UPDATING BEARING CAPACITY – SPT GRAPHS

Report Submitted

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M. Sherif Aggour

Civil and Environmental Engineering Department
University of Maryland
College Park, Maryland
20742

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16. Abstract

The graphs used by the Maryland State Highway Administration (MD SHA) for determining the allowable bearing capacity of shallow foundations related the Standard Penetration Test (SPT) in blows/ft to the maximum advisable presumptive bearing value. The major drawbacks to the use of these graphs are that they do not consider the effects of the depth, shape and the size of the footing, the location of the water table, and the factor of safety. The graphs are very conservative, which suggests that they do not provide accurate, cost effective designs, and in the case of a high water table they provide unsafe values, however, their main advantage is that they are simple to use.

In this study, new graphs were developed to determine the bearing capacity of shallow foundations as a function of the SPT N-value. The new graphs consider the effect of depth, size and shape of the footing, type of soil, factor of safety and the location of the water table. The new graphs are based on the American Association of State Highway and Transportation Officials, (AASHTO) bearing capacity equations.

Although the goal of the present study is to produce charts providing quick estimates of the bearing capacity and not the values to be used in final design, one should not forget that settlement is a controlling mechanism in foundation design and was not addressed within the scope of this project.

17. Key Words Standard Penetration test (SPT), bearing capacity, granular soils, cohesive soils, AASHTO	18. Distribution Statement No restrictions.		
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SUMMARY

The graphs used by the Maryland State Highway Administration (MD SHA) for determining the allowable bearing capacity of shallow foundations are presented in Policy and Procedure Memorandum Memo No. D-79-18-(4), March 29, 1979. The graphs were originally published in Basic Soils Engineering by B.K. Hough, published by the Roland Press Company, 1957. The graphs related the Standard Penetration Test (SPT) in blows/ft to the maximum advisable presumptive bearing value. The major drawbacks to the use of these graphs are that they do not consider the effects of the depth, shape and the size of the footing, the location of the water table, and the factor of safety. The graphs are very conservative, which suggests that they do not provide accurate, cost effective designs, and in the case of a high water table they provide unsafe values, however, their main advantage is that they are simple to use.

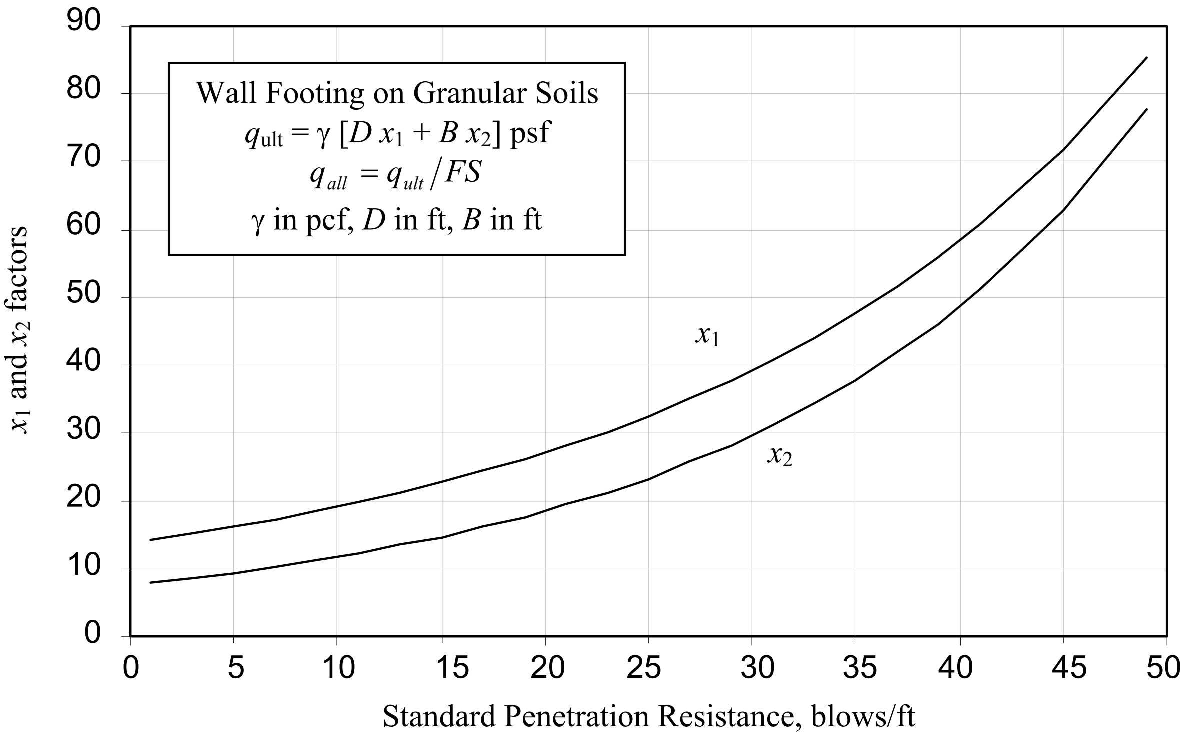
In this study, new graphs were developed to determine the bearing capacity of shallow foundations as a function of the SPT N-value. The new graphs consider the effect of depth, size and shape of the footing, type of soil, factor of safety and the location of the water table. The new graphs are based on the American Association of State Highway and Transportation Officials (AASHTO) bearing capacity equations.

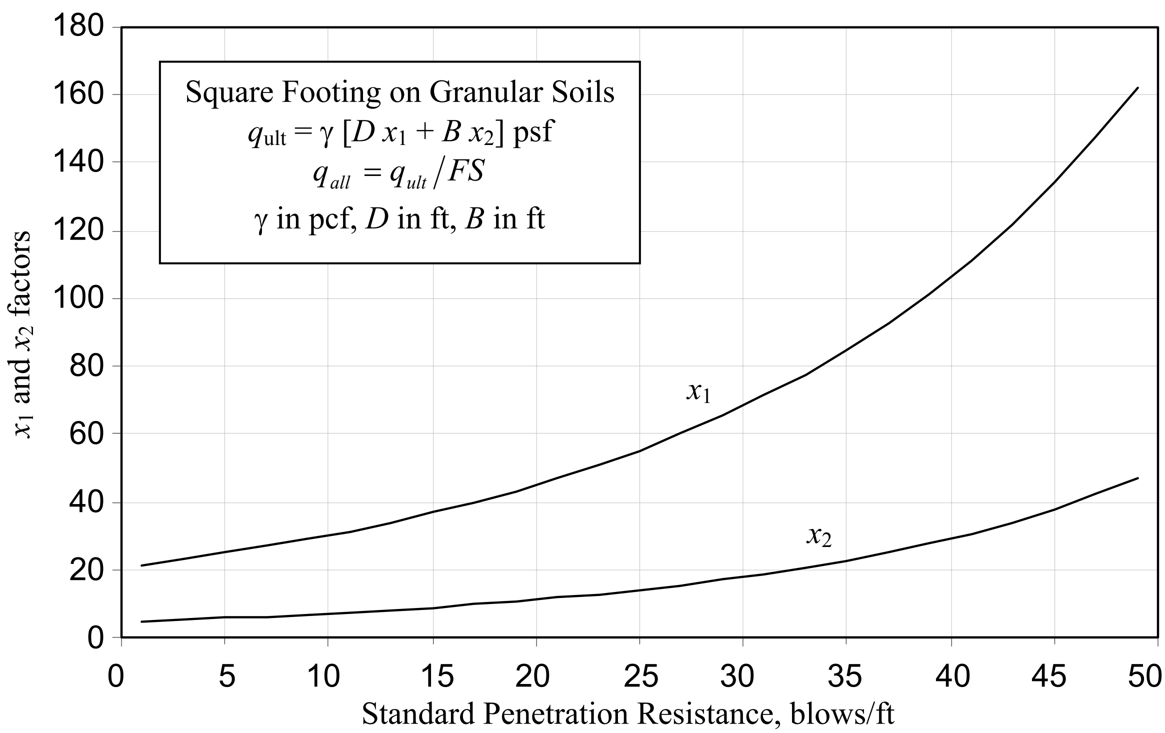
If laboratory testing is performed on the bearing soils, and engineering properties such as unit weight, shear strength, compressive strength, etc, are determined, the bearing capacity can be determined using AASHTO bearing capacity equations, which are included in the report. Laboratory testing is strongly recommended for cohesive soils.

If no shearing strength testing is performed, the soil strength parameters for granular soils can be estimated from knowledge of the SPT N-value and the soil strength parameters for cohesive soils can be estimated from both the soil index parameters and the SPT N-value. Using

these estimations the AASHTO equations were then used to establish a correlation between the bearing capacity and the N-value. These correlations are presented in both table form and graph form. By knowing the SPT N-value, the width of the footing *B*, and the depth of the foundation *D*, the engineer will be able to come up with a quick estimation of the ultimate bearing capacity by using the graphs or the tables. These values are not to be used in final design.

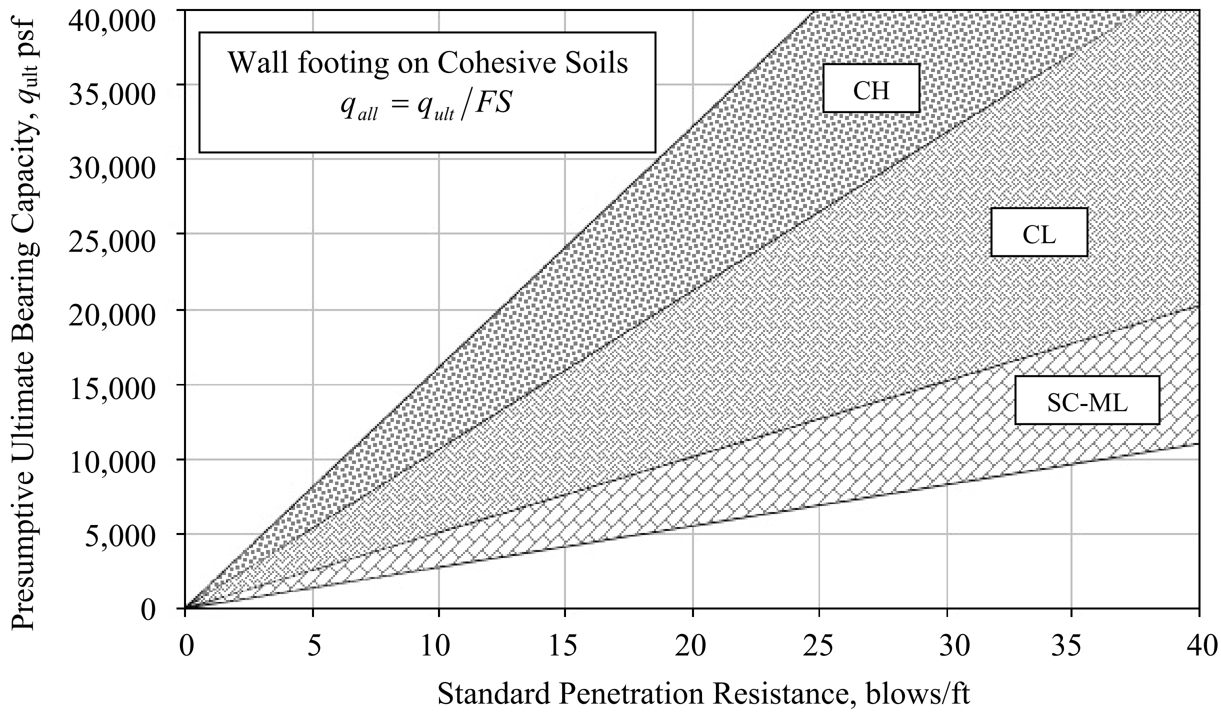
The SPT should be used with discreet judgment when it is used to estimate the bearing capacity of cohesive soils since silt and clay may be stiffened or softened depending on an increase or decrease of their moisture contents. Although the goal of the present study is to produce charts providing quick estimates of the bearing capacity, one should not forget that settlement is a controlling mechanism in foundation design and was not addressed within the scope of this project.

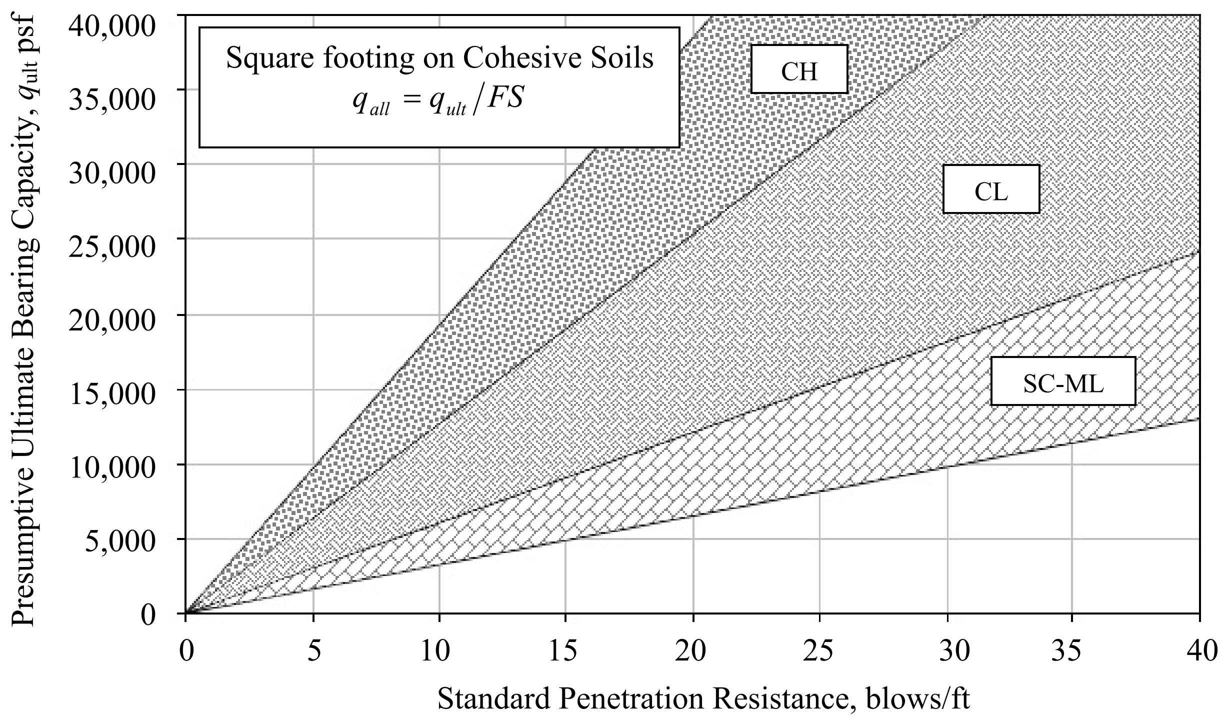




Note: Settlement was not considered in the graphs.

Bearing Capacity for a Footing on Granular Soils





Note: Settlement was not considered in the graphs.

Bearing Capacity for a Footing on Cohesive Soils

x_1 and x_2 Factors as a Function of SPT N-value for a Wall Footing

N	x_1	x_2		N	x_1	x_2
2	14.11	7.91		28	35.08	25.67
4	15.06	8.62		30	37.75	28.15
6	16.09	9.41		32	40.85	31.12
8	17.22	10.28		34	44.12	34.28
10	18.40	11.20		36	47.73	37.83
12	19.74	12.28		38	51.74	41.86
14	21.14	13.42		40	55.96	46.12
16	22.67	14.69		42	60.90	51.27
18	24.34	16.10		44	66.14	56.79
20	26.09	17.59		46	71.96	63.03
22	28.10	19.36		48	78.49	70.18
24	30.21	21.23		50	85.38	77.77
26	32.53	23.32				

x_1 and x_2 Factors as a Function of SPT N-value for a Square Footing

N	x_1	x_2		N	x_1	x_2
2	21.49	4.75		28	60.02	15.40
4	23.13	5.17		30	65.19	16.89
6	24.94	5.65		32	71.20	18.67
8	26.91	6.17		34	77.61	20.57
10	29.02	6.72		36	84.77	22.70
12	31.41	7.37		38	92.77	25.12
14	33.95	8.05		40	101.23	27.68
16	36.73	8.81		42	111.26	30.77
18	39.79	9.66		44	122.03	34.07
20	43.02	10.56		46	134.06	37.82
22	46.76	11.61		48	147.72	42.11
24	50.72	12.74		50	162.22	46.67
26	55.14	13.99				

Ultimate Bearing Capacity as a Function of SPT N-value for a Wall Footing

Soil Type	Range of q_{ult} in psf	Average of q_{ult} in psf
Clays of High Plasticity (CH)	(1059 to 1613) N	1336 N
Clays of Medium Plasticity (CL)	(505 to 1059) N	782 N
Clays of Low Plasticity and Clayey Silt (SC-ML)	(275 to 505) N	390 N

Ultimate Bearing Capacity as a Function of SPT N-value for a Square Footing

Soil Type	Range of q_{ult} in psf	Average of q_{ult} in psf
Clay of High Plasticity (CH)	(1265 to 1927) N	1596 N
Clay of Medium Plasticity (CL)	(603 to 1265) N	934 N
Clay of Low plasticity and Clayey silt (SC-ML)	(329 to 603) N	466 N

