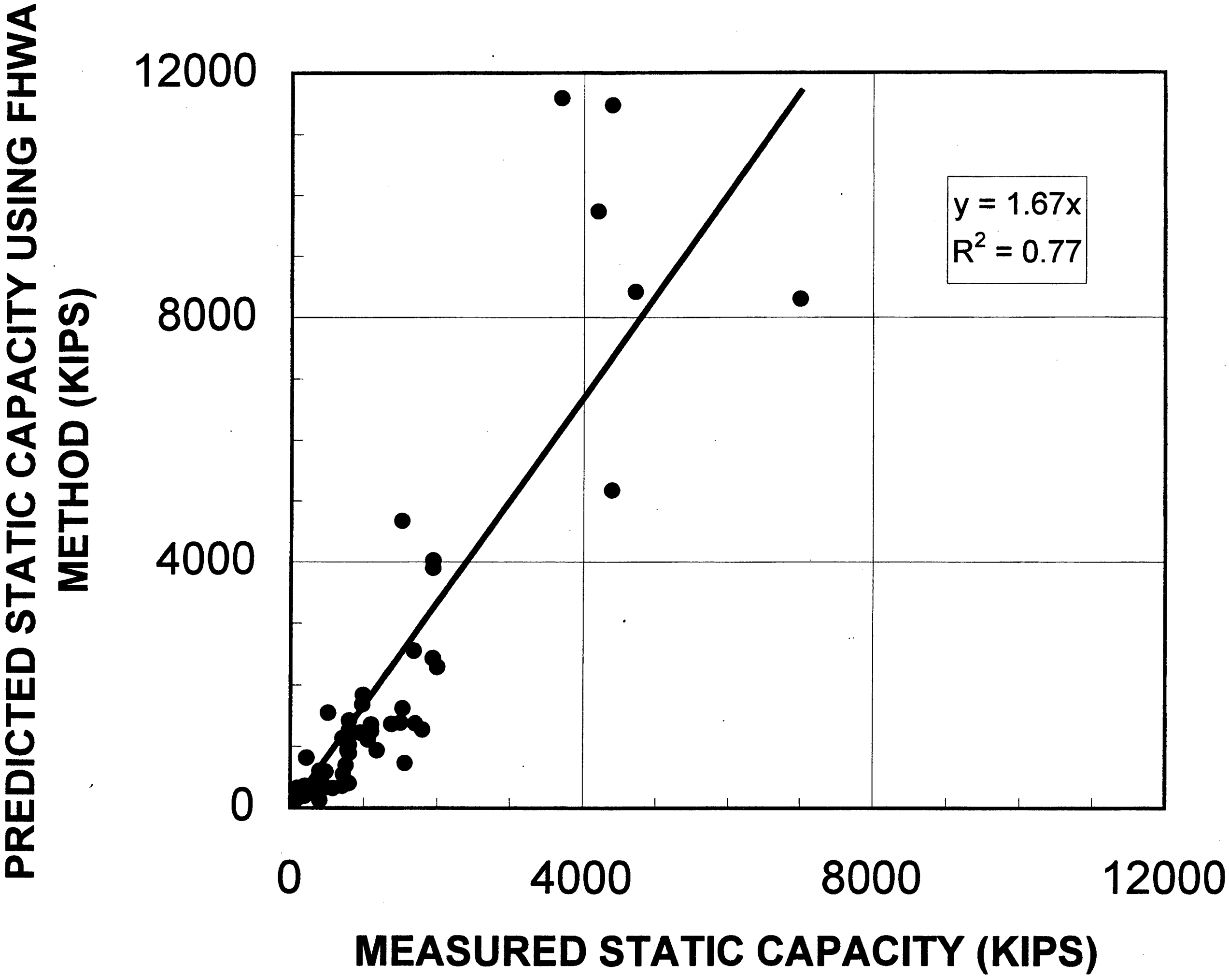

Figure 2

DRILLED SHAFT IN SAND & CLAY

CONSTRUCTION METHOD= MIXED
LOAD TRANSFER= SKIN FRICTION + END BEARING
N= 48
MEAN=1.33
STANARD DEVIATION = 0.60
COEFFICIENT OF VARIATION =0.45

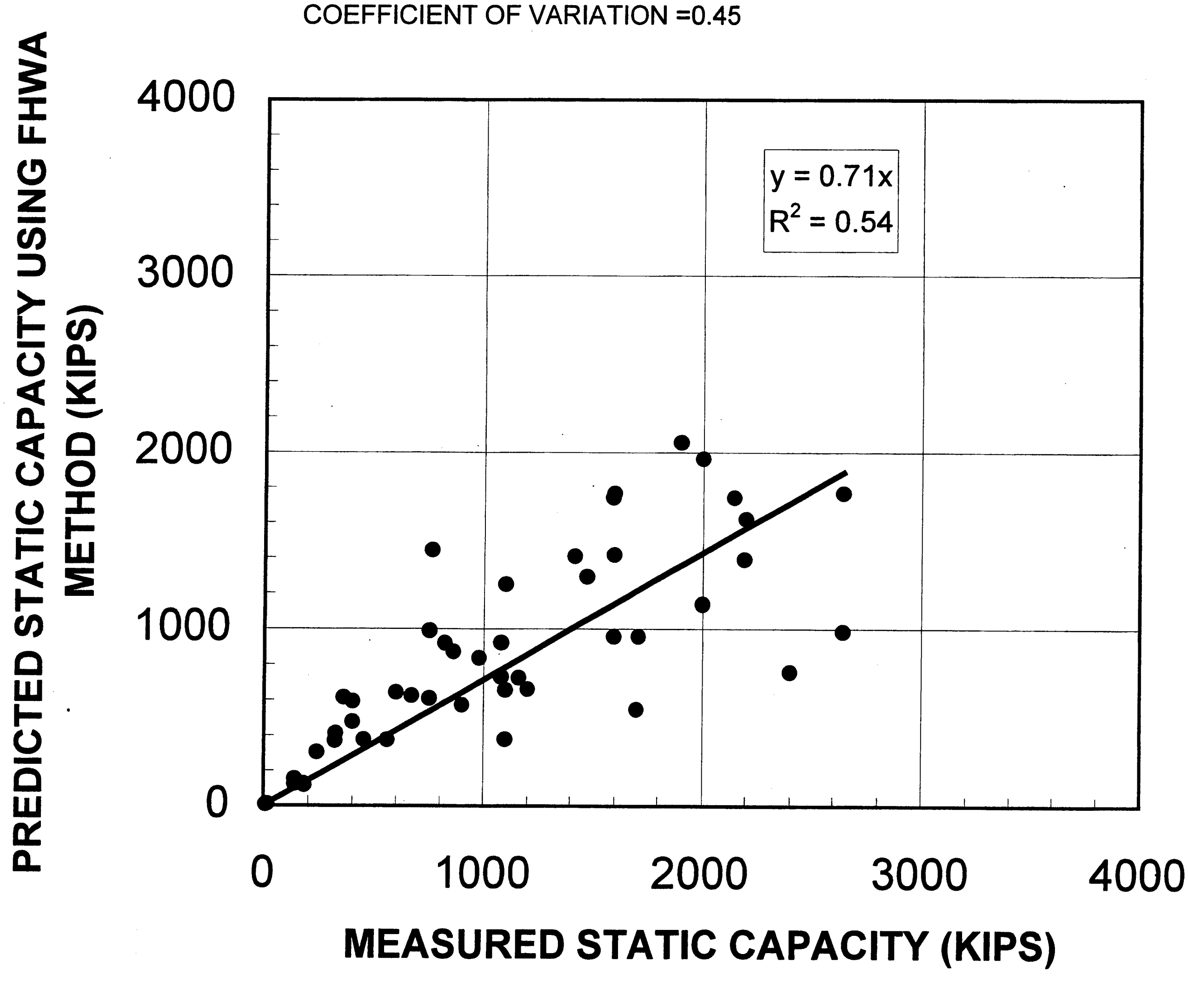

Figure 3

TOTAL DRILLED SHAFT IN SOIL

CONSTRUCTION METHOD= MIXED
LOAD TRANSFER= SKIN FRICTION + END BEARING
N= 136
MEAN=1.32
STANARD DEVIATION =0.91
COEFFICIENT OF VARIATION =0.69

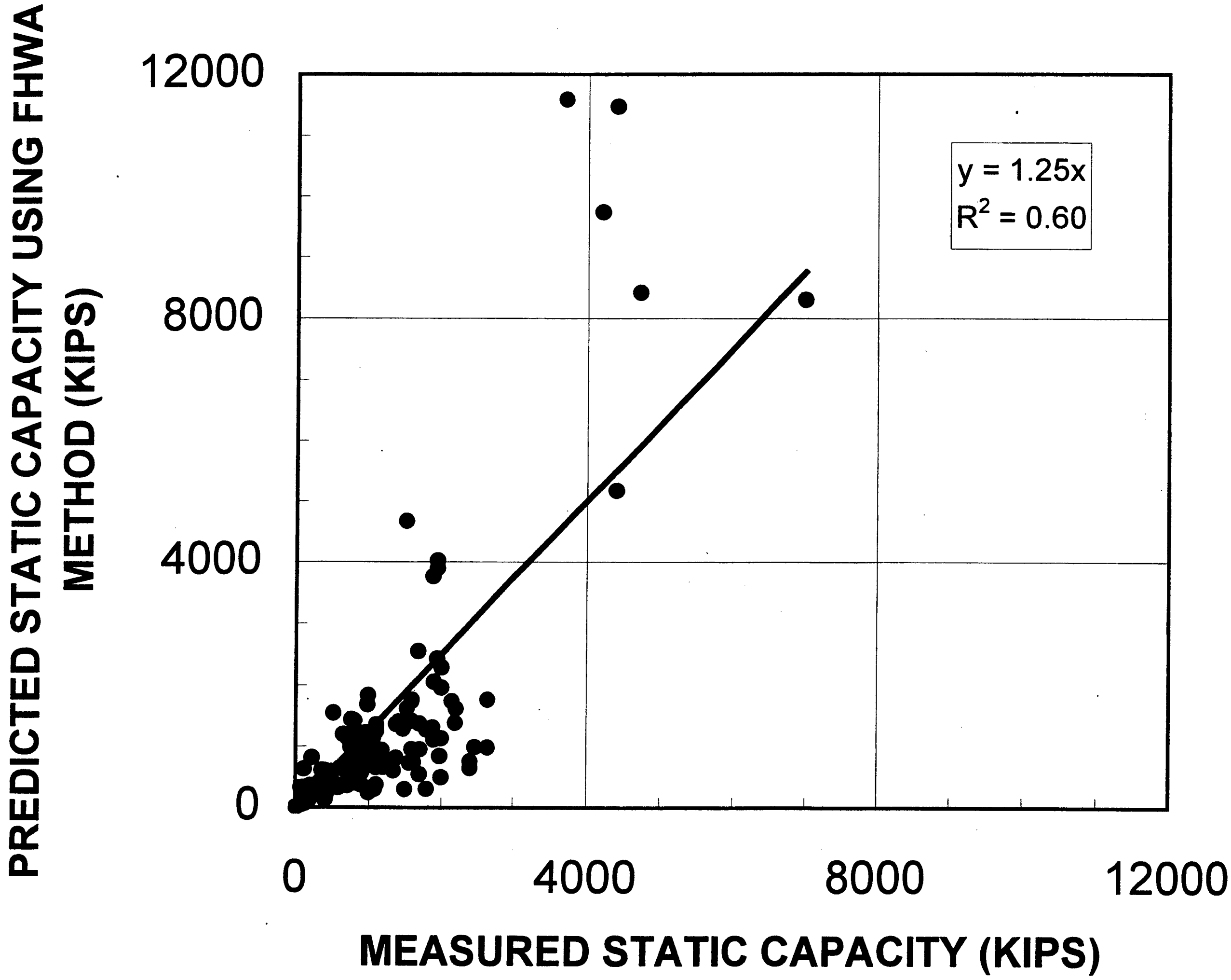

Figure 4

DRILLED SHAFT IN SAND

CONSTRUCTION METHOD=MIXED
LOAD TRANSFER= SKIN FRICTION + END BEARING
N= 34
MEAN=1.42
STANARD DEVIATION = 1.14
COEFFICIENT OF VARIATION =0.81

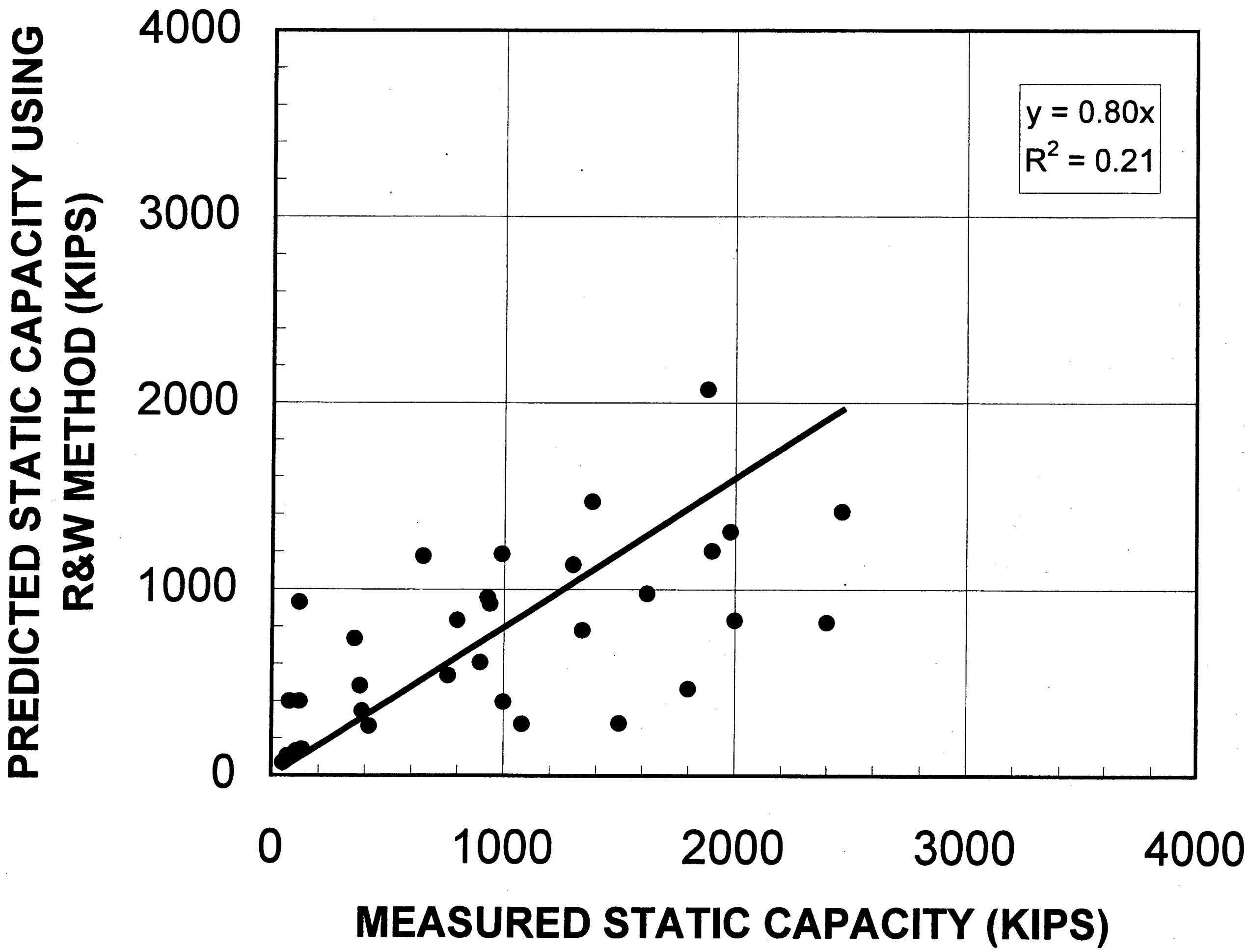

Figure 5

DRILLED SHAFT IN SAND & CLAY

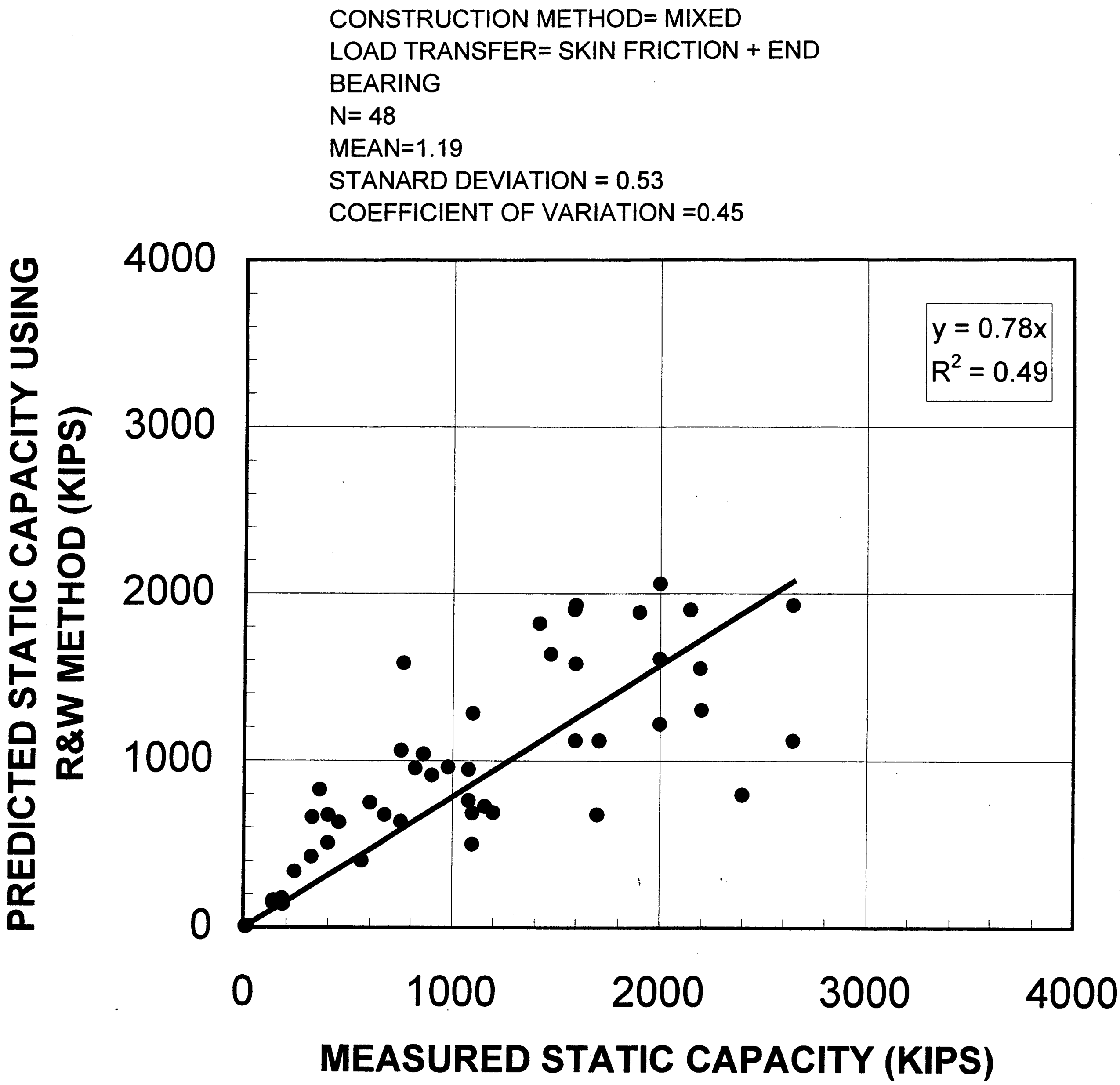

Figure 6

TOTAL DRILLED SHAFT IN SOIL

CONSTRUCTION METHOD= MIXED
LOAD TRANSFER= SKIN FRICTION + END BEARING
N= 82
MEAN=1.28
STANARD DEVIATION =0.84
COEFFICIENT OF VARIATION =0.66

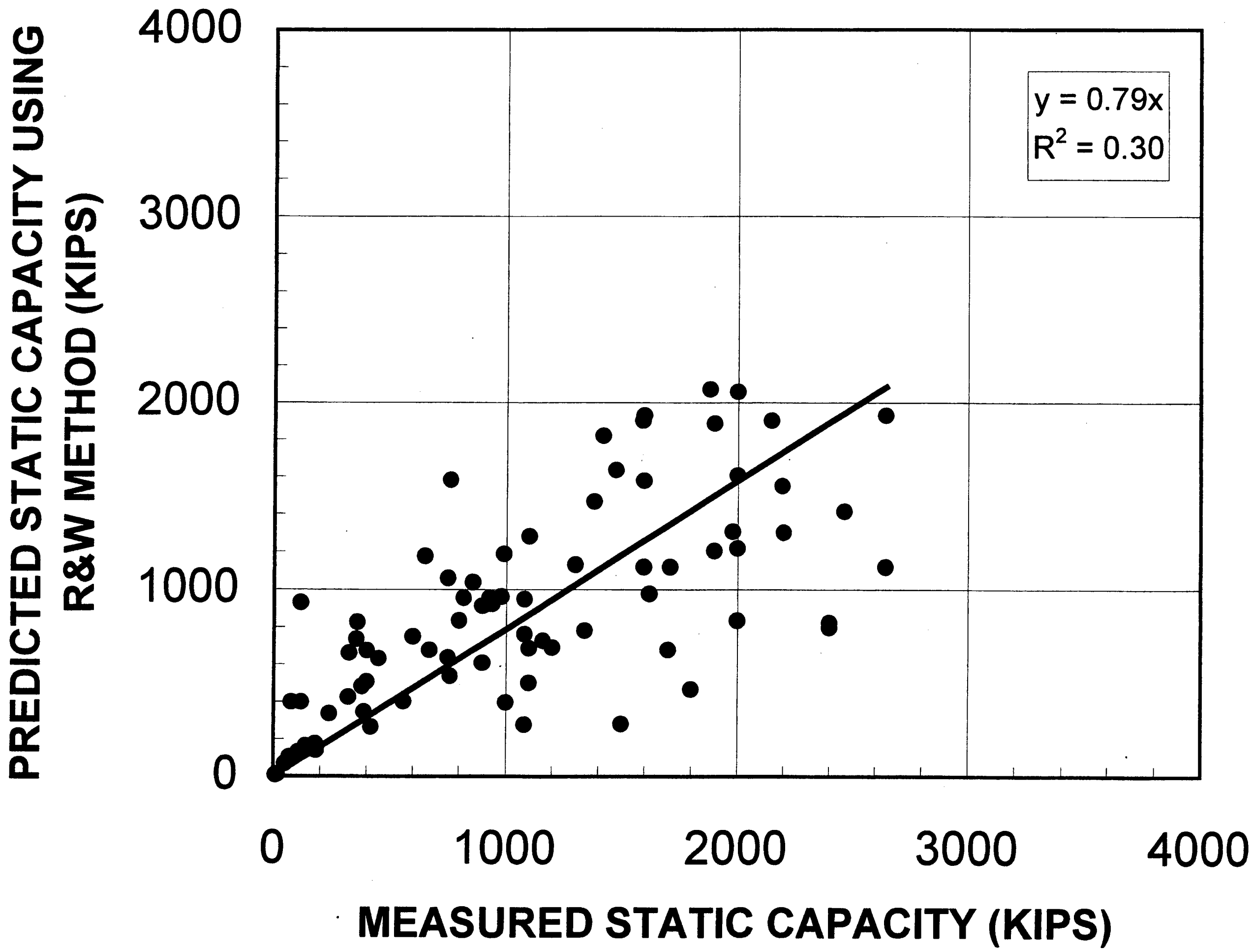

Figure 7

DRILLED SHAFT IN SAND & CLAY

CONSTRUCTION METHOD= CASING
LOAD TRANSFER= SKIN FRICTION + END BEARING
N=23
MEAN=1.21
STANARD DEVIATION = 0.64
COEFFICIENT OF VARIATION =0.53

