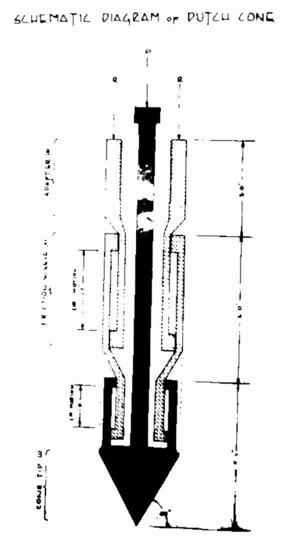
Report No. FHWA/RD-81/113

SENSING SYSTEMS FOR MEASURING MECHANICAL PROPERTIES IN GROUND MASSES

Vol. 5 Dutch Cone Penetrometer Tests- Case Histories

December 1981

Final Report



Prepared for

U.S. Department of Transportation
Federal Highway Administration
Offices of Research & Development
Structures and Applied Mechanics Division
Washington, D.C. 20590

Document is available to the public through
the National Technical Information Service,
Springfield, Virginia 22161

FOREWORD

This report presents research results that demonstrate the usefulness of the Dutch Cone Penetration Test (CPT) for soft ground geotechnical exploration in advance of tunnel construction.

The report indicates that the Dutch Cone penetrometer is capable of detecting thin subsurface material layers which could be missed by routine exploration but which could have very significant effects on the construction of soft ground tunnels. It is believed that the Cone penetration resistance is a measure of the cumulative effect of density and of in situ stress and that CPT is therefore appropriate for exploration of soft ground tunnel sites. The report provides brief descriptions of the use of the Dutch Cone penetrometer test on five projects that demonstrate the value and possible limitations of the method.

This report should serve the needs of geotechnical, structural, and civil engineers who are planning or designing an underground structure.

Copies of the report are being distributed to individual researchers and engineers by FHWA memorandum. Additional copies may be obtained from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

Charles F. Scheffey
Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

NOTICE

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA/RD-81/113		

4. Title and Subtitle	5. Report Date
Sensing Systems for Measuring Mechanical Properties in Ground Masses: Volume 5, Subtitle: Dutch Cone Penetrometer Tests - Case Histories	December 1981
	6. Performing Organization Code
7. Author(s)	8. Performing Organization Report No.
R. A. Waitkus, C. R. Burgin, R. E. Smith	77C731
9. Performing Organization Name and Address	10. Work Unit No. (TRAIS)
Woodward-Clyde Consultants 1300 Piccard Drive, Suite LL-5 Rockville, Maryland 20850	FCP 35B2572
	11. Contract or Grant No. FHWA 7-3-0158
	13. Type of Report and Period Covered Final Report
12. Sponsoring Agency Name and Address	
Office of Research Federal Highway Administration 400 7th Street, S. W. Washington, D. C. 20590	14. Sponsoring Agency Code

15. Supplementary Notes

FHWA Contract Manager: Dr. Don A. Linger, HRS-11

16. Abstract

This report provides case history and applied research information relative to the use of the Dutch Cone penetrometer in the practice of geotechnical engineering. Five case histories are cited that demonstrate the usefulness and possible limitations of the Dutch Cone. Comparisons and correlations are made between the cone and other field and/or laboratory tests. Additionally, the results of analysis involving Dutch Cone data are compared to other analysis methods and in some cases observed behavior.

This is the second volume of the series to be published. The other four volumes are: Volume 1, FHWA/RD-81/109, Subtitle: "Bore Hole Shear, Earth Settlement and Earth Penetrometer Probes;" Volume 2, FHWA/RD-81/110, Subtitle: "Pressuremeter;" Volume 3, FHWA/RD-81/111, Subtitle: "Vane Shear and Cone Piezometer;" Volume 4, FHWA/RD-81/112, "Static Penetrometers."

17. Key Words	18. Distribution Statement
Dutch Cone Penetrometer Test (CPT), Cone Bearing Capacity, Penetration Resistance, Skin Friction, Shear Strength, Overconsolidation Ratio, Settlement Profile	No restrictions. This document is available to the public through the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

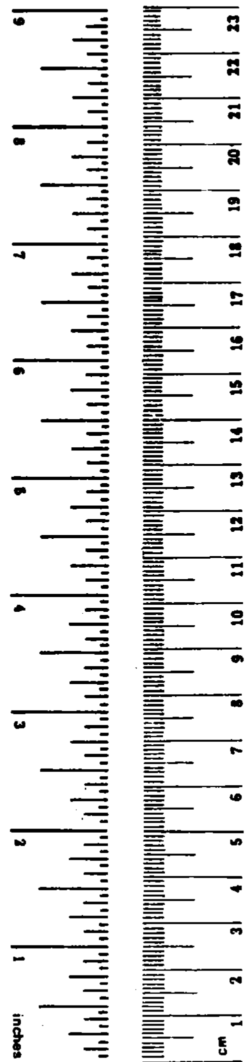
19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	45	

METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in ≡ 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F

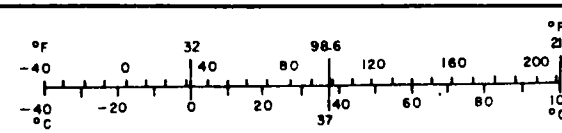


TABLE OF CONTENTS

		Page No.
CASE HISTORY 1:	Compressibility of Sands	1
CASE HISTORY 2:	Evaluation of Floodplain Deposits	11
CASE HISTORY 3:	Settlement Analysis of an Alluvial Profile	21
CASE HISTORY 4:	Densification of Sands	27
CASE HISTORY 5:	Estimating Undrained Shear Strength From Cone Data	34

TABLES

Table 1:	Comparison of Estimated Settlement By CPT and SPT Data with Actual Screw Plate Load Test Results	5
Table 2:	Comparison of qc/N Ratios	17
Table 3:	Shear Strength Estimates From CPT Data	37

FIGURES

Figure 1:	Typical Subsurface Profile (Case History 1)	6
Figure 2:	Correlation of Cone Bearing to Standard Penetration Resistance	7
Figure 3:	Composite CPT Profile (Case History 1)	8
Figure 4:	Vibratory Compaction Results	9
Figure 5:	Screw Plate Load Test Results	10
Figure 6:	Comparison of Profiles Based on Calibration Borings and Adjacent Cone Soundings	15
Figure 7:	Typical Subsurface Profile (Case History 2)	16

FIGURES (Continued)

Figure 8:	Cone Calibration Chart (Case History 2)	18
Figure 9A:	Shear Strength Comparisons Type 1	19
Figure 9B:	Shear Strength Comparisons Type 2	20
Figure 10:	Typical Subsurface Profile (Case History 3)	24
Figure 11:	Composite CPT Profile (Case History 3)	25
Figure 12:	Comparison of Calculated and Observed Settlement versus Fill Height	26
Figure 13:	Typical Subsurface Profile (Case History 4)	30
Figure 14:	Mechanical Analysis	31
Figure 15:	Composite CPT Profile Before Densification	32
Figure 16:	Composite CPT Profile After Densification	32
Figure 17:	SPT and CPT Correlation-Before and After Soil Densification	33
Figure 18:	Typical Subsurface Profile (Case History 5)	38
Figure 19:	Cone Calibration Chart (Case History 5)	39

LIST OF SYMBOLS

CPT	Cone Penetration Test
SPT	Standard Penetration Test
N	SPT blow counts
qc	Cone bearing capacity
El.	Elevation
MSL	Mean Sea Level
ASTM	American Society for Testing and Materials
Es	Deformation Modulus or Soil Compressibility Modulu
S	Settlement of the screw plate
Δp	Net pressure increase
C_1	Correction factor for embedment
C_2	Correction factor for creep
I	Strain influence factor
Z	Soil thickness
σ'_o, σ'_{vo}	Effective overburden
σ_{vm}	Maximum Past Pressure
Dw	Depth of Groundwater
Cn	Correction factor for effective overburden
Cw	Correction factor for groundwater level
D	Depth of the base of the plate
B	Width of the plate
q	Uniform pressure on soil
TSF	Tons per square foot
Su	Undrained shear strength value
Nk	Site specific correlation value or cone factor
NSP	Normalized soil parameters
OCR	Overconsolidation ratio
K	Coefficient relating Su and OCR in (Eq. 2-1)
n	Exponent relating Su and OCR in (Eq. 2-1)
Nc	Bearing capacity factor
OD	Outside diameter
ID	Inside diameter
σ_z	Total overburden pressure
Fr	Friction ratio
Fs	Sleeve friction
CIU	Consolidated-Isotropic-Undrained
Rs	Ratio Settlement Calculated to Settlement Observed

CASE HISTORY 1: COMPRESSIBILITY OF SANDS

The Dutch Cone was one of several methods of in-situ testing performed at a site near the shore of Lake Michigan. The Cone Penetration Test (CPT) was utilized to delineate subsurface stratigraphy and to aid in determining the effectiveness of a soil compaction program. Attempts were made to correlate Screw Plate Load Test results with estimated plate settlement using CPT and Standard Penetration Test (SPT) data.

The site is located inland from the sand dunes fronting Lake Michigan. It is situated within a broad low-lying sandy plain which probably represents the bed of an extinct glacial lake. The existing sediments have been deposited by varied means; shallow marine waters, glacial ice melt water, streams, lake and wind.

Fifty (50) Standard Penetration Test (SPT) borings and 13 CPT soundings were made at the site. The average depth of the borings was 75 feet (23 m) with several extending beyond 150 feet (46 m). The subsurface investigation showed a stratum of loose to medium dense, fine to medium dune sands (SW) underlain by 50 feet (15 m) of dense to very dense, fine to coarse sands (SP-SM). Beneath these sands, sequences of glacial deposits (silty clays and sands) extended to bedrock, approximately 140 feet (43 m) below ground surface. Groundwater was usually encountered seven to 12 feet (2 to 3.7 m) below the existing ground surface. Figure 1 presents a generalized subsurface profile of the site.

CPT soundings were used initially to delineate possible zones of loose, compressible sand existing within the main building area. On-site calibrations with SPT blow counts, N, were made for the upper soil strata at three different locations, and results generally showed that the CPT bearing capacity, qc, was typically between five and six times the SPT values. See Figure 2.

CPT soundings in conjunction with SPT test borings clearly defined the contact between the loose to medium dense dune sands and the underlying dense sands. Figure 3 shows a composite profile developed from the CPT soundings. The distinct break at approximately El. 587 defines the contact between the dune sand and the denser underlying sands. Laboratory testing indicated that the surficial sands, in their natural state, would be more compressible under some of the proposed structures than was tolerable. Therefore a program of vibratory compaction was undertaken to densify the surface soils and reduce the overall compressibility. CPT soundings were employed to determine the influence of vibratory compaction on the upper stratum.

Profiles developed from CPT data showed that after four passes with a vibratory roller, mixed results were achieved. Two CPT soundings revealed successful compaction to a depth of 4.0 feet (1.2 m) and one sounding showed that compaction improved soil density to a depth of 8.9 feet (2.7 m). However, directly below these depths, cone resistance, qc, decreased. Figure 4 shows a before and after compaction profile developed from CPT.

For vibratory compaction it was noticed that the elapsed time between compaction and the cone sounding had a direct bearing on cone resistance values. Cone resistance values showed a temporary decrease if the soundings were performed up to two hours following compaction. This phenomenon is believed to be caused by excess pore water pressure effects generated by the vibratory compaction. With time, the pore pressures dissipated and penetration resistance was slightly greater. As a result a depth of eight feet (2.5 m) was considered to be the extent of measurable densification.

In uncompacted areas, Screw Plate Load Tests were made to determine compressibility of the dune sands when subjected to load. Comparisons were made of the actual screw plate deflections with the plate settlements estimated from static cone bearing capacity, qc, and SPT blow count, N-value. The Screw Plate Load Test was a major investigative tool in the development of the Schmertmann (1970) procedure for estimating settlements of sands. The two screw plates used in these tests were obtained from the University of Florida. The procedures used in making the Screw Plate Tests were developed by Schmertmann and reported in ASTM Special Technical Publication 479. The load deflection curves for the four screw plate tests made are reported on Figure 5.