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CHAPTER I

INTRODUCTION

1.1 General Overview

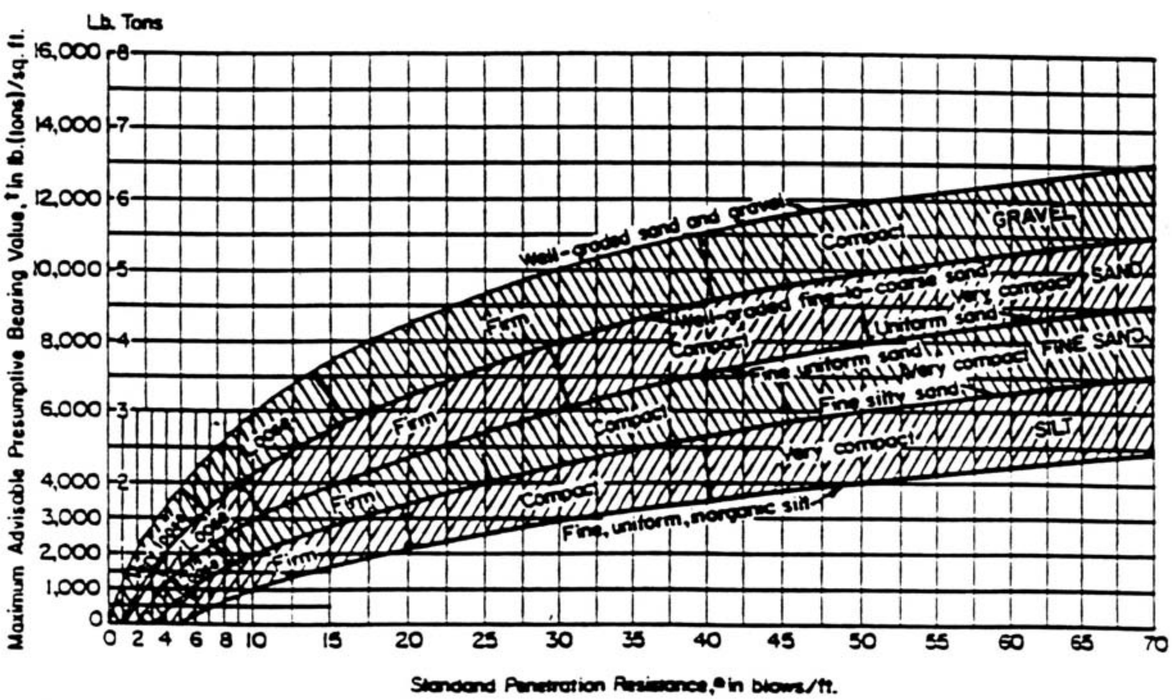
The graphs used by the Maryland State Highway Administration (MD SHA) for determining the allowable bearing capacity of shallow foundations are presented in Policy and Procedure Memorandum Memo No. D-79-18-(4), March 29, 1979. The graphs were originally published in Basic Soils Engineering by B.K. Hough, published by the Roland Press Company, 1957. The graphs, Fig. 1.1 and Fig 1.2, related the Standard Penetration Test (SPT) in blows/ft to the maximum advisable presumptive bearing value. The major drawbacks to the use of these graphs are that they do not consider the effects of the depth, shape and the size of the footing, the location of the water table, or the factor of safety considered. They are also very conservative, which suggests that they do not provide accurate, cost effective designs, and in the case of a high water table they provide unsafe values, however, their main advantage is that they are simple to use.

1.2 Objective of the Study

The objective of the study is to develop new graphs that relate the SPT N-value to the bearing pressure. The new graphs are to use the American Association of State Highway and Transportation Officials (AASHTO) bearing capacity equations. The new graphs will thus consider the effects of the depth, shape and size of the footing, soil properties, factor of safety, and location of the water table. The challenge here is to have the new graphs as simple to use as the current ones.

Maryland Department of Transportation STATE HIGHWAY ADMINISTRATION DIVISION OF BRIDGE DEVELOPMENT POLICY AND PROCEDURE MEMORANDUM	DESIGN MEMO NO. D-79-18(4)
SUBJECT: FOUNDATION BEARING VALUES	DATE: March 29, 1979 REVISED: April 16, 1984 SHEET 2 OF 3

DIRECTIVE (Continued)



* Number of blows of 140-lb. pin-guided weight falling 30 in. per blow required to drive a split-barrel spoon with a 2-in. outside diameter 12 in.

† Values must be corrected for effect of weak substrata, high ground water, and surcharge.

Presumptive bearing values, granular soils.

Fig. 1.1 Maryland SHA graphs for granular soils

Maryland Department of Transportation
STATE HIGHWAY ADMINISTRATION
DIVISION OF BRIDGE DEVELOPMENT

POLICY AND PROCEDURE MEMORANDUM

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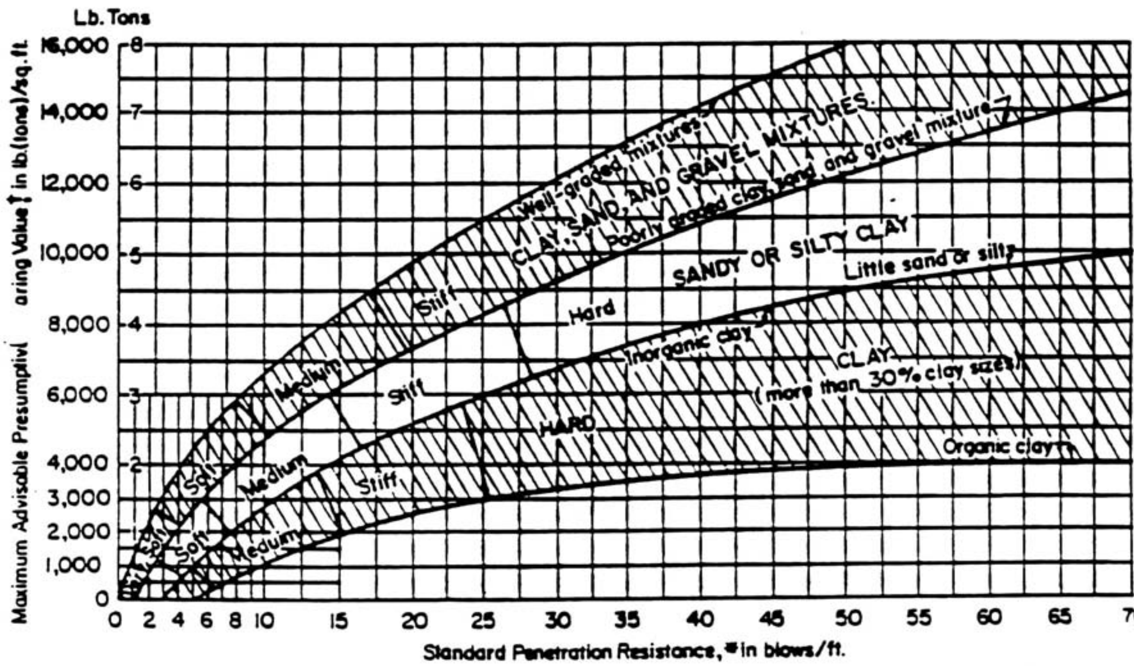


*Number of blows of 140-lb. pin-guided weight falling 30 in. per blow required to drive a split-barrel sample spoon with a 2-in. outside diameter 12 in.

† Higher values may be used for precompressed (or compacted) clays of low sensitivity than for normally loaded or extra sensitive clays.

Presumptive bearing value, clays and mixed soils.

Fig. 1.2 Maryland SHA graphs for clay and mixed soils

Prior to developing the graphs, the correlation between the SPT N-Value and the soil strength parameters based on current research were developed. This is because soil strength parameters are the needed input into the bearing capacity equation.

1.3 Organization of the Report

This report is divided into six chapters. Chapter II presents the relationships between the soil properties and the SPT N values. Chapter III summarizes the available bearing capacity equations, and Chapter IV presents the development of the new graphs. Chapter V shows the application of both the current and new graphs in several examples, and Chapter VI presents the conclusion of the study.

CHAPTER II

SOIL PROPERTIES FROM SPT VALUES

2.1 Overview

To determine the bearing capacity using the bearing capacity equations, the soil properties should be known. Laboratory testing should be performed as necessary to determine engineering properties including unit weight, shear strength, compressive strength and comparisonability. If laboratory testing is not performed, the soil strength parameters for both granular and cohesive soils as a function of the SPT N-value will be needed. In this chapter the correlation between the SPT N-value and the soil strength parameters is presented. As discussed by McGregor and Duncan (1998) the existing correlations generally use the uncorrected SPT blow count, N. However, hammers delivering 60% of the theoretical energy have been the most commonly used hammers for SPT tests, and it seems likely that the data on which these correlations were based was obtained primarily from tests with such hammers. It, therefore, seems logical to use N_{60} with these correlations.

2.2 SPT N-Value Correction

The adoption of the 60% standard energy requires the SPT N-value obtained using any hammer to be corrected. The correction is done in accordance with the following equation:

$$N_{60} = N_f \cdot \left(ER_f / 60\right)$$

where

N_{60} = SPT N-value corrected to 60% of the theoretical free fall hammer energy

N_f = SPT N-value obtained in the field

ER_f = Energy ratio for hammer used in the investigation (measured)

2.3 Friction Angle of Granular Soils

The SPT can be used to estimate the in-situ angle of internal friction ϕ for granular soils. The SPT test is commonly used to estimate the properties of cohesionless soils due to the difficulty in obtaining undisturbed samples. It should be stated that the SPT number may be misleading if large-size gravel is wedged into the split spoon sampler, resulting in apparently high N-values.

The angle of friction of granular soils, φ, has been correlated to the standard penetration number. Peck, Hanson, and Thornburn (1953) gave a correlation between N and φ in a graphical form, Fig 2.1, which can be approximated as (Wolff, 1989)

$$\varphi^\circ = 27.1 + 0.3N - 0.00054N^2 \tag{2.1}$$

In Japan the "Road Bridge Specifications" (Shioi and Fukui 1982) suggests for $N > 5$,

$$\varphi = (15N)^{1/2} + 15 \tag{2.2}$$

and the "Design Standards for Structures" (Shioi and Fukui, 1982):

$$\varphi = 0.3N + 27^\circ \tag{2.3}$$

Table 2.1 shows the values of φ as a function of the SPT N-value. The values from equation 2.3, are very close to the values of equation 2.1. Thus, equation 2.3 is recommended in this study as a good linear approximation of the relationship between the angle φ and the blow count N.

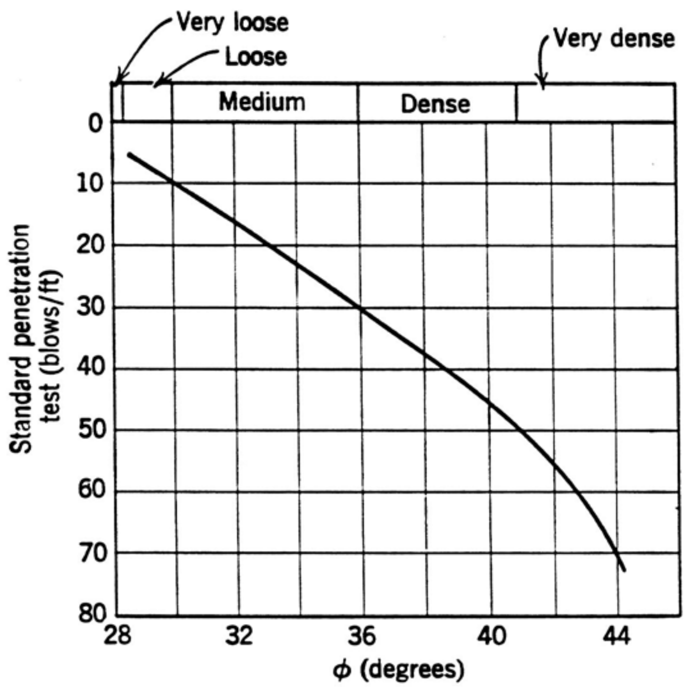


Fig. 2.1 Correlation between friction angle and SPT N-value (from Peck, Hanson, and Thornburn, 1953)

TABLE 2.1 Angle of Friction from Different Equations

N_{60}	Eq. 2.1	Eq. 2.2	Eq. 2.3
2	27.70		27.60
4	28.29		28.20
6	28.88	24.49	28.80
8	29.46	25.95	29.40
10	30.15	27.25	30.00
12	30.62	28.42	30.60
14	31.19	29.49	31.20
16	31.76	30.49	31.80
18	32.33	31.43	32.40
20	32.88	32.32	33.00
22	33.44	33.16	33.60
24	33.98	33.97	34.20
26	34.53	34.75	34.80
28	35.08	35.49	35.40
30	35.61	36.21	36.00
32	36.15	36.91	36.60
34	36.08	37.58	37.20
36	37.20	38.24	37.80
38	37.72	38.87	38.40
40	38.24	39.49	39.00
42	38.75	40.09	39.60
44	39.25	40.69	40.20
46	39.76	41.27	40.80
48	40.26	41.83	41.40
50	40.75	42.39	42.00

2.4 Cohesion of Cohesive Soils

The SPT N-value for a given clay may vary significantly with seasonal fluctuations in the water table. Thus, the values may fall short of providing information on the characteristics of the clay, mainly its strength. There are correlations that estimate the undrained shear strength of clay as a function of the SPT N-value. These correlations are not as meaningful for sensitive and medium to soft clays where effects of disturbance during sampler penetration may cause a lowering in the SPT N-value.

The Japanese "Road Bridge Specifications" (Shioi and Fukui, 1982) offer a correlation between the cohesion, c, and the SPT N-value for cohesive soils:

$$c = (0.061 \text{ to } 0.102)N \text{ tsf}$$

Sowers (1979) presented the relationship between the SPT N-value and the underained shear strength S_u that is shown in Fig. 2.2. The relationship can be represented by:

For clays with high plasticity:

$$S_u = (0.102 \text{ to } 0.179)N \text{ tsf}$$

For clays with medium plasticity:

$$S_u = (0.051 \text{ to } 0.102)N \text{ tsf}$$

For clays of low plasticity and clayey silts:

$$S_u = (0.026 \text{ to } 0.051)N \text{ tsf}$$

In Fig. 2.3 the NAVFAC, 1982 relationships between the SPT N-value and the unconfined compressive strength are presented. They can be summarized as:

An average relationship for all clays by Terzaghi and Peck: $c = 0.066N$ tsf

For clay of high plasticity, Sowers,

$$c = 0.13N \text{ tsf}$$

For clays of Medium plasticity, Sowers,

$$c = 0.076N \text{ tsf}$$

For clays of low plasticity and clayey silts, Sowers,

$$c = 0.038N \text{ tsf}$$

To add further insight into the correlation between the N-value and the shear strength of fine grained soil, a study by Henmueller (2001) was undertaken and is presented in appendix B.

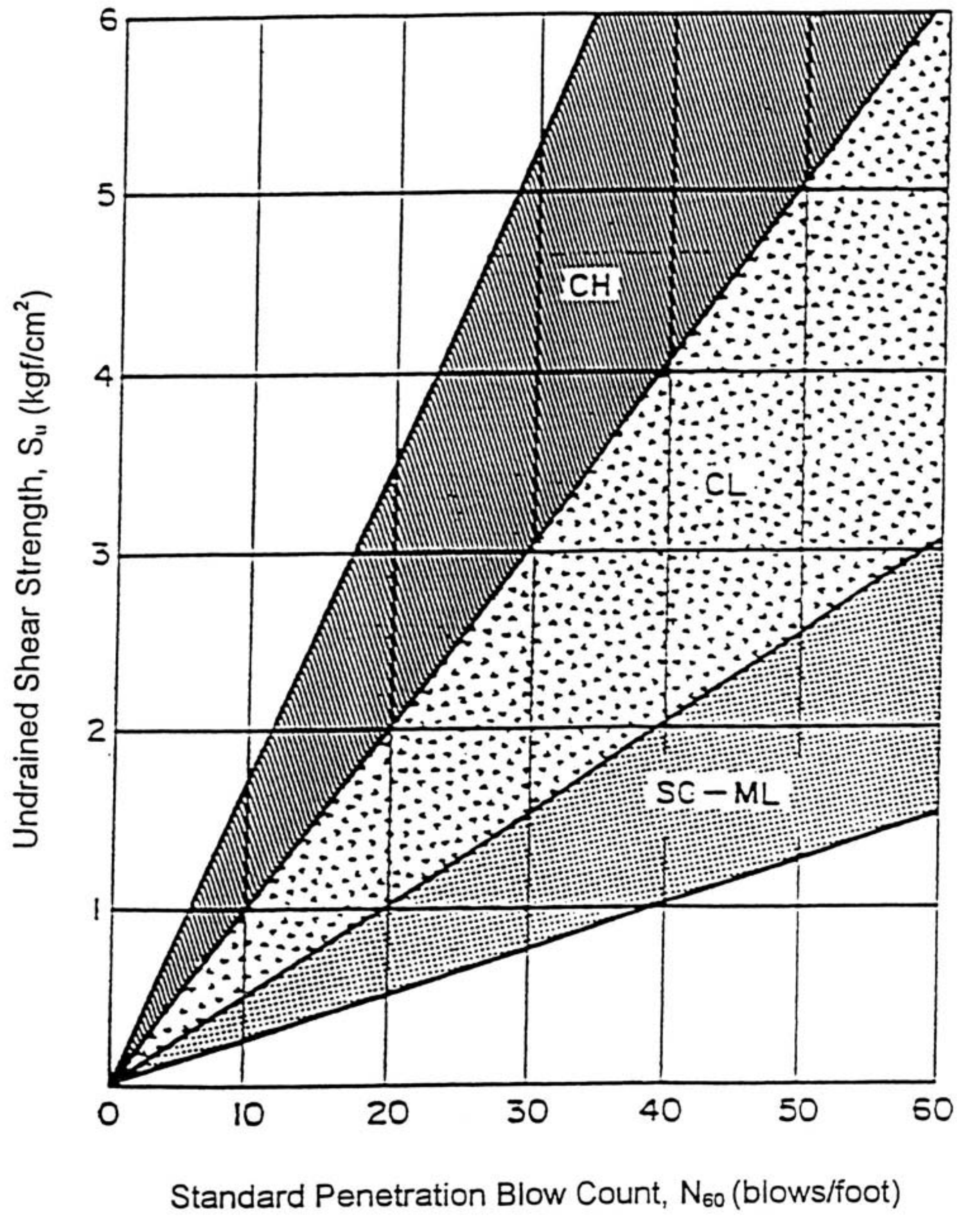


Fig. 2.2 Relationship between SPT N-value and the undrained shear strength (Sowers, 1979)