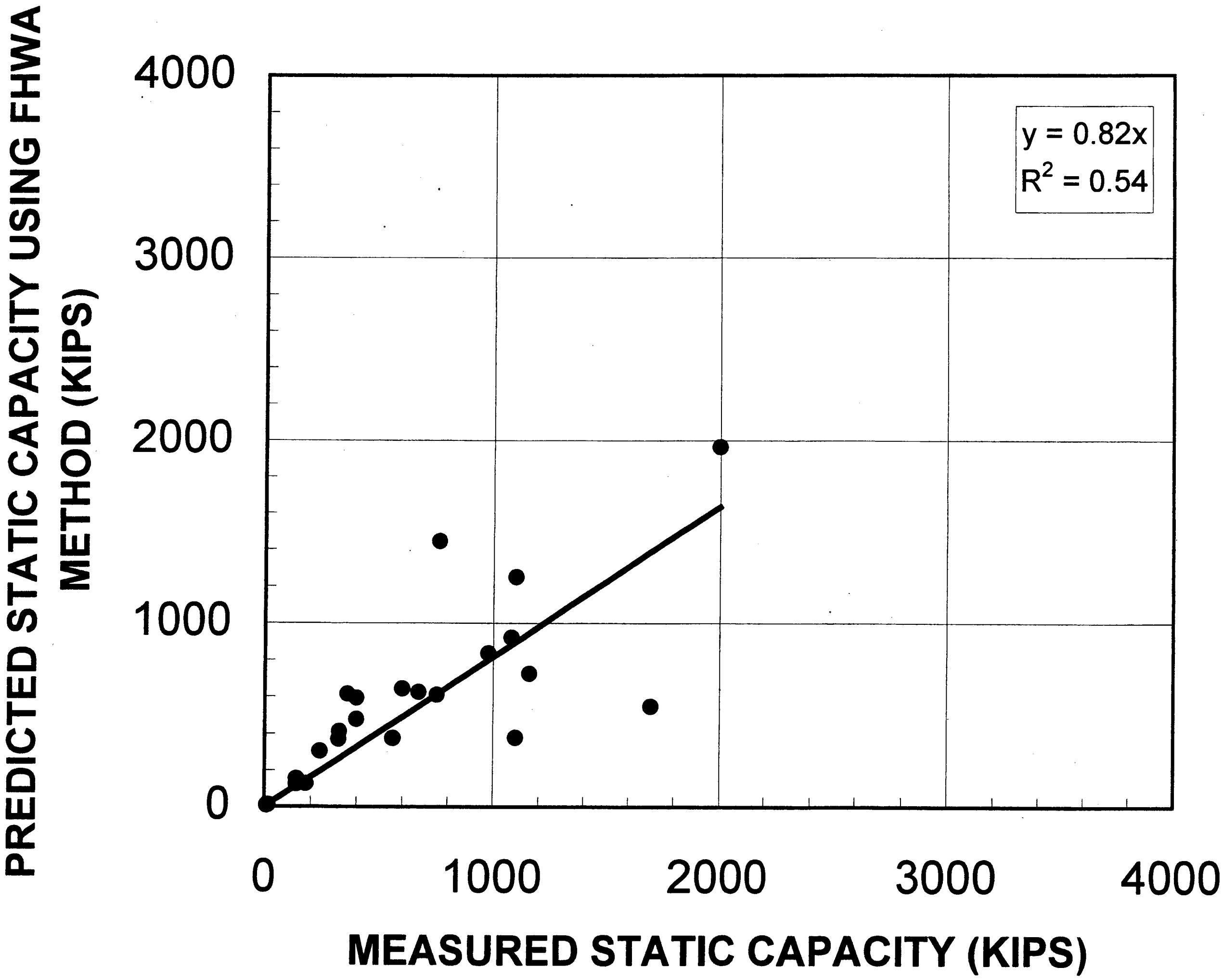

Figure 8

DRILLED SHAFT IN SAND & CLAY

CONSTRUCTION METHOD= CASING
LOAD TRANSFER= SKIN FRICTION + END BEARING
N= 23
MEAN=1.07
STANARD DEVIATION = 0.51
COEFFICIENT OF VARIATION =0.48

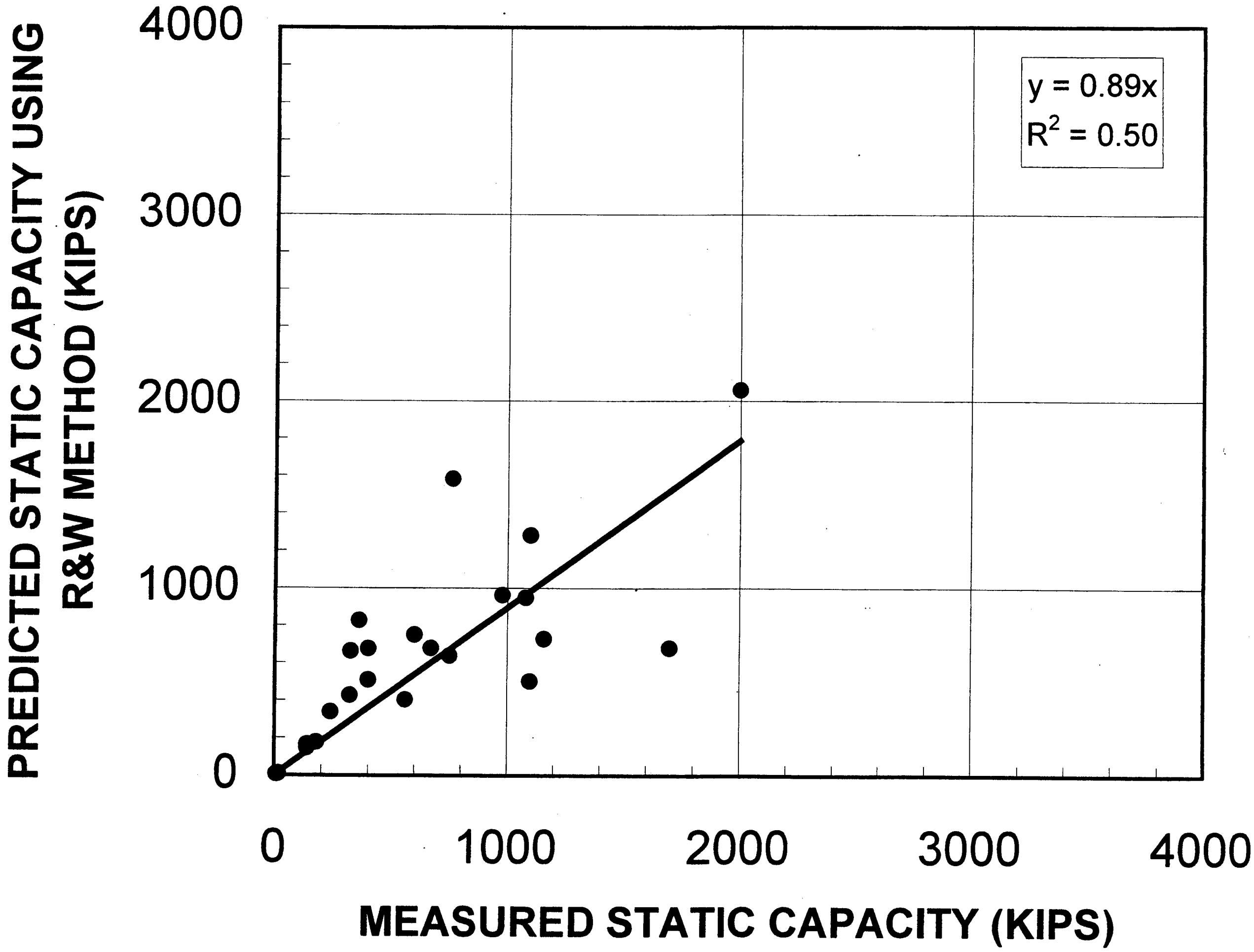

Figure 9

DRILLED SHAFT IN SAND & CLAY

CONSTRUCTION METHOD= DRY
LOAD TRANSFER= SKIN FRICTION + END BEARING
N= 13
MEAN=1.47
STANARD DEVIATION = 0.64
COEFFICIENT OF VARIATION =0.44

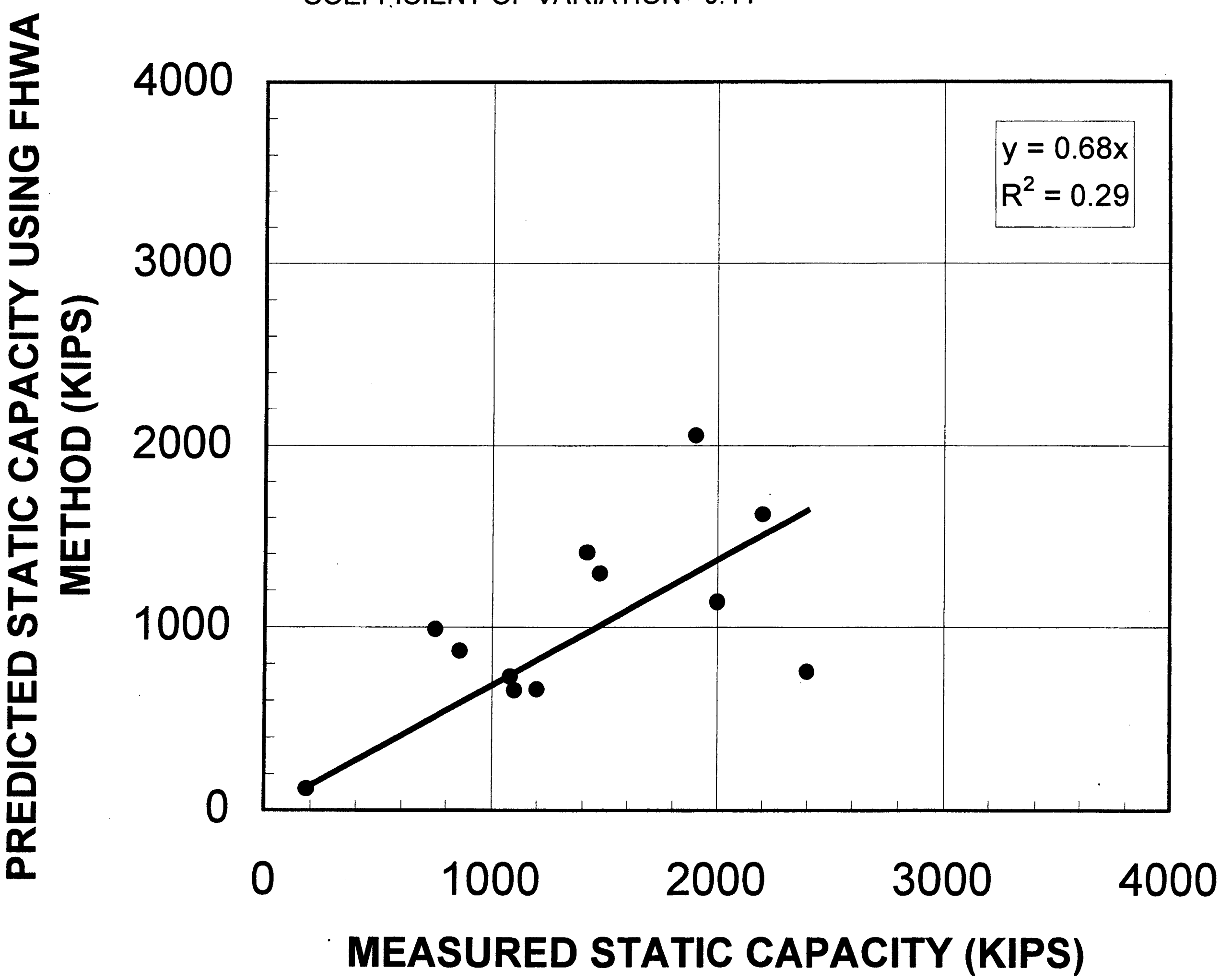

Figure 10

DRILLED SHAFT IN SAND & CLAY

CONSTRUCTION METHOD= DRY
LOAD TRANSFER= SKIN FRICTION + END BEARING
N= 13
MEAN=1.36
STANARD DEVIATION = 0.64
COEFFICIENT OF VARIATION =0.47

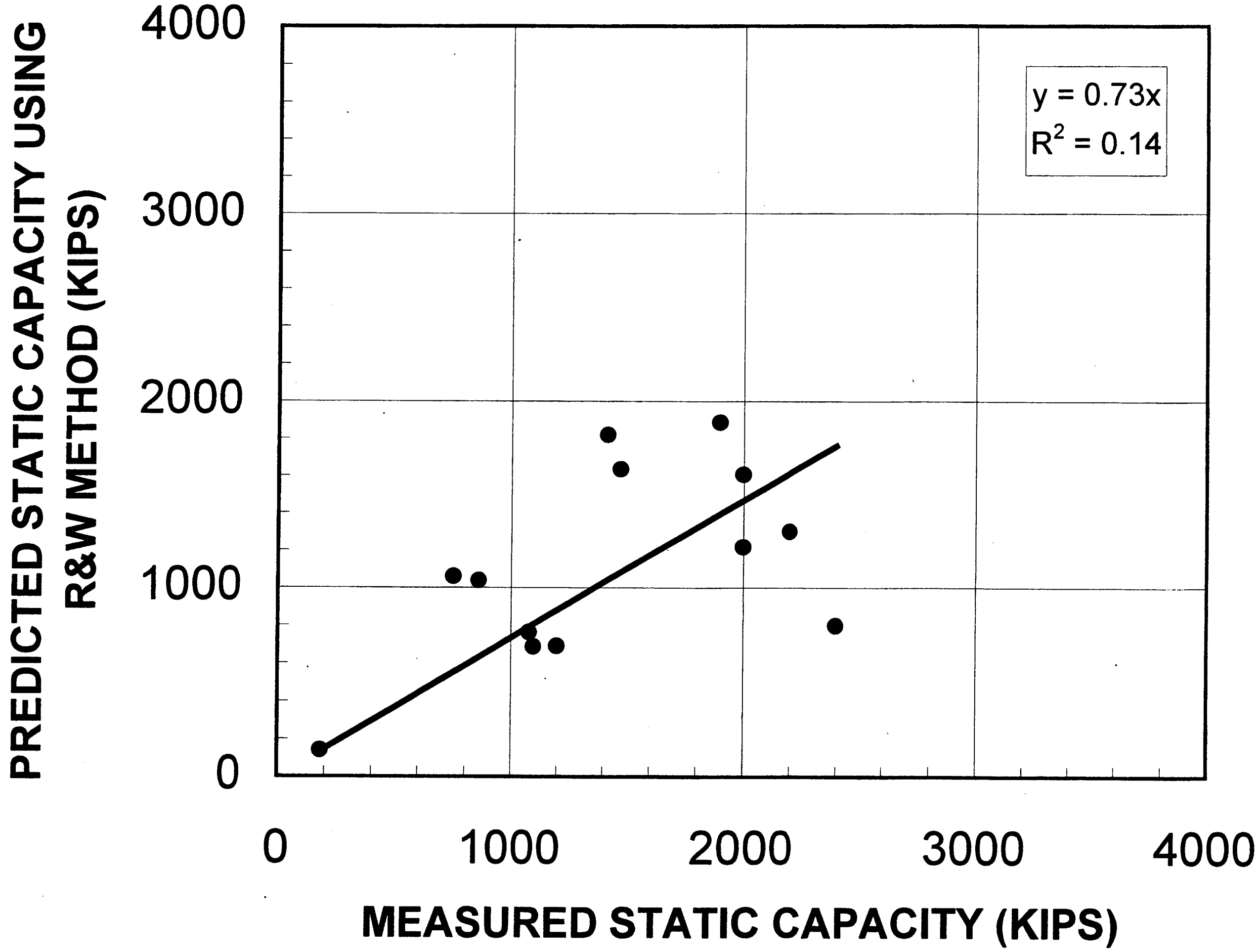

Figure 11

DRILLED SHAFT IN SAND & CLAY

CONSTRUCTION METHOD=SLURRY
LOAD TRANSFER= SKIN FRICTION + END BEARING
N= 12
MEAN=1.43
STANARD DEVIATION = 0.55
COEFFICIENT OF VARIATION =0.39

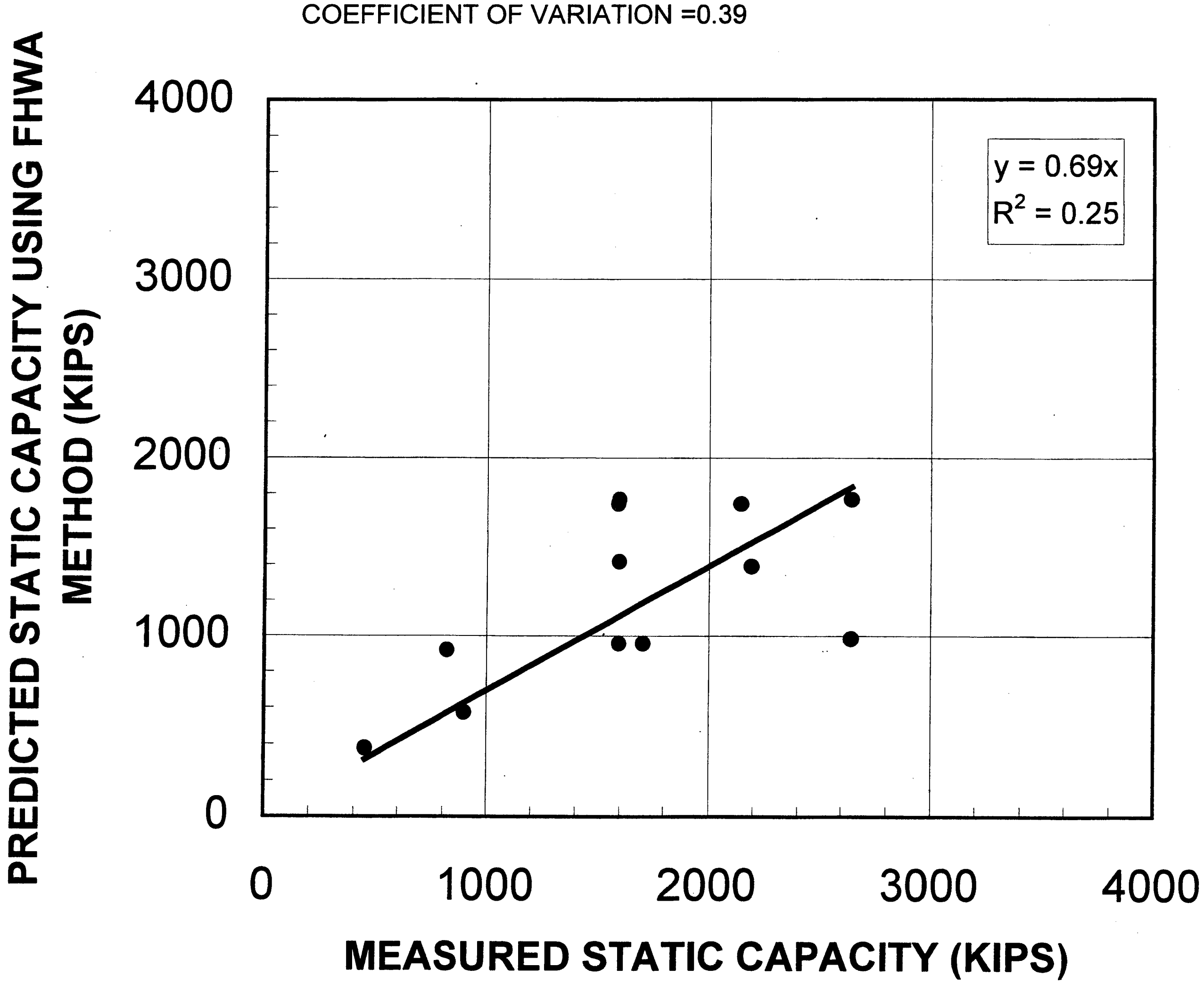

Figure 12

DRILLED SHAFT IN SAND & CLAY

CONSTRUCTION METHOD= SLURRY
LOAD TRANSFER= SKIN FRICTION + END BEARING
N= 12
MEAN=1.28
STANARD DEVIATION = 0.47
COEFFICIENT OF VARIATION =0.37