Using the CPT cone bearing values within the vicinity of each Screw Plate Load Test, soil compressibility modulus values, Es, were determined by the empirical procedure (Schmertmann, 1970) as given by the following equation:

$$Es = 2qc \qquad \text{(Eq. 1-1)}$$

Settlement of the screw plate, S, under a net pressure increase, Δp, at the base of the plate was estimated using the Schmertmann procedure set forth in the following equation:

$$S - C_1 C_2 \ \Delta p \sum_{o}^{2B} \frac{Iz}{Es} \ Z \qquad \text{(Eq. 1-2)}$$

in which:

C_1C_2 = Correction factor for embedment and creep
I_z = Strain influence factor
Z = Soil thickness affected by the load increase, Δp

Likewise, using the SPT blow counts, N, from nearby test borings, settlement of the screw plate was estimated by the following procedure. The N values were first corrected for effective overburden, σ_o', and groundwater, Dw, conditions (Peck, Hanson & Thornburn, 1974) as given by the following equation:

$$N' = N\ Cn\ Cw \qquad \text{(Eq. 1-3)}$$

in which:

$$Cn = 0.77 \log_{10} \frac{20}{\sigma o} \qquad \text{for } \sigma o' \geq 0.25 \text{ tsf}$$

$$Cw = 0.5 + 0.5 \frac{Dw}{D+B} \qquad \text{for } Dw < D+B$$

$$Cw = 1.0 \qquad \text{for } Dw \geq D+B$$

D and B are the depth of the base of the plate (generally the footing) and the width of the plate, respectively. Settlement of the screw plate was estimated by the following equation modified from the Terzaghi and Peck settlement chart (1948).

$$S = \frac{2q}{N'}\left(\frac{2B}{1+B}\right)^2 \qquad \text{(Eq. 1-4)}$$

in which:

q = uniform pressure on soil, in tsf

B = plate width (generally footing)

The results of the comparisons are given on Table 1. The comparative data is highly irregular. The CPT estimates are typically one-third to one-half those from SPT data. When both methods are compared to actual screw plate settlements, the screw plate test at Location 109 is very high and appears to be anomalous. For the other screw plate tests the SPT estimates were conservative, yet closest to the actual settlements in all but one case. The CPT estimates were consistent, however underestimated settlements in three of the four cases. These comparisons are not definitive in correlating the three techniques of evaluating compressibility of sand.

Table 1. Comparison of Estimated Settlements by CPT and SPT Data With Actual Screw Plate Load Test Results.

Nearby Boring/Cone Location No.	Depth(m)	Screw Plate Load (kg/cm^2)	Actual Settlement (cm)	Estimated Settlement by SPT	Estimated Settlement by CPT
109	1.83	1.75	2.54	0.94	0.45
112	2.44	3.0	0.93	1.12	0.31
112	3.66	3.0	0.97	1.32	0.36
116	2.13	1.5	0.38	0.81	0.36

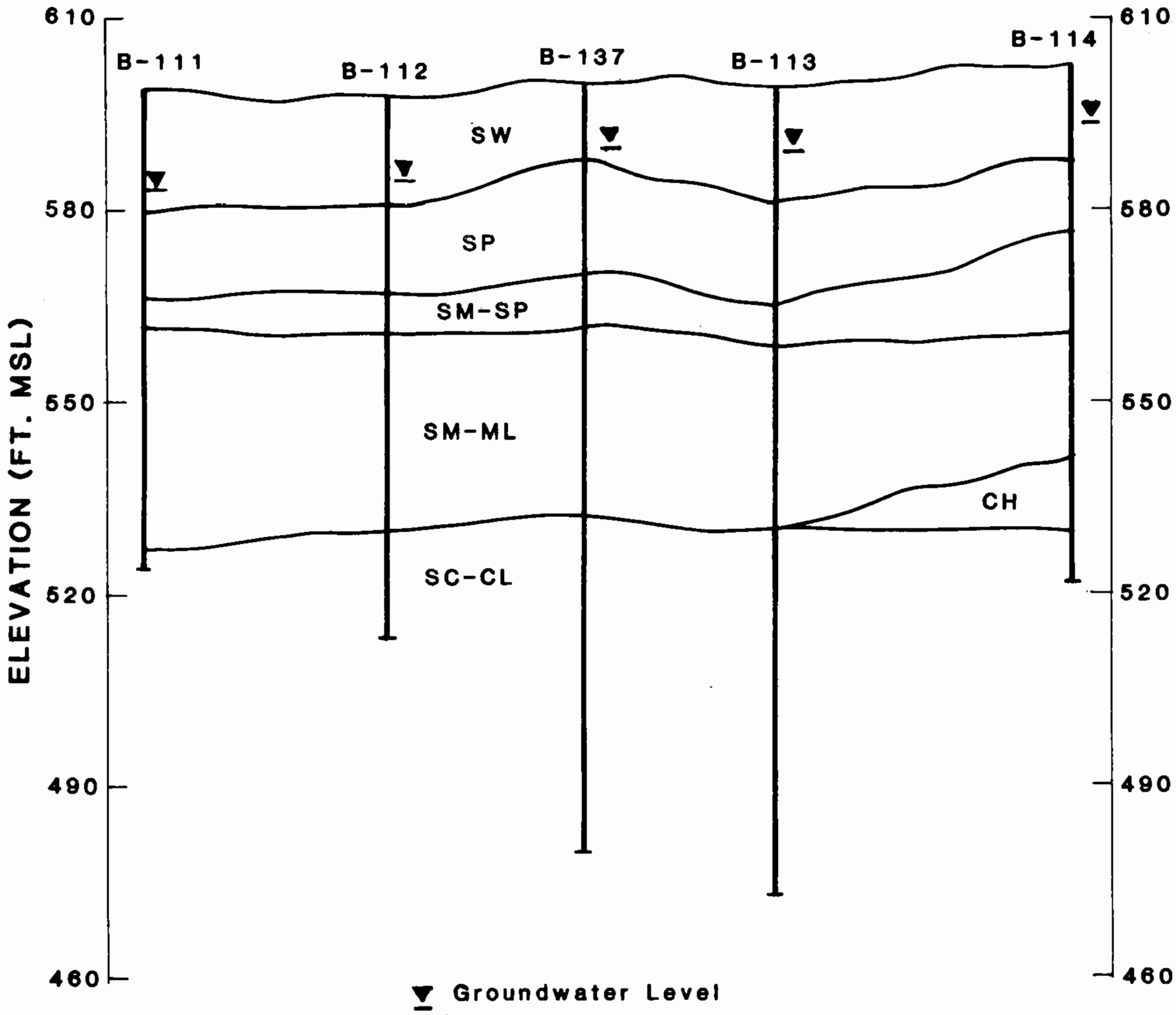


Figure 1. Typical Subsurface Profile

HORIZONTAL SCALE 1''=200'
(1 ft. = 0.3m)

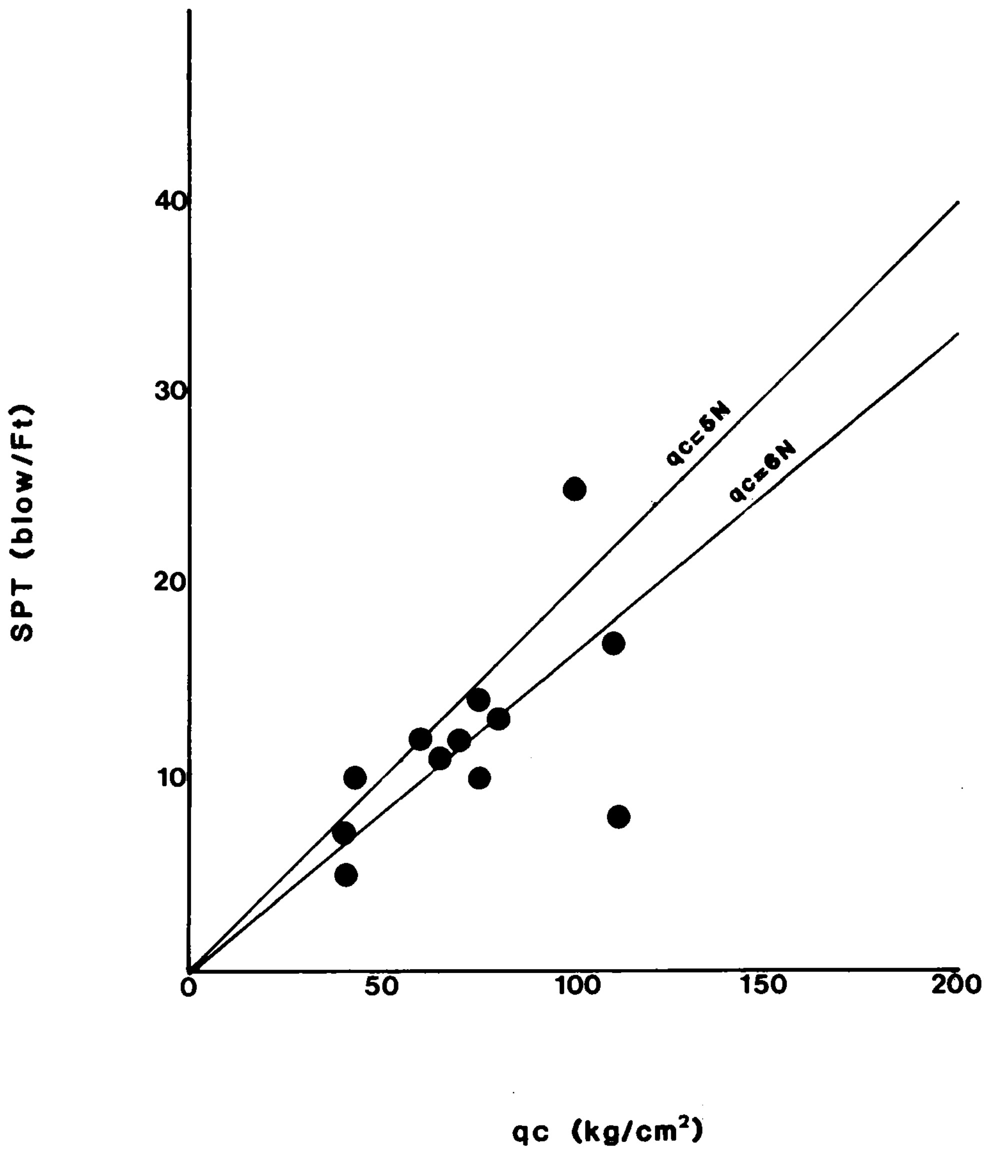


Figure 2. Correlation of Cone Bearing to Standard Penetration Resistance

(1 ft. = 0.3m)

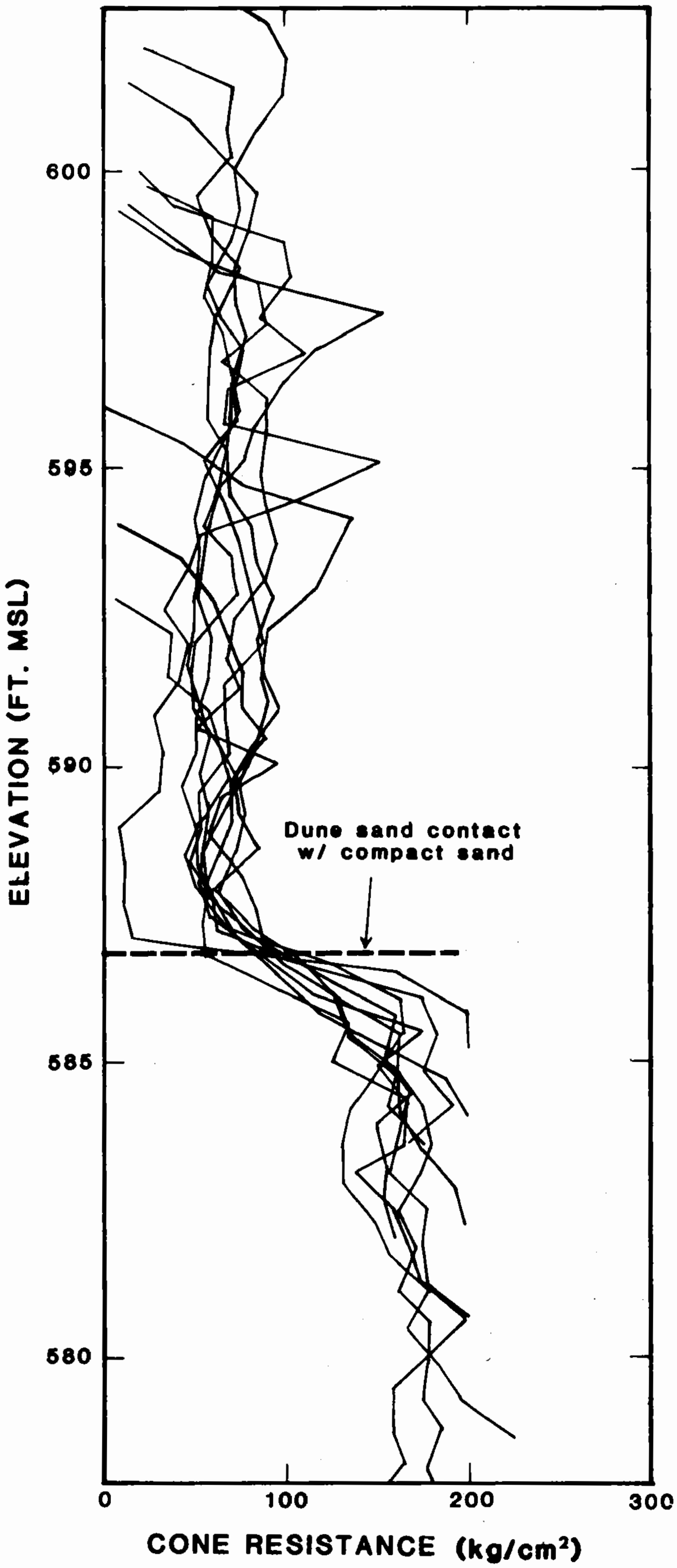


Figure 3. Composite CPT Profile

(1 ft. = 0.3m)