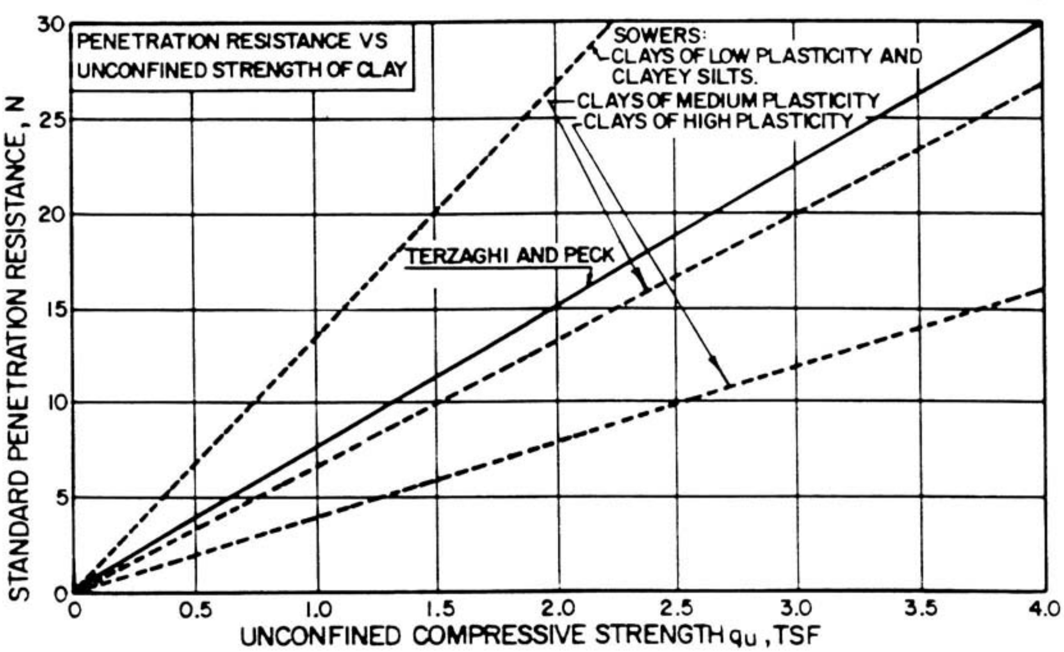


Fig. 2.3 Relationship between SPT N-value and Unconfined Compressive strength (from NAVFAC 1982)

In the study, soil borings used in the design of the metro subway system were used for making comparisons between SPT N-values and the shear strength of fine grained soils. The study concluded that there is a correlation between the SPT N-value and the shear strength of fine grained soils. However, sensitive clays indicate less correlation because of their dependence on moisture content. From the above information, it is concluded that if no test was performed on the cohesive soils, the following relationships in Table 2.2 could be used:

TABLE 2.2 Relationships Between Shear Strength and N-values for Cohesive Soils

Type of Clay	Cohesion (tsf)
High plasticity (CH)	$c = 0.13\ N$
Medium plasticity (CL)	$c = 0.076\ N$
Low plasticity and clayey silt (SC-ML)	$c = 0.038\ N$

CHAPTER III

AVAILABLE BEARING CAPACITY EQUATIONS

Due to extensive research in the topic of bearing capacity, numerous methods of analysis have been developed. The research started by Terzaghi (1943) and was followed by Skempton (1951), Meyerhof (1951), Hansen (1961), De Beer and Ladanyi (1961), Meyerhof (1963), Hansen (1970), Vesic (1973, 1975), and others. The most popular and widely used bearing capacity equations in practice today are the Terzaghi and AASHTO equations. The following is a short description of each.

3.1 Terzaghi's Bearing Capacity Equations

The Terzaghi (1943) set of equations were the first to be proposed. They have been very widely used since then and continue to be in great use mainly because of their relative simplicity. Terzaghi used trial wedges of the type assumed by Prandtl (1921), expanding and improving on Prandtl's theory. The expressions of bearing capacity obtained by Terzaghi are:

Long footings:

$$q_{ult} = cN_c + \gamma DN_q + 0.5\gamma BN_\gamma \qquad (3.1)$$

Square footings:

$$q_{ult} = 1.3cN_c + \gamma DN_q + 0.4\gamma BN_\gamma \qquad (3.2)$$

Circular footings:

$$q_{ult} = 1.3cN_c + \gamma DN_q + 0.3\gamma BN_\gamma \qquad (3.3)$$

where:

c = cohesion of soil

γ = unit weight of soil

D = depth of foundation

B = width of foundation (diameter for a circular foundation)

N_c, N_q, N_γ = bearing capacity factors that are nondimensional and are only functions of the soil friction angle, ϕ. These factors are shown in Table 3.1.

TABLE 3.1 Terzaghi's Bearing Capacity Factors

ϕ	N_c	N_q	N_γ	ϕ	N_c	N_q	N_γ
0	5.70	1.00	0.00	26	27.09	14.21	9.84
1	6.00	1.10	0.01	27	29.24	15.90	11.60
2	6.30	1.22	0.04	28	31.61	17.81	13.70
3	6.62	1.35	0.06	29	34.24	19.98	16.18
4	6.97	1.49	0.10	30	37.16	22.46	19.13
5	7.34	1.64	0.14	31	40.41	25.28	22.65
6	7.73	1.81	0.20	32	44.04	28.52	26.87
7	8.15	2.00	0.27	33	48.09	32.23	31.94
8	8.60	2.21	0.35	34	52.64	36.50	38.04
9	9.09	2.44	0.44	35	57.75	41.44	45.41
10	9.61	2.69	0.56	36	63.53	47.16	54.36
11	10.16	2.98	0.69	37	70.01	53.80	65.27
12	10.76	3.29	0.85	38	77.50	61.55	78.61
13	11.41	3.63	1.04	39	85.97	70.61	95.03
14	12.11	4.02	1.26	40	95.66	81.27	115.31
15	12.86	4.45	1.52	41	106.81	93.85	140.51
16	13.68	4.92	1.82	42	119.67	108.75	171.99
17	14.60	5.45	2.18	43	134.58	126.50	211.56
18	15.12	6.04	2.59	44	151.95	147.74	261.60
19	16.56	6.70	3.07	45	172.28	173.28	325.34
20	17.69	7.44	3.64	46	196.22	204.19	407.11
21	18.92	8.26	4.31	47	224.55	241.80	512.84
22	20.27	9.19	5.09	48	258.28	287.85	650.67
23	21.75	10.23	6.00	49	298.71	344.63	831.99
24	23.36	11.40	7.08	50	347.50	415.14	1072.80
25	25.13	12.72	8.34				

3.2 AASHTO Bearing Capacity Equations

In AASHTO, section 4, Foundations, it states that foundations shall be designed to provide adequate structural capacity, and adequate foundation bearing capacity with acceptable settlements. According to AASHTO the ultimate bearing capacity may be estimated using the following relationship for continuous footing (i.e., L > 5B)

$$q_{ult} = cN_c + 0.5\gamma BN_\gamma + qN_q \qquad \text{(AASHTO 4.4.7.1-1)}$$

The allowable bearing capacity shall be determined as:

$$q_{all} = q_{ult} / FS \qquad \text{(AASHTO 4.4.7.1-2)}$$

the modified form of the general bearing capacity equation that accounts for the effects of footing shape, base inclination, and inclined loads is as follows:

$$q_{ult} = cN_c s_c b_c i_c + 0.5\gamma BN_\gamma s_\gamma b_\gamma i_\gamma + qN_q s_q b_q i_q \qquad \text{(AASHTO 4.4.7.1.1-1)}$$

where N_c, N_γ, and N_q are bearing capacity factors that are functions of the friction angle of the soil ϕ and are shown in Table 3.2; s_c, s_γ, and s_q are footing shape factors, i_c, i_γ, and i_q are inclined load factors; and b_c, b_γ, and b_q are inclined base factors; c is the soil cohesion; γ is the unit weight of soil below the footing base; B is the footing width; q is the surcharge load above the footing base, which is equal to γD where D is the footing depth and γ is the unit weight of the soil above the footing base. AASHTO expressions for shape, and load and base inclination factors are presented in Table 3.3.

AASHTO states that a minimum factor of safety (FS) of 3.0 against a bearing capacity failure should be used.

TABLE 3.2 Bearing Capacity Factors (AASHTO)

ϕ	N_c	N_q	N_γ	ϕ	N_c	N_q	N_γ
0	5.14	1.00	0.00	26	22.25	11.85	12.54
1	5.38	1.09	0.07	27	23.94	13.20	14.47
2	5.63	1.20	0.15	28	25.80	14.72	16.72
3	5.90	1.31	0.24	29	27.86	16.44	19.34
4	6.19	1.43	0.34	30	30.14	18.40	22.40
5	6.49	1.57	0.45	31	32.67	20.63	25.99
6	6.81	1.72	0.57	32	35.49	23.18	30.22
7	7.16	1.88	0.71	33	38.64	26.09	35.19
8	7.53	2.06	0.86	34	42.16	29.44	41.06
9	7.92	2.25	1.03	35	46.12	33.30	48.03
10	8.35	2.47	1.22	36	50.59	37.75	56.31
11	8.80	2.71	1.44	37	55.63	42.92	66.19
12	9.28	2.97	1.69	38	61.35	48.93	78.03
13	9.81	3.26	1.97	39	67.87	55.96	92.25
14	10.37	3.59	2.29	40	75.31	64.20	109.41
15	10.98	3.94	2.65	41	83.86	73.90	130.22
16	11.63	4.34	3.06	42	93.71	85.38	155.55
17	12.34	4.77	3.53	43	105.11	99.02	186.54
18	13.10	5.26	4.07	44	118.37	115.31	224.64
19	13.93	5.80	4.68	45	133.88	134.88	271.76
20	14.83	6.40	5.39	46	152.10	158.51	330.35
21	15.82	7.07	6.20	47	173.64	187.21	403.67
22	16.88	7.82	7.13	48	199.26	222.31	496.01
23	18.05	8.66	8.20	49	229.93	265.51	613.16
24	19.32	9.60	9.44	50	266.89	319.07	762.89
25	20.72	10.66	10.88				

TABLE 3.3 AASHTO Expressions for Footing Shape, Load and Base Inclination Factors

Shape Factor	
For continuous footing (L > 5B):	$s_c = s_q = s_\gamma = 1$
For rectangular footing where L < 5B:	$s_c = 1 + (B/L)(N_q/N_c)$
	$s_q = 1 + (B/L)\tan\phi$
	$s_\gamma = 1 - 0.4(B/L)$
For circular footings:	$B = L$
Load Inclination Factors	
For $\phi > 0$: $i_c = i_q - [(1 - i_q)/N_c \tan\phi]$	
For $\phi = 0$: $i_c = 1 - (n\,P/B\,L\,c\,N_c)$	
$i_q = [1 - P/(Q + B\,L\,c\cot\phi)]^n$	
$i_\gamma = [1 - P/(Q + B\,L\,c\cot\phi)]^{(n+1)}$	
$n = [(2 + L/B)/(1 + L/B)]\cos^2\theta + [(2 + B/L)/(1 + B/L)]\sin^2\theta$	
P = applied shear load	
Q = applied normal load	
Base Inclination Factors	
$b_q = b_\gamma = (1 - \alpha\tan\phi)^2$	
For $\phi > 0$: $b_c = b_\gamma - (1 - b_\gamma)/N_c \tan\phi$	
For $\phi = 0$: $b_c = 1 - [2\alpha/(\pi + 2)]$	
α = base inclination angle	
However, footings with inclined bases are not recommended	

CHAPTER IV

DEVELOPMENT OF THE NEW GRAPHS

4.1 Graphs for Footing on Granular Soils

4.1.1 Wall Footing

The AASHTO equation:

$$q_{ult} = cN_c s_c b_c i_c + 0.5\gamma BN_\gamma s_\gamma b_\gamma i_\gamma + qN_q s_q b_q i_q$$

will have: i) $c = 0$ for granular soil; ii) $s_c = s_q = s_\gamma = 1$ for $L > 5B$; iii) $i_c = i_q = i_\gamma = 1$ for vertical load, and iv) $b_c = b_q = b_\gamma = 1$ for horizontal base, thus the AASHTO equation will be:

$$\begin{aligned} q_{ult} &= \gamma DN_q + 0.5\gamma BN_\gamma \\ &= \gamma\left[DN_q + 0.5N_\gamma B\right] \\ &= \gamma\left[Dx_1 + Bx_2\right] \end{aligned}$$

where $x_1 = N_q$ and $x_2 = 0.5\ N_\gamma$

Figure 4.1 and Table 4.1 show the values of x_1 and x_2 as a function of N_{60}. Appendix A-1 shows the determination of x_1 and x_2 as a function of φ.

4.1.2 Square Footing

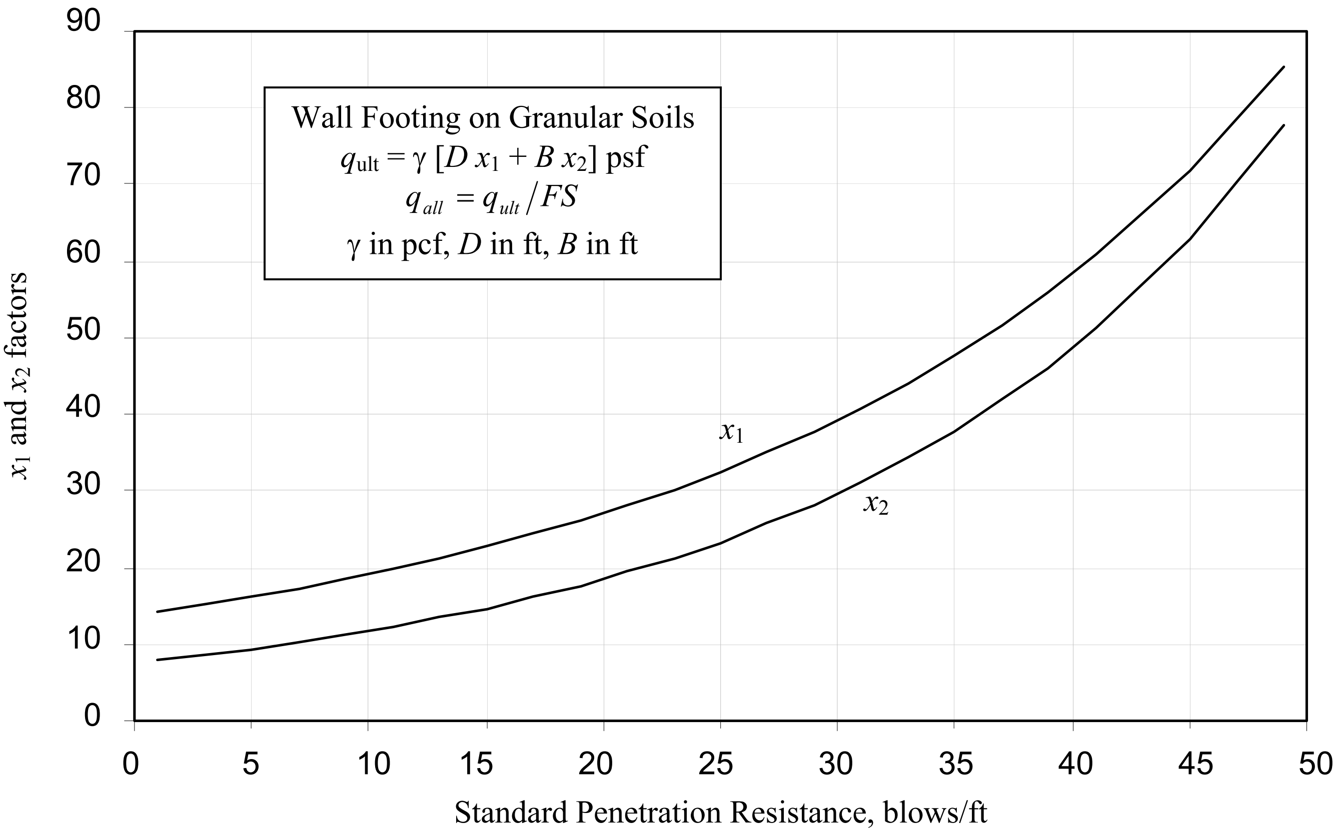
The AASHTO equation:

$$q_{ult} = cN_c s_c b_c i_c + 0.5\gamma BN_\gamma s_\gamma b_\gamma i_\gamma + qN_q s_q b_q i_q$$

will have: i) $c = 0$ for granular soil; ii) $s_q = 1 + \tan\varphi$, $s_\gamma = 0.6$; iii) $i_c = i_q = i_\gamma = 1$ for vertical load, and iv) $b_c = b_q = b_\gamma = 1$ for horizontal base, thus the AASHTO equation will be:

$$\begin{aligned} q_{ult} &= \gamma DN_q(1+\tan\phi) + 0.5\gamma BN_\gamma 0.6 \\ &= \gamma\left[D(1+\tan\phi)N_q + 0.3N_\gamma B\right] \\ &= \gamma\left[Dx_1 + Bx_2\right] \end{aligned}$$