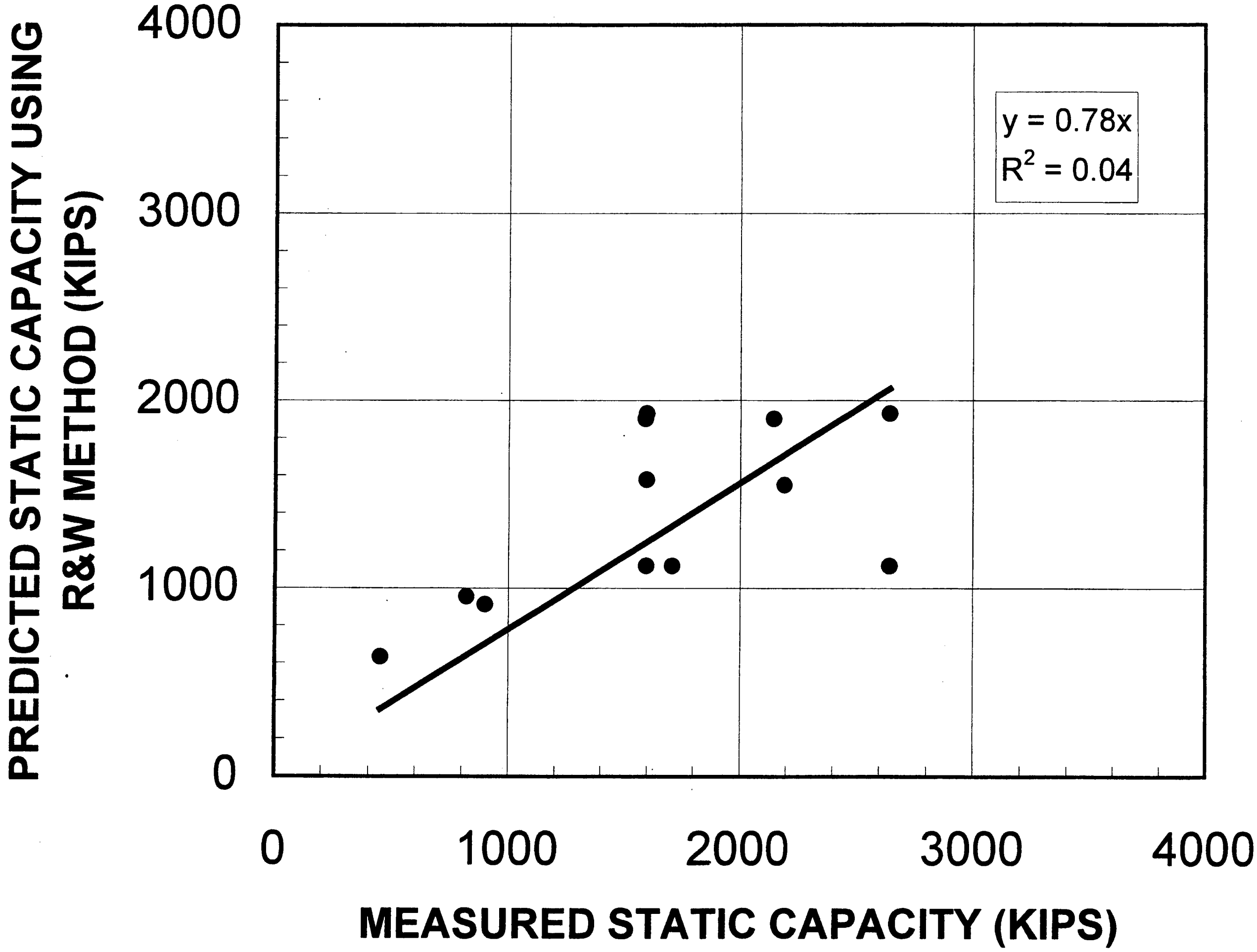

Figure 13

DRILLED SHAFT IN SAND

CONSTRUCTION METHOD= CASING
LOAD TRANSFER= SKIN FRICTION + END BEARING
N= 14
MEAN=2.47
STANARD DEVIATION = 1.24
COEFFICIENT OF VARIATION =0.509

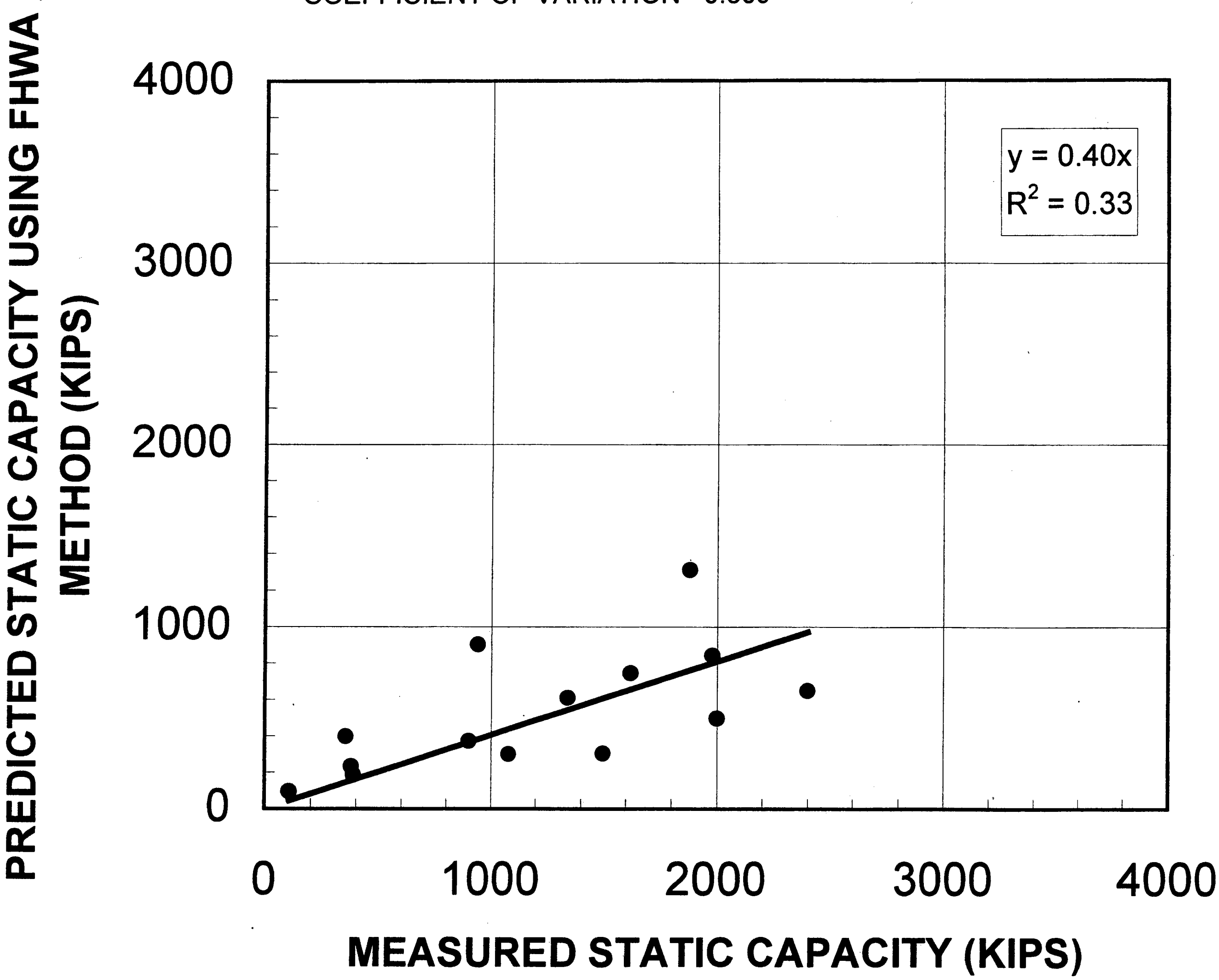

Figure 14

DRILLED SHAFT IN SAND

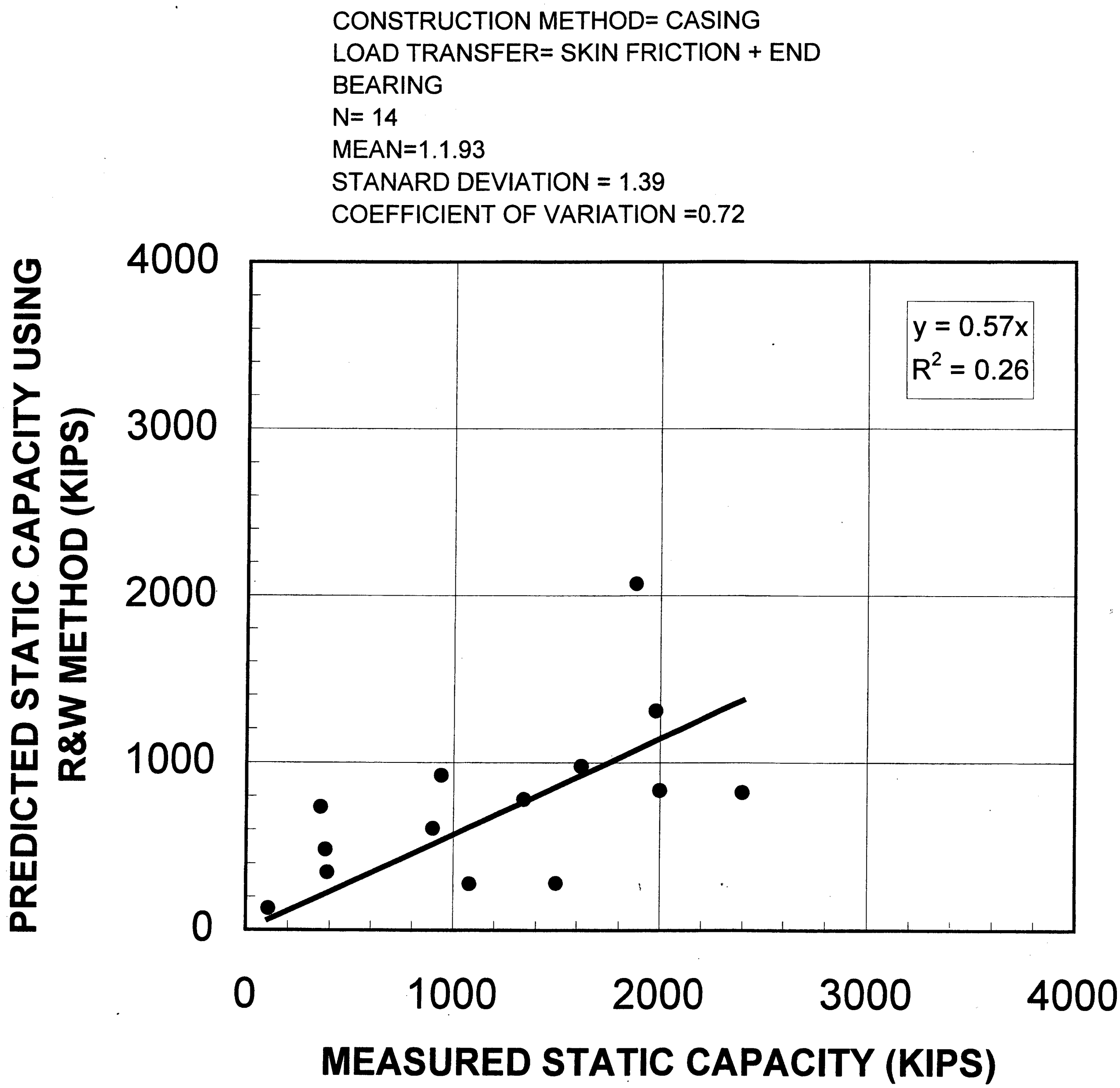

Figure 15

DRILLED SHAFT IN SAND

CONSTRUCTION METHOD= SLURRY
LOAD TRANSFER= SKIN FRICTION + END BEARING
N= 14
MEAN=2.04
STANARD DEVIATION = 1.75
COEFFICIENT OF VARIATION =0.86

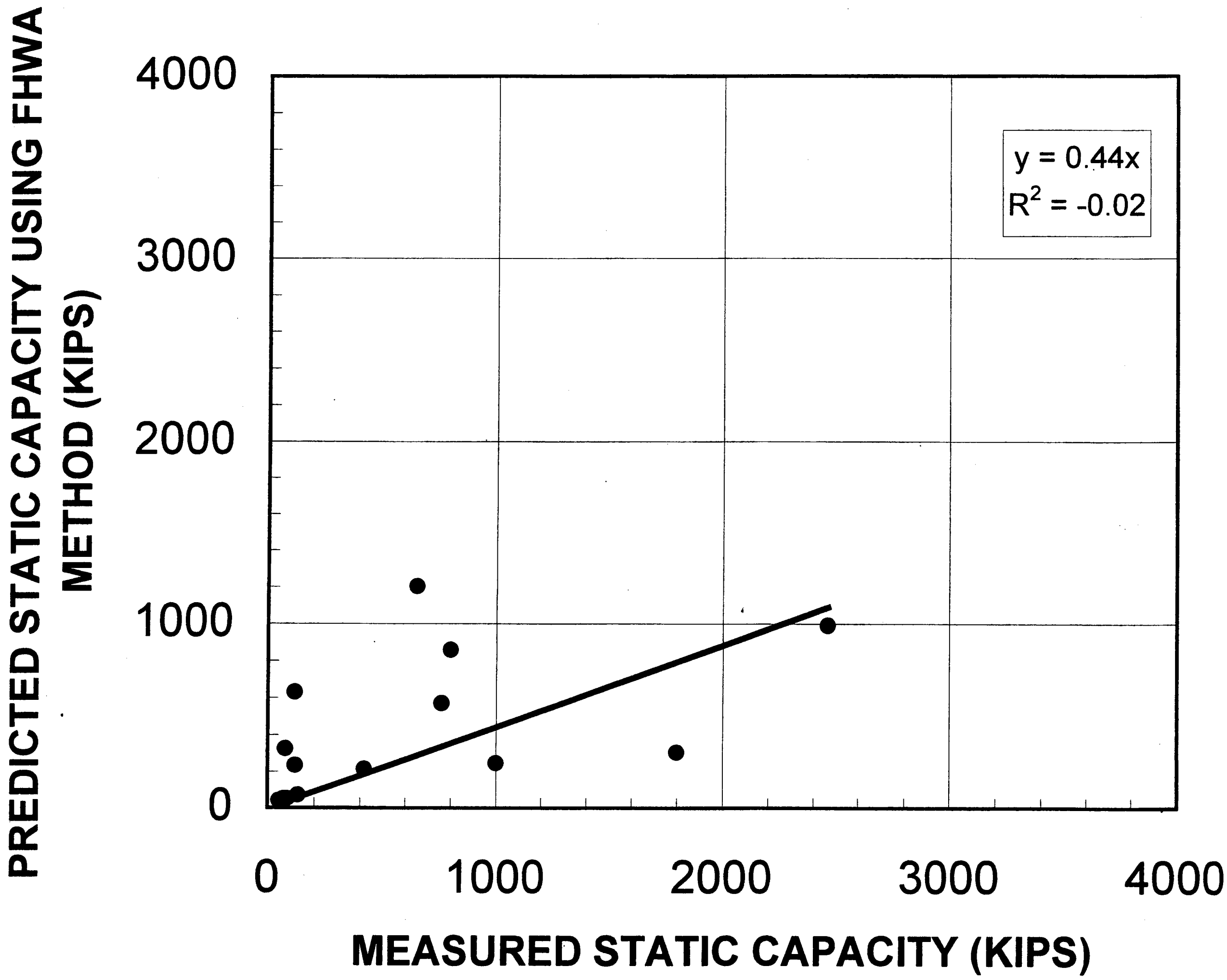

Figure 16

DRILLED SHAFT IN SAND

CONSTRUCTION METHOD=SLURRY
LOAD TRANSFER= SKIN FRICTION + END BEARING
N= 14
MEAN=1.32
STANARD DEVIATION = 1.55
COEFFICIENT OF VARIATION =1.18

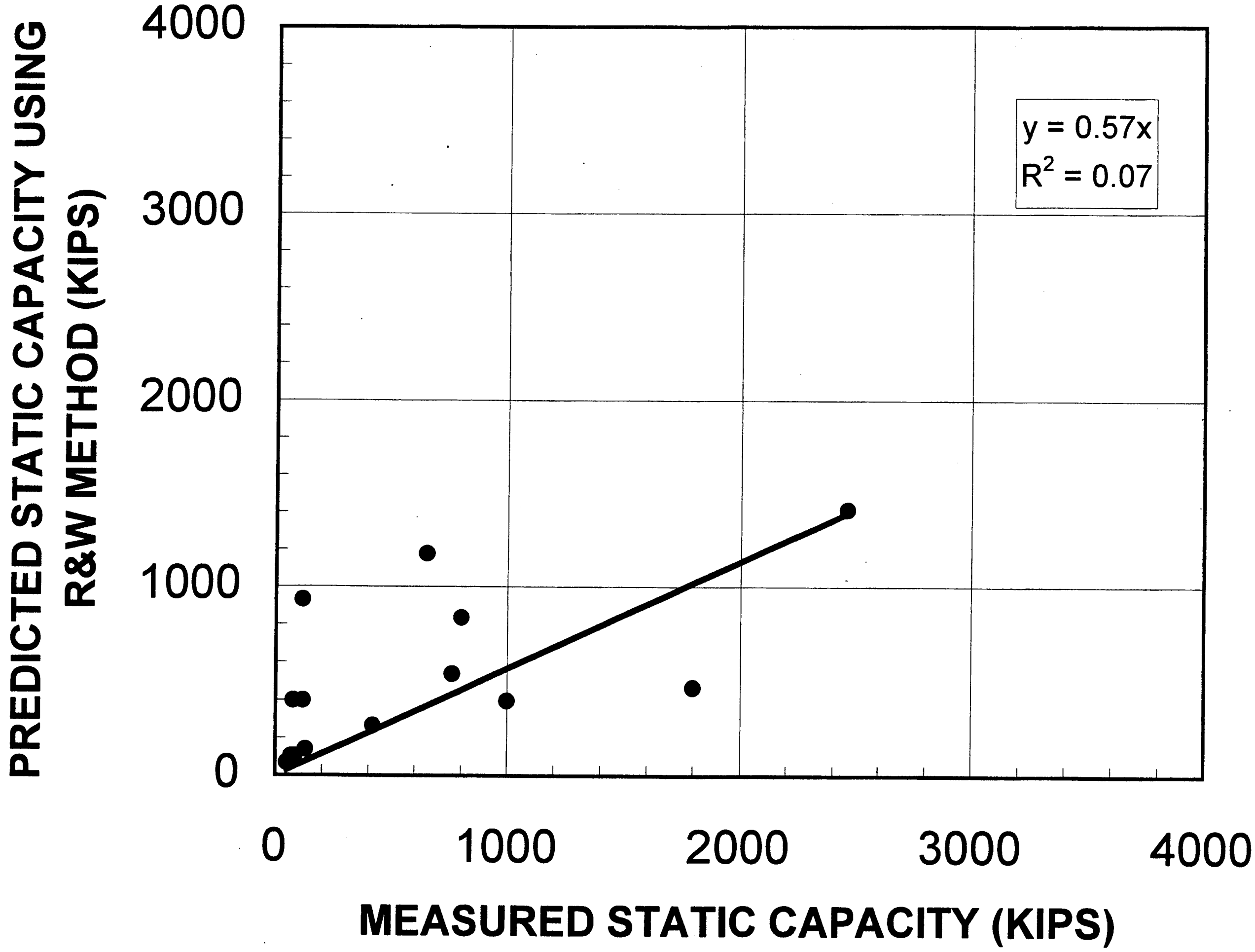

Figure 17

DRILLED SHAFT IN CLAY

CONSTRUCTION METHOD= CASING
LOAD TRANSFER= SKIN FRICTION + END BEARING
N= 14
MEAN=0.99
STANARD DEVIATION = 0.70
COEFFICIENT OF VARIATION =0.71

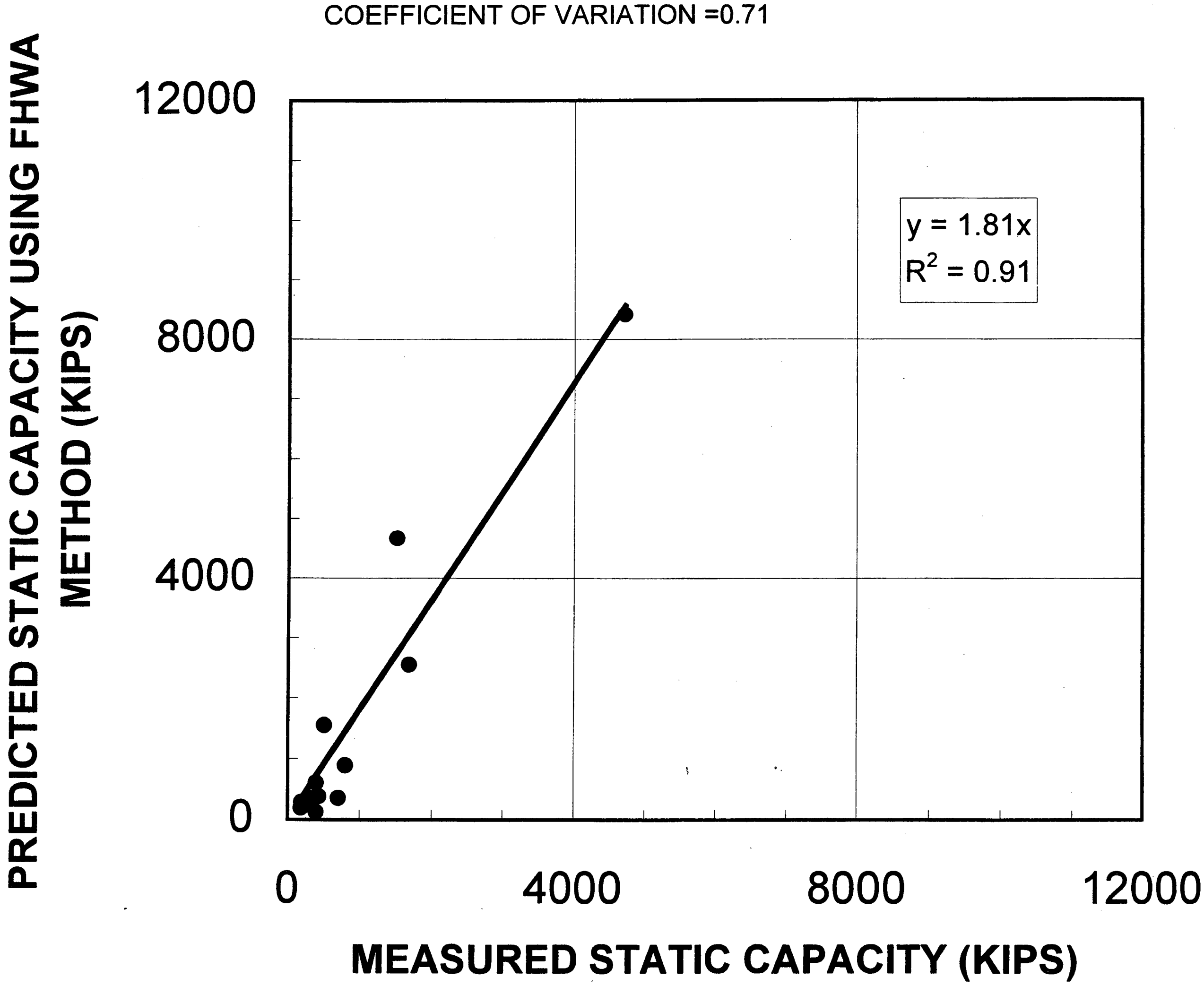

Figure 18

DRILLED SHAFT IN CLAY

CONSTRUCTION METHOD= DRY
LOAD TRANSFER= SKIN FRICTION + END BEARING
N= 40
MEAN=0.88
STANARD DEVIATION = 0.42
COEFFICIENT OF VARIATION =0.48