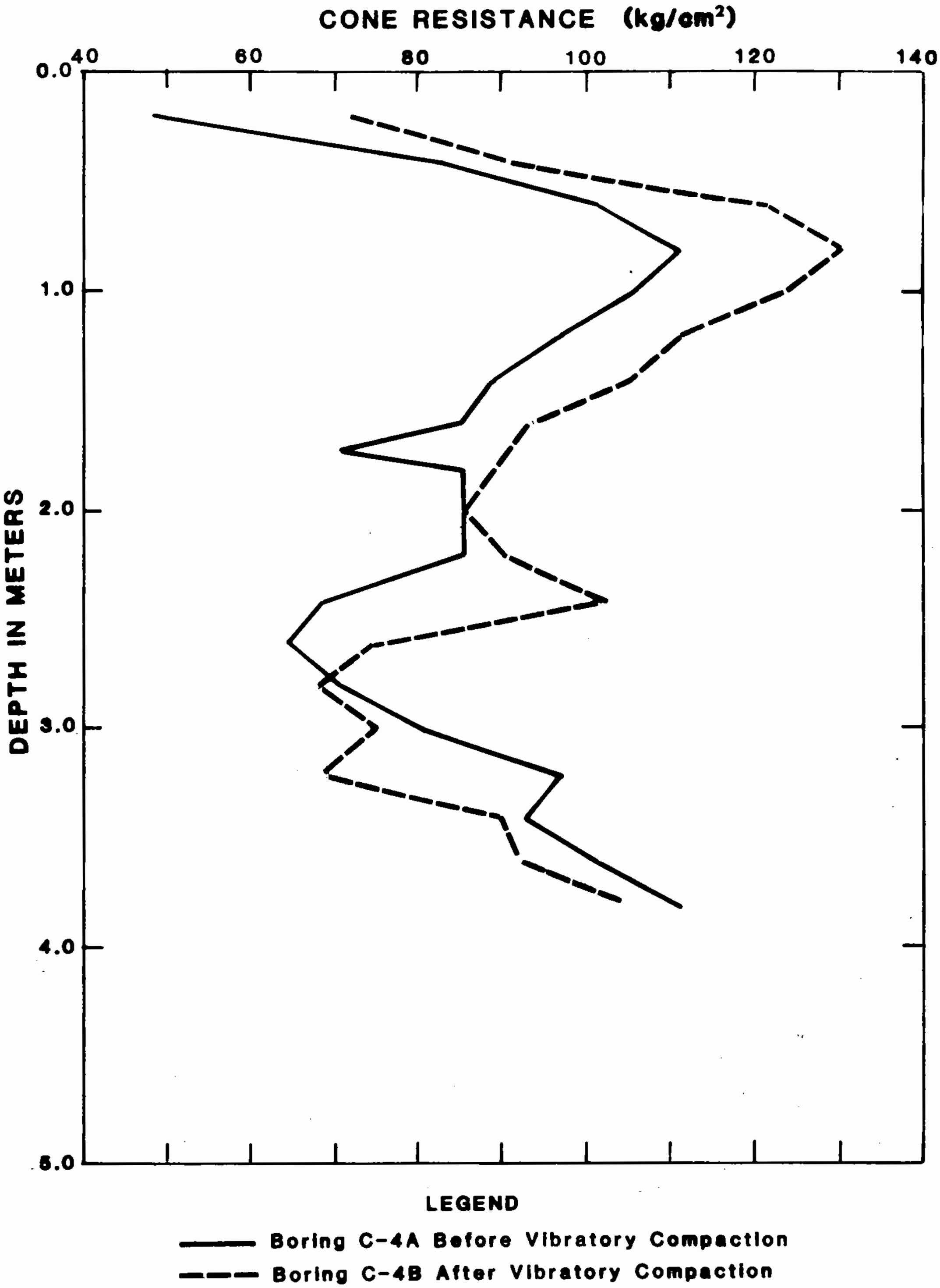


Figure 4. Vibratory Compaction Results

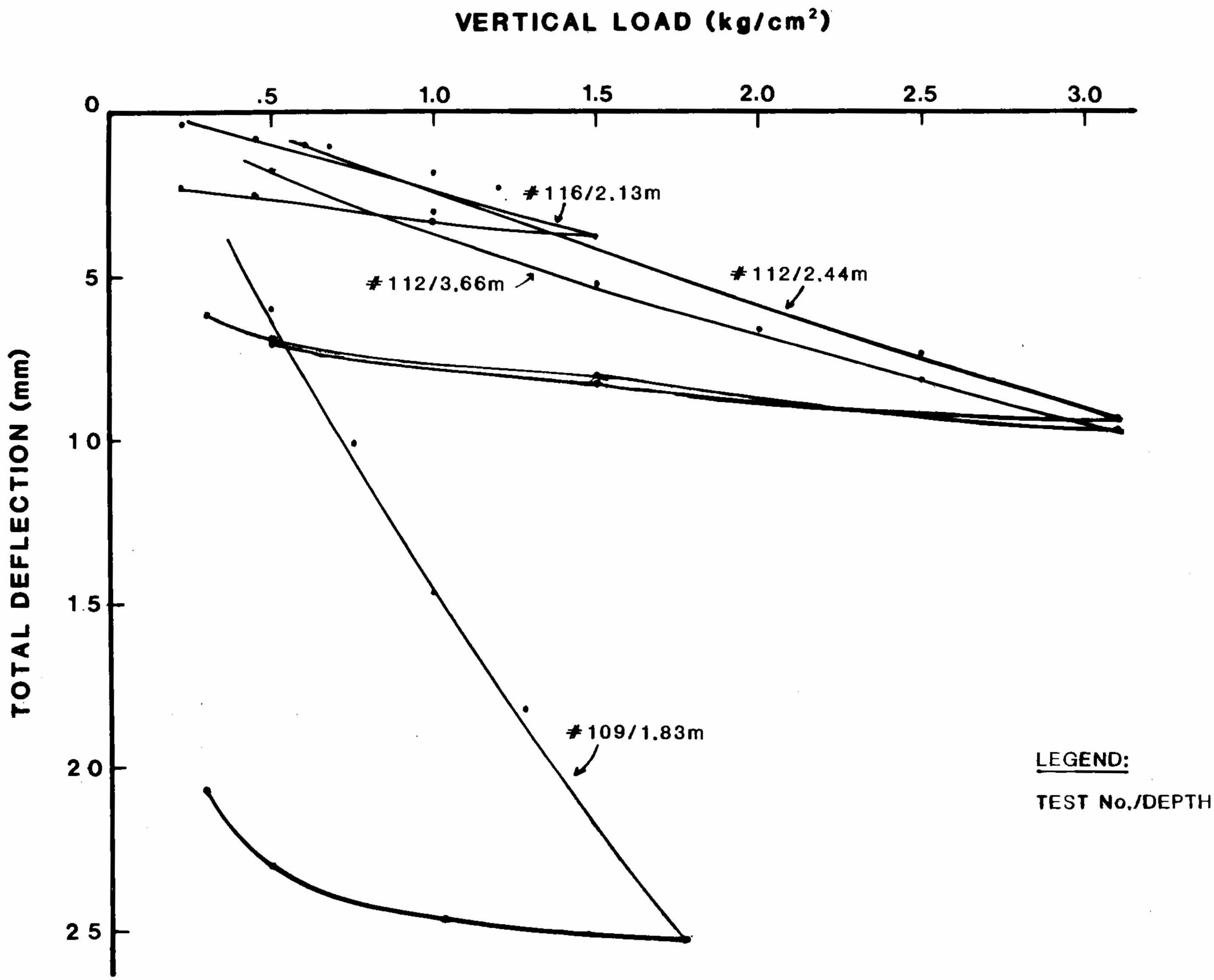


Figure 5. Screw Plate Load Test Results

CASE HISTORY 2: EVALUATION OF FLOODPLAIN DEPOSITS

A geotechnical investigation of a 100 acre (40 ha) site along the Savannah River involved use of the Dutch Cone. Subsurface conditions, including the geotechnical engineering properties of micaceous sands and organic clays, were established for design purposes. Empirical correlations between Dutch Cone Penetration Tests (CPT) and Standard Penetration Tests (SPT) were made along with comparisons to laboratory test results.

The site is located in the Atlantic Coastal Plain Physiographic Province. Most of the materials in the upper 30 feet (9 m) are flood plain or back swamp deposits of sands (SP-SW), silts (SM) and clays (CL-CH). The underlying materials consist of stiff clays, sandy clays (SC), and gravel beds (GW). Groundwater levels were found to be on the order of five feet (1.5 m) below the surface. Air photo interpretation was used to identify the probable limits of an old river meander through the site. The subsurface investigation test locations were arranged to verify this feature. Twenty-five (25) SPT test borings and forty-two (42) CPT soundings were made at the site. The average depth of the test borings and cone soundings was 75 feet (23 m), although five were drilled to depths of 100 feet (31 m). Typically, continuous sampling was employed until the blow counts exceeded 10 blows per foot (.3 m), which usually occurred within the upper 25 feet (8 m). Undisturbed samples were obtained in both clayey and sandy soils. CPT soundings measured both penetration resistance and side friction on the cone sleeve.

CPT soundings were advanced within five feet (1.5 m) of each of three SPT test borings in order to calibrate the two methods of investigation. The soil profiles developed from one of these calibrations are compared in Figure 6. Figure 7 presents a generalized soil profile through the site, as developed from the SPT test borings and CPT soundings.

The ratio of CPT cone resistance to SPT resistance, qc/N, was calculated for the various soil types encountered throughout the site. These ratios are presented in Table 2 along with general limits identified by Schmertmann, (1970). The ratios from this investigation are generally higher than those presented by Schmertmann.

Considerable scatter was found when comparing qc/N for various parts of the site. For examples, qc/N for the micaceous fine sand was found to vary from about four to 13. This appeared to be affected by the mica content. The range of calibration results for the various soils types encountered throughout the site area is shown on Figure 8. Although the general results compared reasonably well with Schmertmann's previous findings, some differences exist, indicating the need for individual site calibration.

Figure 6 shows that identification of the buried organic clay deposit on the cone log is relatively clear. The CPT cone point resistance, qc, drops considerably and the friction ratio increases significantly. It is of interest to note that the location of the buried organic clay deposit as defined by the CPT and SPT is consistent with the interpretation of aerial photograph stereo pairs.

An attempt was made to correlate CPT data to engineering properties determined by conventional laboratory tests. Two zones of clayey soil were investigated. The first zone was a clayey silt to silty clay (MH-CH) from which undisturbed samples were taken at a depth of approximately five feet (1.5 m). Undisturbed samples for testing were also taken from an organic silty clay (OH) at a depth of approximately 18 feet (5.5 m).

A normal course of action with such laboratory and field data would be to compare laboratory undrained shear strength values, Su, to CPT resistance to develop a site specific correlation Nk value. (Su=qc/Nk). Figures 9a and 9b present diagramatically the logic for comparing values of undrained shear strength (Su) derived from lab testing and CPT data. A typical average Nk of 15 was used to reduce the CPT data. Laboratory shear strength included both torvane and triaxial testing. For both material types, each of the three methods: cone, torvane and triaxial produced significantly different Su values. These findings indicated a need for an independent measure of Su that could be used to judge the reliability of the various test results.