where $x_1 = (1 + \tan\phi)\ N_q$ and $x_2 = 0.3\ N_\gamma$



Note: Settlement was not considered in the graph

Fig. 4.1 x_1 and x_2 factors as a function of N-value for a wall footing

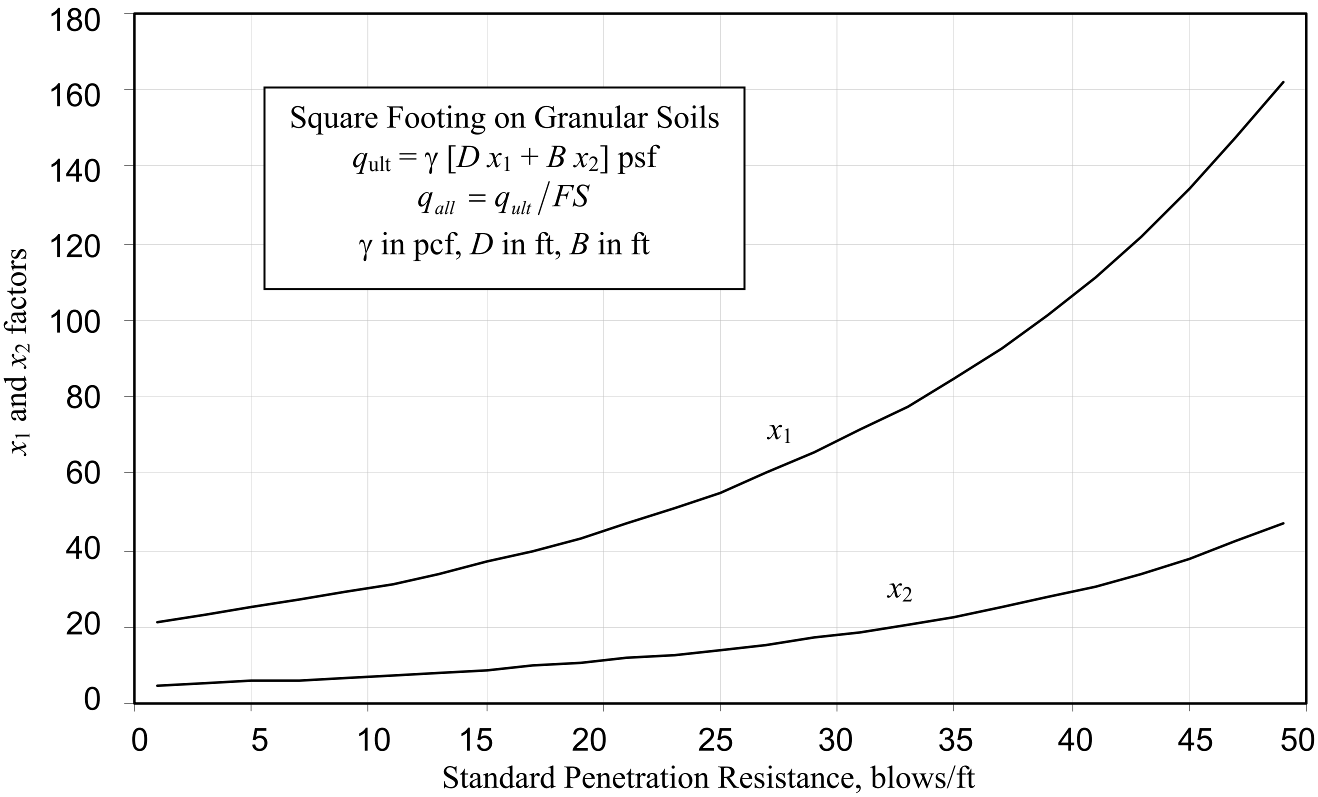
Figure 4.2 and Table 4.2 show the values of x_1 and x_2 as a function of N_{60}. Appendix A-2 shows the determination of x_1 and x_2 as a function of φ.

TABLE 4.1 x_1 and x_2 Factors as a Function of N_{60} for a Wall Footing

N_{60}	x_1	x_2		N_{60}	x_1	x_2
2	14.11	7.91		28	35.08	25.67
4	15.06	8.62		30	37.75	28.15
6	16.09	9.41		32	40.85	31.12
8	17.22	10.28		34	44.12	34.28
10	18.40	11.20		36	47.73	37.83
12	19.74	12.28		38	51.74	41.86
14	21.14	13.42		40	55.96	46.12
16	22.67	14.69		42	60.90	51.27
18	24.34	16.10		44	66.14	56.79
20	26.09	17.59		46	71.96	63.03
22	28.10	19.36		48	78.49	70.18
24	30.21	21.23		50	85.38	77.77
26	32.53	23.32				

TABLE 4.2 x_1 and x_2 Factors as a Function of N_{60} for a Square Footing

N_{60}	x_1	x_2		N_{60}	x_1	x_2
2	21.49	4.75		28	60.02	15.40
4	23.13	5.17		30	65.19	16.89
6	24.94	5.65		32	71.20	18.67
8	26.91	6.17		34	77.61	20.57
10	29.02	6.72		36	84.77	22.70
12	31.41	7.37		38	92.77	25.12
14	33.95	8.05		40	101.23	27.68
16	36.73	8.81		42	111.26	30.77
18	39.79	9.66		44	122.03	34.07
20	43.02	10.56		46	134.06	37.82
22	46.76	11.61		48	147.72	42.11
24	50.72	12.74		50	162.22	46.67
26	55.14	13.99				



Note: Settlement was not considered in the graph.

Fig. 4.2 x_1 and x_2 factors as a function of N-value for a square footing

4.2 Effect of Water Table

Ultimate bearing capacity should be determined using the highest anticipated ground water level at the footing location. Depending on the relative position of the water table level to the level of the base of the footing, three cases can be considered, case 1, depth of water table below the footing Z_w is larger than the footing width B as shown in Fig. 4.3.; case 2, depth of water table is smaller than B, and case 3, the water table is above the base of the footing.

The equation for bearing capacity is:

$$q_{ult} = \gamma[x_1 D + x_2 B]$$

can be written as

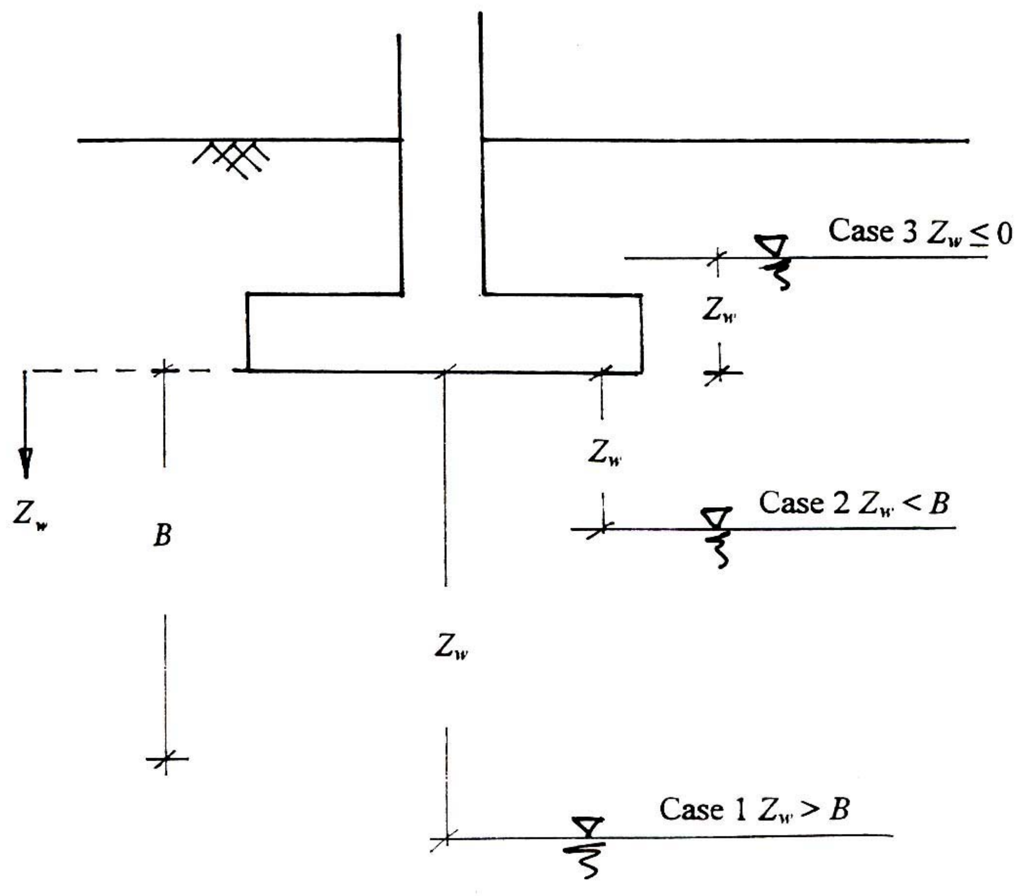


Fig. 4.3 Water Table Level

$$q_{ult} = \gamma_1 x_1 D + \gamma_2 x_2 B$$

where γ_1 represent the unit weight of soils above the footing base

and γ_2 represent the unit weight of soils below the footing base

Case 1 $Z_w > B$

For both γ_1 and γ_2 use the dry unit weight γ_d, water table has no effect on the bearing capacity.

Case 2 $Z_w < B$

For γ_1, use γ_d, and for γ_2 use

$$\gamma_2 = \gamma_b + \frac{Z_w}{B}(\gamma_d - \gamma_b)$$

where γ_b, buoyant unit weight of soils equal the saturated unit weight γ_{sat} minus the unit weight of water γ_w, i.e.,

$$\gamma_b = \gamma_{sat} - \gamma_w$$

Case 3 $Z_w \leq 0$

For γ_1 use γ_d up to the water table elevation, and γ_b up to the elevation of the footing base, and for γ_2 use γ_b.

4.3 Graphs for Footing on Cohesive Soils

4.3.1 Wall Footing

The AASHTO equation:

$$q_{ult} = cN_c s_c b_c i_c + 0.5\gamma B N_\gamma s_\gamma b_\gamma i_\gamma + qN_q s_q b_q i_q$$

will have: i) for $\varphi = 0$, $N_c = 5.14$, $N_q = 1.0$ and $N_\gamma = 0.0$

ii) $s_c = s_q = s_\gamma = 1$ for $L > 5B$

iii) $i_c = i_q = i_\gamma = 1$ for vertical load

and iv) $b_c = b_q = b_\gamma = 1$ for horizontal base thus the AASHTO equation will be:

$$q_{ult} = c5.14 + \gamma D$$

by neglecting γD, the equation will be

$$q_{ult} = 5.14c$$

For clay of high plasticity:

$$q_{ult} = 5.14(0.13N) = 0.668N \text{ tsf}$$

$$= 1336N \text{ psf}$$

For clay of medium plasticity:

$$q_{ult} = 5.14\ (0.076N)$$

$$= 0.391N \text{ tsf}$$

$$= 782N \text{ psf}$$

For clay of low plastic and clayey silts:

$$q_{ult} = 5.14\ (0.038N)$$

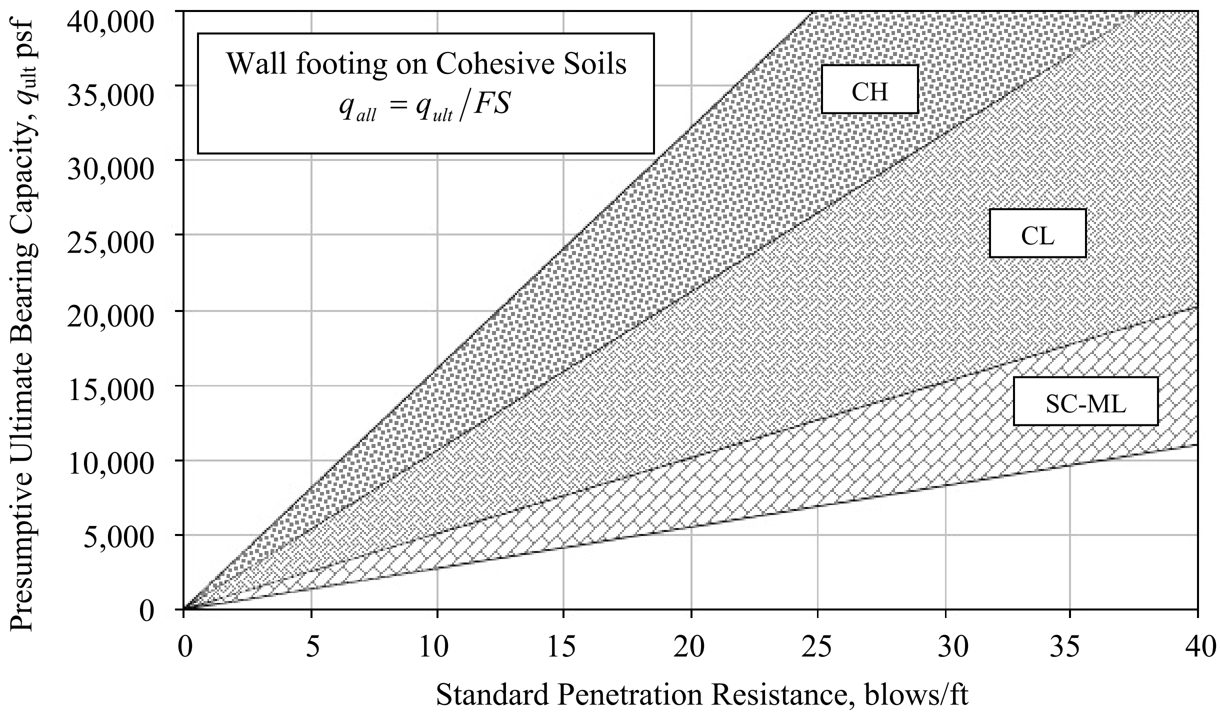
$$= 0.195N \text{ tsf}$$

$$= 390N \text{ psf}$$

Figure 4.4 and Table 4.3 show the ultimate bearing capacity as a function of SPT N-value for a wall footing.

TABLE 4.3 Ultimate Bearing Capacity as a Function of SPT N-value for a Wall Footing

Soil Type	Range of q_{ult} in psf	Average of q_{ult} in psf
Clays of High Plasticity (CH)	(1059 to 1613) N	1336 N
Clays of Medium Plasticity (CL)	(505 to 1059) N	782 N
Clays of Low Plasticity and Clayey Silt (SC-ML)	(275 to 505) N	390 N



Note: Settlement was not considered in the graph.

Fig. 4.4 Ultimate Bearing Capacity as a function of N-value for a wall footing

4.3.2 Square Footing

The AASHTO equation

$$q_{ult} = cN_c s_c b_c i_c + 0.5\gamma BN_\gamma s_\gamma b_\gamma i_\gamma + qN_q s_q b_q i_q$$

will have: i) for $\varphi = 0$, $N_c = 5.14$, $N_q = 1.0$ and $N_\gamma = 0.0$

ii) $s_c = 1 + \left(\dfrac{1.0}{5.14}\right) = 1.195$, $s_q = 1$, $s_\gamma = 1 - 0.4 = 0.6$

iii) $i_c = i_q = i_\gamma = 1$ for vertical load

and iv) $b_c = b_q = b_\gamma = 1$ for horizontal base

thus the AASHTO equation will be:

$$q_{ult} = c5.14(1.195) + \gamma D$$

$$q_{ult} = 6.14c + \gamma D$$

by neglecting γD, the equation will be

$$q_{ult} = 6.14c$$

For clay of high plasticity:

$$q_{ult} = 6.14(0.13N) = 0.798N \text{ tsf}$$

$$= 1596N \text{ psf}$$

For clay of medium plasticity:

$$q_{ult} = 6.14(0.076N) = 0.467N \text{ tsf}$$

$$= 934N \text{ psf}$$

For clay of low plasticity and clayey silts

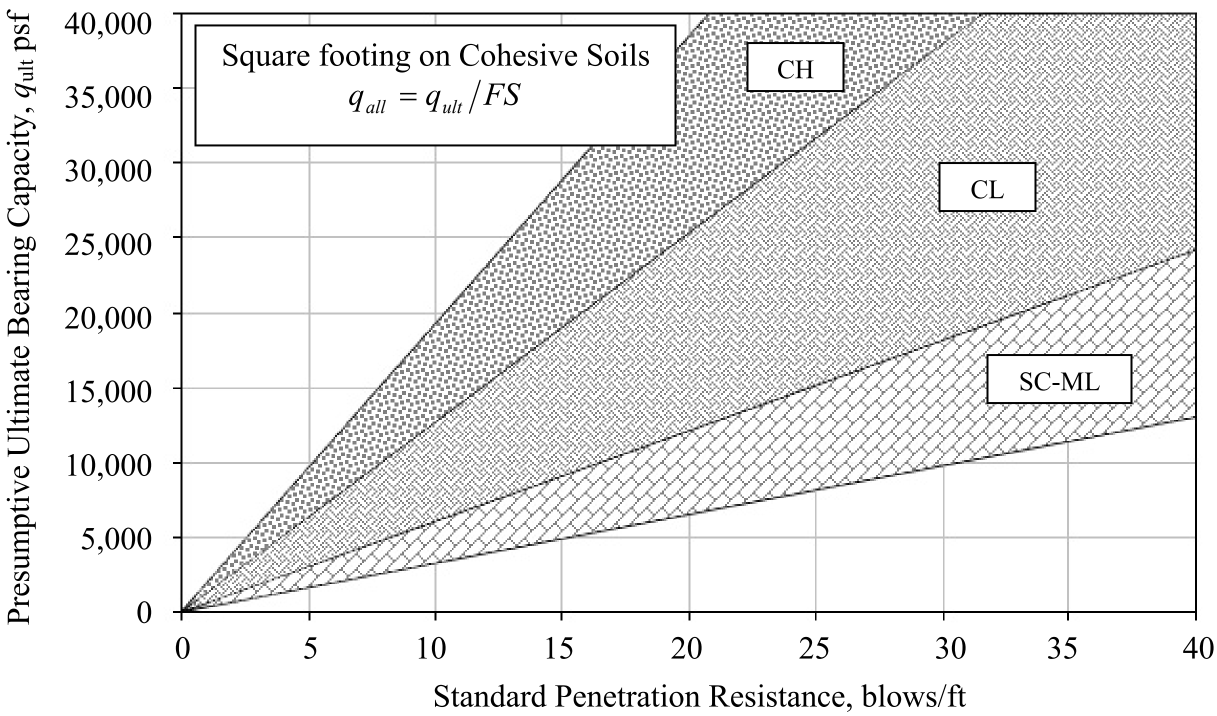
$$q_{ult} = 6.14(0.038N) = 0.233N \text{ tsf}$$

$$= \quad 466N \text{ psf}$$

Figure 4.5 and Table 4.4 show the ultimate bearing capacity as a function of SPT N-value for a square footing.