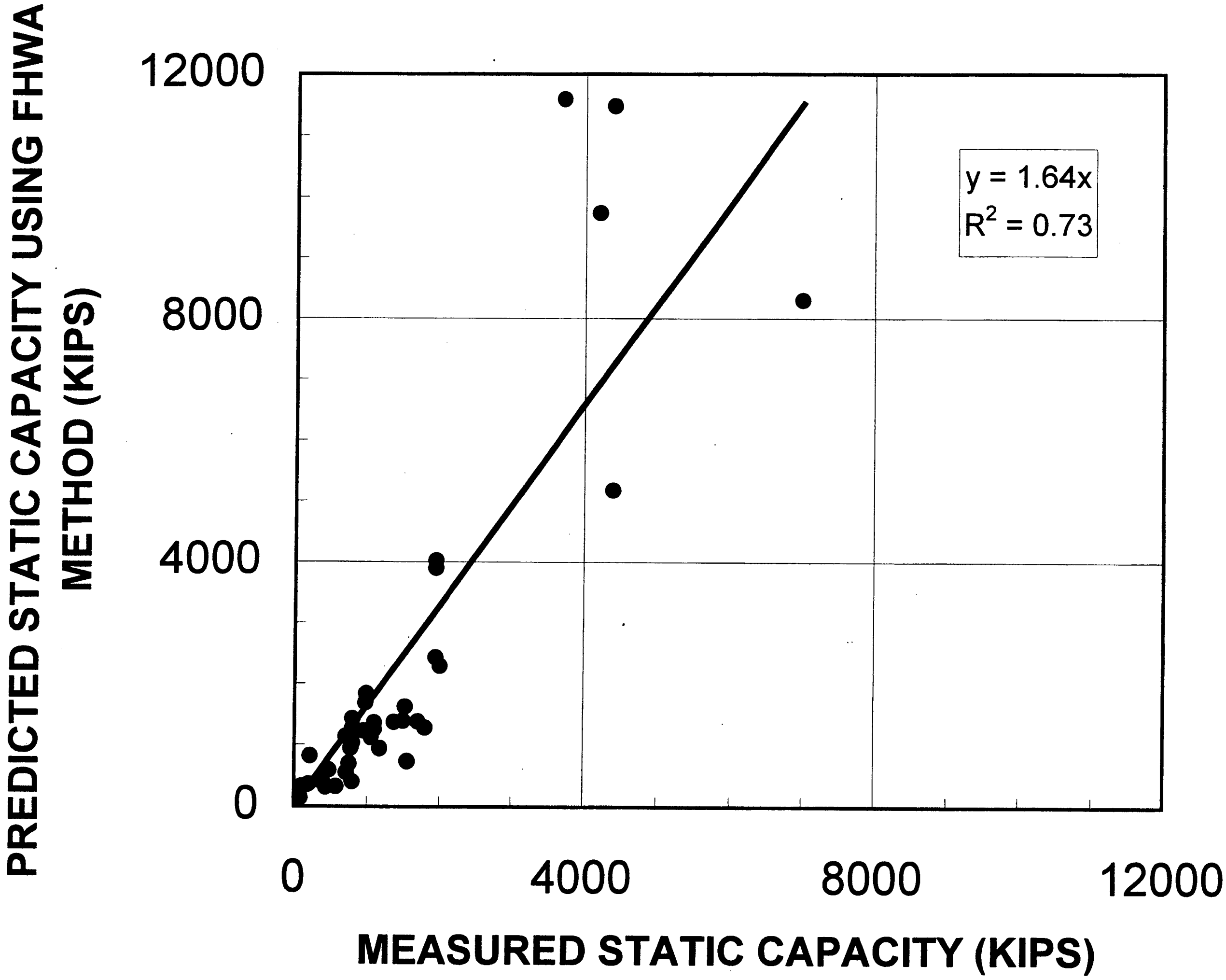

Figure 19

Table		Failure		FHWA	FHWA	W&R	W&R
SHAFT #	Construction	Load (kips)	Soil Type	Predict(kips)	Ratio	Predict(kips)	Ratio
5	Casing	180	A	126.82	1.42	175.56	1.03
6	Casing	322	A	370	0.87	426.00	0.76
12	Casing	10.8	A	8.62	1.25	7.60	1.42
13	Dry	18	A	12	1.50	12.06	1.49
14	Dry	140	A	154.3	0.91	163.90	0.85
18	Dry	1100	A	1250	0.88	1282.00	0.86
19	Dry	1160	A	725.66	1.60	727.08	1.60
20	Dry	1080	A	923.24	1.17	949.72	1.14
22	Dry	1700	A	548	3.10	679.24	2.50
25	Dry	360	A	616.12	0.58	827.00	0.44
31	Dry & Casing	980	A	837.04	1.17	963.20	1.02
38	Slurry	1474	A	1293.28	1.14	1635.78	0.90
85	Slurry	184	A	118.58	1.55	141.40	1.30
95	Slurry	2400	A	758	3.17	798.00	3.01
153	Slurry	1200	A	662	1.81	691.30	1.74
155	Slurry	1100	A	658.26	1.67	686.92	1.60
32	Casing	356	B	397.2	0.90	735.96	0.48
34	Casing	940	B	904	1.04	924.00	1.02
58	Casing	1620	B	746.6	2.17	977.94	1.66
66	Casing	930	B	688	1.35	956.00	0.97
67	Casing	1380	B	814	1.70	1466.00	0.94
123	Casing	70	B	52.3	1.34	103.48	0.68
164	Casing & Slurry	76	B	326	0.23	400.00	0.19
165	Dry	118	B	234	0.50	400.00	0.30
175	Slurry	650	B	1202	0.54	1176.00	0.55
176	Slurry	800	B	860	0.93	836.00	0.96
177	Slurry	760	B	572	1.33	538.00	1.41
61	Casing	300	C	362	0.83		
185	Casing	108	C	332	0.33		
197	Casing	1100	C	1360	0.81		
198	Dry	1180	C	942	1.25		
201	Dry	440	C	316	1.39		
203	Dry	1800	C	1276	1.41		
207	Dry	800	C	1426	0.56		
209	Dry	1380	C	1368	1.01		

SKIN FRICTION SHAFT in SOIL

FHWA Method	**40**			
	SOIL	**TYPE**		
	A	**B**	**C**	**ALL**
Mean	**1.49**	**1.09**	**0.87**	**1.18**
Stdv	**0.72**	**0.56**	**0.32**	**0.62**
Var	**0.49**	**0.51**	**0.37**	**0.53**

R&W Method	**27**			
	SOIL	**TYPE**		
	A	**B**		**ALL**
Mean	**1.35**	**0.83**		**1.14**
Stdv	**0.66**	**0.45**		**0.63**
Var	**0.49**	**0.54**		**0.55**

NOTE:
Soil type A = Sand & Clay
Soil type B = Sand
Soil type C = Clay

210	Dry	780	C	946	0.82		
211	Dry	950	C	1230	0.77		
212	Dry	800	C	1032	0.78		
213	Dry	988	C	1838	0.54		
242	Dry	4400	C	5178	0.85		

DRILLED SHAFT IN SAND

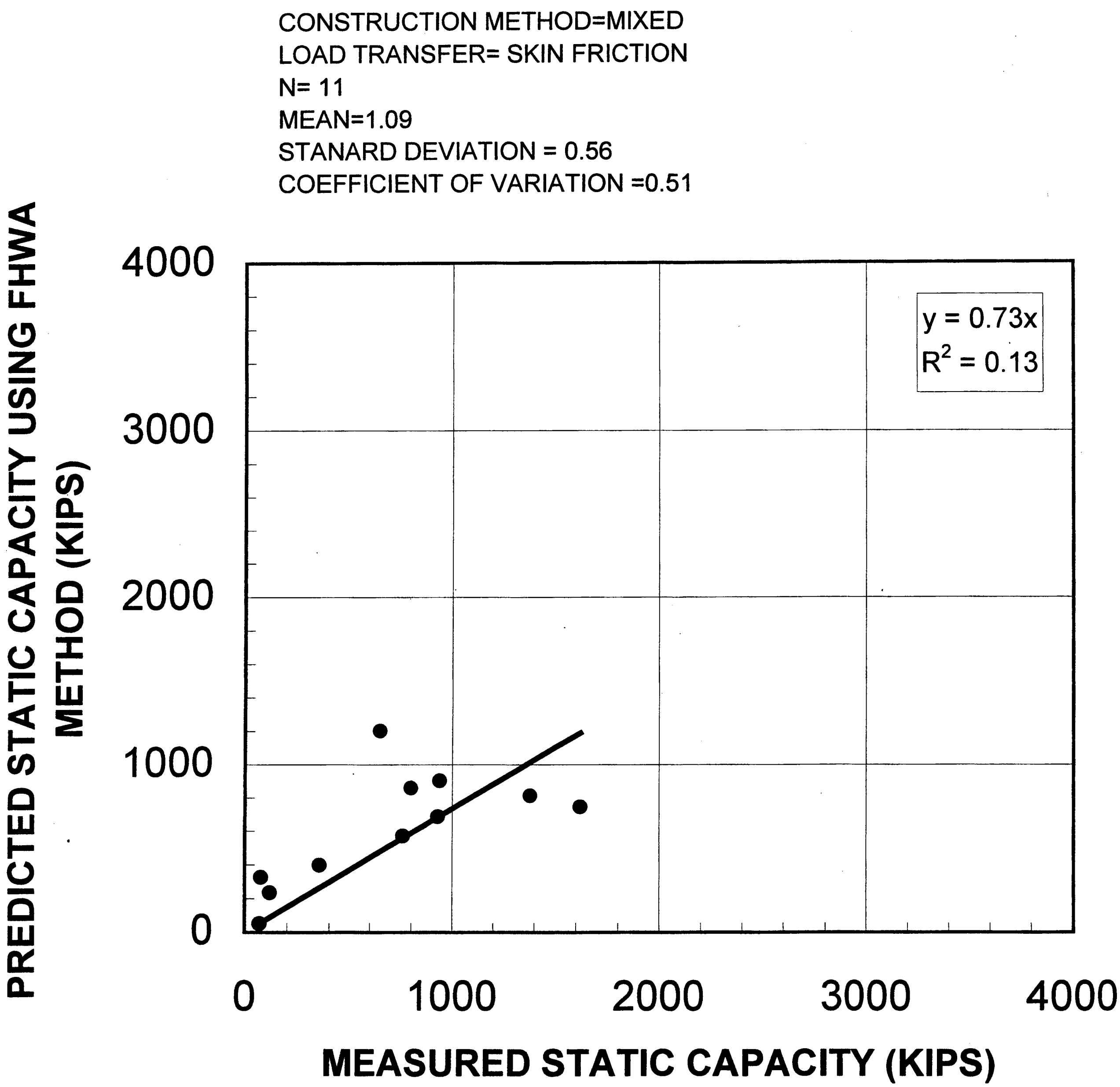

Figure 20

DRILLED SHAFT IN CLAY

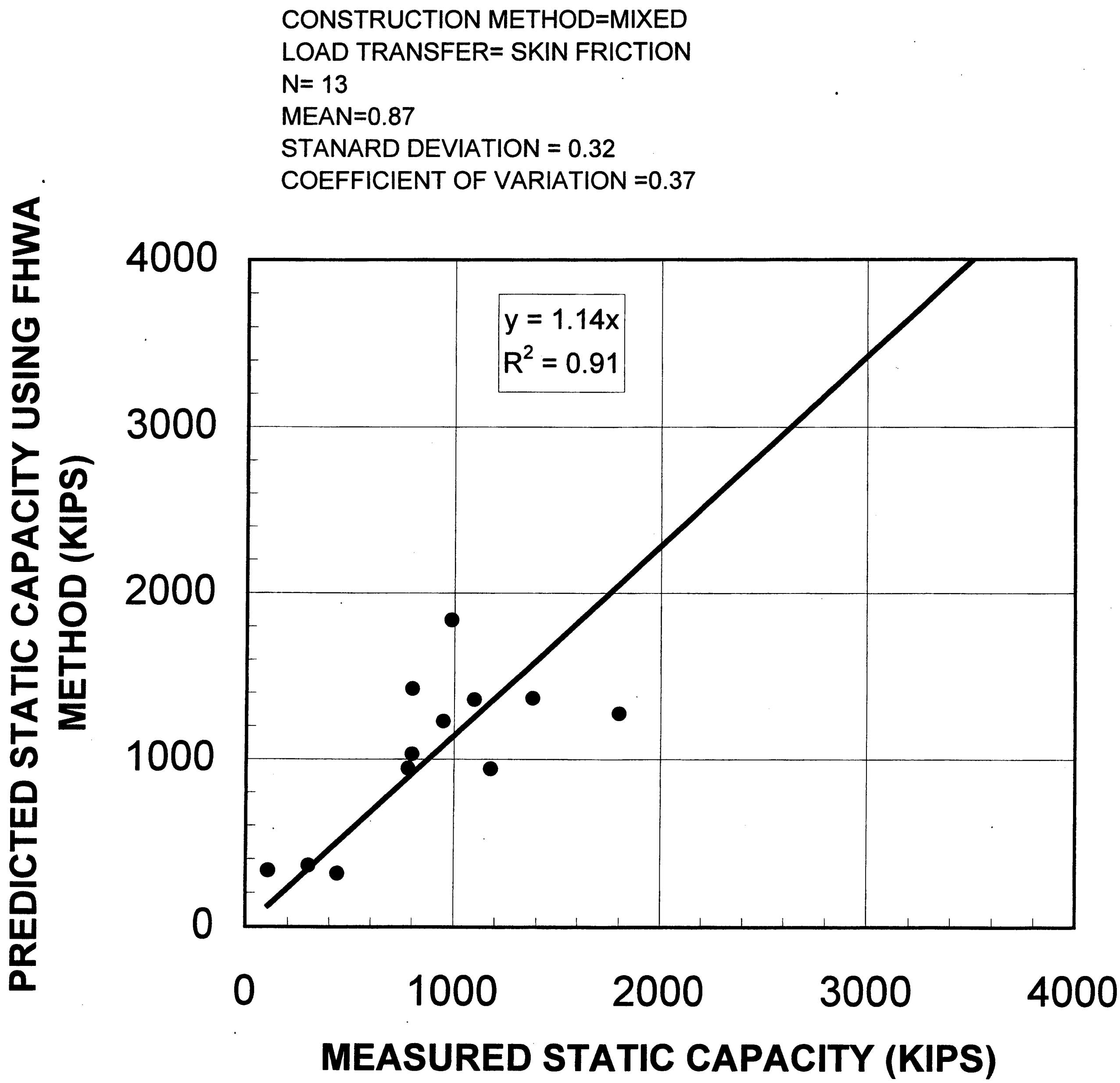

Figure 21

DRILLED SHAFT IN SAND & CLAY

CONSTRUCTION METHOD=MIXED
LOAD TRANSFER= SKIN FRICTION
N= 16
MEAN=1.49
STANARD DEVIATION = 0.72
COEFFICIENT OF VARIATION =0.49

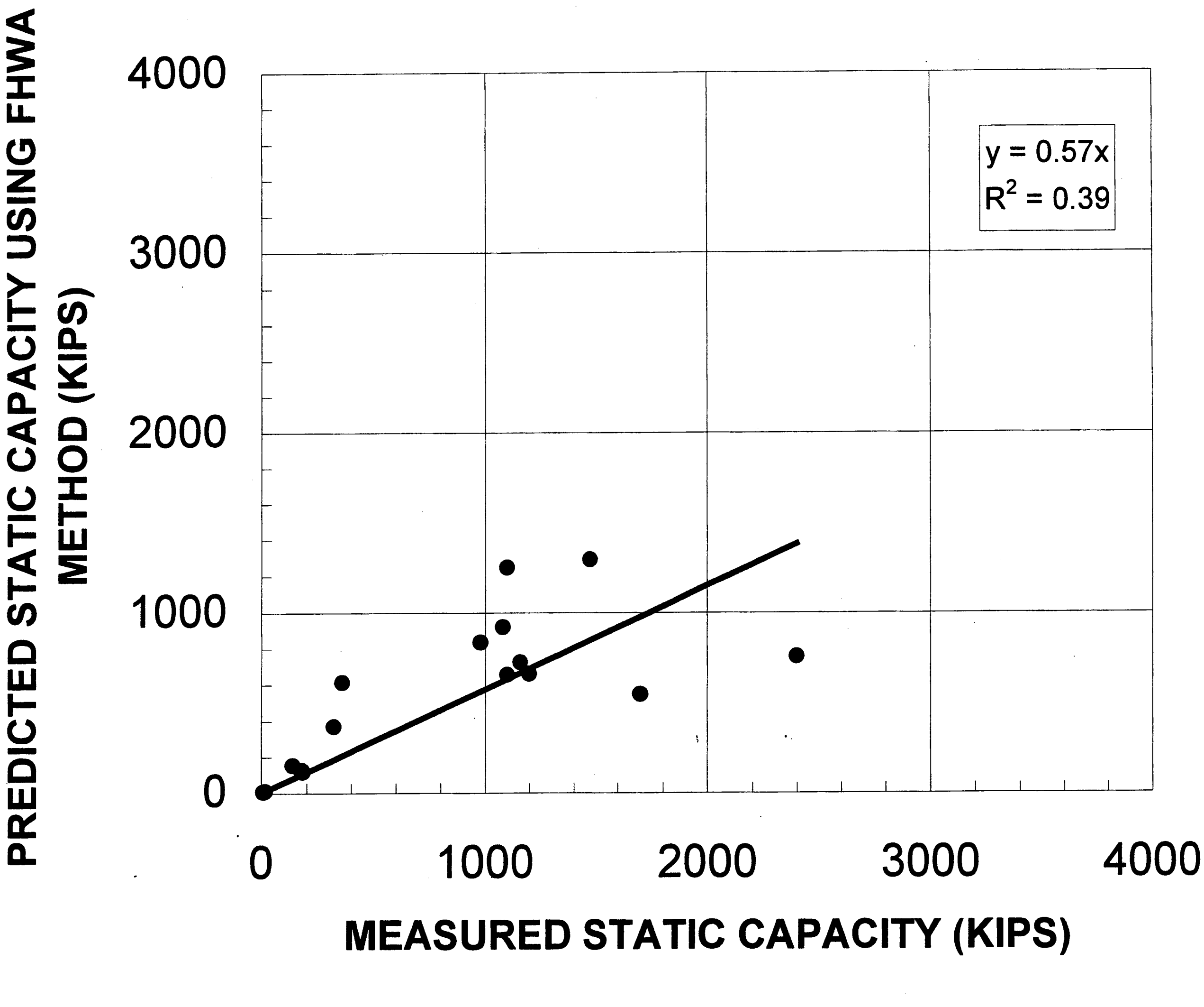

Figure 22

DRILLED SHAFT IN SOILS

CONSTRUCTION METHOD=MIXED
LOAD TRANSFER= SKIN FRICTION
N= 40
MEAN=1.16
STANARD DEVIATION = 0.62
COEFFICIENT OF VARIATION =0.53

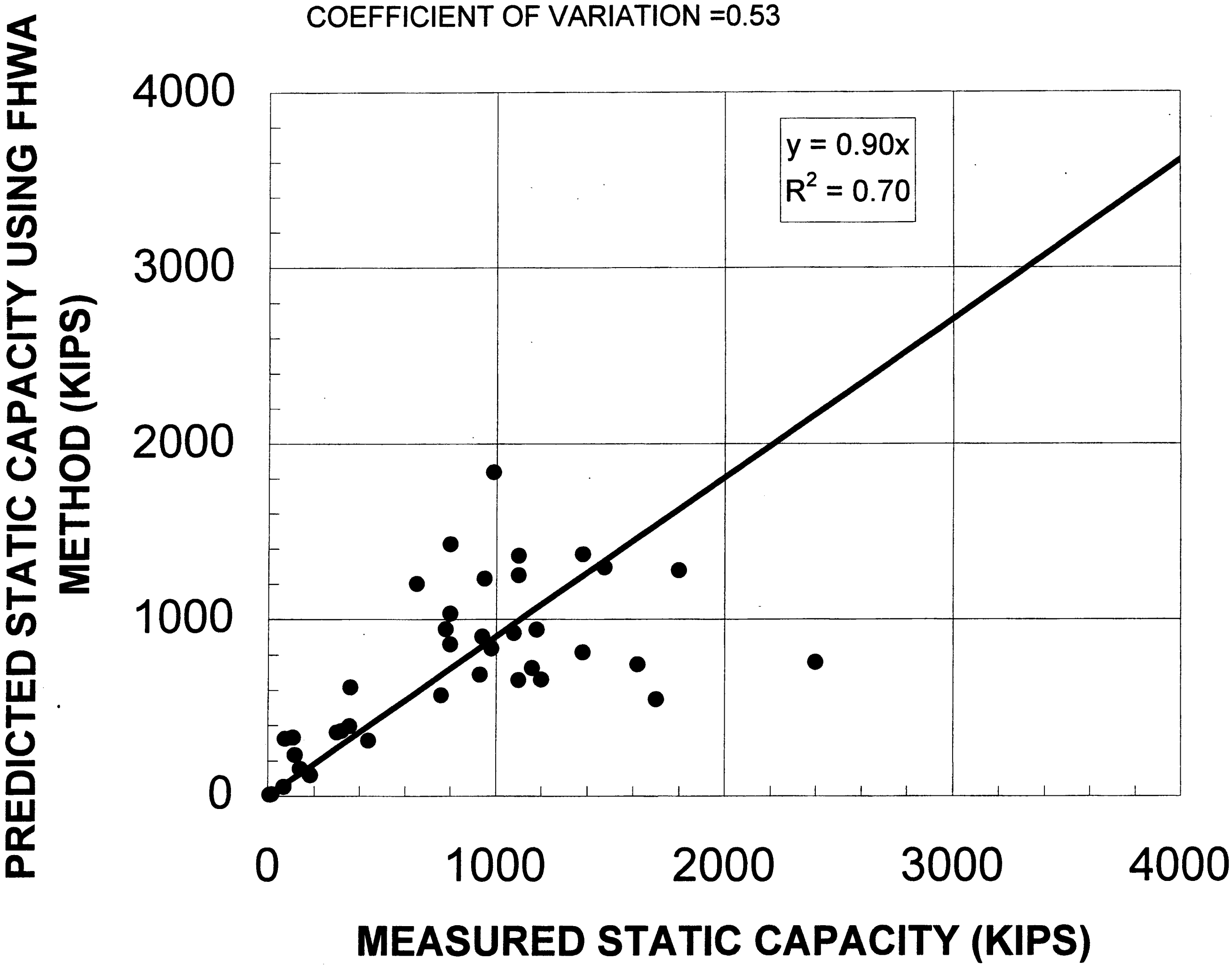

Figure 23

DRILLED SHAFT IN SAND

CONSTRUCTION METHOD=MIXED
LOAD TRANSFER= SKIN FRICTION
N= 11
MEAN=0.83
STANARD DEVIATION = 0.45
COEFFICIENT OF VARIATION =0.54