To achieve an independent measure of Su, we turned to the consolidation test results and the concept of Normalized Soil Parameter (NSP). Two consolidation tests were conducted on each of the subject materials, resulting in consistent values of maximum preconsolidation pressures. The overconsolidation ratios (OCR) were computed. This enabled values of undrained shear strength (Su) to be computed for each material on the basis of Eq. 1. (After Ladd and Foote, 1974).

$$Su = K\ (OCR)^{n} \qquad \text{(Eq. 2-1)}$$

This empirical relationship between normalized undrained shear strength Su and OCR is expressed in terms of a coefficient K and an exponent n, which have been evaluated as 0.34 and 0.67 respectively, based on extensive triaxial test results. This approach produced an independent Su to which the torvane value compared well in the MH-CH material case and the triaxial value compared well in the case of the OH materials. The Su values calculated from cone data (using Nk = 15) were significantly different in both cases. It is interesting to note that for the upper MH-CH zone, the CPT-computed Su value was on the order of two to five times greater than laboratory values, whereas for the OH material the CPT-derived Su was 0.5 to 0.3 times the laboratory values.

To estimate undrained shear strength from CPT data, a cone factor, Nk, must be assumed. An average Nk of 16 has been proposed by many researchers as a typical value for overconsolidated clays. Using the Su values derived based on evaluations of consolidation tests, the Nk values for the clayey silt to silty clay (MH-CH) and organic silty clay (OH) are 32 and 6, respectively.

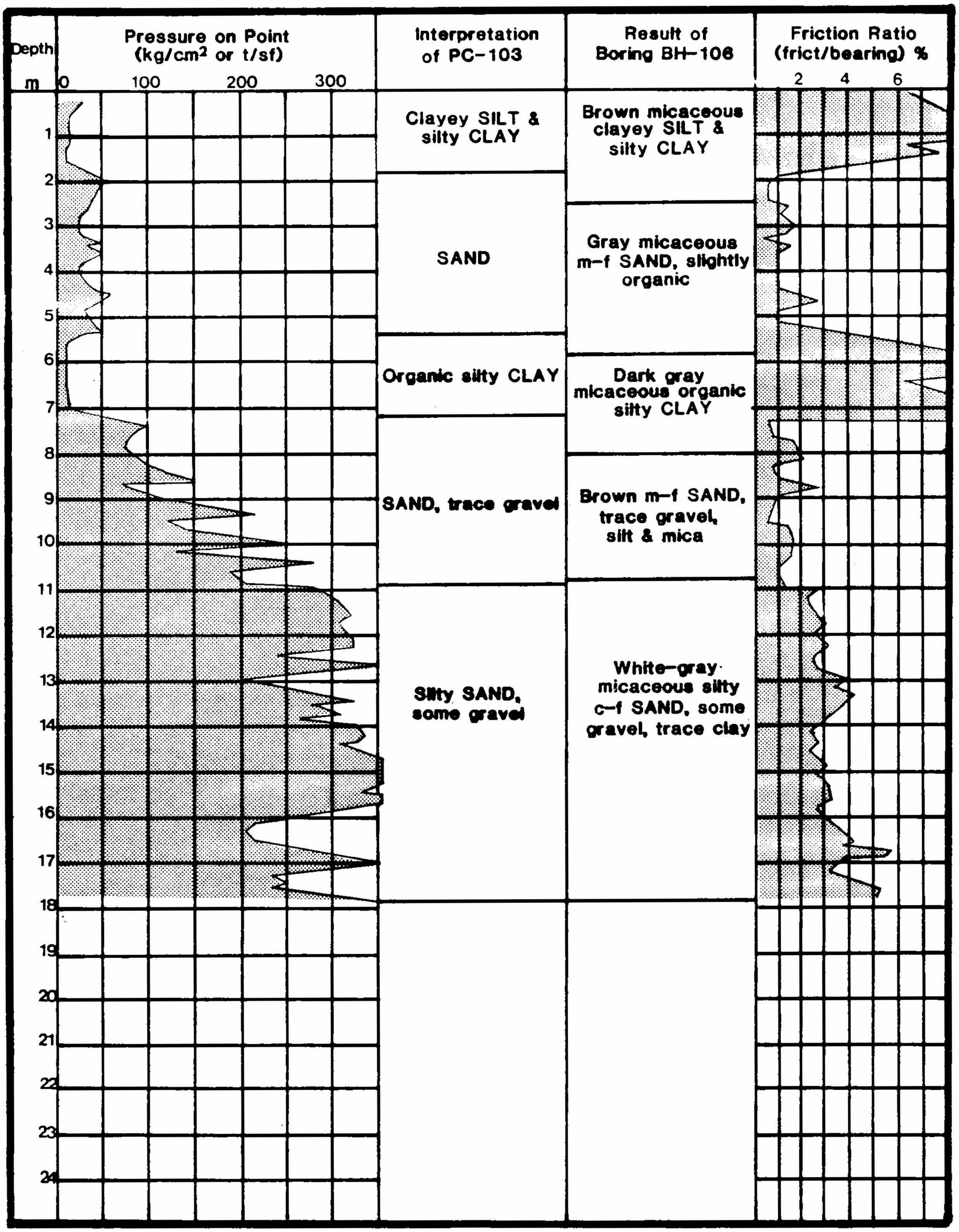


Figure 6. Comparison of Profiles Based on Calibration Borings and Adjacent Cone Soundings

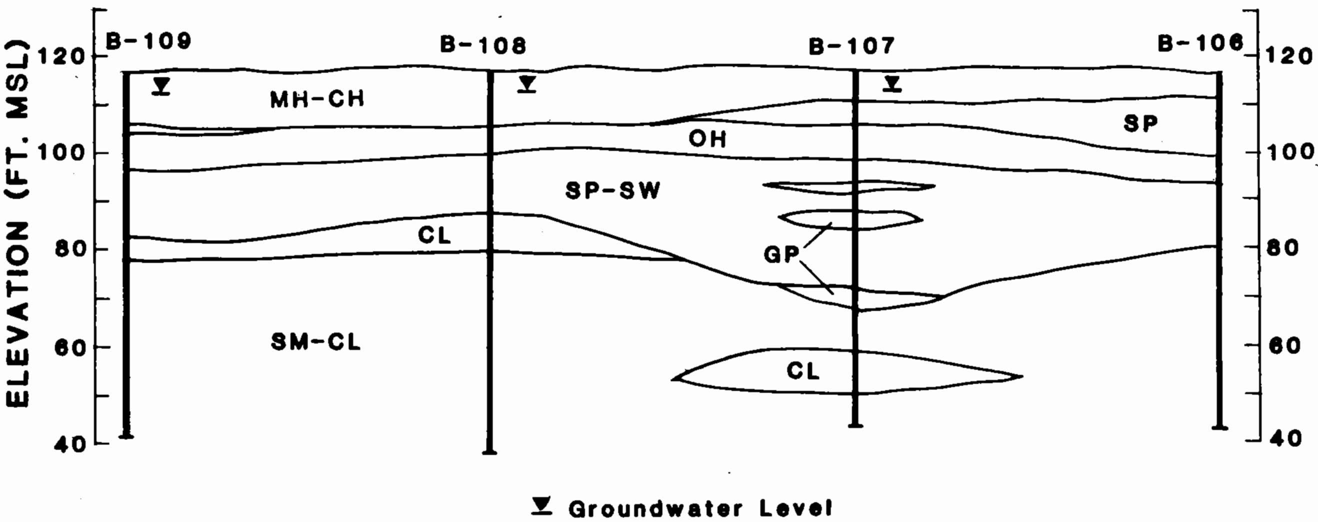


Figure 7. Typical Subsurface Profile

HORIZONTAL SCALE 1''=200'

(1 ft. = 0.3m)

Table 2. Comparison of qc/N Ratios (CPT Cone Resistance/SPT Resistance)

THIS STUDY			AFTER SCHMERTMANN
MATERIAL TYPE	qc/N	qc/N	MATERIAL TYPE
Clay, occasional lenses of gravel (CL)	1.5		
Organic silty clay (OH)	2		
Clayey silt to silty clay (MH-CH)	3.5	2	Silts, sandy silts slightly cohesive silty-sand mixtures
Micaceous, fine sand, some silt (SP-SM)	5.5	3-4	Clean fine to medium sands and slightly silty sands
Coarse to fine sandy, some gravel (SP-SW)	9	5-6	Coarse sands and sands with little gravel
Micaceous, silty coarse to fine sand, some clay (SM-SC)	10	8-10	Sandy gravels and gravel

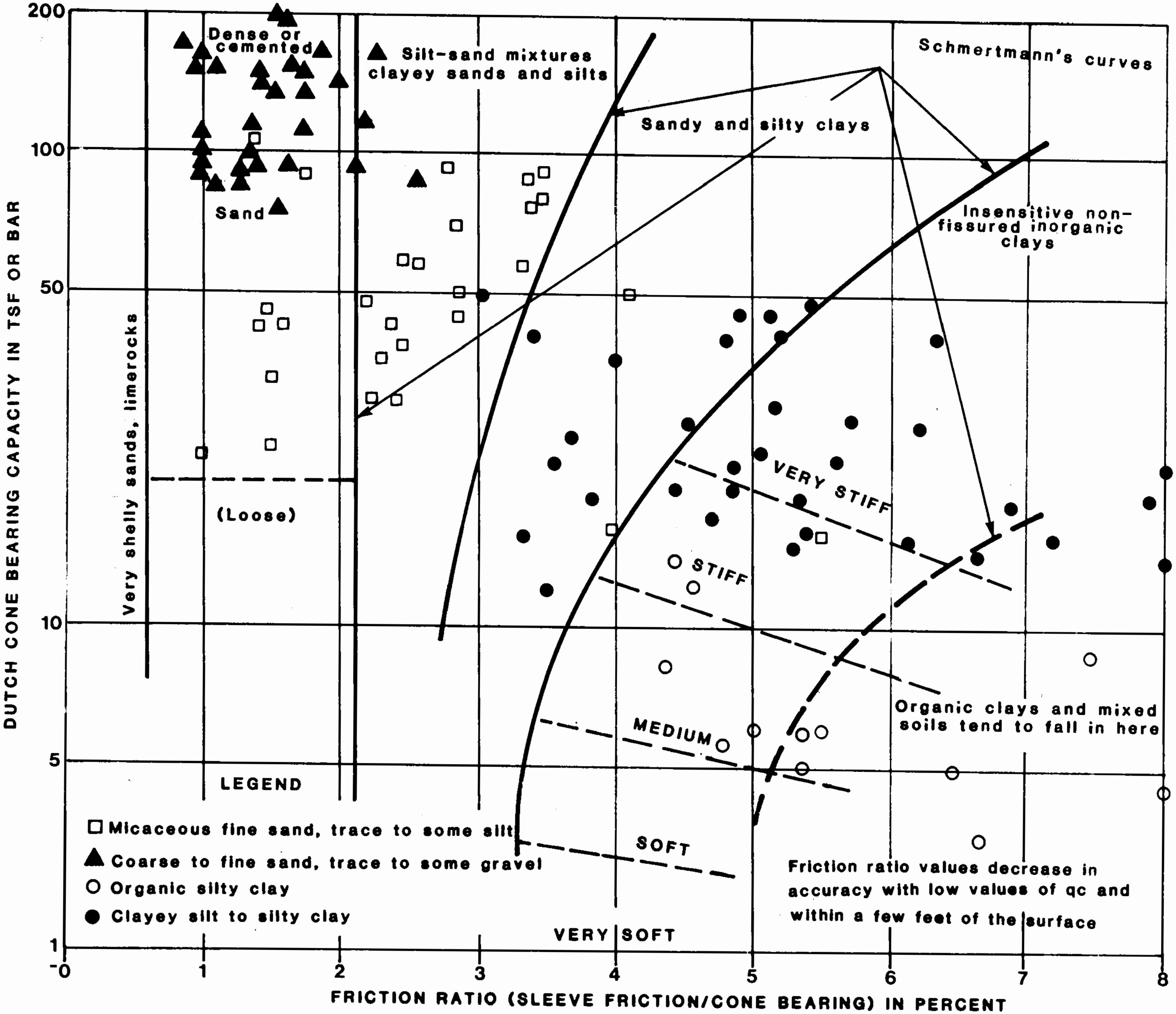


Figure 8. Cone Calibration Chart

SOIL TYPE 1 MH–CH

CONSOLIDATION TEST RESULTS	TORVANE	TRIAXIAL	CONE DATA
$\sigma'_{vm} = 5.15$ where σ'_{vm} is the maximum past pressure $\sigma'_{vo} = .25$ where σ'_{vo} is the effective overburden $OCR = \frac{\sigma'_{vm}}{\sigma'_{vo}}$ $OCR = 20.5$			$q_c = 21$
$\frac{S_u}{\sigma'_{vo}} = .34\,(OCR)^{.67}$ (Using NSP relationship) $S_u = .65$	$S_u = .77$	$S_u = \frac{\sigma_1 - \sigma_3}{2}$ $S_u = .30$	$S_u = \frac{q_c}{N_K}$ if $N_K = 15$ * $S_u = 1.4$
Backfiguring $N_K = \frac{q_c}{S_u}$ for site specific cone correlation * $N_K = 15$ is the average value reported in literature			
$\frac{N_K}{32}$	$\frac{N_K}{27}$	$\frac{N_K}{70}$	