TABLE 4.4 Ultimate Bearing Capacity as a Function of SPT N-value for a Square Footing

Soil Type	Range of q_{ult} in psf	Average of q_{ult} in psf
Clay of High Plasticity (CH)	1265 N to 1927 N	1596 N
Clay of Medium Plasticity (CL)	603 N to 1265 N	934 N
Clay of Low plasticity and Clayey silt (SC-ML)	329 N to 603 N	466 N



Note: Settlement was not considered in the graph.

Fig. 4.5 Ultimate Bearing Capacity as a function of N-value for a square footing.

CHAPTER V

APPLICATIONS OF CURRENT AND NEW GRAPHS

To show the difference between bearing capacity values determined using the State graphs and the new graphs that are based on AASHTO equations, the following examples are presented.

The examples will assume that a footing for a retaining wall rests on level ground and is subjected to vertical loading. For the retaining wall, the footing can be considered continuous with its length L larger than its width B (at least L > 5B).

Example 1

For the footing of a retaining wall, calculate the allowable bearing capacity if the footing width is *B* ft and its base rests 3 ft below the ground surface. Assume the soil below the footing is a uniform fine sand with an average value of standard penetration resistance (blows/ft) of 6.

Solution:

a) Using the MD, SHA charts (D-79-18(4)), (Fig. 1.1) the allowable bearing capacity for the uniform fine sand with 6 blows/ft is 2000 lb/ft^2.

b) Using the new graphs, the allowable bearing capacity is a function of the width *B*, hence, it will be calculated in this example for B = 2, 4, 6, 8 ft. Assume also that γ = 110 lb/ft^3.

For a Wall Footing:

$$q_{ult} = \gamma\left[Dx_1 + Bx_2\right]$$

From Table 4.1, for N = 6, x_1 = 16.09 and x_2 =9.41.

Thus: $q_{ult} = 110\left[D \times 16.09 + B \times 9.41\right]$

For B = 2 ft and D = 3 ft

$$q_{ult} = 110\left[3 \times 16.09 + 2 \times 9.41\right] = 7380 \text{ psf}$$

For a factor of safety of 3, the allowable q is

$$q_{all} = \frac{7380}{3} = 2460 \text{ lb/ft}^2$$

The computations for the other widths use the same equation, with the following results:

B (ft)	q_{ult} (psf)	q_{all} (psf)
2	7380	2460
4	9450	3150
6	11520	3840
8	13590	4530

Figure 5.1 shows the allowable bearing pressure as a function of the width B plotted using the state graphs and using the new graphs. As can be seen, the bearing pressure from the State graphs is very conservative. The result of this is that their use will not provide a cost effective design.

<u>Example 2</u>

In Example 1, a low blow count was used for the sand. In this example, all parameters are the same as in Example 1, except the blow count is 30.

Solution:

a) Using the MD, SHA chart, (D-79-18(4)), the allowable bearing capacity for the uniform fine sand with 30 blows/ft is 6000 lb/ft^2.

b) Using the new charts, for a blow count of 30, $x_1 = 37.75$ and $x_2 = 28.15$.

Then

$$q_{ult} = 110[D \times 37.75 + B \times 28.15]$$

For $B = 2$ ft

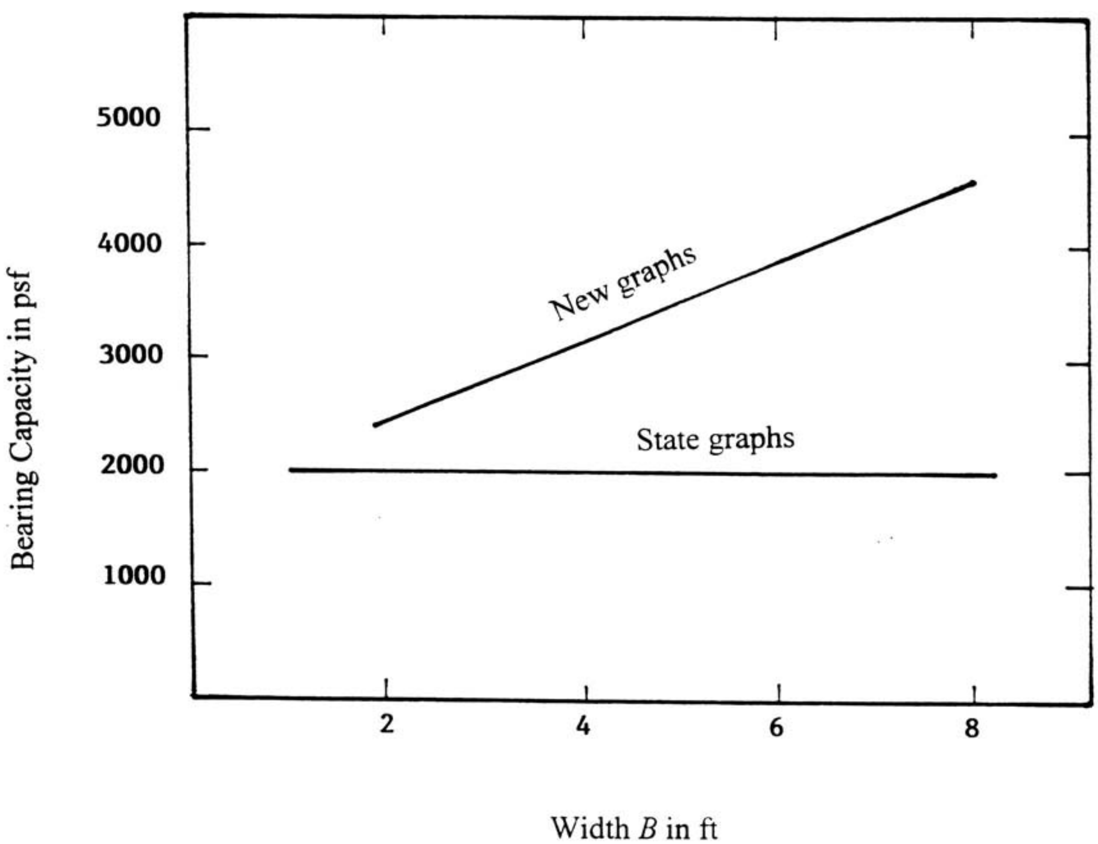


Fig. 5.1 Comparison between the current and new graphs for Example 1.

$$q_{ult} = 110[3 \times 37.75 + 2 \times 28.15] = 18650$$

For a factor of safety of 3

$$q_{all} = \frac{18,650}{3} = 6217 \text{ lb/ft}^2$$

The computations for the other widths are thus:

B (ft)	q_{ult} (psf)	q_{all} (pfs)
2	18,650	6,217
4	24,844	8,281
6	31,036	10,346
8	37,230	12,410

Figure 5.2 shows the allowable bearing pressure as a function of the width B plotted using the State graphs and using the new graphs. Again the State charts produce very conservative results.

Example 3

In this example, the footing of the retaining wall of Example 1, is now resting on clayey soils with an average blow count of 10 blows/ft.

Solution:

a) Using the MD, SHA charts (D-79-18(4)), (Fig. 1.2) the allowable bearing capacity will range from 1000 psf to 2800 psf.

b) Using the new graphs, the allowable bearing capacity is a function of the type of clay. If the clay is of medium plasticity, q_{all} for a factor of safety of 3 will vary from 1680 to 3913 psf. If the clay is of high plasticity, q_{all} will vary from 3913 to 5967 psf. Again the State charts produce very conservative results.

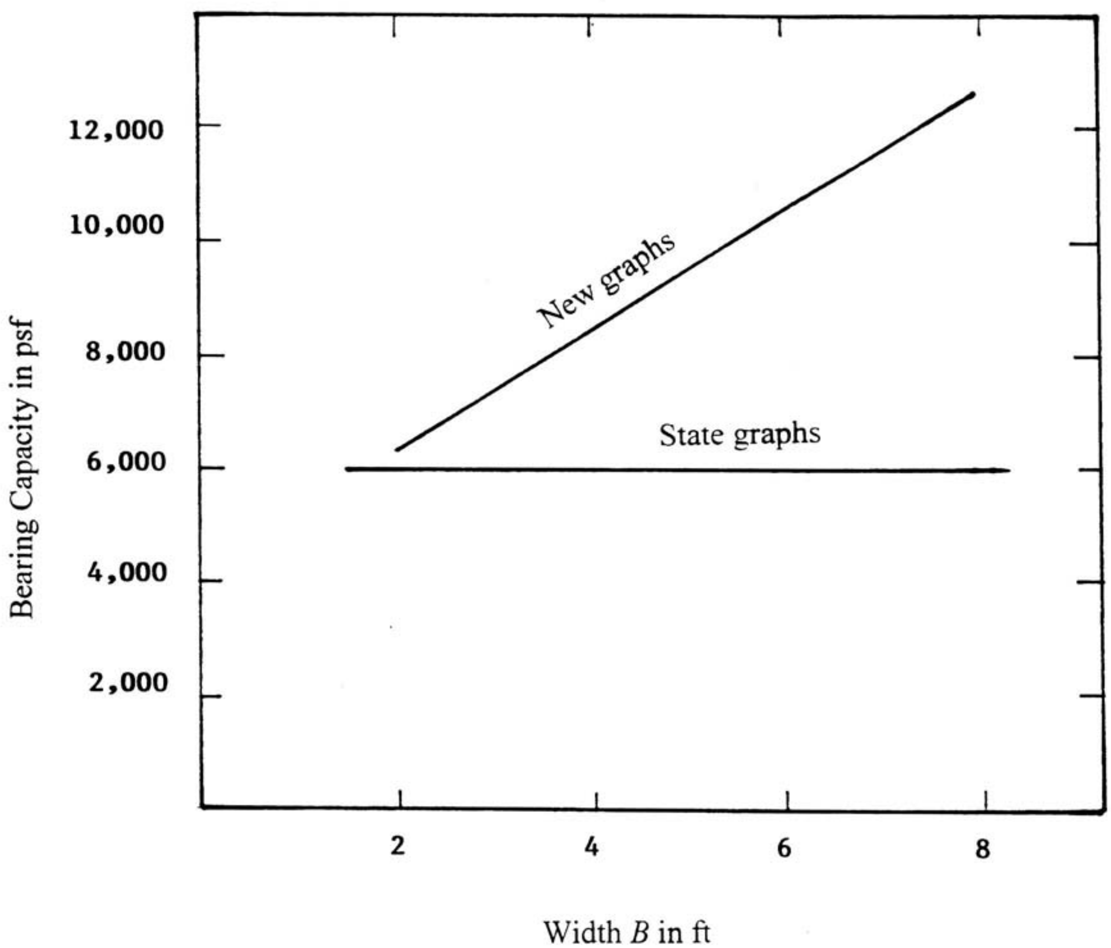


Fig. 5.2 Comparison between the current and new graphs for Example 2.

CHAPTER VI

CONCLUSIONS

In this study, new graphs were developed to replace MD SHA graphs for determining the allowable bearing capacity of shallow foundations, Policy and Procedures, Memorandum Memo No. D-79-18-(4), March 29, 1979, relating the SPT N-value to the maximum advisable presumptive bearing value. The new graphs consider the effect of depth, size and shape of the footing, type of soil, factor of safety, and the location of the water table. The new graphs are based on the AASHTO bearing capacity equations. By knowing the SPT N-value, the width of the footing B, the depth of the foundation, D, the engineer will be able by using the graphs to came up with a quick estimation of the ultimate bearing capacity.

The SPT should be used with discrete judgment when it is used to estimate the bearing capacity of cohesive soils since silt and clay may be stiffened or softened depending on an increase or decrease of their moisture contents. Additionally, the SPT number may be misleading if large-size gravel is wedged into the split spoon sampler resulting in apparently high *N*-values.

Although the goal of the present study is to produce charts providing quick estimates of the bearing capacity, one should not forget that settlement is a controlling mechanism in foundation design and was not addressed within the scope of this project.

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APPENDIX A

x_1 and x_2 as a function of φ

A.1. x_1 and x_2 factors as a function of N_{60} and φ for a wall footing

N_{60}	φ	$x_1 = N_q$	N_γ	$x_2 = 0.5N_\gamma$
2	27.60	14.11	15.82	7.91
4	28.20	15.06	17.24	8.62
6	28.80	16.09	18.82	9.41
8	29.40	17.22	20.56	10.28
10	30.00	18.40	22.40	11.20
12	30.60	19.74	24.55	12.28
14	31.20	21.14	26.84	13.42
16	31.80	22.67	29.37	14.69
18	32.40	24.34	32.21	16.10
20	33.00	26.09	35.19	17.59
22	33.60	28.10	38.71	19.36
24	34.20	30.21	42.45	21.23
26	34.80	32.53	46.64	23.32
28	35.40	35.08	51.34	25.67
30	36.00	37.75	56.31	28.15
32	36.60	40.85	62.24	31.12
34	37.20	44.12	68.56	34.28
36	37.80	47.73	75.66	37.83
38	38.40	51.74	83.72	41.86
40	39.00	55.96	92.25	46.12
42	39.60	60.90	102.55	51.27
44	40.20	66.14	113.57	56.79
46	40.80	71.96	126.06	63.03
48	41.40	78.49	140.35	70.18
50	42.00	85.38	155.55	77.77

A.2. x_1 and x_2 factors as a function of N_{60} and φ for a square footing

N_{60}	φ	1 + tan φ	N_q	x_1	N_γ	x_2
2	27.60	1.523	14.11	21.49	15.82	4.75
4	28.20	1.536	15.06	23.13	17.24	5.17
6	28.80	1.550	16.09	24.94	18.82	5.65
8	29.40	1.563	17.22	26.91	20.56	6.17
10	30.00	1.577	18.40	29.02	22.40	6.72
12	30.60	1.591	19.74	31.41	24.55	7.37
14	31.20	1.606	21.14	33.95	26.84	8.05
16	31.80	1.620	22.67	36.73	29.37	8.81
18	32.40	1.635	24.34	39.79	32.21	9.66
20	33.00	1.649	26.09	43.02	35.19	10.56
22	33.60	1.664	28.10	46.76	38.71	11.61
24	34.20	1.679	30.21	50.72	42.45	12.74
26	34.80	1.695	32.53	55.14	46.64	13.99
28	35.40	1.711	35.08	60.02	51.34	15.40
30	36.00	1.727	37.75	65.19	56.31	16.89
32	36.60	1.743	40.85	71.20	62.24	18.67
34	37.20	1.759	44.12	77.61	68.56	20.57
36	37.80	1.776	47.73	84.77	75.66	22.70
38	38.40	1.793	51.74	92.77	83.72	25.12
40	39.00	1.809	55.96	101.23	92.25	27.68
42	39.60	1.827	60.90	111.26	102.55	30.77
44	40.20	1.845	66.14	122.03	113.57	34.07
46	40.80	1.863	71.96	134.06	126.06	37.82
48	41.40	1.882	78.49	147.72	140.35	42.11
50	42.00	1.900	85.38	162.22	155.55	46.67

APPENDIX B

Shear Strength vs. SPT-values

ENPM 808
Advanced Topics in Engineering
Independent Study - University of Maryland
Shear Strength of Fine-Grained Soils
vs.
SPT-Values

By:

Eric C. Henmueller, P.E.
May 15, 2001

Advisor:

Dr. M.S. Aggour
University of Maryland
Dept. of Geotechnical Engineering

Abstract

The in-situ shear strength of fine-grained-cohesive soils is difficult to predict with the Standard Penetration Test (SPT). While many papers have been published regarding the prediction of bearing capacity and settlement characteristics of coarse-grained soils by SPT, relatively few attempts have been made to draw a correlation between shear strength of fine-grained soils and SPT's. This study attempts to find a correlation between shear strength and SPT N-values, and provide a comparison with published correlations.

Introduction

The body of data used for this analysis is drawn from soil borings performed by Mueser, Rutledge, Wentworth & Johnston, New York, New York for design of the Washington Metropolitan Area Transit Authority (WMATA) or Metro. The design of the Metro subway system required hundreds of soil borings. This study was limited in scope to four Metro lines located primarily in southeast and southwest Washington, D.C. and portions of Prince George's County. Many more soil borings are available, however, the geology of the northern and western portion of the Metro is primarily Piedmont. The Piedmont soils are generally residual silt and sand overlying disintegrated rock and bedrock. The scope of this study was limits to the fine grained soils generally found interlayered in Terrace deposits or Sedimentary deposits of southern D.C.

Summary of the Metro Line Location and General Geology

The boundary between two major geographic provinces of significantly different characteristics passes through the District of Columbia. The southeastern portion of the District and Prince Georges County, Maryland lies in the "coastal plain" which consists of a broad belt of flat-lying sediments over deep bedrock. The northwestern portion of the District lies within the "Piedmont" province which comprises of a relatively thin cover of overburden above crystalline bedrock, the surface of which dips to the southeast beneath the coastal plain deposits. The "fall line" which is the boundary between these geologic units extends southwest from a line along the Montgomery County boundary through Farragut Square crossing the Potomac River just north of Roosevelt Island and passing to the west roughly on the line of Sprout Run in North Rosslyn.

The Metro routes that were selected for this study reside within the coastal plain portion of Washington, D.C. The four routes detailed in this report include the Branch Route, B & O Route, New Carrollton Route, and the L'Enfant-Pentagon Route. Refer to Figure 1, Metro Route Locations, for a plan view of the location for each route.

A summary of the subsurface conditions and general geology for each Metro route selected follows:

New Carrollton Route

The section of the New Carrollton route summarized herein is located between Minnesota Avenue and the District line along the trackage of the Penn Central and B&O Railroads. This area is the community of Kenilworth in southeastern District of Columbia.