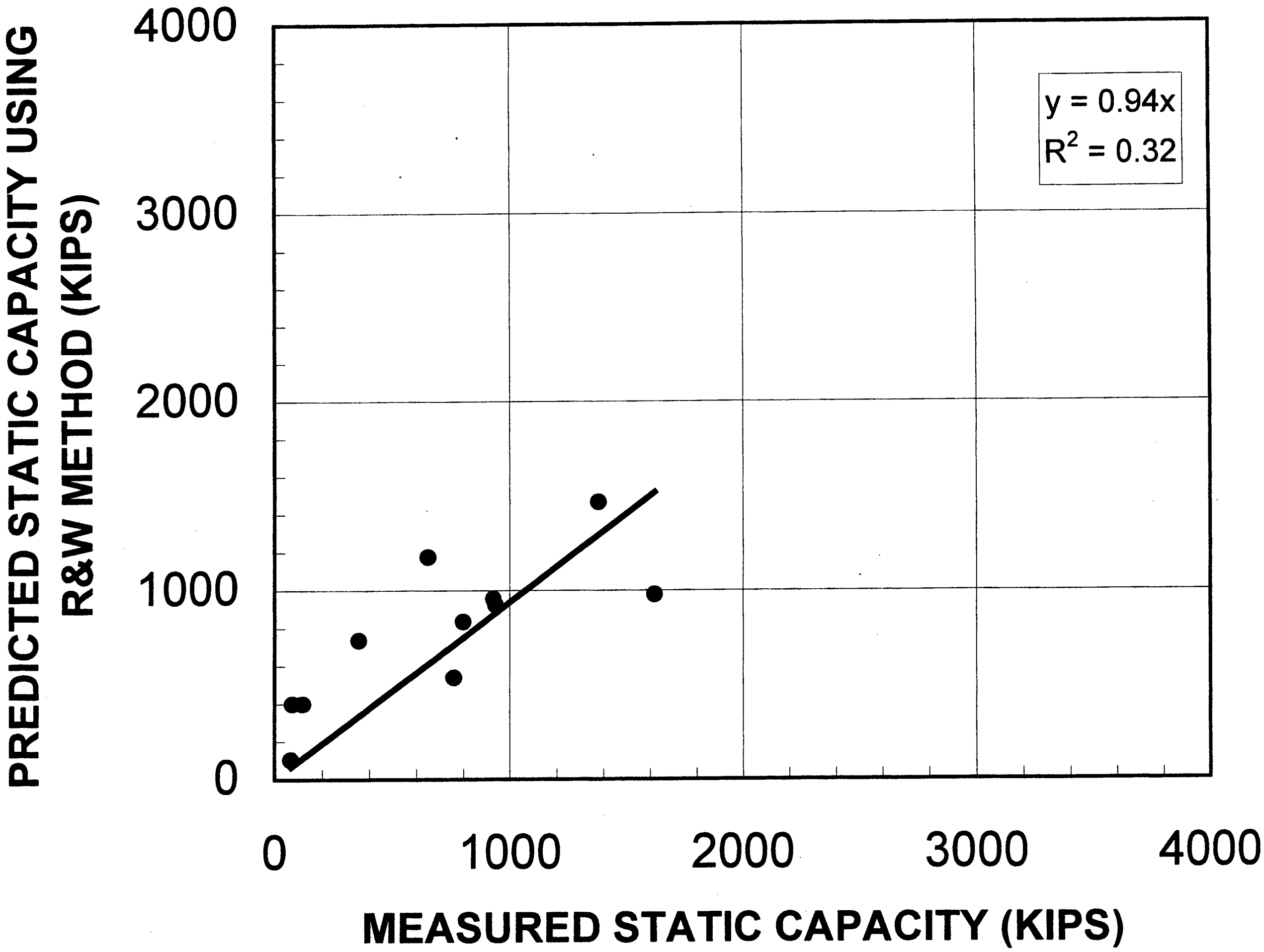

Figure 24

DRILLED SHAFT IN SAND & CLAY

CONSTRUCTION METHOD=MIXED
LOAD TRANSFER= SKIN FRICTION
N= 16
MEAN=1.35
STANARD DEVIATION = 0.66
COEFFICIENT OF VARIATION =0.49

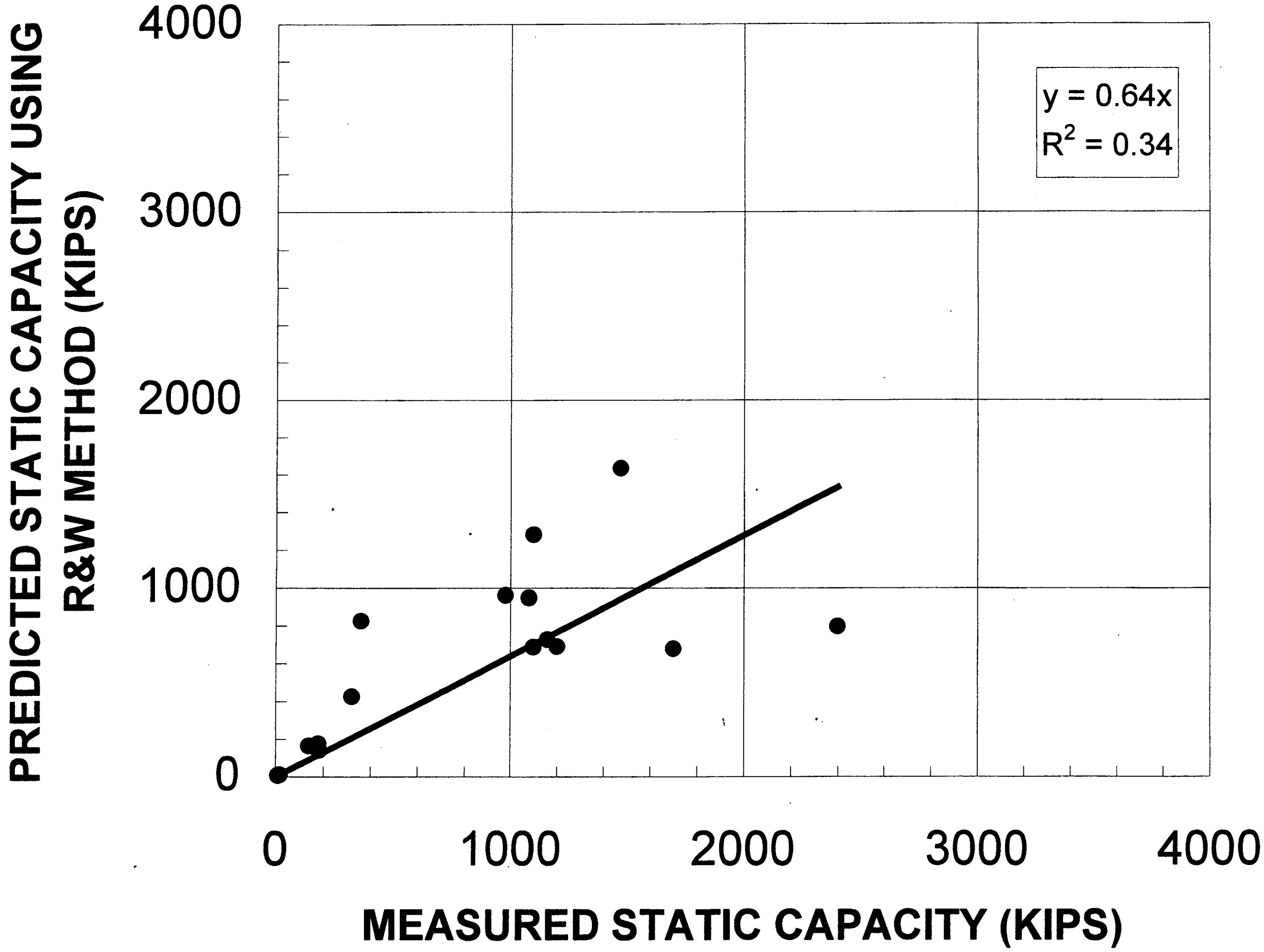

Figure 25

DRILLED SHAFT IN SOILS

CONSTRUCTION METHOD=MIXED
LOAD TRANSFER= SKIN FRICTION
N= 27
MEAN=1.14
STANARD DEVIATION = 0.63
COEFFICIENT OF VARIATION =0.55

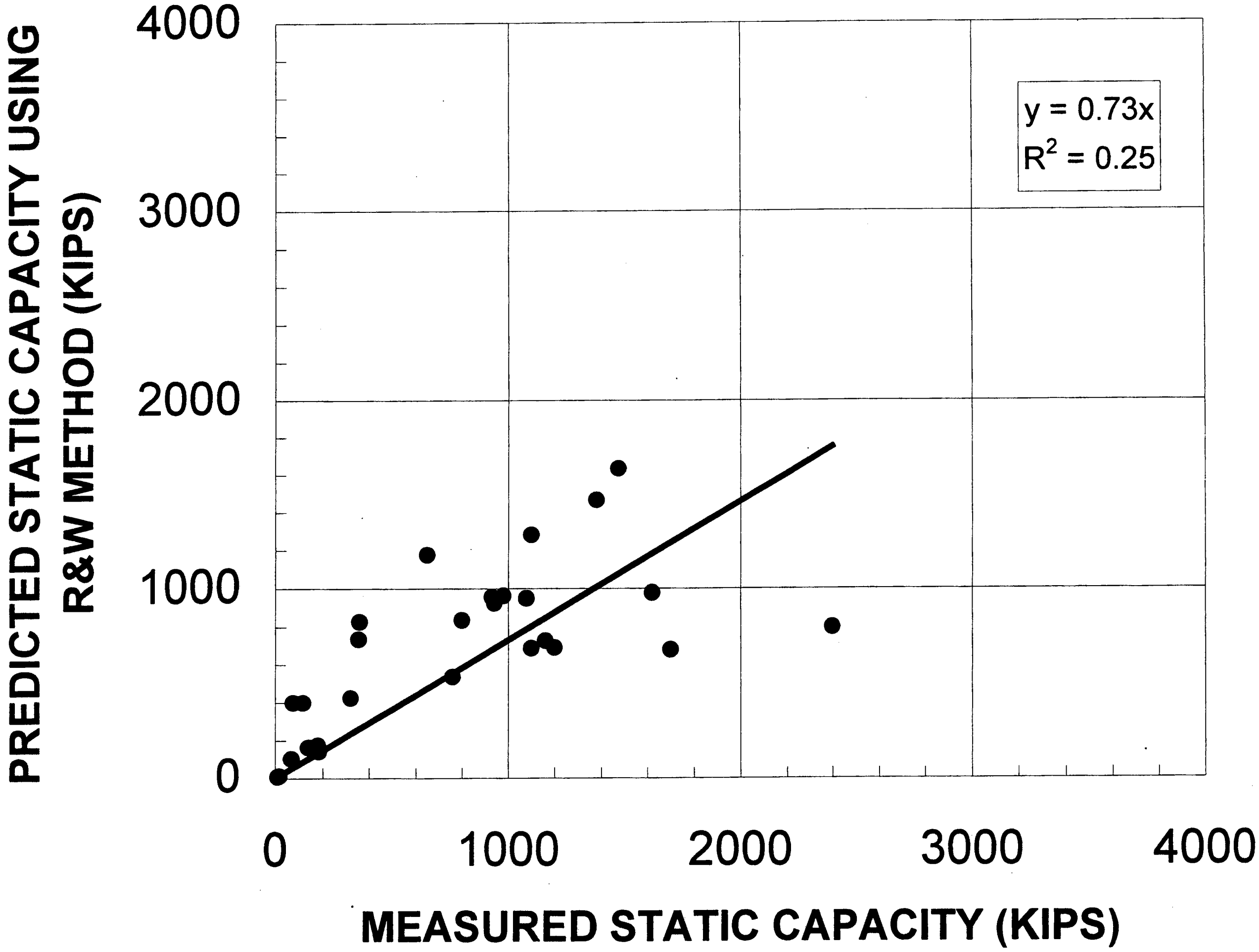

Figure 26

Table SHAFT #	Construction	SOIL	Failure Load(kips)	C&K Predict(kips)	C&K Ratio	IGM Predict(kips)	IGM Ratio
2	Casing	silt/clay/rock	2000	2236.00	0.89	2204.00	0.91
8	Casing	sand/clay/rock	1500	4332.00	0.35	3269.00	0.46
42	Casing	sand/rock	2000	2319.00	0.86	1771.00	1.13
44	Casing	sand/rock	2000	1150.00	1.74	1021.00	1.96
45	Casing	sand/rock	1980	1343.00	1.47	1184.00	1.67
46	Casing	silt/sand/rock	600	867.00	0.69	784.00	0.77
47	Dry	sand/rock	1816	2348.00	0.77	1497.00	1.21
48	Dry	sand/rock	1714	2709.00	0.63	1638.00	1.05
49	Dry	sand/rock	760	693.00	1.10	633.00	1.20
50	Dry	rock	644	537.00	1.20	728.00	0.88
51	Dry	rock	716	537.00	1.33	728.00	0.98
55	Dry	sand/rock	2800	1976.00	1.42	1384.00	2.02
81	Dry	rock	776	537.00	1.45	728.00	1.07
111	Dry	clay/rock	2060	547.00	3.77	562.00	3.67
112	Dry	sand/rock	1000	535.00	1.87	729.00	1.37
113	Dry	sand/rock	1800	1169.00	1.54	1281.00	1.41
115	Dry	silt/rock	700	395.00	1.77	416.00	1.68
116	Dry	silt/rock	750	324.00	2.31	336.00	2.23
117	Dry	silt/rock	450	236.00	1.91	245.00	1.84
118	Dry	silt/rock	320	159.00	2.01	166.00	1.93
119	Dry	silt/rock	350	250.00	1.40	270.00	1.30
120	Dry	silt/rock	460	236.00	1.95	245.00	1.88
122	Dry	silt/rock	700	236.00	2.97	252.00	2.78
136	Dry	rock	1080	1444.00	0.75	1037.00	1.04
137	Dry	rock	1854	2973.00	0.62	1787.00	1.04
138	Dry	rock	1438	1500.00	0.96	1066.00	1.35
139	Dry	rock	1832	1473.00	1.24	1052.00	1.74
140	Dry	rock	1686	1444.00	1.17	1037.00	1.63
146	Dry	sand/rock	2000	4521.00	0.44	3064.00	0.65
147	Dry	sand/rock	2000	4816.00	0.42	3258.00	0.61
157	Dry	clay/rock	2918	2003.00	1.46	1833.00	1.59
158	Dry	clay/rock	3144	2830.00	1.11	2477.00	1.27
159	Dry	clay/rock	3092	2830.00	1.09	2477.00	1.25
161	Dry	clay/rock	2688	5425.00	0.50	4258.00	0.63
162	Dry	clay/rock	1528	864.00	1.77	976.00	1.57

TOTAL SHAFT in ROCK

TOTAL	**49**	
	C&K	**IGM**
MEAN	**1.37**	**1.42**
STDV	**0.75**	**0.64**
VAR	**0.55**	**0.45**

TOTAL DRY METHOD	**32**	
	C&K	**IGM**
MEAN	**1.50**	**1.53**
STDV	**0.83**	**0.69**
VAR	**0.56**	**0.45**

169	Dry	clay/rock	1980	513.00	3.86	607.00	3.26
170	Dry	sand/rock	2406	1258.00	1.91	1583.00	1.52
171	Dry	clay/gravel	600	422.00	1.42	462.00	1.30
172	Dry & Casing	clay/gravel	2650	1764.00	1.50	2195.00	1.21
173	Dry & Casing	clay/gravel	2200	1327.00	1.66	1641.00	1.34
174	Slurry	clay/gravel	2440	1794.00	1.36	2233.00	1.09
179	Slurry & Casing	clay/gravel	1060	640.00	1.66	663.00	1.60
214	Slurry & Casing	sand/Shale	1500	816.00	1.84	646.00	2.32
257	Slurry & Casing	Sand/rock	1935.6	1974.00	0.98	1895.00	1.02
258	Slurry & Casing	sand/rock	2096	1590.00	1.32	1530.00	1.37
261	Slurry & Casing	sand/rock	2720	2636.00	1.03	2186.00	1.24
262	Slurry & Casing	sand/rock	1962	2624.00	0.75	2214.00	0.89
263	Slurry & Casing	sand/rock	4828	9545.00	0.51	5626.00	0.86
264	Slurry & Casing	sand/rock	3078	7805.00	0.39	4870.00	0.63

DRILLED SHAFT IN ROCK

CONSTRUCTION METHOD= MIXED
LOAD TRANSFER= SKIN FRICTION + END BEARING
N= 49
MEAN=1.37
STANARD DEVIATION = 0.75
COEFFICIENT OF VARIATION =0.55

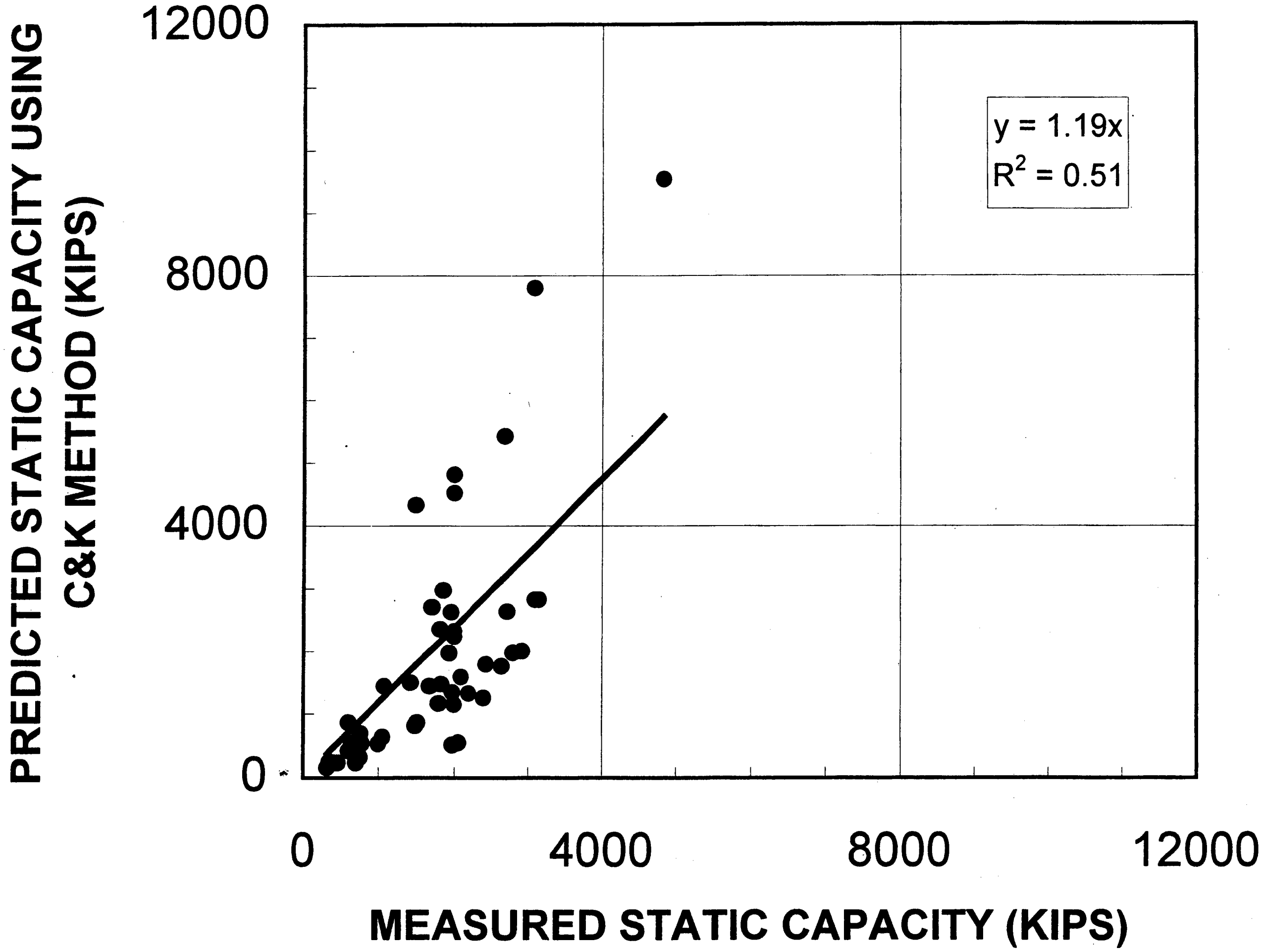

Figure 27

DRILLED SHAFT IN ROCK

CONSTRUCTION METHOD= MIXED
LOAD TRANSFER= SKIN FRICTION + END BEARING
N= 49
MEAN=1.42
STANARD DEVIATION = 0.64
COEFFICIENT OF VARIATION =0.46

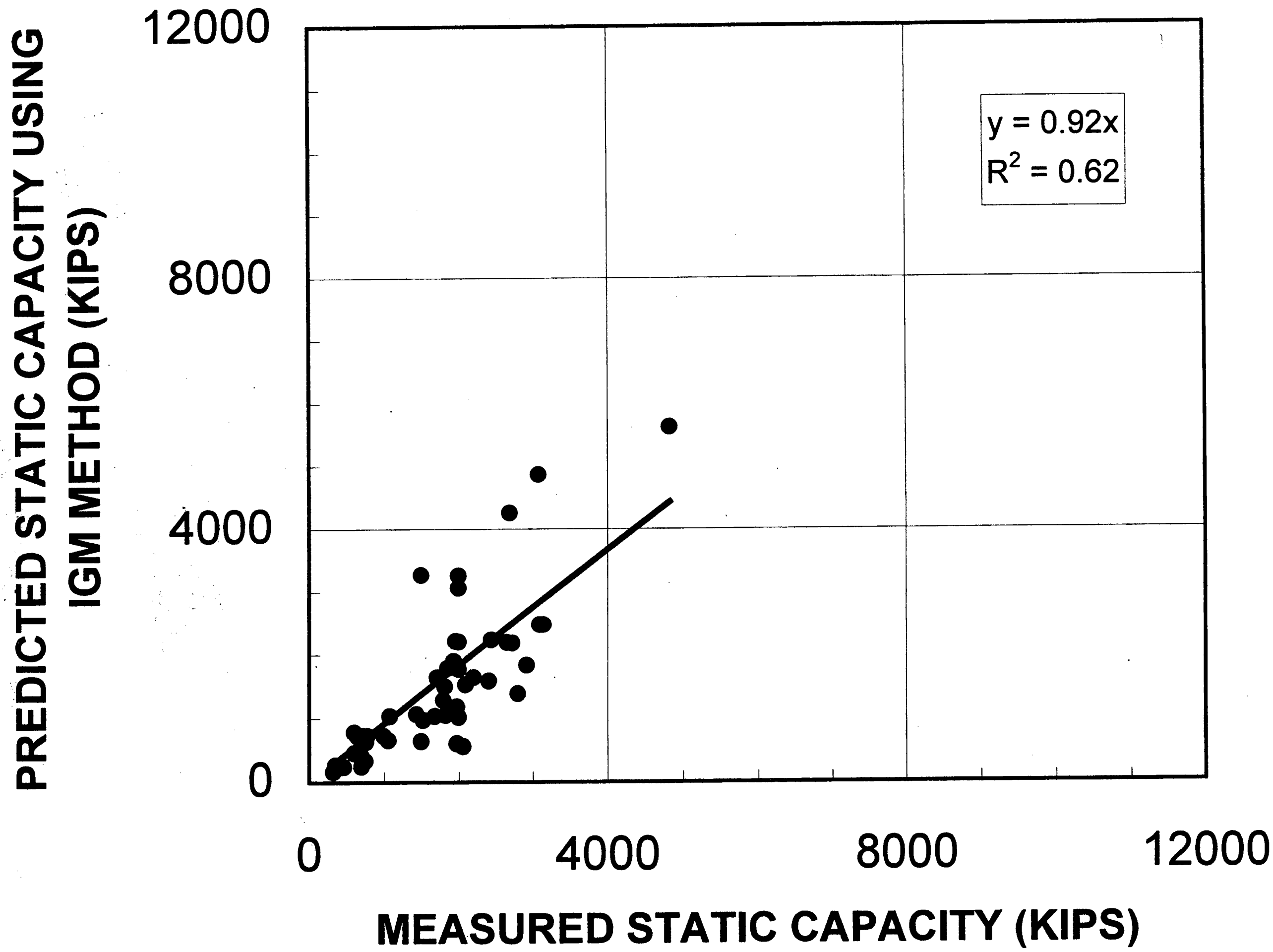

Figure 28

DRILLED SHAFT IN ROCK

CONSTRUCTION METHOD= DRY
LOAD TRANSFER= SKIN FRICTION + END BEARING
N= 32
MEAN=1.50
STANARD DEVIATION = 0.83
COEFFICIENT OF VARIATION =0.56