Figure 9A. Shear Strength Comparisons Type 1

SOIL TYPE 2 OH

CONSOLIDATION TEST RESULTS	TORVANE	TRIAXIAL	CONE DATA
$\sigma'_{vm} = 1.4$ where σ'_{vm} is the maximum past pressure $\sigma'_{vo} = .552$ where σ'_{vo} is the effective overburden $OCR = \dfrac{\sigma'_{vm}}{\sigma'_{vo}}$ $OCR = 2.54$			$q_c = 2$
$\dfrac{S_u}{\sigma'_{vo}} = .34\,(OCR)^{.67}$ (Using NSP relationship) $S_u = .35$	$S_u = .48$	$S_u = \dfrac{\sigma_1 - \sigma_3}{2}$ $S_u = .35$	$S_u = \dfrac{q_c}{N_K}$ if $N_K = 15$* $S_u = .13$
Backfiguring $N_K = \dfrac{q_c}{S_u}$ for site specific cone correlation * $N_K = 15$ is the average value reported in literature			
$\dfrac{N_K}{5.7}$	$\dfrac{N_K}{4.2}$	$\dfrac{N_K}{5.7}$	

Figure 9B. Shear Strength Comparisons Type 2

CASE HISTORY 3: SETTLEMENT ANALYSIS OF AN ALLUVIAL PROFILE

The Dutch Cone was utilized in conjunction with Standard Penetration Testing during a subsurface investigation for a site along the Missouri River. The study was conducted to:

a) determine the general subsurface conditions at the site,
b) select procedures to improve the site for foundation support and provide foundation design criteria, and
c) point out the special precautions that should be taken in designing the foundations because of existing subsurface conditions.

This case history focuses on one portion of the site where considerable cone penetrometer data was generated.

The site is located on the flood plain of the Missouri River. Soil deposits in the area consist primarily of Pleistocene glacial till, loess, and various types of recent river deposited alluvium. Groundwater levels fluctuate seasonally and with variations in the level of the adjacent Missouri River; however, the groundwater table generally existed between two and 10 feet (0.6 and 3.1 m) from ground surface.

One hundred twenty-seven (127) Standard Penetration Test (SPT) borings and 44 Dutch Cone Penetrometer Tests (CPT) were made at the site. The average depth of the test borings was 60 feet (18 m), with several reaching 100 feet (31 m). Samples were obtained in standard 2-inch (50 mm) OD split spoons, 2-inch (50 mm) ID California spoon samples, and 3-inch (75 mm) OD Shelby tube samples. CPT soundings were made in the upper 60 feet (18 m) below ground surface using a truck-mounted rig.

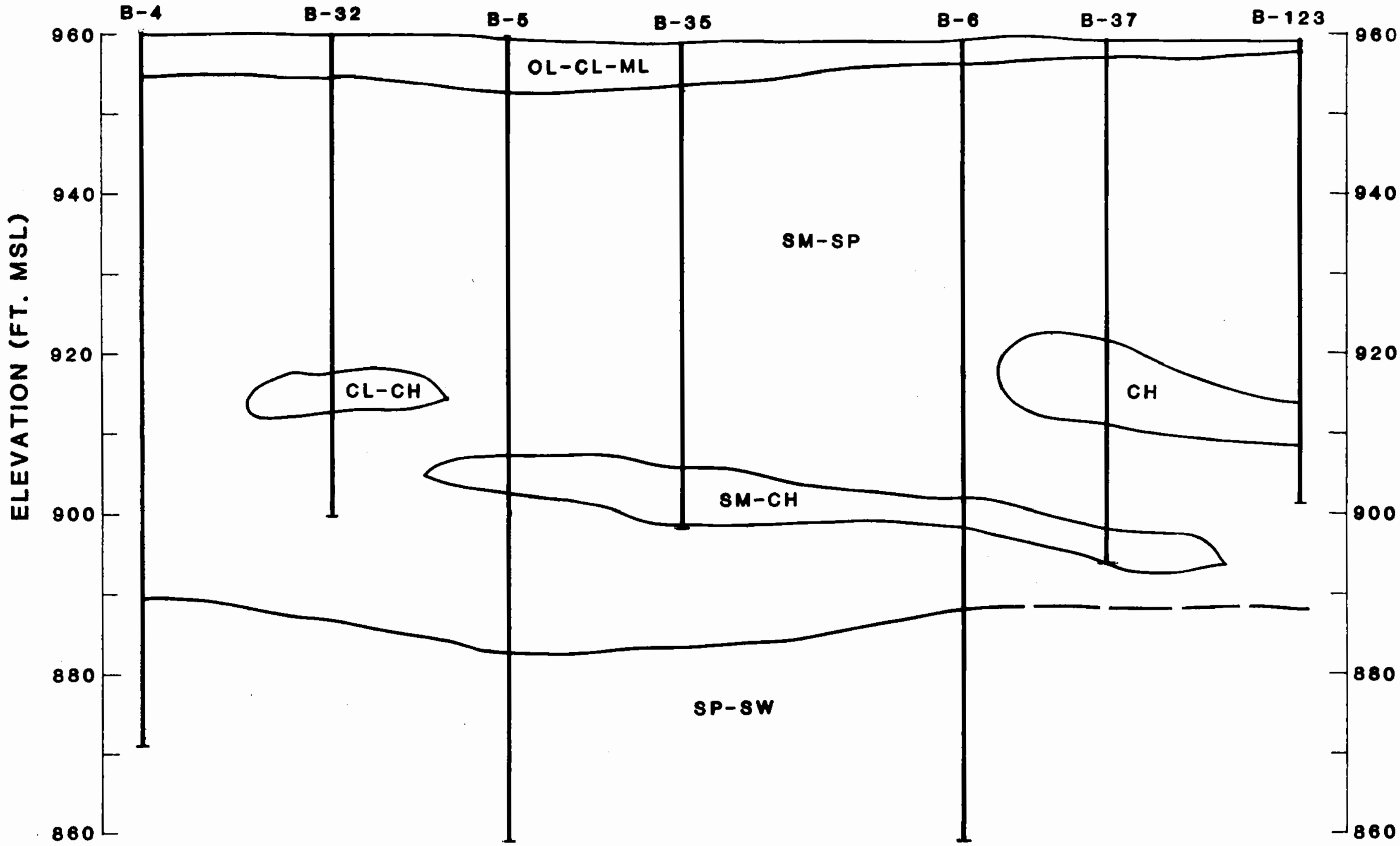
The subsurface conditions, as determined by the field investigation, are of various types of river deposited alluvium over bedrock. The alluvium is mainly coarse grained soils (SP-SW), with several layers of fine grained soils encountered at various depths. In addition, there are localized deposits of soft soils of limited horizontal extent which consist of highly plastic clay or very loose sand with wood and fine organic material. Figure 10 presents a generalized soil profile through the site.

Several potentially compressible strata were identified by the field investigation. One such stratum existed within the upper five feet (1.5 m) of the profile. To minimize post-construction settlements, a site preparation program was undertaken that included excavation of the soft near-surface soils and surcharging of deeper compressible zones. Analyses using CPT data were performed to predict potential settlements in the deeper sands. To estimate the potential settlement that would occur due to compression and deformation of the sand formation, methods introduced by Schmertmann and Buismann-DeBeer were followed. These analyses involve the utilization of CPT cone resistance (qc) values to predict compressibility moduli. Results of these analyses show that predicted settlements were well in excess of the tolerable limits. The site preparation program was then continued by surcharge loading consisting of controlled fill. Settlement plates were installed at a number of locations on the site. As actual settlement was monitored, an attempt was made to correlate observed settlements with predicted settlements.

A composite profile of the six cone penetrometer soundings made within the study area is shown on Figure 11. Superimposed on the composite is the simplified cone bearing profile that was used in the settlement prediction calculations. Predicted settlements were calculated using both the Buisman-DeBeer and Schmertmann techniques. Figures 12 shows a plot of fill height versus settlement for the study area. Included on this plot are the two settlement predictions and the actual settlements that were monitored during and following fill placement. Settlements varied among the seven settlement plates. This range of observed settlements is shown on Figure 12 by a series of three short vertical lines. The connecting line indicates the average settlement exhibited by the seven settlement plates.

The results of both the Buisman-DeBeer and Schmertmann settlement analyses were conservative values for predicted settlement. The Buisman-DeBeer method overestimated settlements by a factor of 2.0, while the Schmertmann method overestimated by a factor of 1.7.

The results obtained from this study concur with others that have been previously documented; namely, that settlement predictions made with the use of cone penetrometer data can be significant aids when designing foundations. The Buisman-DeBeer method, in its present form, presents conservative results that must be corrected by a factor of approximately one-half. This has previously been suggested by Meyerhof. The Schmertmann method, although more closely paralleling actual results, still provides a conservative settlement prediction. Correction factors of 0.6 to 0.7 appear appropriate.



HORIZONTAL SCALE 1''=300'

(1 ft. = 0.3m)

Figure 10. Typical Subsurface Profile

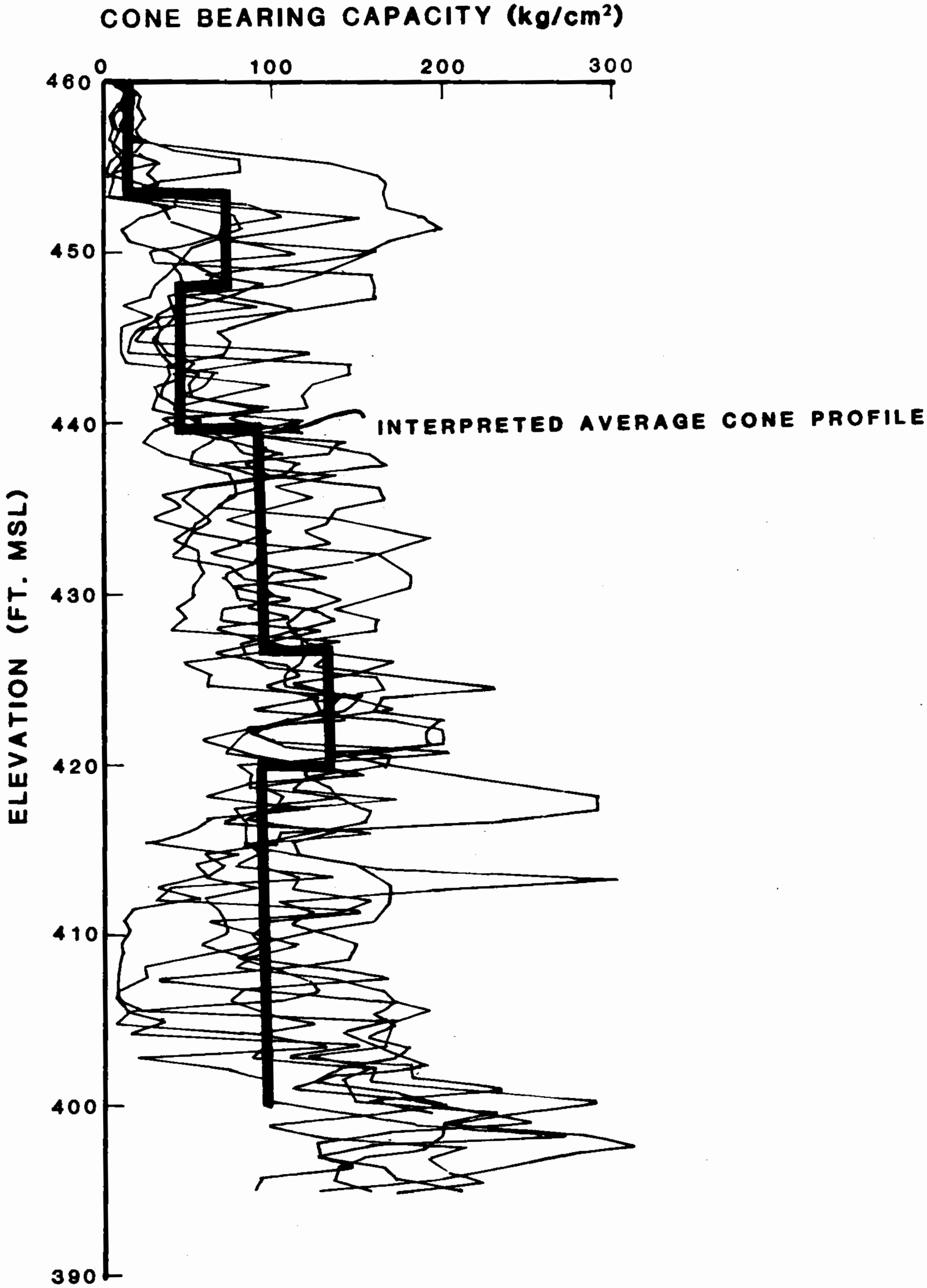


Figure 11. Composite CPT Profile

(1 ft. = 0.3m)