The geologic profile appears fairly consistent with typically about 50 to 70 feet of Pleistocene terrace deposits overlying hard Cretaceous clay. The surface of the clay typically appears between El –10 and –30. The Pleistocene soils are made up of a complex interlayering of clay lenses and sandy gravelly material. Soil borings generally indicated the Pleistocene soil to be medium stiff and of moderate compressibility. As the track runs at or near existing grade for this section, laboratory testing was concentrated in the Pleistocene soils.

Branch Route

The section of the Branch route summarized in this report is bounded by the approximate intersection of Half Street and M Street in southwestern Washington and the intersection of Branch Avenue and Henson Creek. The line extends east from its origin to cross the Anacostia River near the Washington Navy Yard. The line continues to extend southeast into Prince Georges County to its termination point. The total length of the section is about 6.5 miles.

L'Enfant-Pentagon Route

The L'Enfant-Pentagon Route is comprised of two sections. Section one addresses the Potomac River crossing and section two addresses the overland route. The route begins at the intersection of Frontage Road and 7th Street, SW, turns to the west to cross Washington Channel, East Potomac Park, and the Potomac River and continues west to join the Huntington Route just east of Pentagon Station.

B & O Route

The B & O Route begins at 12th and G Streets, NW in the District of Columbia and extends a distance of about 10 miles to Brookville Road in Maryland. The geologic conditions in the downtown portion of the line between 12th and G Streets and extending through Judiciary Square consist of 50-foot terrace deposits overlying Cretaceous Potomac material. Shallower terrace deposits of about 25 feet are found between D Street, NW and the engine and coach yards serving Union Station. Also in this area is Tiber Creek and tributaries. From Union Station north to Blair Park, the line generally follows the B & O Rail line and alternates between fill soil profiles in stream channels and cuts in Cretaceous materials. As the line extends north of Blair Park, the Piedmont terrain is encountered with the track elevation generally residing in decomposed rock above bedrock.

Laboratory and Field Test Summary

Three types of shear strength testing was generally performed on soil samples recovered from Metro borings. These tests included the unconfined compression test and the unconsolidated-undrained triaxial shear test. This study focused on the shear strength results from the two former tests or the unconfined compression and the unconsolidated-undrained triaxial shear tests. These tests are primarily performed on fine-grained soils.

The tests were performed on 2 inch and 3 inch tube samples recovered from the borings as essentially unaltered moisture content. The triaxial tests were performed in a triaxial cell with a confining pressure applied without permitting drainage. The confining pressure restricts failure that may occure at low stresses due to sand pockets, seams, or fractures that may exist in the sample. Confinement also some what compensates for sample disturbance. In theory, undisturbed, homogeneous, saturated samples of clay from a singular stratum sheared at different confining pressures would all exhibit the same deviator stress at failure. As such, the Mohr circle envelope for such a series of tests should plot as a straight line. However, since homogeneous samples do not exist in nature and instead will have traces of sand or incomplete saturation, the shear strength recorded for a sample tested at increasing confining pressure will often return a slightly increased test strength.

The triaxial shear strength values plotted in this report are typically an average of two or three triaxial shear strength values recorded for a particular sample. In several cases, an unusually high or low strength value was reported in the testing. In such cases, the unusually high or low value was omitted from the average value recorded in the graphs. The unusually high or low value may have been a result of sample disturbance, testing error, or a combination of the two.

Laboratory testing performed on selected samples from the four routes selected for this study are summarized in Appendix B. The summary includes the following index, classification, and physical properties:

Sample Identification

- Boring Number
- Sample Number
- Sample Depth
- Stratum Designation
- N-Value

Classification Properties

- Natural Water Content for Entire Sample
- Liquid Limit
- Plasticity Index
- Natural Water Content for Limit Sample
- Specific Gravity
- Soil Type (Unified Soil Classification System)
- Percent Sand
- Percent Finer than 200 Sieve

Physical Properties

- Compressive Strength (TSF)
- Water Content at End of Test (%)
- Strain at Failure (%)
- Type of Test
- Deviator Stress
- Confining Pressure
- Natural Water Content
- Natural Water Content at End of Test

Note that all properties were not measured for each sample. Particularly in the case of specific gravity, percent sand, and percent finer that the 200 sieve.

Test Procedures and Data Collection

Standard Penetration Test

The soil boring standard is ASTM D 1586 – Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils. The test standard is presented in Appendix C.

N-values were obtained from the Metro borings at approximately the depth at which shear strength testing was performed. N-values indicate the penetration resistance in blows per foot of a 2 inch O.D., 1-3/8 inch I.D. split-spoon sampler driven with a 140 pound hammer falling 30 inches. After the initial set of 6 inches to set the sampler in undisturbed material, the number of blows required to drive the sampler an additional 12 inches has been taken as the N value. In the event 30 or more blows were required to drive the sampling spoon the initial 6-inch interval, the sampling spoon is driven to a total penetration resistance of 100 blows or 18 inches, whichever occurs first. The sampling operation is generally terminated after a total of 100 hammer blows and the depth of penetration is recorded.

The standard penetration test is traditionally performed using a rope and pulley apparatus to raise and drop the 140 pound hammer. However, ASTM D-1586 also allows for the option of an automatic hammer apparatus. Testing for the two methods has shown that the auto hammer method generally imparts about 30 percent more energy to the split sampling spoon. As such, blow counts are generally lower on average for the auto hammer method when compared with the rope and pulley method.

The test method used for the Metro borings was consistently the rope and pulley method of sampling. As such, corrections should not be required when comparing data collected from different Metro lines.

Analysis of Shear Strength vs. N-Values

A problem that is inherent with comparisons of shear strength and N-values is the fact that when an undisturbed tube sample is recovered, N-values are typically not obtained. As such, some amount of interpolation is required to arrive at a relatively accurate estimate of N-value for a particular sample depth. In most cases the N-value can be obtained from a SPT taken 5 feet above or 5 feet below the depth of the tube sample. In some cases however, as many as 5 or 6 tube samples may be recovered from a single boring. This situation results in few SPT results from that boring. In such cases, the boring immediately up-station or down-station may indicate the same stratum at the tube sample depth. If the stratum is continuous and the depth of sample is equal, the SPT may be interpolated for the adjacent boring.

In several borings the undisturbed sample depth was 80 feet or more. Error may be introduced in the N-value measurement at these depths, however, the N-value is generally accepted as accurate without adjustment to a depth of about 100 feet. The method of sampling will often vary between drilling operators. Within the standard test procedure, there are presented two methods for raising and dropping the 140 lb. anvil for the N-value count. These procedures are discussed above. Use of both procedures on the same project may result in error if the drop method is not taken into account. Since the Metro borings were performed under the oversight of Mueser Rutledge, I believe the sampling method remained relatively consistent for all borings. Therefore, I believe that this resulted in a reasonably consistent body of data from which to draw data.

Discussion of Results

The data collected from the four Metro lines has been reduced to six graphs. The graphs are divided into two groups – Unconfined Compression Test vs. N-value and Triaxial Shear Test vs. N-value. Within each group, the data has been divided into fat clay, lean clay, and organic silt, respectively, vs N-value. The graphs of each soil group are presented in Appendix A. Also presented on the graphs exhibited in Appendix A are lines which indicate minimum and maximum values of cohesion as demonstrated by the equations for cohesion by Sowers, 1979.

A relationship between N-values and undrained shear strength (after Sowers, 1979) is also presented in Appendix C. The equations for cohesion by Sowers may be demonstrated as:

c = (0.102 to 0.179) N tsf - for fat clay soils

c = (0.051 to 0.102) N tsf - for lean clay soils

c= (0.026 to 0.051) N tsf – for silt soils

A shear strength equation was also developed by Shioi, Y. and Fukui, J. (1982). The equation for cohesion by Shioi, Y. and Fukui, J. may be demonstrated as:

c = (0.061 to 0.102) N tsf, where N = N-value for the sample tested.

The Shioi, Y. and Fukui, J. equation is applicable for lean clay soils and compares closely with Sowers equation for lean clay.

Correlations between shear strength and N-values have also been demonstrated in several articles by Ladd et al. (1977), Casagrande (1966), de Mello (1971), Schmertmann (1971) and Mitchell et al. (1978). Schmertmann and Mitchel note that lower blow counts may be a result during sampling of sensitive clays due to strength loss during sampling. Ladd indicates that shear strength values are of little value unless the clay soils are relatively stiff and insensitive.

The graphical results prepared by Sowers generally indicate shear strength increasing with increased N-value. Shear strength also increases relative to N-value from a relatively low shear strength for silt, to a relatively high shear strength for fat clay.

The graphs prepared from the Metro borings indicate a similar pattern for the organic silt and lean clay data. However, the fat clay data resulted in a relatively scattered data collection. This scattering may be a result of interpolation during data collection. The samples classified as fat clay may also have had sand seams or fissures that may result in a lower than expected shear strength. Moisture content varied considerably for fat clay samples tested. It is likely that high moisture content resulted in lower than average shear strength values.

Variations also exist within the Unified Soil Classification System's definition of a particular soil type. This aspect of the classification system may result in variation of shear strengths for a particular soil classification.

Conclusions

Graphical results for the lean clay and organic silt classifications indicate a correlation with Sowers (1979) and Shioi, Y. and Fukui, J. (1982). However, the fat clay soils indicate less correlation, in part, due to the high moisture content of a large number of samples tested. This is in general agreement with Ladd (1977) in that sensitive clays tend to return erratic results that appear to be dependant on moisture content.

I believe the Metro boring data is a valuable base of information for making comparisons between SPT values and shear strength of fine-grained soils. The Metro borings may also be used to develop general subsurface outlines for selected areas throughout Washington, D.C. I believe a comparison between N-values and shear strength may be more useful if additional borings were performed with the explicit purpose of obtaining N-values at the same elevation as the tube sample extraction.

Figure 1: WMATA Metro Site Map

Appendix A:

Shear Strength vs. N-Value Graphs

1) Lean Clay – Unconfined Compression Test
2) Lean Clay – Triaxial Shear Test
3) Organic Silt – Triaxial Shear Test
4) Fat Clay - Triaxial Shear Test
5) Fat Clay – Unconfined Compression Test
6) Organic Silt - Unconfined Compression Test

Appendix B:

Summary of Metro Data (16 Pages)

Appendix C:

Shear Strength vs. N-value (after Sowers, 1979)

Appendix D:

ASTM D 1586 – Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils.

Appendix E:

References

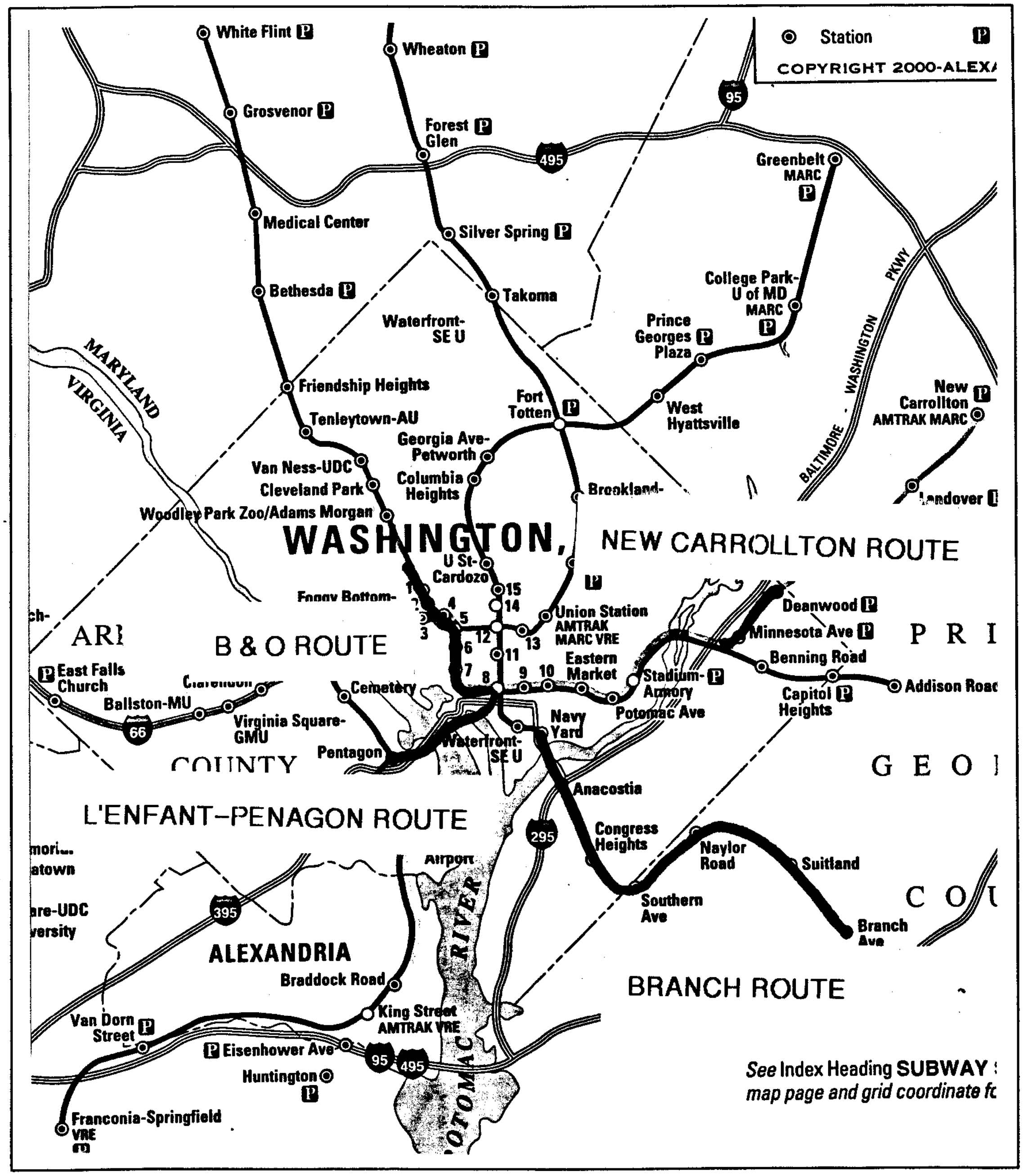
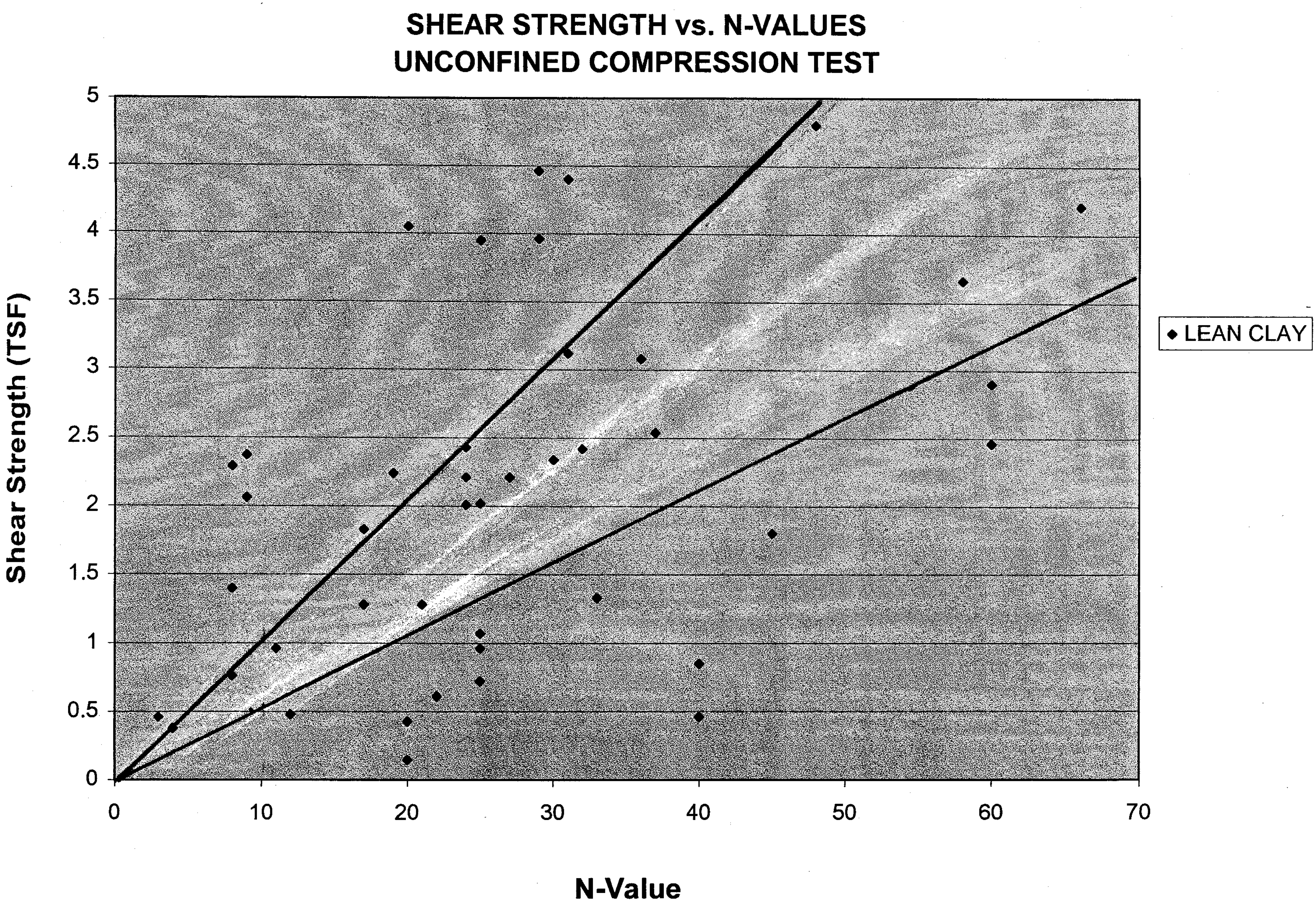


FIGURE 1

WMATA METRO SITE MAP
WASHINGTON, D.C.