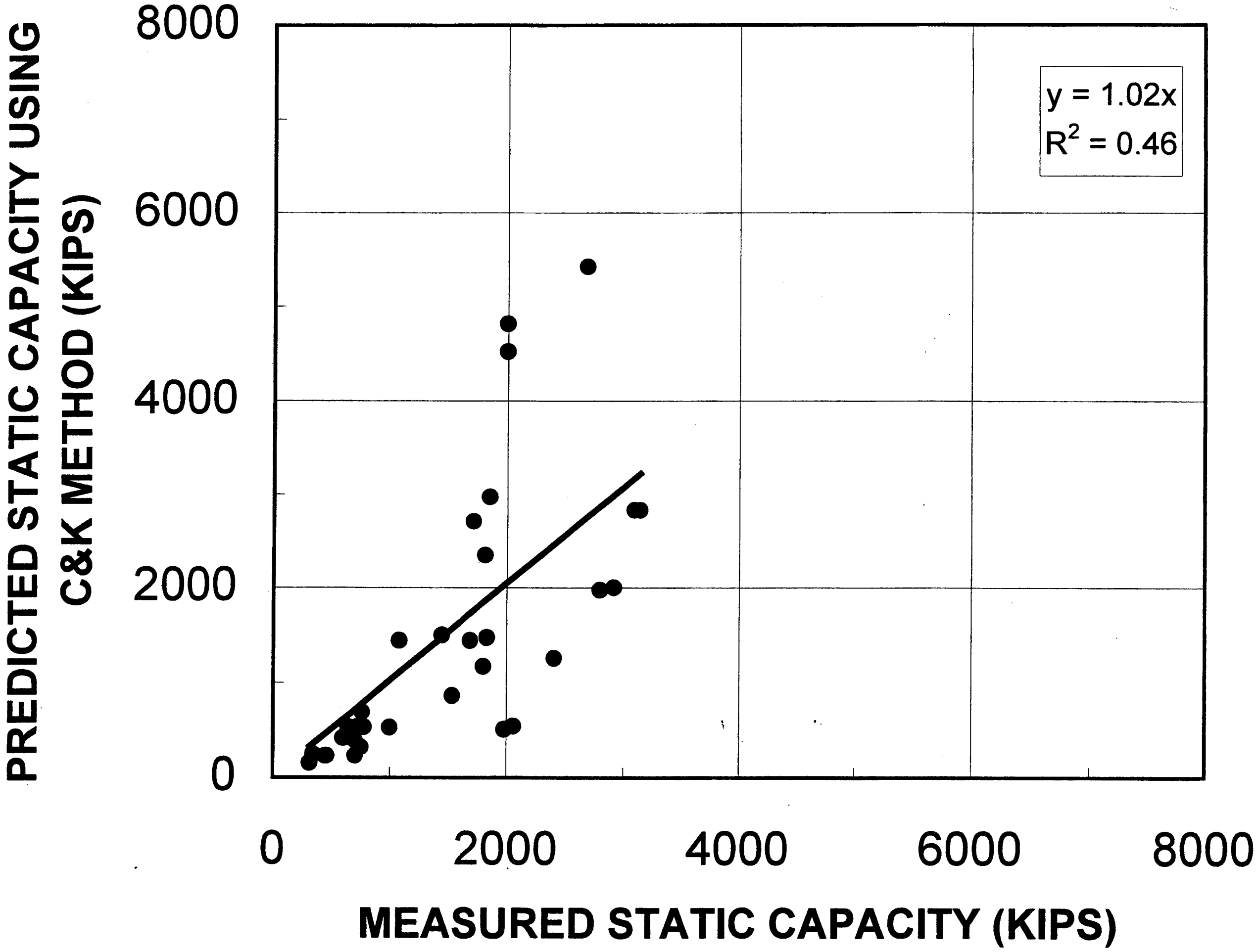

Figure 29

DRILLED SHAFT IN ROCK

CONSTRUCTION METHOD= DRY
LOAD TRANSFER= SKIN FRICTION + END BEARING
N= 32
MEAN=1.39
STANARD DEVIATION = 0.74
COEFFICIENT OF VARIATION =0.53

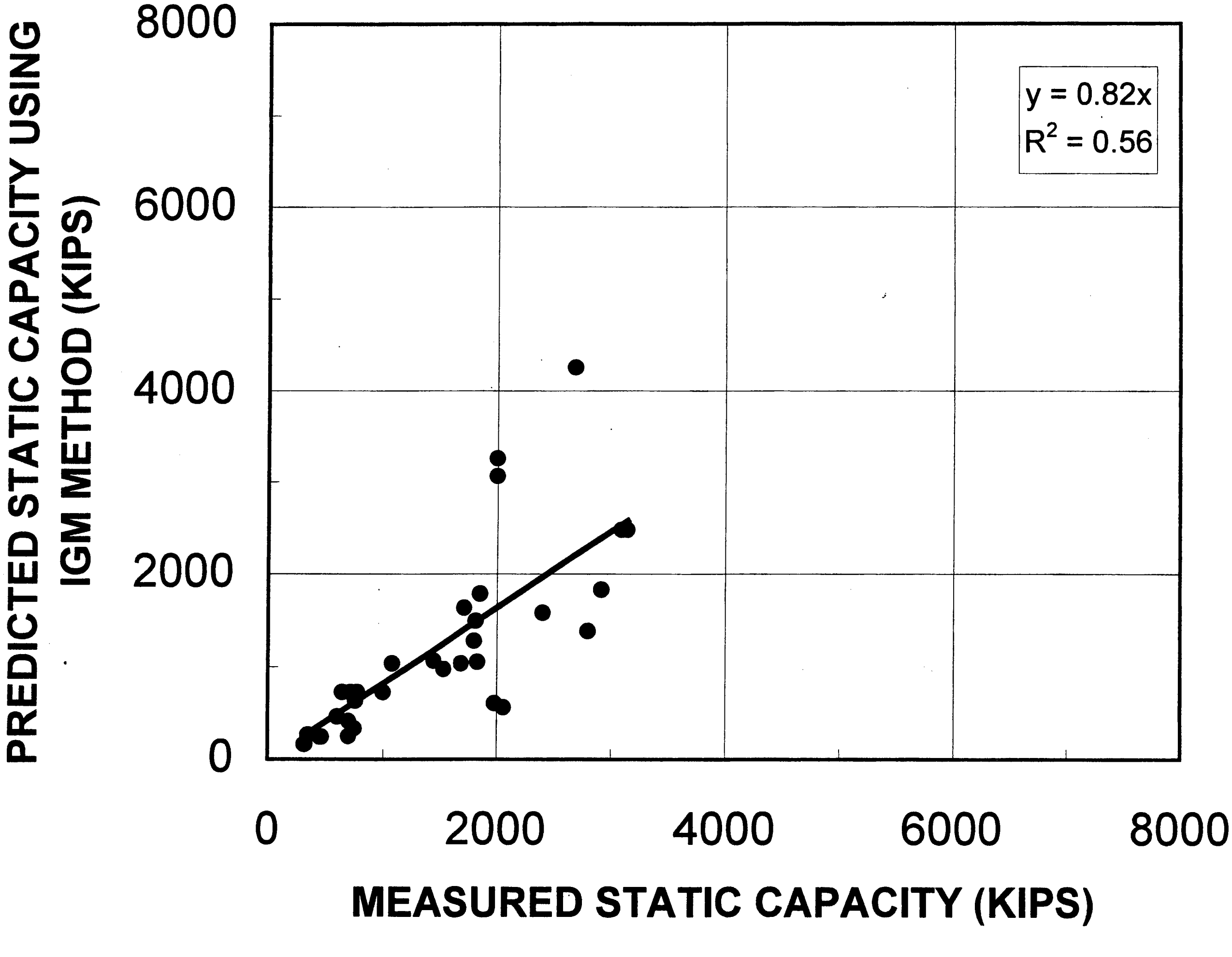

Figure 30

Table

SHAFT #	Construction	SOIL	Failure Load(kips)	C&K Predict(kips)	C&K Ratio	IGM Predict(kips)	IGM Ratio
8	Casing	sand/clay/rock	1500	4332.00	0.35	3269.00	0.46
47	Casing	sand/rock	1816	2348.00	0.77	1497.00	1.21
49	Casing	sand/rock	760	693.00	1.10	633.00	1.20
50	Casing	rock	644	537.00	1.20	728.00	0.88
55	Casing	sand/rock	2800	1976.00	1.42	1384.00	2.02
111	Dry	clay/rock	2060	547.00	3.77	562.00	3.67
112	Dry	sand/rock	1000	535.00	1.87	729.00	1.37
115	Dry	silt/rock	700	395.00	1.77	416.00	1.68
118	Dry	silt/rock	320	159.00	2.01	166.00	1.93
120	Dry	silt/rock	460	236.00	1.95	245.00	1.88
171	Dry	clay/gravel	600	422.00	1.42	462.00	1.30
257	Dry	Sand/rock	1935.6	1974.00	0.98	1895.00	1.02
258	Slurry & Casing	sand/rock	2096	1590.00	1.32	1530.00	1.37
261	Slurry & Casing	sand/rock	2720	2636.00	1.03	2186.00	1.24
262	Slurry & Casing	sand/rock	1962	2624.00	0.75	2214.00	0.89
263	Slurry & Casing	sand/rock	4828	9545.00	0.51	5626.00	0.86
264	Slurry & Casing	sand/rock	3078	7805.00	0.39	4870.00	0.63

SKIN SHAFT in ROCK

TOTAL	17	
	C&K	**IGM**
MEAN	**1.33**	**1.39**
STDV	**0.82**	**0.74**
VAR	**0.62**	**0.53**

DRILLED SHAFT IN ROCK

CONSTRUCTION METHOD= MIXED
LOAD TRANSFER= SKIN FRICTION
N= 17
MEAN=1.33
STANARD DEVIATION = 0.82
COEFFICIENT OF VARIATION =0.62

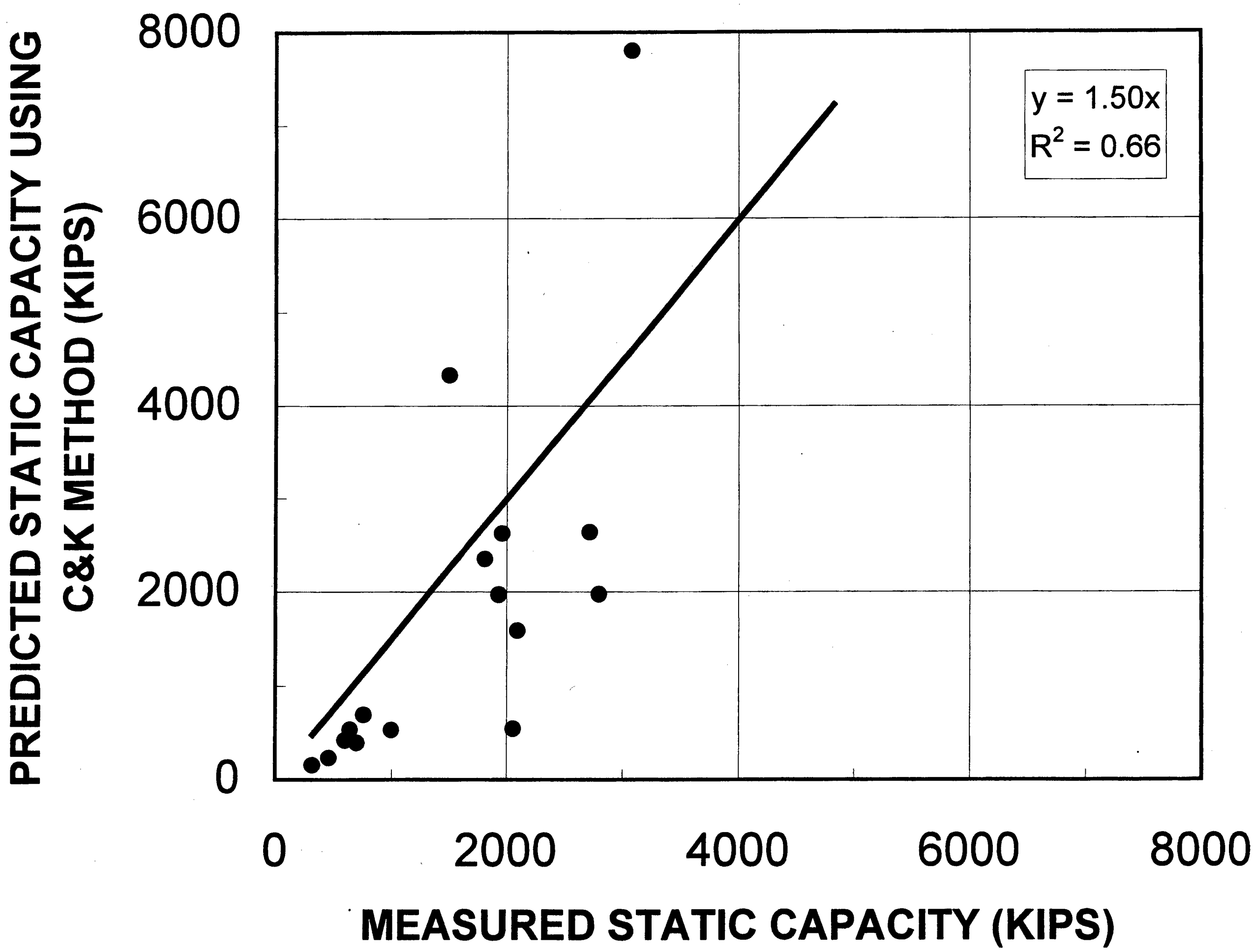

Figure 31

DRILLED SHAFT IN ROCK

CONSTRUCTION METHOD= MIXED
LOAD TRANSFER= SKIN FRICTION
N= 17
MEAN=1.39
STANARD DEVIATION = 0.74
COEFFICIENT OF VARIATION =0.53

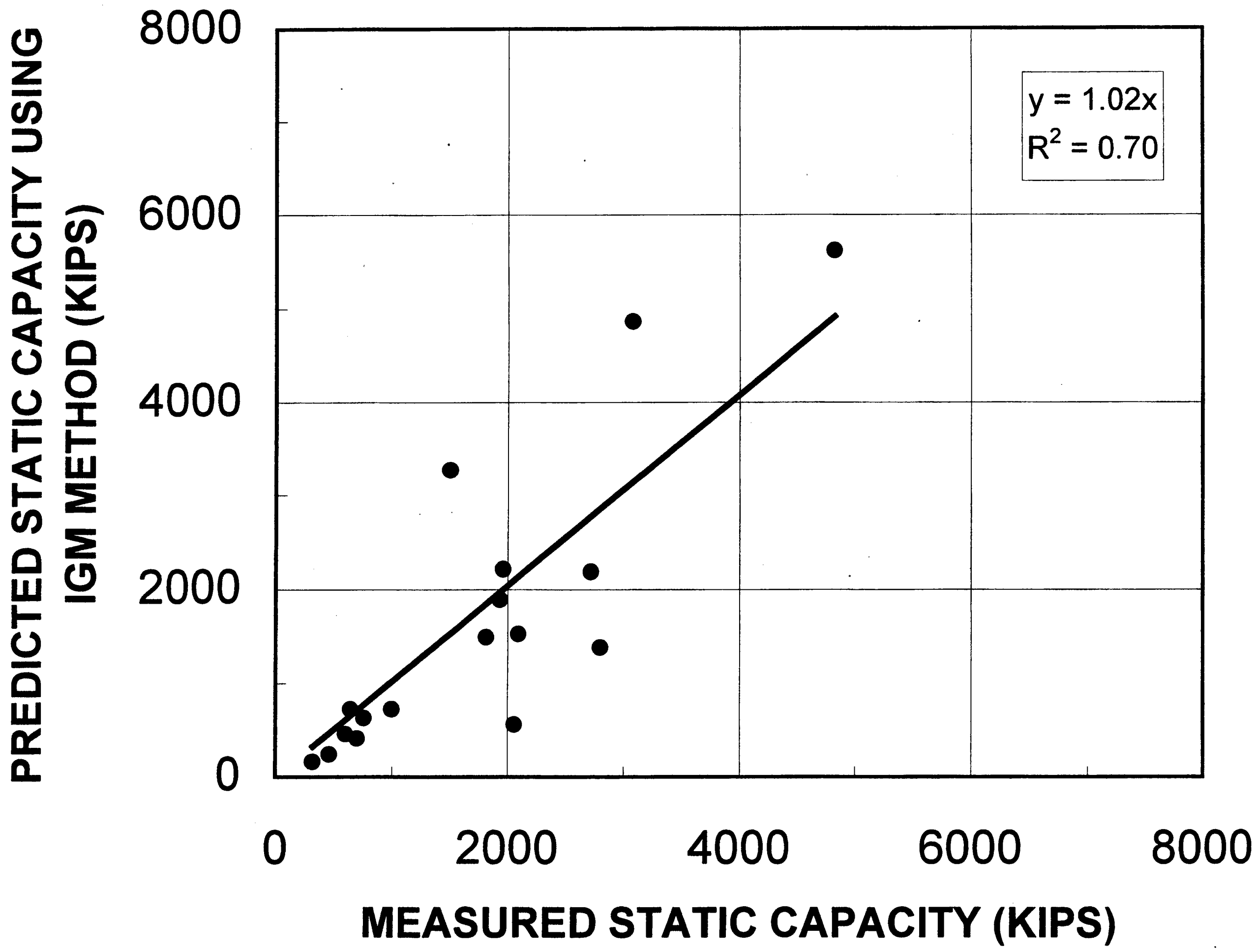

Figure 32

DRILLED SHAFTS - STATIC ANALYSIS

DESCRIPTION OF STATIC CAPCITY

EVALUATION METHODS

Federal Highway Administration's Design of Drilled Shafts

The Federal Highway Administration (FHWA) design of drilled shafts founded in sand, clay, and intermediate geomaterial (i.e. soft rock) follows O'Neill and Reese (1999) and O'Neil et. al (1996).

According to FHWA, the axial capacity of a drilled shaft may be calculated as:

$$Q_t = Q_s + Q_b \quad \text{(Eqn 1)}$$

where:

Q_t = failure shaft capacity
Q_s = skin friction capacity
Q_b = end bearing capacity

In Eqn. 1, shaft capacity or failure is defined as the applied load which will result in settlement of the top of the drilled shaft equal to five percent the diameter of the shaft. An explanation of the computation of Qs and Qb for each material (i.e. sand, clay, and intermediate geomaterials) is presented below with examples given in Appendix A.

Skin Friction, Qs, and End Bearing, Q_b, for Clay

Skin Transfer - The load transfer in side resistance for drilled shafts founded in clay is a variant of Tomlinson's Alpha (α) method (19). The undrained shear strength C_u of clay (found from laboratory tests or insitu correlations) is multiplied by alpha, α, to compute the unit skin friction (stress) at the depth z below the ground surface as follows,

$$f_{su} = \alpha \, C_u \qquad \text{(Eqn 2)}$$

where

f_{su} = unit skin friction (stress) at depth z
α = empirical factor that varies with depth, (see Table 1) and
C_u = undrained shear strength at depth z

Table 1 Recommended Values for α for Drilled Shafts in Clay

Location along Drilled Shaft	Value of α	Maximum Value of fsu (tsf)
From ground surface to depth of 5 ft. (1.52 m.)	0.0	0.0
From ground surface to length of casing	0.0	0.0
Bottom 1 diameter of shaft or 1 stem diameter above top of bell	0.0	0.0
All other points along drilled shaft sides	0.55	2.75 tsf (275 kPa)

Due to disturbances (i.e. drilling tool entering and exiting the hole frequently), the unit skin friction for the top five foot (see Fig. 1 and Table 1)) is neglected (i.e. set to zero). The setting of $\alpha = 0$ for a distance of one diameter above the base is from the work of Ellison et al. (1971). They showed that the downward movement of the base of the shaft can result in the development of a tensile crack in the soil near the base resulting in a lateral stress reduction. Consequently the unit skin friction in this zone (i.e. one diameter above the base) is set to zero.

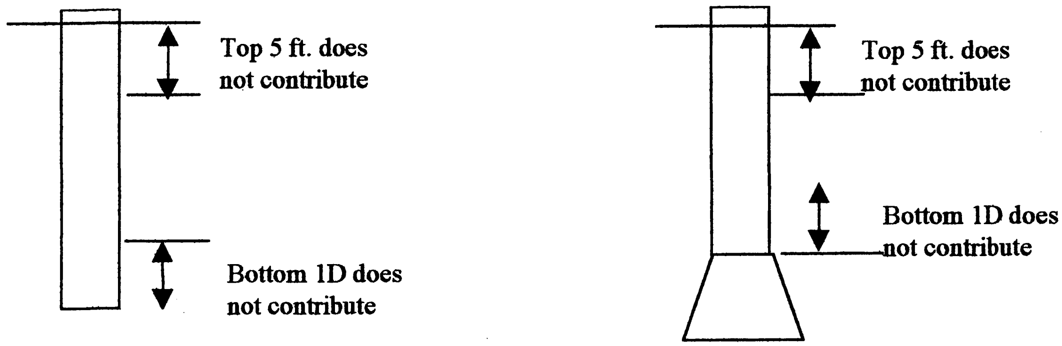

Figure 1 Portions of Drilled Shaft Non-Contributory in Friction

The total side resistance (i.e. force), Q_s for a given layer located at depths L_1 and L_2 below the ground surface is given as:

$$Q_s = \int_{L_1}^{L_2} f_{su} dA \quad \textbf{(Eqn 3)}$$

where dA = differential area of the perimeter along the side over a specific depth.

End Bearing – FHWA's unit end bearing (stress) for a drilled shaft founded in clay is based on the work of Skempton (1951):

$$\mathbf{q_b = N_c\, C_u,\quad q_b < 40\ tsf\ (4000\ kPa)\ (Eqn\ 4)}$$

where:

q_b = unit end bearing for drilled shafts in clay
$N_c = 6.0[1 + 0.2(L/B)]$ (bearing capacity factor) $N_c < 9$
C_u = average undrained shear strength of clay for 1.0 B below the tip
L = total embedment length of shaft in the ground
B = diameter of shaft at the base.