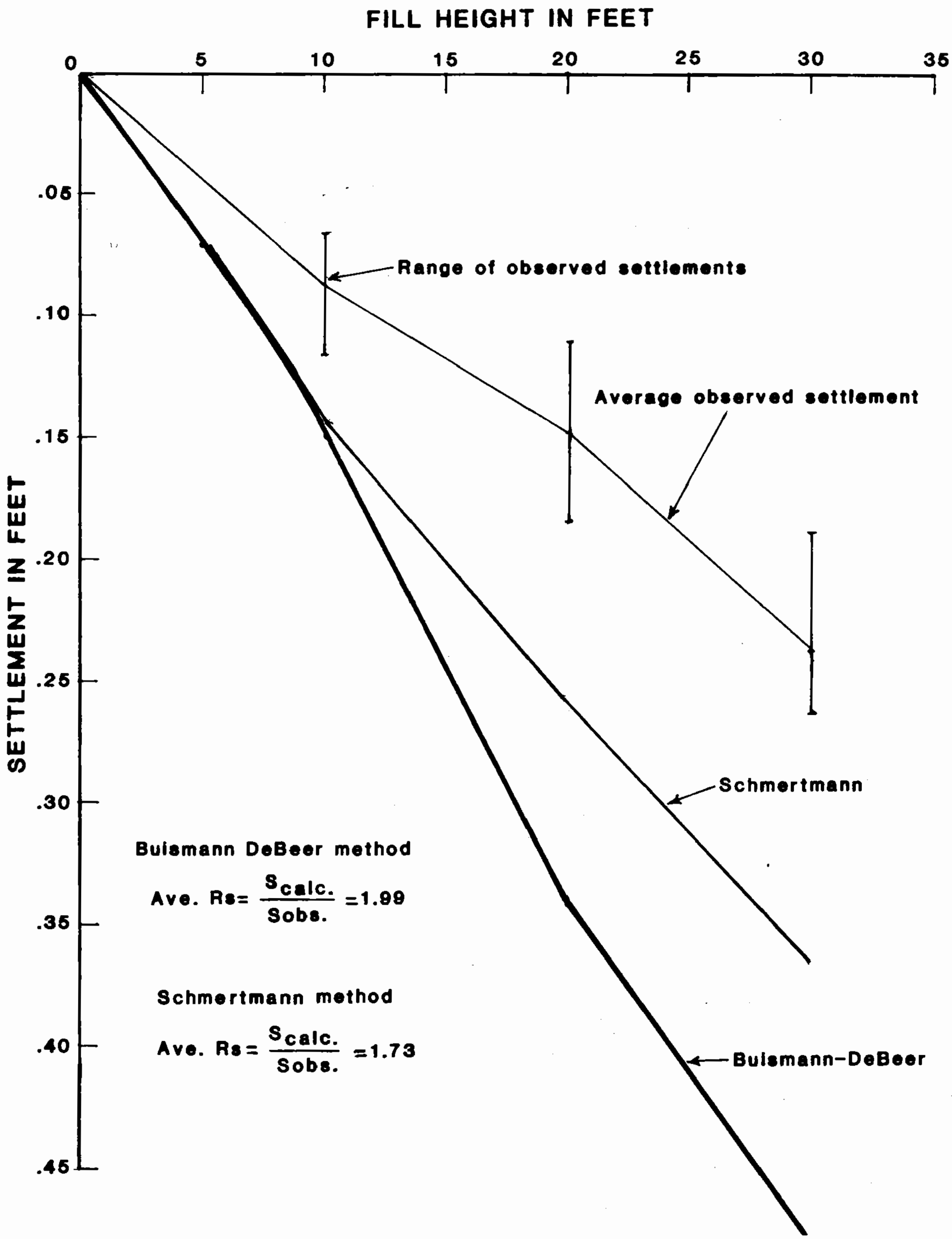


Figure 12. Comparison of Calculated and Observed Settlements Versus Fill Height

(1 ft. = 0.3m)

CASE HISTORY 4: DENSIFICATION OF SANDS

The Dutch Cone has been successfully utilized on projects involving vibroflotation; however, unusual behavior patterns were experienced at an industrial site in northeastern Florida. It was originally recommended that Dutch Cone Penetration Tests (CPT) be used in evaluating the effectiveness of vibroflotation compaction. A conventional interpretation of the findings at this site would be that the subsurface materials were loosened as a result of vibroflotation.

Standard Penetration Tests (SPT) were performed during the initial foundation investigation at the site. The upper 20 to 30 feet (6 to 9 m) of the subsurface profile beneath critical building areas consisted of very loose to loose, fine slightly silty sand (SM) with isolated clayey pockets (SC) and zones of medium dense sand (SP). Groundwater levels were found to be one to three feet (0.3 to 0.9 m) below the surface. A typical profile is presented in Figure 13 and a series of soil gradation curves are presented in Figure 14. Densification of the loose sands was chosen for foundation preparation as opposed to the more costly use of timber piles driven to bedrock.

Following the decision to use vibroflotation, a program was developed to evaluate the effectiveness of densification by utilizing the CPT. Series of CPT soundings and SPT test borings were made at several locations to provide data for comparisons. A composite diagram of cone penetration resistances, qc, before compaction is presented in Figure 15. CPT soundings were generally made to a depth of 15 feet (4.6 m). Scatter is evident, however the heaviest concentration of penetration resistance is within a band between 50 and 100 kg/cm^2. Breaks in the curves can be attributed to lenses of clay or medium dense sand. Figure 16 is a composite diagram of penetration resistance after vibroflotation. In contrast to that shown on Figure 15, the heaviest concentration of penetration resistances falls below 50 kg/cm^2. When it was noted that the CPT data were indicating that the vibroflotation process was loosening rather than densifying, it was decided to try other techniques to evaluate post-compaction densities.

Post-compaction Standard Penetration Tests (SPT) were conducted at several locations, providing data for correlations between CPT cone resistance, qc, and SPT blow counts, N, before and after vibroflotation. The Meyerhof correlation for fine or silty medium dense to loose sands is qc=4N where qc is in tons per square foot (tsf). Figure 17 shows the respective plots of three CPT soundings performed before and after vibroflotation. Test borings were made adjacent to each of the CPT probes, and SPT blow counts were plotted on each figure to a scale which reflects the qc=4N relationship. A reasonable correlation is shown for both cases; however, the post-compaction comparison is the closest.

Subsequent to both the CPT and SPT results indicating a loosening of the soils after vibroflotation, it was decided to determine in-place densities by direct techniques. Undisturbed bulk samples were taken from test pits and an Osterberg sampler at selected depths. Dry densities of the sands indicated that the relative compaction of the site was suitable for the proposed building loads.

It can only be speculated as to what caused the CPT and SPT results to be lower after vibroflotation. First, it is possible that an actual loosening occurred. Other explanations for lower penetration resistance after vibroflotation could be:

- Cementation of the sand prior to vibroflotation contributed added penetration resistance more than the added density after vibroflotation.
- The vibroflotation caused relief of locked-in horizontal stresses which tend to reduce penetration resistance values even though the material was more dense.

- Dispersion of silt or clay particles during vibroflotation tended to cause lubrication of the sand particles and therefore lowered the post-vibroflotation test results.

This case study has been presented to document the fact that anomalies in test results do occur. It is hoped that others who experience similar unresolved situations can identify with the problems associated with such an occurrence. More publicized evidence of anomalies would undoubtably lead to a better understanding of limitations within an engineering practice.

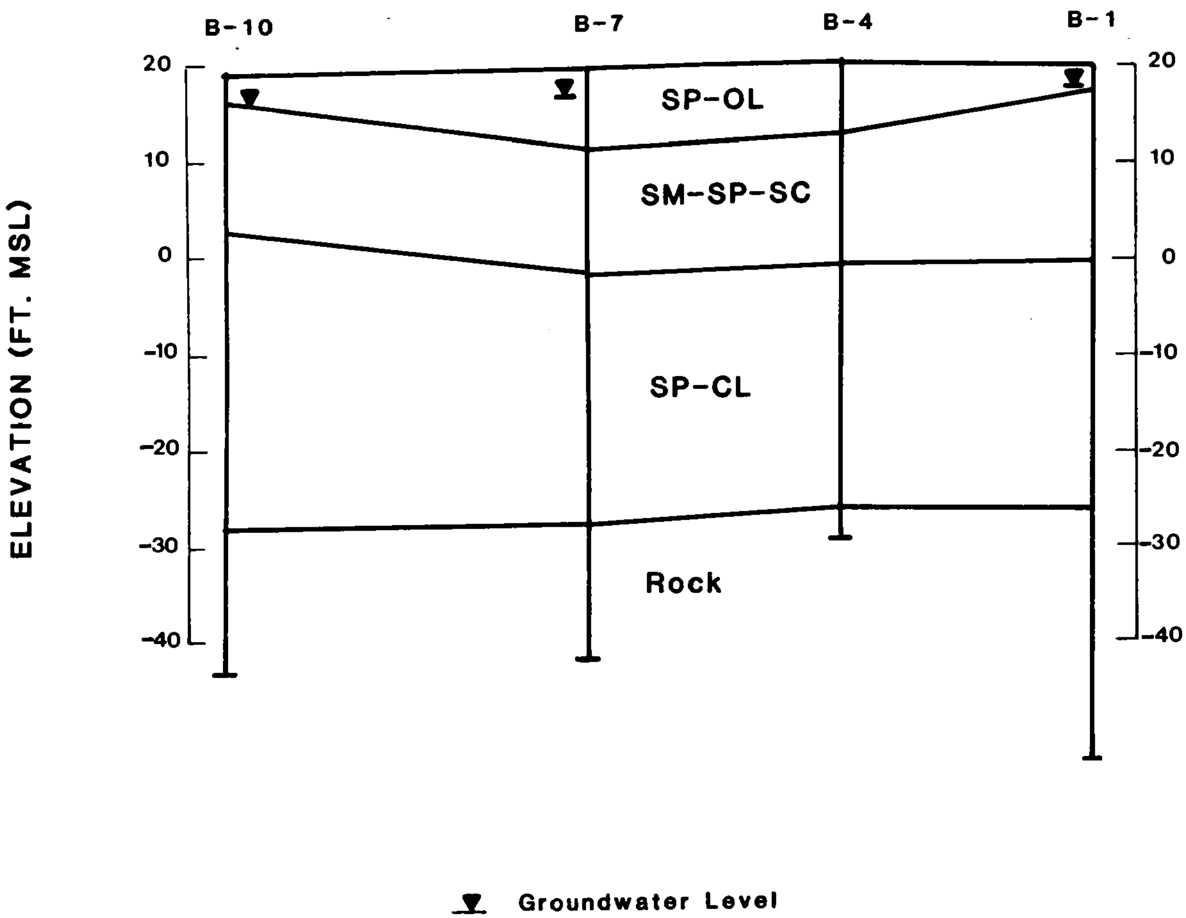
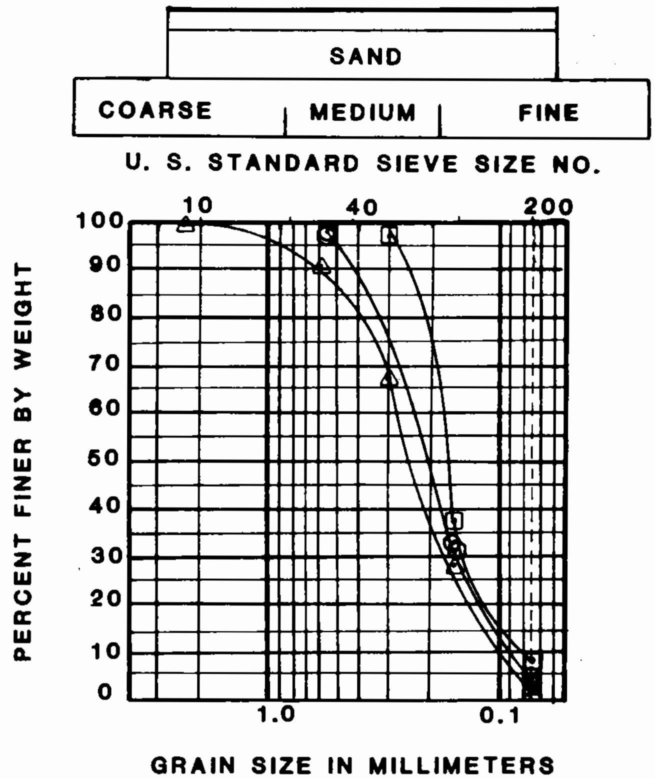