APPENDIX A
Shear Strength vs. N-Value Graphs

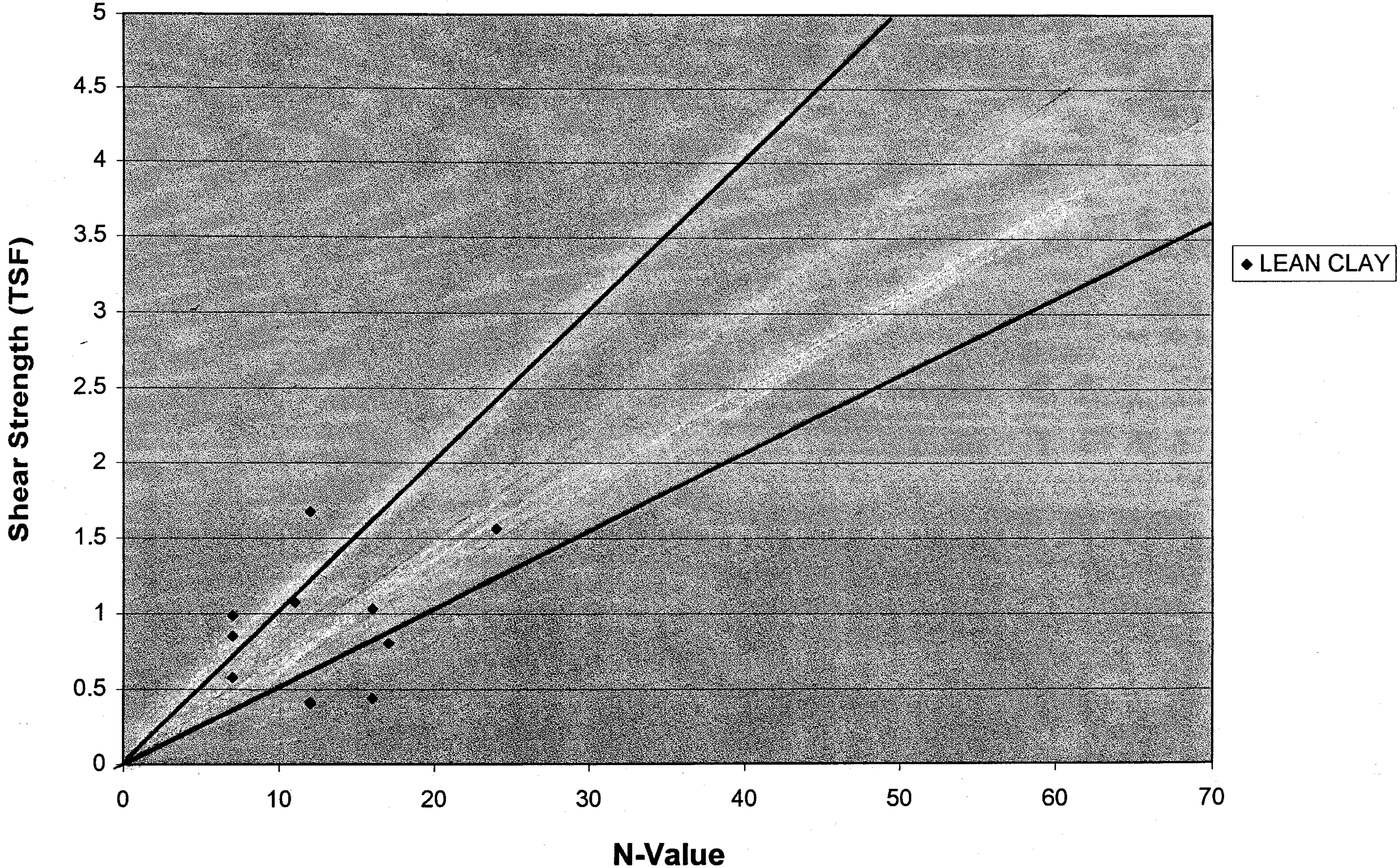
- Lean Clay – Unconfined Compression Test
- Lean Clay – Triaxial Shear Test
- Organic Silt – Triaxial Shear Test
- Fat Clay - Triaxial Shear Test
- Fat Clay – Unconfined Compression Test
- Organic Silt - Unconfined Compression Test



SHEAR STRENGTH vs. N-VALUES
UNCONFINED COMPRESSION TEST
Shear Strength (TSF)
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
0
10
20
30
40
50
60
70
N-Value
LEAN CLAY

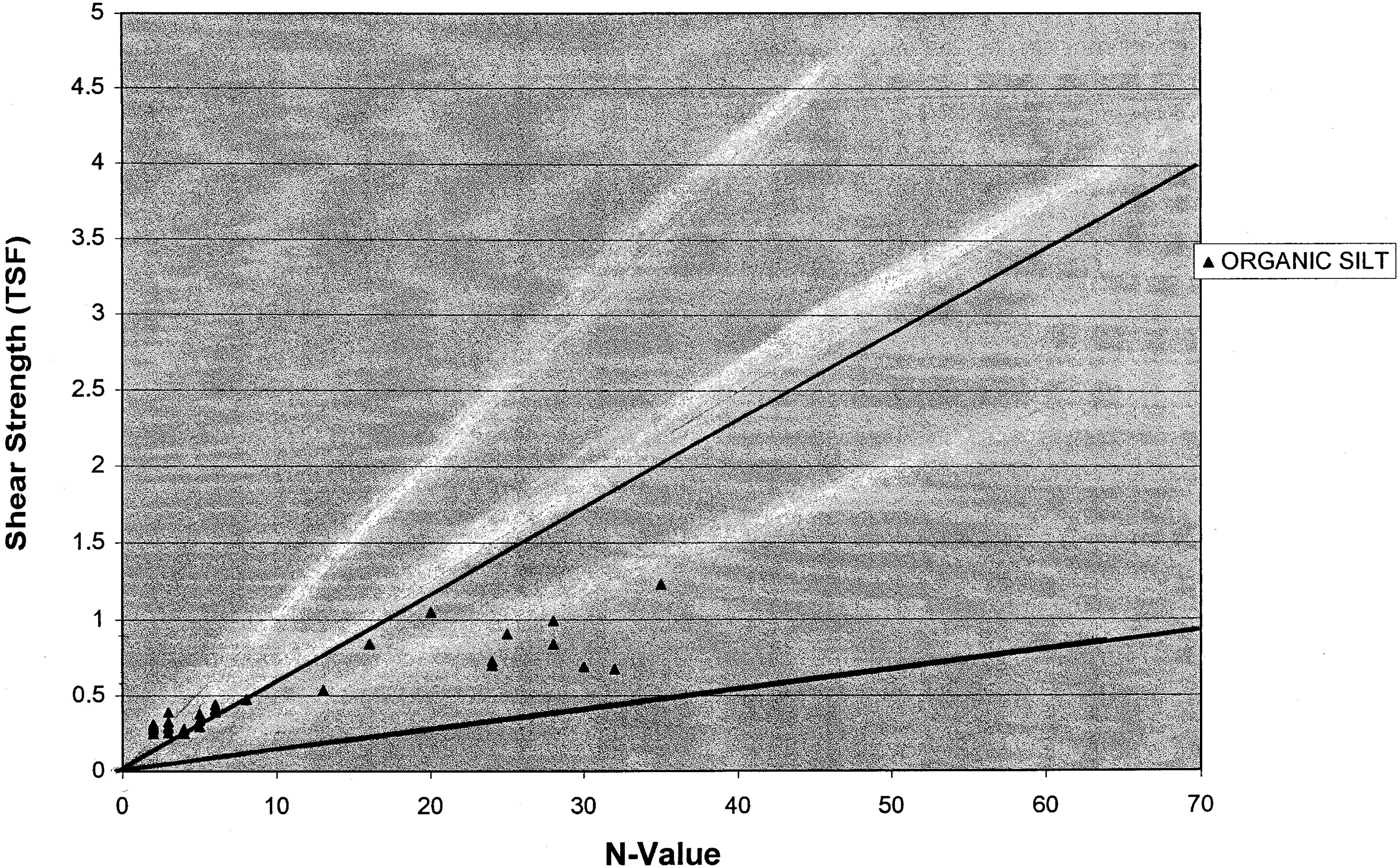
GRAPH NO.1

SHEAR STRENGTH vs. N-VALUES
TRIAXIAL SHEAR TEST



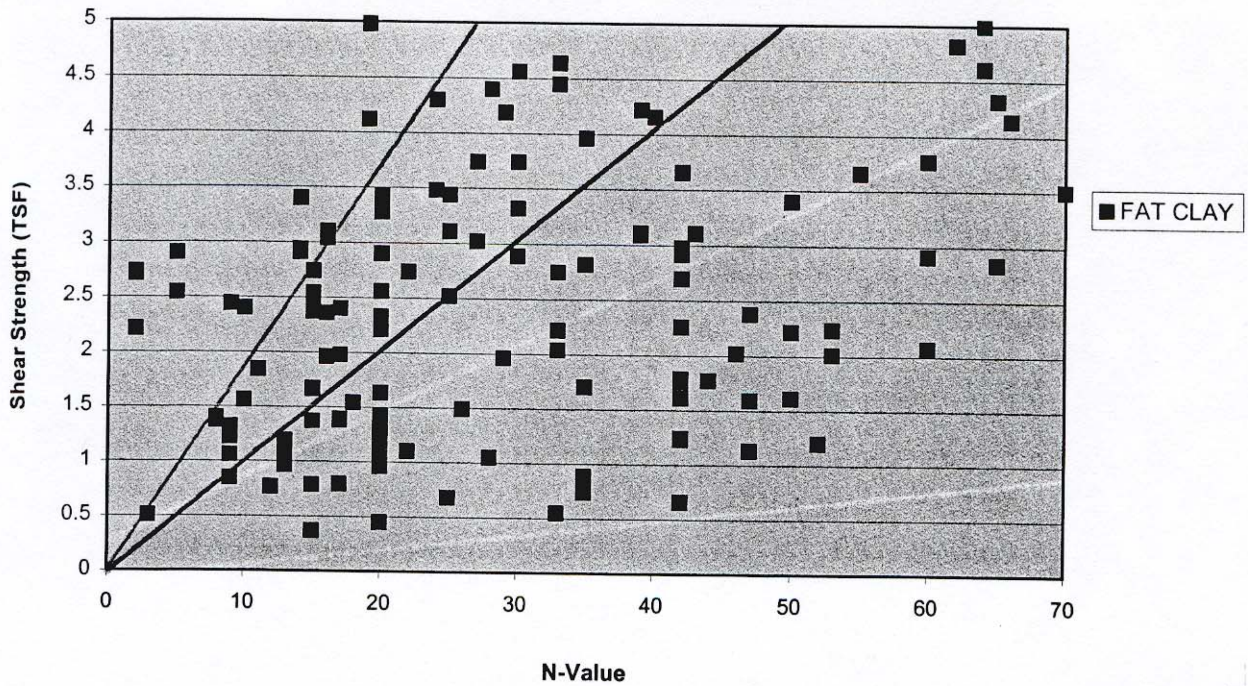
GRAPH NO.2

SHEAR STRENGTH vs. N-VALUES
TRIAXIAL SHEAR TEST



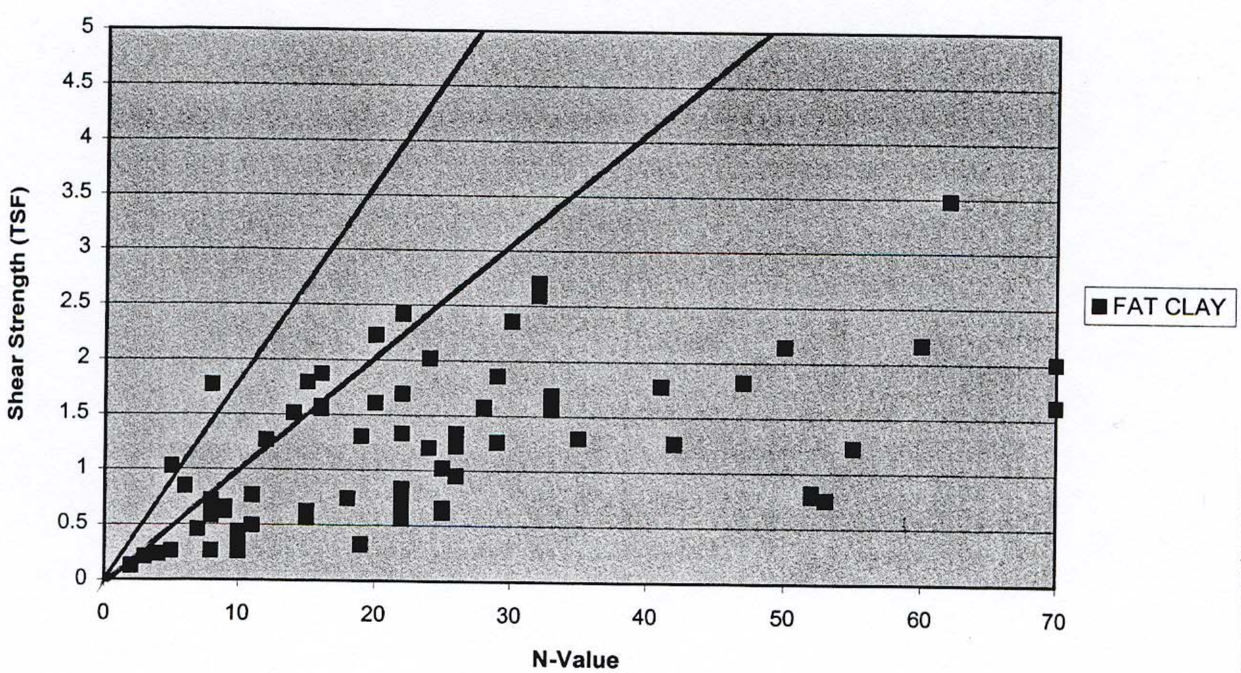
GRAPH NO.3

SHEAR STRENGTH vs. N-VALUES
TRIAXIAL SHEAR TEST



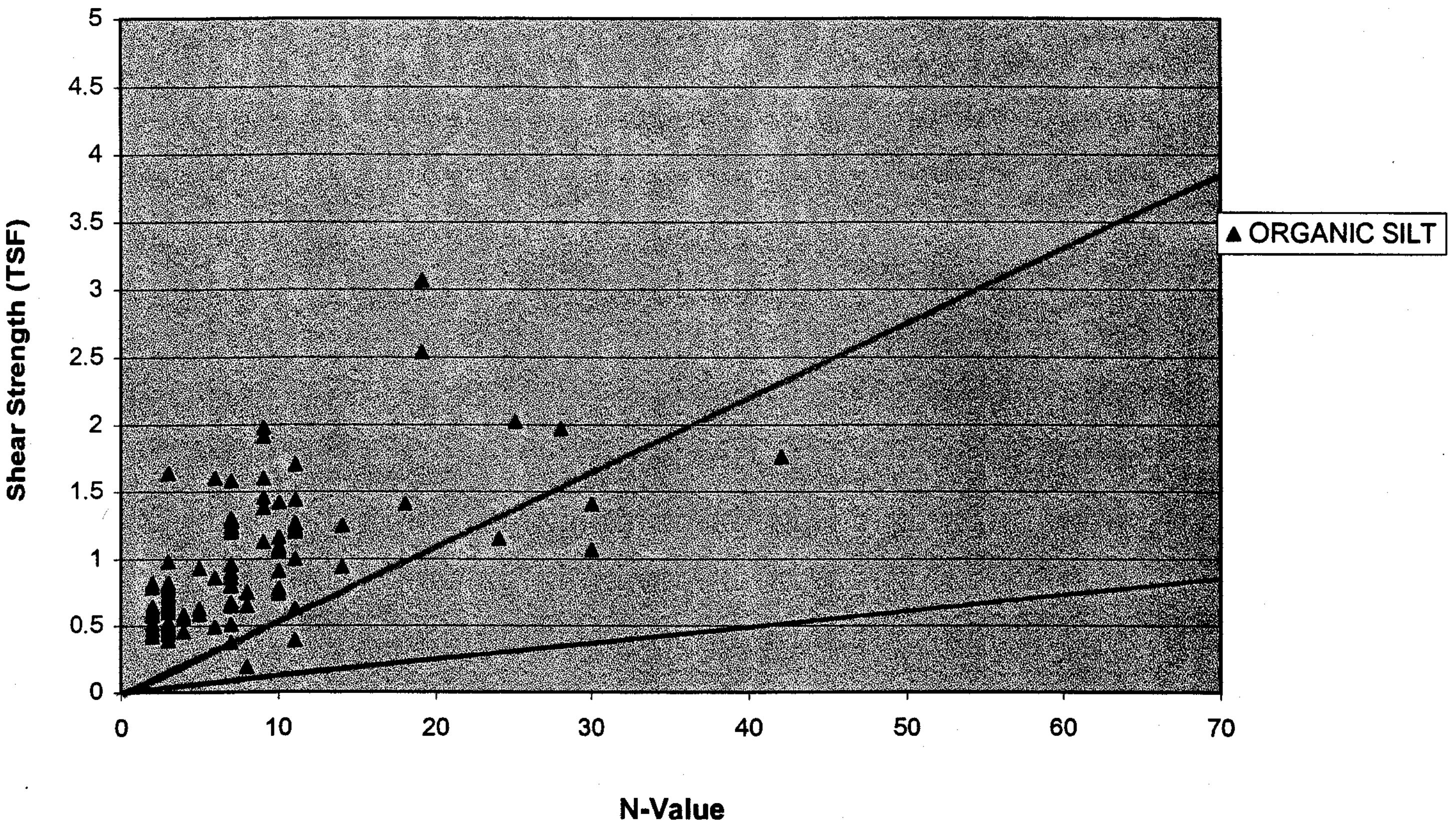
GRAPH NO.4

SHEAR STRENGTH vs. N-VALUES
UNCONFINED COMPRESSION TEST



GRAPH NO.5

SHEAR STRENGTH vs. N-VALUES
UNCONFINED COMPRESSION TEST



APPENDIX B

- Summary of Metro Data (16 Pages)