It should be noted that the limiting value of q_b (40 tsf) given in Eqn. 4 is the largest measured end bearing recorded for drilled shafts and not a theoretical limit (Engling and Reese, 1974)

In the case of drilled shaft diameters (at the base: B_b) exceeding 75 inches (1.9 m), the FHWA reduces the unit end bearing, q_b, to q_{br} to ensure tolerable settlements under service load conditions. Again, failure capacity of a shaft is the load, which develops settlements equal to five percent the diameter of the shaft. The reduced bearing resistance is defined as:

$$q_{br} = Fr\ q_b \qquad \textbf{(Eqn 5)}$$

where:

$F_r = 2.5/[aB_b \text{ (inches)} + 2.5\ b] \quad F < 1.0$

in which

$a = 0.0071 + 0.0021\ (L/B_b), \qquad a < 0.015$

$b = 0.45\ (C_u)^{0.5} \qquad 0.5 < b < 1.5$ and C_u in ksf

The latter expressions were based upon load tests of large under-reamed drilled shafts in very stiff clay (O'Neill and Sheikh, 1985). The reduced bearing resistance, q_{br}, gave similar results to the measured net bearing stress at a base settlement of 2.5 inches (6.35 cm). In addition, when more than half the design load is carried by end bearing, a global factor of safety greater than 2.5 is recommended by FHWA, unless site specific load tests are performed.

The failure end-bearing load, Q_b, is computed as q_b or q_{br} times the cross-sectional area of the drilled shaft's base. An example of capacity prediction for a drilled shaft founded in clay is given in Appendix A.

Skin Friction, Qs, and End Bearing, Q_b, for Sand

Skin Transfer - The unit side resistance on a drilled shaft founded in sand is based on Coulombic friction, i.e. equal to the normal (horizontal) effective stress times a coefficient of friction (tan ϕ_c).

$$f_{sz} = K \sigma_z \tan\phi_c \qquad \textbf{(Eqn 6)}$$

The total side resistance (i.e. force), Q_s for a given layer located at depths L below the ground surface is given as

$$Q_s = \int_0^L K \sigma_z \tan\phi_c \, dA \qquad \textbf{(Eqn 7)}$$

where

f_{sz} = ultimate unit side shear resistance in sand at depth z,
K = a parameter that combines the lateral pressure coefficient
σ_z = vertical effective stress at depth z
ϕ_c = interface friction angle for soil-concrete
L = depth of embedment for drilled shaft in sand
dA = differential area of perimeter along sides of drilled shaft

Generally, the normal stress at the interface of the drilled shaft and the soil is relatively low when the excavation is completed; however the fluid stress from the fresh concrete will impose a normal stress that is dependent on the characteristics of the concrete. Experiments have shown that concrete with moderate slump (up to 6 inches, 15 cm.) act hydrostatically over a depth of 10 to 15 ft. (3 to 4.5 m.) followed by a leveling off of lateral stress at greater depths, probably due to arching (Bernal and Reese, 1983). Concrete with higher slump (about 9 inches, 23 cm.) act hydrostatically to a depth of 32 ft. (10 m.). Thus, construction procedures and the concrete characteristics

will probably have a strong influence on the magnitude of the lateral stress at the soil-concrete interface.

As a result of the drilling and concreting influences, the K tan ϕ in Eqn. 6 is replaced by a simple constant, β, as a function of depth to account for variation in lateral stresses (i.e. K):

$$\beta = 1.5 - 0.135\sqrt{z} \qquad 1.2 > \beta > 0.25 \qquad \text{(Eqn 8)}$$

Consequently, the unit skin friction (stress) is given by

$$f_{sz} = \beta \sigma_z \qquad \text{(Eqn. 9)}$$

The total side resistance (i.e. force), Q_s for a given layer located at depths L below the ground surface is given as

$$Q_s = \int \beta \sigma_z dA \qquad \textbf{(Eqn 10)}$$

It should be noted that the limiting unit skin friction (Eqn. 9) is again not a theoretical limit, but rather is merely the largest value that has been measured (Owens and Reese, 1982). Higher values can be used if justified via a load test.

End Bearing - Generally, an experimental tip resistance curve for a drilled shaft in sand shows that the end bearing is still increasing at settlements equal to five percent the diameter of the shaft (i.e. FHWA defined shaft capacity). For instance, settlements of more than fifteen percent the diameter have been recorded. However, since such large settlement is not tolerated for most structures; FHWA limits end bearing and settlements to five percent of the shaft's base diameter.

The values of the unit end bearing (stress) q_b are tabulated as a function of N_{SPT} (uncorrected field values) in Table 3 for shaft diameters less than 50 inches. In the case of large diameter shafts [i.e. Shaft diameter, D> 50 in. (1.3m)], equation 11 is used:

$q_{br} = 50 * (q_b/B_b)$; B_b in inches

or $\mathbf{q_{br} = 1.3 * (q_b/B_b)}$**;** $\mathbf{B_b}$ **in meters (Eqn 11)**

Table 3 Recommended Unit End Bearing Values for Cohesionless Soils

N_{SPT} Values (Uncorrected)	Value of q_b (TSF) [kPa]
0 to 75	($0.60\ N_{SPT}$) [$60\ N_{SPT}$]
above 75	(45) [4500]

Table 3 limits the unit end bearing to 45 tsf (4500 kPa) at a settlement of 5 percent of the base diameter. Higher values, i.e. 58 tsf (5800 kPa) was measured for a settlement of 4 percent of the base diameter in Florida (Owens and Reese, 1982), are viable with load testing. An example of capacity prediction for a drilled shaft founded in sand is given in Appendix A.

Skin Friction, Qs, and End Bearing, $\mathbf{Q_b}$, for Intermediate Geomaterials (Soft Rock)

FHWA's determination of skin and tip resistance for a drilled shaft founded in soft rock is based on a recent publication of O'Neill and et. al (1996). The equations for unit skin friction and end bearing are presented separately.

Side Resistance – Requires a six step approach as identified:

1. Find the average E_m (mass modulus of rock) and f_{su} (ultimate unit skin friction) along the side of the rock socket:

$$E_m = \Sigma E_{mk} L_k / \Sigma L_k \tag{Eqn 12}$$

where $E_m = 115\ q_u$ and

$$f_{su} = \Sigma f_{su} L_k / \Sigma L_k \tag{Eqn 13}$$

where f_{su} = ultimate side friction.

The values selected for f_{su} depend whether the socket is considered "smooth" and failure occurs at the interface (α values) or "rough" where failure occurs through the rock. The rough assumption was used in this study and f_{su} was set equal to $0.5\sqrt{q_u}\sqrt{q_t}$.

2. Calculate Ω given by Eqn. 14:

$$\Omega = 1.14(\frac{L}{D})^{0.5} - 0.05[(\frac{L}{D})^{0.5}\ \log_{10}(\frac{E_c}{E_m}) - 0.44 \tag{Eqn. 14}$$

where L = socket length and modulus of concrete is given as $E_c(\Psi) = 57{,}000\sqrt{q_{uc}}$

3. Calculate Γ given by Eqn. 15

$$\Gamma = 0.37\sqrt{(\frac{L}{D})} - 0.15[\sqrt{(\frac{L}{D})} - 1]\log_{10}(\frac{E_c}{E_m}) + 0.13 \tag{Eqn. 15}$$

4. Find n (socket surface roughness)

For "rough" sockets;

$n = \sigma / q_u$ where σ = normal stress of concrete = $\gamma_c Z_c M$ (Eqn. 16)

where γ_c . 130 pcf or 20.5 kN/m^3

and M is given in Table 4 below based on concrete slump and socket depth

Table 4 Values of M

Socket Depth (m)	Slump (mm)		
	125	175	225
4	0.50	0.95	1.0
8	0.45	0.75	1.0
12	0.35	0.65	0.9

Also, if a water table is present, then $\sigma_n = \gamma_c(Z_c - Z_w) + \gamma_c Z_w$, where Z_c = depth to water table.

In the case of a "smooth" socket, n is estimated from Figure 2.

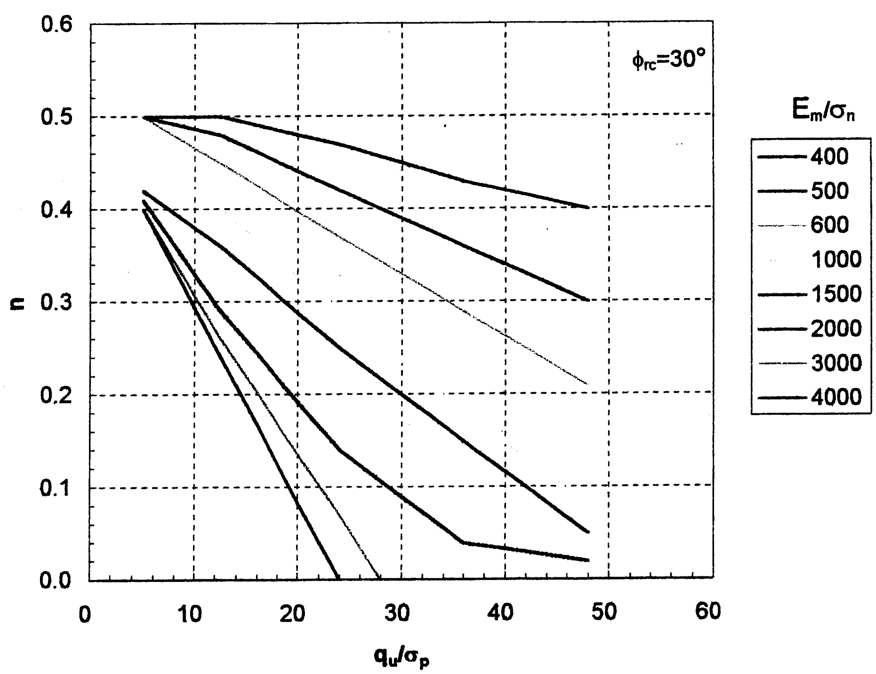

Figure 2 N Factors for Smooth Sockets

5. Next calculate Θ_f and K_f as given below:

$$\Theta_f = \frac{E_m \Omega}{\pi L \Gamma} W_t \qquad \text{(Eqn. 17)}$$

$$K_f = n + \frac{(\Theta_f - n)(1-n)}{\Theta_f - 2n + 1} < 1 \qquad \text{(Eqn. 18)}$$

where

W_t = deflection at top of rock socket

6. Finally, calculate the side shear load transfer vs. deformation from:

$$Q_s = \pi D L \Theta_f f_{su} \quad \Theta_f < n \qquad \text{(Eqn. 19)}$$

$$Q_f = \pi D L K_f f_{su} \quad \Theta > n \qquad \text{(Eqn. 20)}$$

End Bearing – The tip resistance as a function of displacement according to O'Neill et al. (1996) is found from :

$$Q_b = \frac{\pi D^2}{4} q_b \qquad \text{(Eqn. 21)}$$

where $q_b = \Lambda\, W_t^{0.67}$, and

$$\Lambda = 0.0134 E_m \frac{(L/D)}{(1+L/D)} \left\{ \frac{[200(L/D)^{0.5} - \Omega][1+(L/D)]}{\pi L \Gamma} \right\}^{0.67} \qquad \text{(Eqn. 22)}$$

The total shaft resistance, Qt, for a rock socket is the sum of $Q_s + Q_b$. An example of a drilled shaft design in soft rock is given in Appendix A.

Reese and Wright & Carter and Kulhway Design of Drilled Shafts

Similar to the FHWA design, Reese and Wright (1977) and Carter and Kulhway (1988) proposed that the general equation for computing the failure capacity of a drilled shaft be as follows.

$$Q_T = Q_B + Q_S = q_b \; A_b + q_s \; A_s \qquad \text{(Eqn. 23)}$$

Where Q_T is ultimate capacity
Q_B is ultimate tip resistance
Q_S is ultimate side resistance
q_b is unit end bearing
q_s is unit skin friction
A_b is base area
A_s is perimeter surface area

In Eqn. 23, shaft capacity or failure is defined as the applied load which will result in settlement of the top of the drilled shaft equal to five percent the diameter of the shaft. An explanation of the computation of Q_B and Q_S for each material (i.e. sand, clay, and intermediate geomaterials) is presented below

Reese and Wright

In 1977 Reese and Wright (1977) proposed a semi-empirical method to estimate the unit skin friction (q_s) and unit end bearing (q_b) for drilled shafts founded in sands using uncorrected SPT blow count, N. A discussion of unit skin friction and end bearing follows:

Unit skin friction, q_s, for sands:

For $N \leq 53$, $\quad q_s \text{ (MPa)} = 0.0028\ N$
For $53 < N \leq 100$, $\quad q_s \text{ (MPa)} = 0.00021\ (N - 53) + 0.15 \qquad$ (Eqn. 24)

In the case of multiple soil layers, i.e. sand with clay, the FHWA approach for clay, i.e. αCu was employed.

Unit end bearing q_b for sands:

For $N \leq 60$, q_b (MPa) = 0.064 N
For $N > 60$, q_b (MPa) = 3.8 (Eqn. 25)

Where N is the uncorrected SPT N value (blows/300 mm). The unit end bearing is based on a settlement equal five percent the diameter of the base of the drilled shaft.

Carter and Kulhawy

Carter and Kulhawy (1987) investigated drilled shafts socketed into weak rock and developed an expression for unit skin friction. In the case of end bearing, the method proposed in the Canadian Foundation Manual (1978) was used. A discussion of each follows.

Unit skin friction, q_s is based on average unconfined compressive strength of the rock,

$$q_s = 0.15\ q_u \qquad \text{(Eqn. 26)}$$

where q_u is uniaxial compressive strength of the rock or concrete, whichever is less.

Unit end bearing q_b - Although Carter and Kulhawy (1987) didn't propose an equation to estimate the unit end bearing, the data base indicated that significant amount of end bearing could be mobilized under relatively small deformation. Consequently, the unit end bearing was computed by the Canadian Foundation Manual (1978) approach:

$q_b = K_{sp} \text{ X } q_u$

$$K_{sp} = \frac{9 + \frac{3c_s}{B_b}}{10(1 + 300\,\delta/c_s)^{0.5}} \qquad \text{(Eqn. 27)}$$

where q_b is ultimate end bearing
K_{sp} is empirical coefficient
q_u is uniaxial compressive strength of the rock
c_s is spacing of discontinuities
δ is thickness of individual discontinuities
B_b is diameter of socket

A value of 1 was used for δ/c_s when no information on discontinuities was available for the site.

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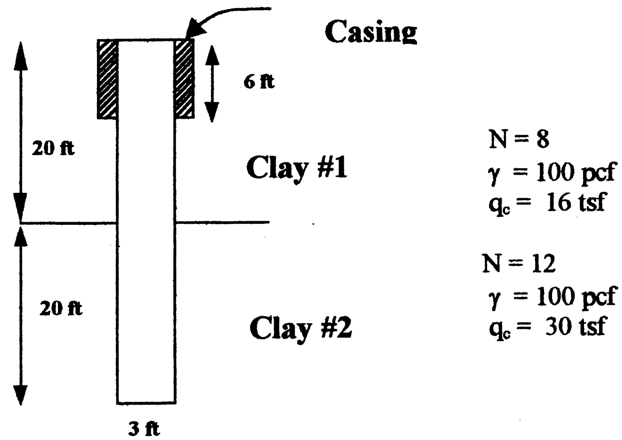
APPENDIX A - Examples

CLAYS:

Consider Two Cases:

1. Multi Layer Clay with Casing
2. Multi Layer Clay with Casing B > 75 "



$$c = \frac{q_c - \sigma_0}{15}$$

Clay Layer # 1 :c = $\dfrac{16*2000-10*100}{15}$ = $2{,}066.67\ psf\ \ (1.0333\ tsf)$

Clay Layer # 2 :c = $\dfrac{30*2000-30*100}{15}$ = $3{,}800\ psf\ \ (1.90\ tsf)$

1. ***Multi Layer Clay with Casing:*** Full Capacity (40 ft Shaft)

a) Skin Friction:

$Q_s = \pi * 3.0 * [(20' - 6')(0.55 * 1.033) + (20' - 3')(0.55 * 1.9)]$

$= 9.4248 * [7.9567 + 17.765]$

$= 242.42\ Tons$

b) End Bearing:

$Q_b = q_b \cdot \dfrac{\pi\ b^2}{4},$

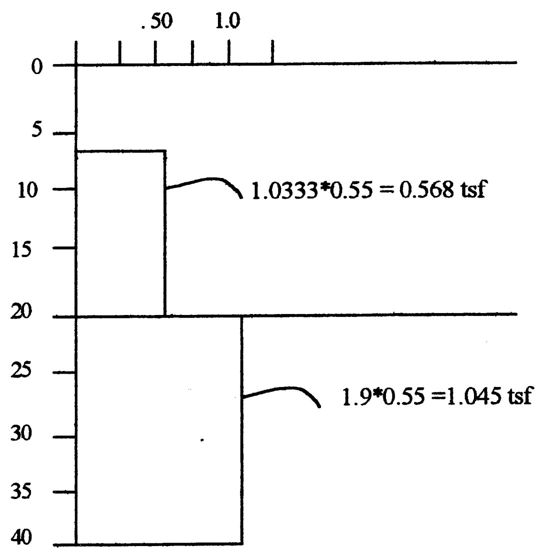
$$q_b = N_c C_u,$$

$$N_c = 6.0 * \left[1 + 0.2\frac{40}{3}\right] = 22 \;>\; 9 \;(use\; 9)$$

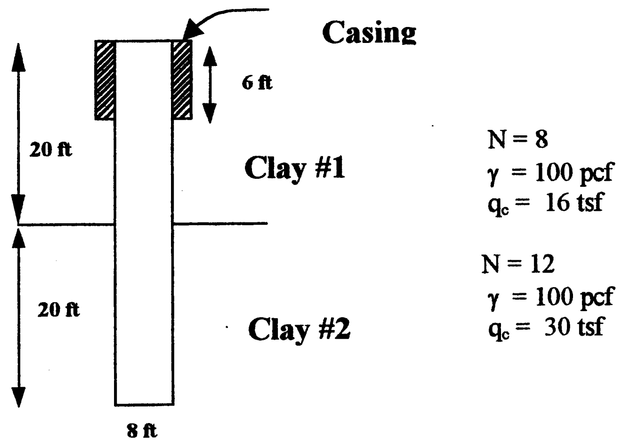
$$Q_b = (9 * 1.9 tsf).\frac{\pi \; 3^2}{4} = 120.87 \; Tons$$

c) Total Capacity = Skin Friction + End Bearing
= 242.42 + 120.87
= 363.29 Tons (ultimate)

d) Unit Skin Friction with depth:



2. Multi Layer Clay with Casing, but B>75" (1.9m):



a) Skin Friction: $Q_s = \pi * 8.0 * [(20' - 6')(0.55 * 1.033) + (20' - 8')(0.55 * 1.9)]$

$= 25.1327 * [7.9567 + 12.5]$

$= 515.14\ Tons$

b) End Bearing: If $B > 75$ ", then $q_{br} = F_r\, q_b$

$$F_r = \frac{2.5}{[a\ B_b(inches) + 2.5\ b]}$$

$a = 0.0071 + 0.0021(L / B_b)$
$= 0.0071 + 0.0021(40' / 8')$
$= 0.0176$, but $a < 0.015$

$b = 0.45\sqrt{C_u} = 0.45\sqrt{1.9 * 2.0}$, C_u in ksf

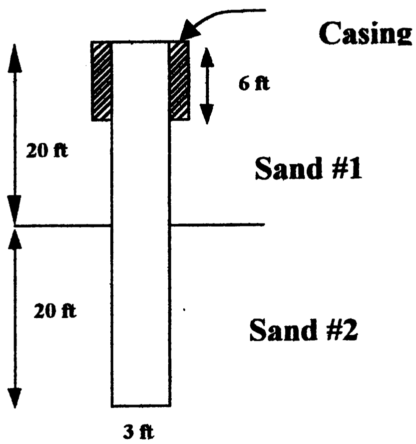
$= 0.8772$, $0.5 < b < 1.5$

$$F_r = \frac{2.5}{[0.015\ (96") + 2.5\ (0.8772)]} = 0.6881$$

$$Q_b = \frac{\pi * 8^2}{4}(0.6881)(9 * 1.9) = 591.48\ Tons$$

$Q_t = 515.14 + 591.48 = 1106.62\ Tons$

SANDS:



Depth	N	γ,pcf
0	0	100
10	10	100
20	10	100
25	15	100
60	15	100

1. Skin Friction:

$\beta = 1.5 - 0.135\sqrt{z}$ $\quad 0.25 < \beta(\text{tsf}) < 1.2$

or Z<4.94ft, β = 1.2 tsf, and Z>85.73ft, β = 0.25

$$\int_6^{40} \beta\sigma_v dz = \int_6^{40} 150Z - 13.5Z^{3/2}\, dZ = \frac{150Z^2}{2} - 13.5Z^{5/2} * \frac{2}{5} \Big|_6^{40}$$

$$= 65{,}355.84 - 2{,}223.82 = 18{,}116.37 * \frac{3\pi}{2000} = 297.50^T$$

2. End Bearing: above 8*B and below 3.5*B ,

above: 40.0 - 8*B = 40.0 - 8*(3)= 16' ;

below: 40.0 + 3.5*B = 40.0 + 3.5*(3) = 50.5'

for z = 16' q_b = 0.6*N = 0.6*(10) = 6 tsf

z = 20' q_b = 0.6*N = 0.6*(10) = 6 tsf

z = 25' q_b = 0.6*N = 0.6*(15) = 9 tsf

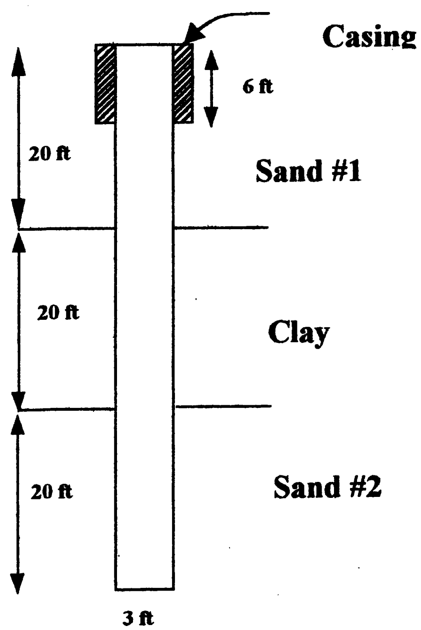
z = 60' q_b = 0.6*N = 0.6*(15) = 9 tsf

$$\therefore\ q_b = \left[\frac{\left[6*(20-16)+\dfrac{9+6}{2}*(25-20)+9*(50.5-25)\right]}{[50.5-16]}\right] = 8.4348$$

So, $$Q_b = 8.4348*\left[\frac{\pi.3^2}{4}\right] = 59.622^T$$

$$Q_T = 297.5 + 59.62 = 357.12$$

MULTILAYER- SAND-CLAY-SAND:



Depth	N	γ,pcf	C_u ,tsf
0	0	100	
10	10	100	
20	10	100	
20		100	1.90
40		100	1.90
40	15	100	
70	15	100	

1. Skin Friction (6-20ft) : $Q_s = \dfrac{3.\pi}{2000}\displaystyle\int_6^{20}(1.5-0.135\sqrt{z})\ \gamma\ z\ dz$

$$= 0.0047\left[\frac{150*z^2}{2} - 13.5*z^{5/2}*\frac{5}{2}\right]_6^{20}$$

$$=0.0047\left|75*(20^2-6^2)-5.4*(20^{5/2}-6^{5/2})\right|$$

$$= 0.0047[27,300\text{-}9,183.6]$$

$$= 85.371^T$$

2. Skin Friction (20-40ft) : $Q_s = 3.\pi[(40-20)(0.55*1.9)]$

$$= 196.978^{T}$$

3. Skin Friction (40-60ft) : $Q_s = \frac{3.\pi}{2000}\int_{40}^{60}(1.5-0.135\sqrt{z})\ \gamma\ z\ dz$

$$= 0.0047\left[\frac{150*z^2}{2} - 13.5*z^{5/2}*\frac{5}{2}\right]_{40}^{60}$$

$$= 0.0047\left(75*(60^2-40^2) - 5.4*(60^{5/2}-40^{5/2})\right)$$

$$= 0.0047[150{,}000 - 95{,}937.4]$$

$$= 254.764^{T}$$

$\Sigma\ Q_s = 85.371+196.978+254.764 = 537.11$ tons

4. Tip Resistance : above 8*B and below 3.5*B,

Above: 60.0 - 8*B = 60.0 - 8*(3)= 36 ft ;
Below: 60.0 + 3.5*B = 60.0 + 3.5*(3) = 70.5 ft

For z = 40 ft q_b = 0.6*N = 0.6*(15) = 9 tsf

z = 60 ft q_b = 0.6*N = 0.6*(15) = 9 tsf

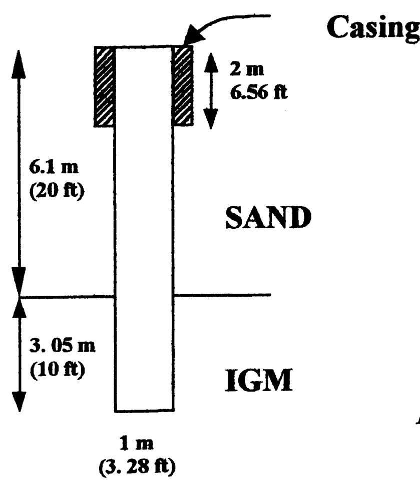
z = 75 ft q_b = 0.6*N = 0.6*(15) = 9 tsf

So, $Q_b = \left[\frac{\pi.3^2}{4}\right]*9 = 63.62^{T}$

Check q_b of overlaying Clay:

$q_b = 9*C_u = 9*1.9 = 17.1$ tsf stronger, $\therefore$ stop @ 40ft.