Figure 13. Typical Subsurface Profile

HORIZONTAL SCALE 1'' = 200'

(1 ft. = 0.3m)



BORING	SAMPLE	DEPTH	SYMBOL	CLASSIFICATION
—	3	6.5	⊙	Fine Sand (SP)
—	3	21.5	△	Fine Sand (SP)
—	4	6.5	⊡	Fine, Slightly Silty Sand (SP)
—	4	26.5	⊙	Fine Sand (SP)

Figure 14. Mechanical Analysis

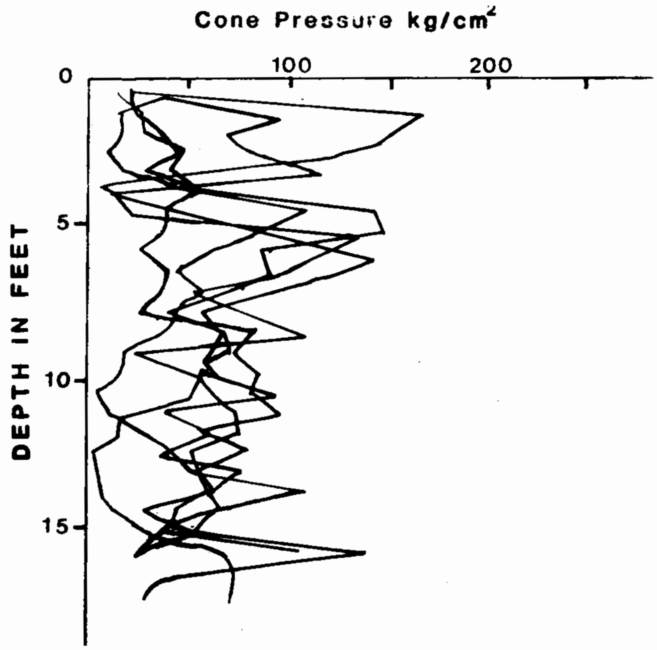


Figure 15. Composite CPT Profile Before Densification

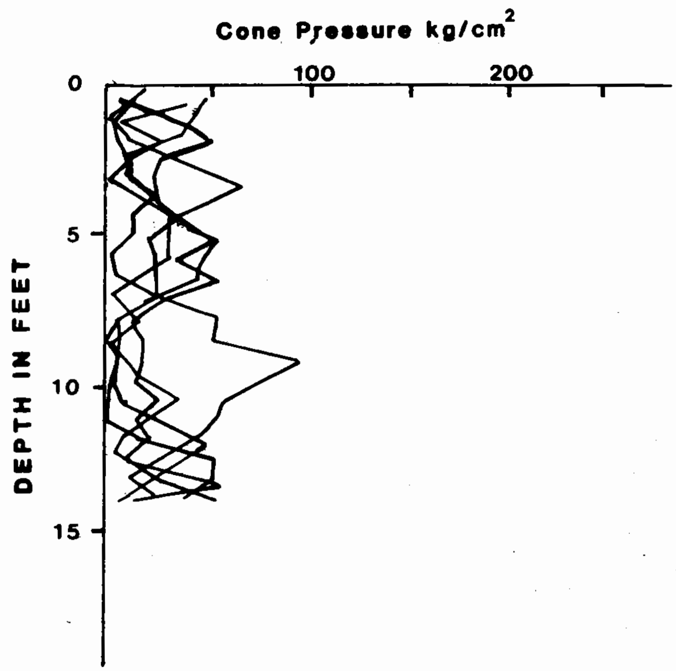
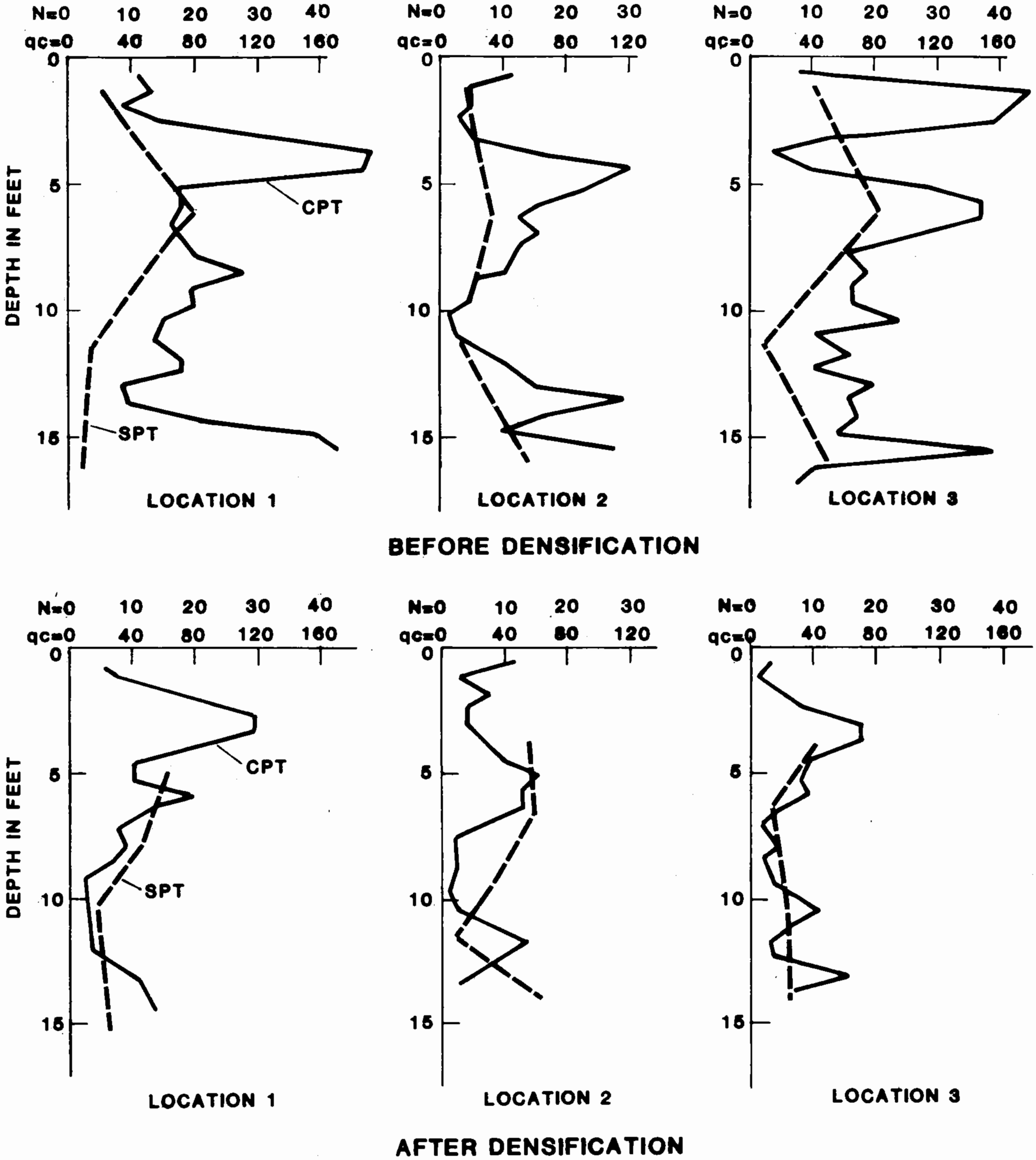


Figure 16. Composite CPT Profile After Densification

(1 ft. = 0.3m)



LEGEND

N-SPT Blows per 12 inches

qc-Cone bearing in Kg/cm^2

(1 ft. = 0.3m)

Figure 17. SPT and CPT Correlation Before and After Soil Densification

CASE HISTORY 5: ESTIMATING UNDRAINED SHEAR STRENGTH FROM CONE DATA

For the construction of a facility in Texas, a geotechnical investigation was conducted to define the foundation conditions and formulate parameters to be used in the design of slopes and vertical bracing for a deep excavation. The Dutch Cone Penetrometer (CPT) was used to supplement Standard Penetration Test (SPT) borings. An on-site correlation was made between SPT blow counts, N, and CPT cone bearing resistance, qc. The resulting relationship showed qc approximately equal to 3N. In addition to measuring cone resistance, qc, local friction was measured by means of a mechanically operated friction sleeve. The dimensionless ratio, Fr, of sleeve friction, Fs, to cone bearing, qc, provided a means to help identify the soil types penetrated. A analysis of combined cone and laboratory data was used to define the in-situ properties of the clay soils.

The site is part of the flat coastal plain located approximately 10 miles (17 km) north of the Gulf Coast. It is a nearly featureless depositional plain composed of deltaic sediments of Pleistocene age. The regional stratigraphy typically shows clays interbedded with silts, sandy silts, and fine to medium grained sand. Groundwater levels vary from one to 10 feet (0.3 to 3.0 m) below the surface of the site. Sixty (60) SPT borings and 13 CPT soundings were performed at the site. The subsurface profile consists of interbedded layers of sandy silts (SM-ML) and plastic silty clays (CL-CH). A generalized soil profile through the site is given on Figure 18.

The use of a friction sleeve during cone soundings increased the value of the CPT data. At the Texas site, lenses of sands and silts were bedded between clay layers. With the addition of a friction ratio profile to the cone bearing profile it was possible to identify the existence of virtually all soil layers. Plotting cone bearing capacity against friction ratio, Fr, on a diagram such as proposed by Schmertmann (1969), an attempt was made to correlate cone data with soil type. Actual soil types were determined from SPT samples. Figure 19 presents CPT data from the Texas site plotted on the proposed soil identification diagram.

In this case, the use of Figure 19 yields a somewhat misleading representation of the soils encountered. The use of Figure 19 indicated higher sand contents than the actual SPT samples showed. This is indicated by most data points plotting too far to the left of the diagram. Schmertmann recommends that local correlations be made to improve results; however, this cannot be accomplished by applying one common factor to the data shown on Figure 19. For instance, by multiplying friction ratios by two (2), several of the sandy clays and silty clays can be shifted to within their designated areas on Figure 19. The drawback to this approach is that the multiplication factor is not appropriate for all data points. Soils with friction ratios greater than four would plot off the diagram when multiplied by a correction factor of two. Accurate results can only be achieved if a site specific correlation is developed between both elements of the cone data and the actual soil samples. By following this practice at the initial stages of an investigation, a useful correlation could be developed between cone data and soil type.

A laboratory testing program was conducted to determine the physical characteristics of the different soils. General index property and consolidated-isotropic-undrained (CIU) triaxial compression tests were performed on selected thin-wall samples obtained from various depths in boreholes located adjacent to CPT soundings. Total stress and effective stress strength parameters were determined from the triaxial test results.

Comparative analyses were made of the laboratory test data and CPT data to evaluate the relationship between undrained shear strength, Su, of cohesive soils and qc. The approach involved cone bearing values and a bearing capacity factor for clay, Nc, as in Eq. 5-1. (Schmertmann, 1977).

$$Su = \frac{qc - \sigma z}{Nc} \qquad \text{(Eq 5-1)}$$

where σz = total overburden pressure at depths where qc was measured.

To estimate Su, values for qc were taken from CPT profiles, and a value of Nc equal to 16 was used. The Nc value of 16 has been shown in many previous investigations to be a reasonable choice as a bearing capacity factor. Estimated Su values are shown on Table 3. These results were compared to laboratory shear strength values, determined from CIU triaxial tests, by the ratio Su Lab/Su Cone. Values approaching unity represent agreement between the laboratory strength results and those developed from cone data and Eq. 5-1. Comparison shows that calculated Su values generated from cone data are generally conservative.

It seems appropriate to make local correlations for Nc values, thus maximizing the accuracy of calculated strength values. This is necessary, since it has been recognized that Nc is not a constant, but varies with a number of factors including clay composition, plasticity and loading history. Development of local correlations for the different soils encountered at the Texas site involved substituting Su values determined from triaxial tests into Eq. 5-1 and backfiguring the appropriate Nc values. These backfigured values, presented in Table 3, show how much Nc values may deviate at one particular site.

Table 3. Shear Strength Estimates From CPT Data

Depth (ft.)	Soil Classification	Laboratory Shear Strength, Su (Kg/cm^2)	Estimated Su From Cone Data (Kg/cm^2)	$\frac{\text{SuLAB}}{\text{SuCONE}}$	Nc Required for $\frac{\text{SuLAB}}{\text{SuCONE}} = 1$
6.5	CH	.44	.59	0.8	21.3
7.0	CH	.80	.63	1.3	12.6
7.5	CH	1.58	.86	1.8	8.7
14.0	CH	2.90	.74	3.2	4.1
33.5	ML-CL	9.62	.46	21.0	0.8
34.0	CL	2.12	1.09	1.9	8.3
34.5	ML-CL	1.38	.63	2.2	7.2
34.5	CL	3.56	1.86	1.9	8.4
34.5	CH	3.19	.73	4.4	3.6
					AVERAGE 8.3

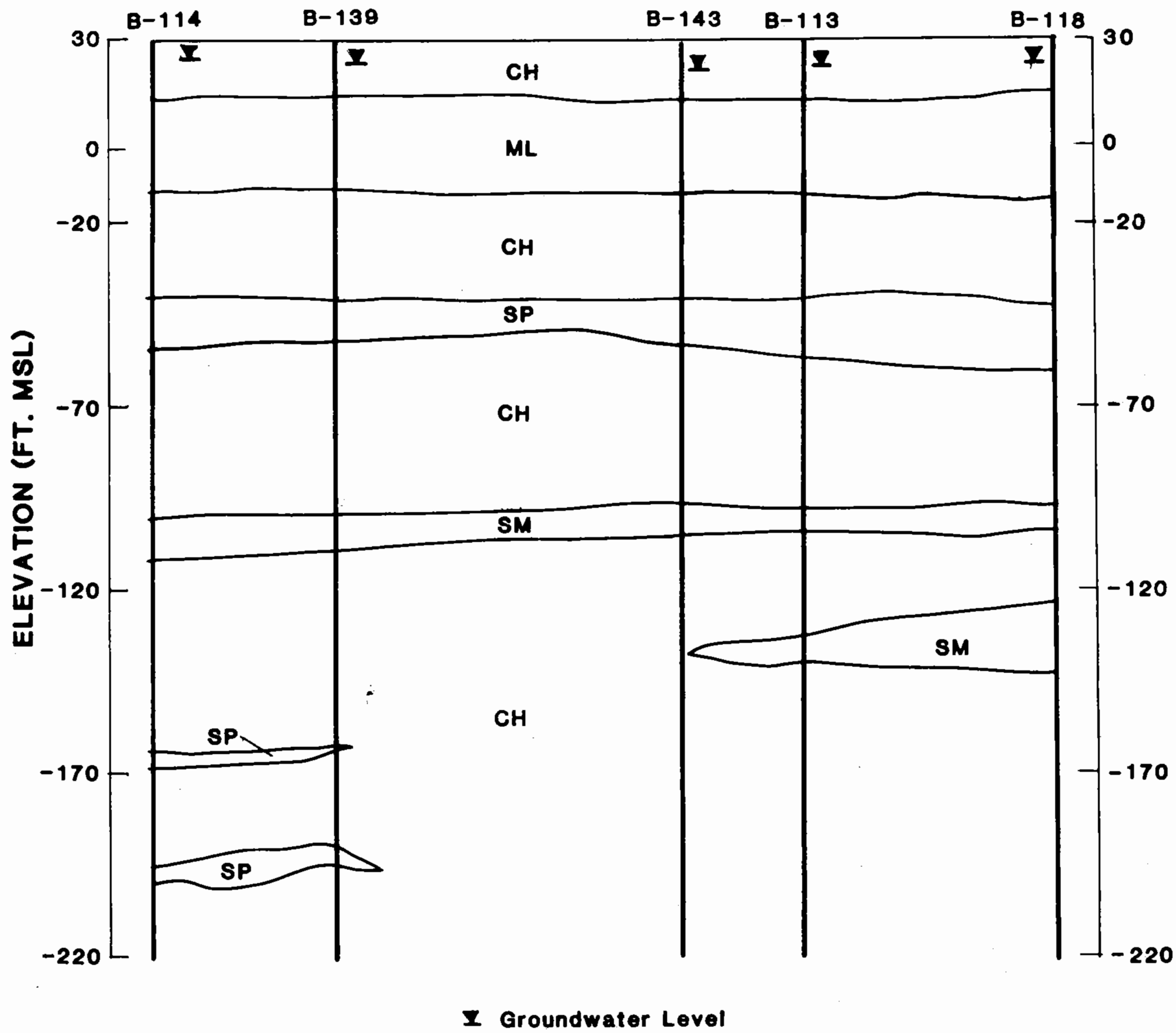


Figure 18. Typical Subsurface Profile

HORIZONTAL SCALE 1"=160'

(1 ft. = 0.3m)

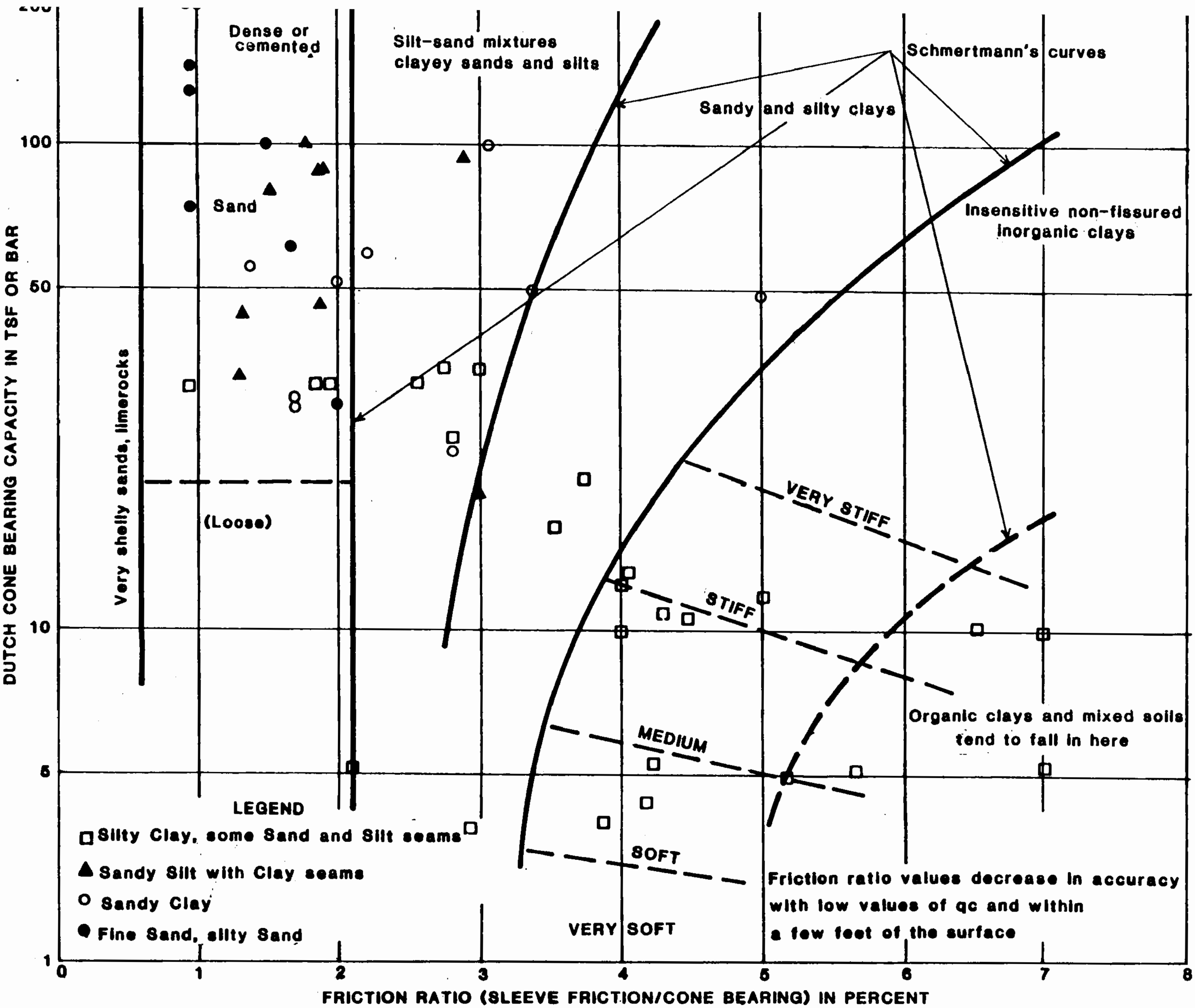


Figure 19. Cone Calibration Chart

REFERENCES

Bazaraa, A.R., "Use of The Standard Penetration Test For Estimating Settlements of Shallow Foundations on Sand," Ph.D. Thesis, University of Illinois, Urbana, 1967.

DeBeer, E.E., "Bearing Capacity and Settlement of Shallow Foundations on Sand," Proceedings, Bearing Capacity and Settlement of Foundations Symposium, Duke University, Durham, N.C., 1967

Ladd, C.C., and Foott, R., "New Design Procedure for Stability of Soft Clays," Journal of Geotechnical Engineering Division, ASCE, Vol. 100. No. GT7, 1974.

Meyerhof, G.G., "Penetration Tests and Bearing Capacity of Cohesionless Soils," Journal of Soil Mechanics and Foundation Division, Proceedings ASCE, Vol. 82, SM 1, 1956.

Peck, R.B., Hanson, W.E., and Thornburn, T.H., Foundation Engineering, John Wiley & Sons, New York, 1974.

Schmertmann, J.H., "Guidelines for CPT Performance and Design", Report Prepared for FHWA, 1977.

Schmertmann, J.H., "Static Cone to Compute Settlements Over Sand, "Journal of Soil Mechanics and Foundation Division, ASCE, Vol. 96, No. SM3, 1970.

FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH AND DEVELOPMENT

The Offices of Research and Development (R&D) of the Federal Highway Administration (FHWA) are responsible for a broad program of staff and contract research and development and a Federal-aid program, conducted by or through the State highway transportation agencies, that includes the Highway Planning and Research (HP&R) program and the National Cooperative Highway Research Program (NCHRP) managed by the Transportation Research Board. The FCP is a carefully selected group of projects that uses research and development resources to obtain timely solutions to urgent national highway engineering problems.*

The diagonal double stripe on the cover of this report represents a highway and is color-coded to identify the FCP category that the report falls under. A red stripe is used for category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, green for categories 6 and 7, and an orange stripe identifies category 0.

FCP Category Descriptions

1. Improved Highway Design and Operation for Safety

Safety R&D addresses problems associated with the responsibilities of the FHWA under the Highway Safety Act and includes investigation of appropriate design standards, roadside hardware, signing, and physical and scientific data for the formulation of improved safety regulations.

2. Reduction of Traffic Congestion, and Improved Operational Efficiency

Traffic R&D is concerned with increasing the operational efficiency of existing highways by advancing technology, by improving designs for existing as well as new facilities, and by balancing the demand-capacity relationship through traffic management techniques such as bus and carpool preferential treatment, motorist information, and rerouting of traffic.

3. Environmental Considerations in Highway Design, Location, Construction, and Operation

Environmental R&D is directed toward identifying and evaluating highway elements that affect the quality of the human environment. The goals are reduction of adverse highway and traffic impacts, and protection and enhancement of the environment.

4. Improved Materials Utilization and Durability

Materials R&D is concerned with expanding the knowledge and technology of materials properties, using available natural materials, improving structural foundation materials, recycling highway materials, converting industrial wastes into useful highway products, developing extender or substitute materials for those in short supply, and developing more rapid and reliable testing procedures. The goals are lower highway construction costs and extended maintenance-free operation.

5. Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

Structural R&D is concerned with furthering the latest technological advances in structural and hydraulic designs, fabrication processes, and construction techniques to provide safe, efficient highways at reasonable costs.

6. Improved Technology for Highway Construction

This category is concerned with the research, development, and implementation of highway construction technology to increase productivity, reduce energy consumption, conserve dwindling resources, and reduce costs while improving the quality and methods of construction.

7. Improved Technology for Highway Maintenance

This category addresses problems in preserving the Nation's highways and includes activities in physical maintenance, traffic services, management, and equipment. The goal is to maximize operational efficiency and safety to the traveling public while conserving resources.

0. Other New Studies

This category, not included in the seven-volume official statement of the FCP, is concerned with HP&R and NCHRP studies not specifically related to FCP projects. These studies involve R&D support of other FHWA program office research.

* The complete seven-volume official statement of the FCP is available from the National Technical Information Service, Springfield, Va. 22161. Single copies of the introductory volume are available without charge from Program Analysis (HRD-3), Offices of Research and Development, Federal Highway Administration, Washington, D.C. 20590.

HDV-14/12-81(250)