<u>IGM:</u> (Sand & Limestone)



γ = 100 pcf (15.708 kN/m^3)
N = 10

LimeStone:
q_u = 10 tsf (957.6 kPa, 0.96 Mpa)
q_t = 1 tsf (95.76 kPa, 0.096 Mpa)
γ = 135 pcf (21.2 kN/m^3), γ_c = 20.4 kN/m^3

$$E_c = 57{,}000\sqrt{f_y'} = 57{,}000\sqrt{5000psi}$$

$$= 4.03E6psi\ \ (27.77E6\ kPa)$$

Because of unit comparison problems, calculate Sand using English and Rock using SI units.

1. Skin Friction (Sand): $Q_s = \dfrac{3.28 * \pi}{2000} \displaystyle\int_{6.56}^{20} (1.5 - 0.135\sqrt{z})\ \gamma\ z\ dz$

$$= \frac{3.28 * \pi}{2000}\left[\frac{150 * z^2}{2} - 13.5 * z^{5/2} * \frac{5}{2}\right]_{6.56}^{20}$$

$$= \frac{3.28 * \pi}{2000}\left[75 * (20^2 - 6.56^2) - 5.4 * (20^{5/2} - 6.56^{5/2})\right]$$

$$= 0.00515[26{,}772.5\text{-}9064.6]$$
$$= 91.23^{T} = 91.23*2000/224.809 = \mathbf{811.66}\ \text{kN}$$

3. Analysis of Rock resistance is based on O'Neill (FHWA) intermediary geo-materials method, which is deformation based.

4. O'Neill IGM: (Note: Must enter values for E_c, slump, E_m/E_I, E_m, and IGM_Type = 2)

   a. $E_m = 115\ q_u = 115\ (0.96\ \text{MPa}) = 110.4\ \text{MPa}.$

   b. $\Omega = 1.14\left(\dfrac{L}{D}\right)^{1/2} - 0.05\left(\left\{\dfrac{L}{D}\right\}^{1/2} - 1\right)\log(\dfrac{E_c}{E_m}) - 0.44$

$$\Omega = 1.14(3.05)^{1/2} - 0.05(3.05^{1/2} - 1)\log(\frac{27,777}{110.4}) - 0.44 = 1.46$$

c. $$\Gamma = 0.37\left(\frac{L}{D}\right)^{1/2} - 0.15\left(\left\{\frac{L}{D}\right\}^{1/2} - 1\right)\log(\frac{E_c}{E_m}) + 0.13$$

$$\Gamma = 0.37(3.05)^{1/2} - 0.15(3.05^{1/2} - 1)\log(\frac{27,777}{110.4}) + 0.13 = 0.507$$

d. $$\frac{\theta}{w} = \frac{E_m \Omega}{\pi\ L \Gamma f_{su}}; \quad f_{su} = \frac{1}{2}\sqrt{q_u}\sqrt{q_t}$$

$$= \frac{110.4 * 1.46}{\pi * 3.05 * 0.507 * (\frac{1}{2}\sqrt{0.96}\sqrt{0.96})} = \frac{161.18}{0.7374} = 218.586 / m$$

e. $$\Lambda = 0.0134\ E_m \frac{(\frac{L}{D})}{(\frac{L}{D} + 1)} \left\{ \frac{200\ \left[\sqrt{\frac{L}{D}} - \Omega\right]\left[1 + \frac{L}{D}\right]}{\pi\ L\ \Gamma} \right\}^{0.67}$$

$$\Lambda = 0.0134\ \ (110.4 MPa)\frac{3.05}{4.05}\left\{ \frac{200\ \left[\sqrt{3.05} - 1.46\right]\left[1 + 3.05\right]}{\pi * 3.05 * 5.07} \right\}^{0.67}$$

$= 1.1141[4.7757]^{0.67}$

$= 3.159$ MPa $m^{-0.67}$

$$\Lambda = (1114.1\ \ kPa)\left\{ \frac{200\ \left[\sqrt{3.05} - 1.46\right]\left[1 + 3.05\right]}{\pi * 3050 * 0.507} \right\}^{0.67}$$

$= 1.1141[0.1316]$

$= 146.65$ kPa $mm^{-0.67}$

f. Determine n for deformation criteria Fig (2) $\frac{q_u}{\sigma_p} = \frac{957.6\ kPa}{100} = 9.576$

$$\frac{E_m}{\sigma_n}; \quad \sigma_n = M\,\gamma_c\,Z_c; \quad Since \quad Z_c = 6.1 + \frac{3.05}{2} = 7.625m \quad (use\ 8m)$$

$$For\ a\ slump = 175\ mm, \quad M(Fig3.5) = 0.78$$

$$\therefore \sigma_n = 0.78 * 20.4 * 7.625 = 121.33 \quad kPa$$

$$\therefore \frac{E_m}{\sigma_n} = \frac{110{,}400}{121.33} = 909.9 \quad \therefore n \approx 0.42$$

g. Select values of 'w' for calculating

$$Q_t = \pi\ D\ L\,\theta\ f_{su} + \frac{\pi D^2}{4} q_b \quad for \quad \theta < n\ ; \quad q_b = \Lambda\ w^{0.67}$$

$$Q_t = \pi\ D\ L\,k\ f_{su} + \frac{\pi D^2}{4} q_b \quad for \quad \theta > n$$

1) Let w = 2 mm; θ / w =218.586,

$$\therefore \theta = 218.586 * 0.002m = 0.437 < n = 0.45$$

$$Q_t = \pi * 1 * 3.05 * 0.437 * (151.4\ kPa) + \frac{\pi * 1^2}{4} * 146.65 * 2^{0.67}$$

$$= 634 + 182.8$$

$$= 816.7\ kPa$$

2) Let w = 5 mm; θ / w =218.586,

$$\therefore \theta = 218.586 * 0.005m = 1.093 > n = 0.45$$

$$k = n + \frac{(\theta - n)(1-n)}{(\theta - 2n + 1)} = 0.45 + \frac{(1.093 - 0.45)(1 - 0.45)}{(1.093 - 2(0.45) + 1)} = 0.7706$$

$$Q_t = \pi * 1 * 3.05 * 0.77 * (151.4\ kPa) + \frac{\pi * 1^2}{4} * 146.65 * 5^{0.67}$$

$$= 1118 + 336.8$$

$$= 1455\ kPa$$

h. Now go back and calculate sand capacity using trend lines when w = 2mm and 5mm.

1. S = (s*100/B);

@ 2mm $S=(0.2cm*100/100cm) = 0.2$

@ 5mm $S=(0.5cm*100/100cm) = 0.5$

2. $q_{st} / Q_s = -2.16*S^4 + 6.34*S^3 - 7.36*S^2 + 4.15*S$

$= -2.16*(0.2)^4 + 6.34*(0.2)^3 - 7.36*(0.2)^2 + 4.15*(0.2)$

$= 0.5829$ for w = 2mm

$q_s = 0.5829 * (811.66\ kN)$

$= 473.1\ kN$ for 2 mm

$q_{st} / Q_s = -2.16*S^4 + 6.34*S^3 - 7.36*S^2 + 4.15*S$

$= -2.16*(0.5)^4 + 6.34*(0.5)^3 - 7.36*(0.5)^2 + 4.15*(0.5)$

$= 0.892$ for w = 5mm

3. $q_s = 0.892 * (811.66\ kN)$

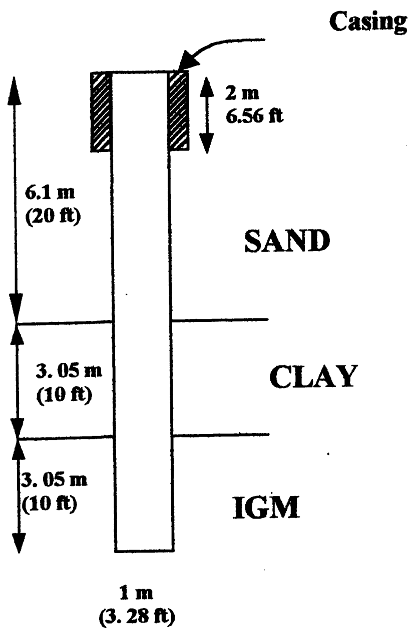
$= 724.4\ kN$ for 5 mm

i. Total Shaft Capacity (Sand + Rock)

1) @ 2mm $Q_T = 473.1\ kN + 634\ kN + 182.8\ kN = 1289.9\ kN$

2) @ 5mm $Q_T = 724.4\ kN + 1118\ kN + 336.8\ kN = 2179.2\ kN$

IGM: (Sand, Clay & Limestone)



γ = 100 pcf (15.708 kN/m^3)
N = 10

γ = 100 pcf (15.708 kN/m^3)
c = 1.9 tsf (181.94 kPa)

LimeStone:
q_u = 10 tsf (957.6 kPa, 0.96 Mpa)
q_t = 1 tsf (95.76 kPa, 0.096 Mpa)
γ = 135 pcf (21.2 kN/m^3), γ_e = 20.4 kN/m^3

$$E_c = 57{,}000\sqrt{f_y^{'}} = 57{,}000\sqrt{5000 psi}$$

$$= 4.03E6 psi \;\; (27.77E6 \;\; kPa)$$

$$f_{su} = \tfrac{1}{2}\sqrt{q_u}\sqrt{q_t} = 151.41 \;\; kPa$$

Smooth socket IGM_type = 2.0

1. Skin Friction (Sand): $Q_s = \dfrac{3.28 * \pi}{2000} \displaystyle\int_{6.56}^{20} (1.5 - 0.135\sqrt{z}) \; \gamma \; z \; dz$

$$= \frac{3.28 * \pi}{2000}\left[\frac{150 * z^2}{2} - 13.5 * z^{5/2} * \frac{5}{2}\right]_{6.56}^{20}$$

$$= \frac{3.28 * \pi}{2000}\left[75 * (20^2 - 6.56^2) - 5.4 * (20^{5/2} - 6.56^{5/2})\right]$$

= 0.00515[26,772.5-9064.6]

= 91.23^T = 91.23*2000/224.809 = 811.66 kN

2. Skin Friction (Clay): $Q_s = \pi$ D L α C_u = π (1) (3.05) (0.55*181.94)

= 958.85 kN (107.78^T)

3. FHWA IGM Calculations: (Note: Must enter values for E_c, slump, E_m/E_I, E_m, and IGM_Type = 2)

a. E_m = 115 q_u = 115 (957.6 kPa) = 110.4 MPa.

b. $\Omega = 1.14\left(\frac{L}{D}\right)^{1/2} - 0.05\left(\left\{\frac{L}{D}\right\}^{1/2} - 1\right)\log(\frac{E_c}{E_m}) - 0.44$

$$\Omega = 1.14(3.05)^{1/2} - 0.05(3.05^{1/2} - 1)\log(\frac{27{,}777}{110.4}) - 0.44 = 1.46$$

c. $\Gamma = 0.37\left(\frac{L}{D}\right)^{1/2} - 0.15\left(\left\{\frac{L}{D}\right\}^{1/2} - 1\right)\log(\frac{E_c}{E_m}) + 0.13$

$$\Gamma = 0.37(3.05)^{1/2} - 0.15(3.05^{1/2} - 1)\log(\frac{27{,}777}{110.4}) + 0.13 = 0.507$$

d. $\frac{\theta}{w} = \frac{E_m \Omega}{\pi\ L \Gamma f_{su}}; \quad f_{su} = \frac{1}{2}\sqrt{q_u}\sqrt{q_t}$

$$= \frac{110.4 * 1.46}{\pi * 3.05 * 0.507 * (\frac{1}{2} * 0.151 MPa)} = \frac{161.18}{0.7336} = 219.73 / m$$

e. $\Lambda = 0.0134\ E_m \frac{(\frac{L}{D})}{(\frac{L}{D} + 1)}\left\{\frac{200\ \left[\sqrt{\frac{L}{D}} - \Omega\right]\left[1 + \frac{L}{D}\right]}{\pi\ L\ \Gamma}\right\}^{0.67}$

$$\Lambda = 0.0134\ \ (110{,}112.5 kPa)\frac{3.05}{4.05}\left\{\frac{200\ \left[\sqrt{3.05} - 1.46\right]\left[1 + 3.05\right]}{\pi * 3050 * 0.507}\right\}^{0.67}$$

=146.27 kPa $mm^{-0.67}$

f. Determine n for deformation criteria Fig 36 $\frac{q_u}{\sigma_p} = \frac{957.6\ kPa}{100} = 9.576$

$\frac{E_m}{\sigma_n}; \quad \sigma_n = M\ \gamma_c\ Z_c; \quad Since \quad Z_c = 6.1 + 3.05 + \frac{3.05}{2} = 10.675 m$

$For\ a\ slump = 175\ mm, \quad M(Fig 3.5) = 0.68$

$\therefore \sigma_n = 0.68 * 20.4 * 10.675 = 148.1 \quad kPa$

$\therefore \frac{E_m}{\sigma_n} = \frac{110{,}112.5}{148.1} = 743.6 \quad \therefore n \approx 0.4 \quad < \quad n = 0.45$

g. Select values of 'w' for calculating

$$Q_t = \pi\ D\ L\,\theta\ f_{su} + \frac{\pi D^2}{4} q_b \quad for \quad \theta < n\ ; \quad q_b = \Lambda\ w^{0.67}$$

$$Q_t = \pi\ D\ L\ k\ f_{su} + \frac{\pi D^2}{4} q_b \quad for \quad \theta > n$$

1) Let w = 2 mm; θ / w =219.73 m^{-1},

$\therefore \theta = 219.73 * 0.002m = 0.439 < n = 0.45$

$$Q_t = \pi * 1 * 3.05 * 0.439 * (151.4\ kPa) + \frac{\pi * 1^2}{4} * 146.27 * 2^{0.67}$$

$= 636.85 + 182.8$

$= 819.2\ kPa$

2) Let w = 5 mm; θ / w =219.73 m^{-1},

$\therefore \theta = 219.73 * 0.005m = 1.099 > n = 0.45$

$$k = n + \frac{(\theta - n)(1-n)}{(\theta - 2n + 1)} = 0.45 + \frac{(1.099 - 0.45)(1-0.45)}{(1.099 - 2(0.45) + 1)} = 0.75$$

$$Q_t = \pi * 1 * 3.05 * 0.75 * (151.4\ kPa) + \frac{\pi * 1^2}{4} * 146.27 * 5^{0.67}$$

$= 1084.6 + 335.9$

$= 1420.5\ kPa$

h. Now go back and calculate sand capacity using trend lines when w = 2mm and 5mm.

1. S = (s*100/B);

   @ 2mm S=(0.2cm*100/100cm) = 0.2 , and

   @ 5mm S=(0.5cm*100/100cm) = 0.5

2. $q_{st} / Q_s = -2.16*S^4 + 6.34*S^3 - 7.36*S^2 + 4.15*S$

   $= -2.16*(0.2)^4 + 6.34*(0.2)^3 - 7.36*(0.2)^2 + 4.15*(0.2)$

   $= 0.5829$ for w = 2mm

3. q_s = 0.5829 * (811.66 kN)

$= 473.1$ kN for 2 mm

2. $q_{st} / Q_s = -2.16*S^4 + 6.34*S^3 - 7.36*S^2 + 4.15*S$

$= -2.16*(0.5)^4 + 6.34*(0.5)^3 - 7.36*(0.5)^2 + 4.15*(0.5)$

$= 0.892$ for w = 5mm

3. $q_s = 0.892 * (811.66 \text{ kN})$

$= 724.4$ kN for 5 mm

4. Clay: S = s*100/B; @ 2 mm S=0.2 & 0.5 @ 5 mm $0.12 < S < 0.74$

$$\frac{q_{st}}{Q_s} = \frac{S}{[0.095155 + 0.892937 * S]} = \frac{0.2}{0.2737} = 0.731$$

$$= \frac{0.5}{0.5416} = 0.9232$$

$q_s = 0.7310*958.85 = 700.55$ kN @ 2 mm

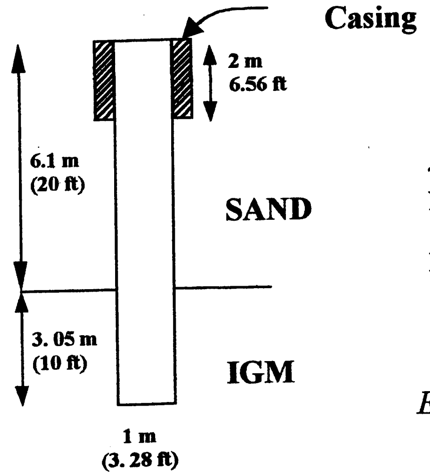
$q_s = 0.9232*958.85 = 885.16$ kN @ 5 mm

i. **Total Shaft Capacity (Sand + Rock)**

1) @ 2mm $Q_T = 473.1$ kN + 700.5 kN+ 636.85 kN + 182.4 = 1992.8 kN

2) @ 5mm $Q_T = 724.4$ kN + 885.16 kN+ 1084.6 kN + 335.9 kN = 3030.1 kN

IGM: (Sand & Limestone) Consider "Rough" Socket:



γ = 100 pcf (15.708 kN/m^3)
N = 10

LimeStone:
q_u = 10 tsf (957.6 kPa, 0.96 Mpa)
q_t = 1 tsf (95.76 kPa, 0.096 Mpa)
γ = 135 pcf (21.2 kN/m^3), γ_c = 20.4 kN/m^3

$$E_c = 57{,}000\sqrt{f_y'} = 57{,}000\sqrt{5000 psi}$$

$$= 4.03E6 psi \ (27.77E6 \ kPa)$$

$$f_{su} = \tfrac{1}{2}\sqrt{q_u}\sqrt{q_t} = 151.41 \ kPa$$

1. From Previous Example,

a) Skin Friction (Sand): $Q_s = \dfrac{3.28*\pi}{2000}\int_{6.56}^{20}(1.5 - 0.135\sqrt{z})\ \gamma\ z\ dz$

$$= \frac{3.28*\pi}{2000}\left[\frac{150*z^2}{2} - 13.5*z^{5/2}*\frac{5}{2}\right]_{6.56}^{20}$$

$$= \frac{3.28*\pi}{2000}\left[75*(20^2 - 6.56^2) - 5.4*(20^{5/2} - 6.56^{5/2})\right]$$

=0.00515[26,772.5-9064.6]

=91.23^T = 91.23*2000/224.809 = 811.66 kN

2. 2. O'Neill (FHWA) Rock - Rough Socket: (Note: Must enter values for E_c, slump, E_m/E_I, E_m, and IGM_Type = 1..0)

a) If "Rough" $n = \sigma_n / q_u$

$$\sigma_n = M\ \gamma_c\ Z_c; \quad Since \ \ Z_c = 6.1 + \frac{3.05}{2} = 7.625m \ \ (use \ 8m)$$

$$For \ a \ slump = 175 \ mm, \quad M(Fig3.5) = 0.78$$

$$\therefore \sigma_n = 0.78 * 20.4 * 7.625 = 121.33 \quad kPa$$

b) $n = \sigma_n / q_u = 121.33 / 95.76 = 0.13$

c)

$$Q_t = \pi\ D\ L\ \theta\ f_{su} + \frac{\pi D^2}{4} q_b \quad for \quad \theta < n\ ; \quad q_b = \Lambda\ w^{0.67}$$

$$Q_t = \pi\ D\ L\ k\ f_{su} + \frac{\pi D^2}{4} q_b \quad for \quad \theta > n$$

d) $\theta / w = 218.586\ m^{-1}$

e) Let w = 2 mm; $\therefore \theta = 218.586 * 0.002m = 0.437 > n = 0.13$

$$k = n + \frac{(\theta - n)(1-n)}{(\theta - 2n + 1)} = 0.13 + \frac{(0.437 - 0.13)(1 - 0.13)}{(0.437 - 2(0.13) + 1)} = 0.356$$

$$Q_t = \pi * 1 * 3.05 * 0.356 * (151.4\ kPa) + \frac{\pi * 1^2}{4} * 146.65 * 2^{0.67}$$

$$= 516.48 + 182.83$$

$$= 699.3\ kPa$$

f) Calculate sand capacity using trend lines when w = 2mm
   1. S = (s*100/B); @ 2mm S=(0.2cm*100/100cm) = 0.2

   2. $q_{st} / Q_s = -2.16*S^4 + 6.34*S^3 - 7.36*S^2 + 4.15*S$

   $= -2.16*(0.2)^4 + 6.34*(0.2)^3 - 7.36*(0.2)^2 + 4.15*(0.2)$

   = 0.5829 for w = 2mm

   3. q_s = 0.5829 * (811.66 kN)

   = 473.1 kN for 2 mm

g) Σ Q = 473.1 + 516.48 + 182.83 = 1172.4