SAMPLE IDENTIFICATION					CLASSIFICATION PROPERTIES								PHYSICAL PROPERTIES: Unconfined Compression			PHYSICAL PROPERTIES: Triaxial Compression				
Boring Number	Sample No.	Sample Depth	Stratum Designation	N-Value	Natural Water Content for Entire Sample	Liquid Limit	Plasticity Index	Natural Water Content of Limit Sample	Specific Gravity (G)	Soil Type	% Sand	%Finer Than 200 Sieve	Compressive Strength (TSF)	Water Content at End of Test (%)	Strain at Failure (%)	Type of Test	Deviator Stress	Confining Pressure	Natural Water Content	Water Content at End of Test
NEW CARROLLTON ROUTE																				
BW 2	11S	51.0	TIH	21	21					CL			1.28	21	13					
													0.81	23	9					
BW 3	5S	18.0	TIG	9	26	39	22	26		CL										
	6S	21.0	TIG	9	23					CL										
BW 5U	12U	53.0	TIH	15	48	70	42	48		CH			0.79	46	18					
	14U	59.0	TIH	15	20					CL										
	15U	61.0	TIH	19	37	54	33	27	2.64	CH						Q	0.65	2.0	39.0	39.0
	16U	63.5	TIH	19	45					CH						Q	2.64	0.5	48.0	47.0
																Q	2.58	1.0	41.0	40.0
BW 8U	5U	18.0	TIG	12	29					CL			0.73	27	11					
10	6S	23.0	TIG	4	32	39	20	32		CL			0.38	30	8					
													0.47	33	11					
14	8U	33.0	TIG	16	27					CL						Q	2.14	0.5	25.0	25.0
																Q	0.93	1.0	28.0	28.0
																Q	1.60	2.0	28.0	28.0
BW 18	9S	40.4	TIH	13	32					CL			0.63	29	21					
BW 21	4S	16.0	TIG	17	26	24	7	23		CL										
	7S	31.0	TIH	11	26	45	24	28		CL			0.96	24	16					
													1.32	27	12					
BW 23U	9U	35.0	TIH	6	25					CH						Q	1.64	0.5	25.0	25.0
																Q	1.60	1.0	25.0	24.0
																Q	1.84	2.0	24.0	23.0
BW 24	8S	36.0	TIH	5	25	39	22	26		CL			0.51	26	15					
BW 26	6S	24.0	TIG		22					CL			1.42	21	7					
	7S	26.0	TIG		27					CH			1.08	26	14					
													1.64	25	11					
BW 28	11S	48.7	TIH	24	25	25	7	25		CL										
BW 30	4S	15.8	TIG	12	22	38	19	20		CL			0.48	21	4					
													2.16	19	13					
WR 3	6S	25.9	P1	25	24	61	37	24		CH										
7	5S	18.0	P1	16	24	56	34	25		CH			2.36	24	14					
	6S	26.0	P1	16	22					CH			1.96	23	6					
													3.10	23	12					
	7S	31.0	P1	80	15					CH			4.21	14	12					
													2.90	16	12					
8U	10U	41.0	P1	113	16					CH						Q	3.16	1.0	16.0	16.0
																Q	4.12	2.0	16.0	16.0

SAMPLE IDENTIFICATION					CLASSIFICATION PROPERTIES								PHYSICAL PROPERTIES							
													Unconfined Compression			Triaxial Compression				
Boring Number	Sample No.	Sample Depth	Stratum Designation	N-Value	Natural Water Content for Entire Sample	Liquid Limit	Plasticity Index	Natural Water Content of Limit Sample	Specific Gravity (G)	Soil Type	% Sand	%Finer Than 200 Sieve	Compressive Strength (TSF)	Water Content at End of Test (%)	Strain at Failure (%)	Type of Test	Deviator Stress	Confining Pressure	Natural Water Content	Water Content at End of Test
11	4S	16.0	P1	20	23	83	60	17		CH			0.45	25	2					
													1.63	25	3					
	6S	25.0	P1	25	22					CH			3.11	22	14					
													3.44	22	20					
14U	4U	16.0	P1	22	32					CH						Q	1.54	2.0	32.0	31.0
	7U	31.0	P1	29	22					CH			1.96	22	5	Q	2.52	1.0	23.0	22.0
	8U	36.0	P1	29	19					CH						Q	2.12	0.5	18.0	18.0
																Q	4.56	1.0	18.0	18.0
																Q	3.54	2.0	19.0	18.0
																Q	3.07	0.5	19.0	19.0
23	6S	26.0	P1	46	21	55	31	21		CH			5.10	21	5					
													2.02	21	7					
28U	5U	15.6	P1	31	25					CH										
29	3S	11.0	P1	33	24	74	52	24		CH			2.04	23	9					
													2.22	25	3					
	5S	15.6	P1	33	12					CL			9.14	11	2					
	7S	25.8	P1	96	17					CH			5.48	17	6					
													2.48	17	12					
31	2S	6.0	P1	27	18					CL			2.21	18	12					
													2.08	18	8					
33U	2U	5.0	P1	20	19					CL						Q	1.79	0.5	21.0	21.0
																Q	2.82	1.0	15.0	15.0
																Q	3.54	2.0	20.0	19.0
	6U	22.8	P1	32	16					CH						Q	5.16	1.0	15.0	15.0
																Q	5.65	2.0	16.0	16.0
	8U	35.4	P1	50	19	49	28	19		CL						Q	4.16	0.5	19.0	18.0
36	6S	26.0	P1	27	21					CH			3.02	20	6					
													3.74	21	20					
	8S	35.8	P1	46	18					CH			8.42	19	8					
													####	17	6					
38U	7U	28.0	P1	25	20					CH						Q	2.04	1.0	20.0	20.0
	8U	32.8	P1	70	18					CH						Q	4.09	0.5	18.0	18.0
																Q	3.95	2.0	18.0	18.0
41	8S	43.8	P1	110	21					CH			5.25	24	6					
													5.78	20	9					
42U	8U	35.8		60	20					CH						Q	4.16	0.5	20.0	20.0
																Q	4.92	1.0	20.0	19.0
																Q	3.94	1.0	20.0	18.0
	9U	43.8		86	20					CH						Q	6.21	0.5	20.0	20.0
																Q	5.76	2.0	20.0	20.0
																Q	6.55	2.0	20.0	19.0
43	7S	31.0		31	14	75	54	13		CH										
	8S	36.0		60	20					CH			3.77	20	7					
45	7S	29.0		25	19					CH			5.30	19	9					

SAMPLE IDENTIFICATION					CLASSIFICATION PROPERTIES								PHYSICAL PROPERTIES							
													Unconfined Compression			Triaxial Compression				
Boring Number	Sample No.	Sample Depth	Stratum Designation	N-Value	Natural Water Content for Entire Sample	Liquid Limit	Plasticity Index	Natural Water Content of Limit Sample	Specific Gravity (G)	Soil Type	% Sand	%Finer Than 200 Sieve	Compressive Strength (TSF)	Water Content at End of Test (%)	Strain at Failure (%)	Type of Test	Deviator Stress	Confining Pressure	Natural Water Content	Water Content at End of Test
46U	7U	28.8		26	21					CH						Q	2.67	0.5	21.0	21.0
51	8S	33.0		32	18					CL			2.42	17	6					
													2.43	17	7					
	9S	36.0		24						CL			2.21	16	3					
													2.01	17	5					
	11S	40.8		24	18	59	34	14		CH			3.48	18	4					
													4.30	20	4					
56U	2U	5.6		7	19					CL						Q	2.59	2.0	19.0	19.0
58	5S	18.8		30	15					CH			3.74	15	16					
													5.16	15	9					
	7S	26.0		30	14	22	9	14		CL			2.34	16	11					
													1.66	12	3					
60	4S	16.0		25	17					CH			2.52	17	14					
	6S	26.0		19	13					CH			4.12	13	14					
													4.98	12	3					
76U	5U	16.8		28	19					CH			4.40	19	6					
84U	3U	11.0		15	18					CL						Q	3.94	1.0	18.0	18.0
																Q	3.66	2.0	19.0	18.0
	4U	16.0		15	20	52	32	20		CH										
	7U	29.0		16	19					CH			3.04	18	10	Q	3.14	0.5	19.0	18.0
92	7S	28.0		29	17					CH			4.19	12	12					
93	5S	21.0		18	19					CL			2.76	18	5					
95U	5U	21.0		26	21					CH						Q	2.70	0.5	21.0	21.0
																Q	2.29	1.0	20.0	20.0
																Q	2.33	2.0	20.0	20.0
96	6S	25.9		25	28					CL										
	7S	29.6		25	19					CL			5.00	17	9					
98	5S	21.0		20	21	39	18			CL			4.40	19	7					
	6S	26.0		31	18	34	15			CL			3.12	19	9					
													3.08	17	12					
	7S	31.0		36	18	33	13			CL			2.74	18	6					
102	3S	11.0		4	20					CL			1.83	19	19					
	4S	16.0		17	21					CH			2.40	18	6					
	5S	23.0		11	19					CL			2.13	19	7					
105U	2U	6.0		10	42	61	35	44		CH						Q	0.56	1.0	46.0	46.0
																Q	0.47	2.0	39.0	39.0
	3U	9.0		10	36	66	38	37		CH						Q	0.66	0.5	41.0	41.0
111	9S	42.8		103	15					CL			4.24	15	9					
													4.35	15	4					
112	6S	25.8		66	22	68	41	22		CH										

SAMPLE IDENTIFICATION					CLASSIFICATION PROPERTIES								PHYSICAL PROPERTIES							
													Unconfined Compression			Triaxial Compression				
Boring Number	Sample No.	Sample Depth	Stratum Designation	N-Value	Natural Water Content for Entire Sample	Liquid Limit	Plasticity Index	Natural Water Content of Limit Sample	Specific Gravity (G)	Soil Type	% Sand	%Finer Than 200 Sieve	Compressive Strength (TSF)	Water Content at End of Test (%)	Strain at Failure (%)	Type of Test	Deviator Stress	Confining Pressure	Natural Water Content	Water Content at End of Test
	7S	30.8		63	22					CH			2.30	22	7					
114U	3U	16.0		16	15					CH			4.00	15	12					
BRANCH ROUTE																				
37U	4U	13.0		29	26					CL						Q	0.19	0.5	29.0	28.0
																Q	2.71	1.0	21.0	21.0
	5U	15.0		29						CL						Q	2.32	0.5	19.0	18.0
																Q	2.94	2.0	19.0	19.0
	6U	17.5		29	23	42	18	25		CL						Q	3.04	1.0	25.0	25.0
																Q	3.20	2.0	21.0	21.0
	8U	21.0		24	26					CH						Q	1.82	0.5	33.0	33.0
																Q	3.11	1.0	29.0	29.0
																Q	2.29	2.0	29.0	29.0
	9U	26.0		19	29					CL						Q	0.40	0.5	72.0	71.0
					72					MH						Q	0.47	1.0	28.0	20.0
																Q	1.43	2.0	29.0	28.0
	12U	34.0		27	25					CL						Q	0.91	0.5	30.0	29.0
																Q	1.26	1.0	23.0	23.0
																Q	1.54	2.0	22.0	21.0
38	4S	12.8		18	30	39	12	29		CL			0.89	29	2					
													2.02	29	2					
	5S	17.0		25	24					CL			0.96	25	9					
													1.07	23	5					
	6S	21.0		25	23					CL	27	73	0.72	22	7					
													1.28	23	3					
	9S	33.2		17	43					OH			2.02	43	6					
					44					CH			1.98	49	4					
													0.90	38	12					
	10S	36.0		19	44					OH			2.49	40	4					
													3.07	44	10					
													2.54	46	8					
43U	4U	16.5		35	26	81	59	26		CH			0.74	26	5					
	5U	18.4		35	21					CH			0.89	29	8	Q	2.46	0.5	21.0	20.0
	13U	47.5		39	24					CH						Q	3.94	2.0	22.0	23.0
45U	8U	28.0		7	28					CL						Q	1.10	1.0	34.0	33.0
	14U	42.0		8	87	146	85	79	2.46	OH						Q	1.22	1.0	70.0	70.0
	15U	44.5		5	83					OH	30	70				Q	0.81	2.0	53.0	53.0
	17U	51.0		5	23					CH						Q	1.06	1.0	22.0	22.0
																Q	0.99	2.0	24.0	24.0
	19U	58.0		11	31	103	74	56	2.73	CH										
46	6S	23.0		9	24	48	28	23		CH	4	96	0.85	23	5					
													1.06	22	5					
													1.22	26	9					
	7S	25.5		15	22	58	37	22		CH			1.37	23	6					
													2.37	21	8					
													2.54	21	6					
47	4S	15.6		28	39					CH			1.05	39	10					
	6S	25.4		40	19	50	33	19		CH			4.16	17	11					

SAMPLE IDENTIFICATION					CLASSIFICATION PROPERTIES								PHYSICAL PROPERTIES							
													Unconfined Compression			Triaxial Compression				
Boring Number	Sample No.	Sample Depth	Stratum Designation	N-Value	Natural Water Content for Entire Sample	Liquid Limit	Plasticity Index	Natural Water Content of Limit Sample	Specific Gravity (G)	Soil Type	% Sand	%Finer Than 200 Sieve	Compressive Strength (TSF)	Water Content at End of Test (%)	Strain at Failure (%)	Type of Test	Deviator Stress	Confining Pressure	Natural Water Content	Water Content at End of Test
	7S	30.4		44	22					CH			1.77	21	3					
48U	4U	13.6		22	28	31	45	28	2.77	CH						Q	1.26	0.5	30.0	29.0
																Q	1.38	1.0	25.0	24.0
																Q	1.92	2.0	27.0	26.0
50	9S	37.8		50	23	67	50	23		CH			2.22	24	5					
													1.61	22	3					
													3.40	22	6					
51U	6U	21.0		15	93	211	66	148	2.17	Pt						Q	1.78	0.5	152.0	150.0
																Q	1.31	2.0	34.0	33.0
	7U	23.0		17	113	171	59	119		Pt			0.74	64	11	Q	1.11	1.0	149.0	149.0
	8U	25.0		20	120	89	24	127		Pt			1.00	131	6					
													1.23	101	6					
53	8S	32.0		7	63	70	32	57		OH			0.42	66	9					
													0.38	57	9					
													0.51	70	14					
	9S	34.5		7	62	91	45	62		OH			0.68	62	9					
											1	90	0.65	61	10					
	10S	37.0		7	60	63	29	47		OH			1.20	64	8					
													0.96	65	9					
													0.80	61	11					
54U	8U	30.6		6	46					OL	37	63				Q	0.69	0.5	50.0	50.0
																Q	0.24	1.0	60.0	59.0
																Q	0.41	2.0	61.0	60.0
	9U	33.0		6	64	85	46	68	2.65	OH						Q	0.82	0.5	66.0	66.0
																Q	0.67	2.0	63.0	62.0
																Q	0.50	2.0	56.0	56.0
	10U	36.1		4	67	85	47	71		OH						Q	0.76	0.5	67.0	66.0
																Q	0.80	1.0	75.0	74.0
																Q	0.40	2.0	62.0	62.0
	11U	38.6		2	51	88	47	68		OH						Q	1.11	0.5	25.0	24.0
																Q	0.85	1.0	51.0	51.0
																Q	0.78	2.0	63.0	64.0
56U	2U	6.0		2	104					OH						Q	0.06	0.5	112.0	111.0
																Q	0.08	2.0		112.0
58U	2U	6.0		2	81					OH						Q	0.23	0.5	75.0	75.0
																Q	0.10	1.0	82.0	80.0
																Q	0.11	2.0	88.0	80.0
	3U	9.0		2	81	73	23	74	2.64	OH						Q	0.24	0.5		75.0
																Q	0.12	1.0	76.0	75.0
																Q	0.04	2.0	99.0	99.0
	4U	12.0		4	93					OH						Q	0.24	0.5	102.0	102.0
																Q	0.12	1.0	89.0	89.0
																Q	0.06	2.0	89.0	85.0
	5U	15.0		6	75	72	26	77		OH						Q	0.33	0.5	87.0	87.0
																Q	0.43	1.0	69.0	68.0
																Q	0.37	2.0	67.0	65.0
64U	5U	19.0		10	21					CL						Q	1.65	0.5	21.0	20.0

SAMPLE IDENTIFICATION					CLASSIFICATION PROPERTIES								PHYSICAL PROPERTIES: Unconfined Compression			PHYSICAL PROPERTIES: Triaxial Compression				
Boring Number	Sample No.	Sample Depth	Stratum Designation	N-Value	Natural Water Content for Entire Sample	Liquid Limit	Plasticity Index	Natural Water Content of Limit Sample	Specific Gravity (G)	Soil Type	% Sand	%Finer Than 200 Sieve	Compressive Strength (TSF)	Water Content at End of Test (%)	Strain at Failure (%)	Type of Test	Deviator Stress	Confining Pressure	Natural Water Content	Water Content at End of Test
																Q	1.08	1.0	23.0	23.0
																Q	0.86	2.0	21.0	21.0
	8U	28.0		35	20	31	17	20		CL										
	14U	56.0		32	24	67	48	24		CH										
66U	5U	21.1		12	22					CL			0.63	20	11	Q	0.75	0.5	24.0	24.0
																Q	0.88	2.0	25.0	24.0
	6U	26.0		12	27	33	19	22		CL			0.96	29	18	Q	0.70	0.5	25.0	25.0
																Q	0.96	1.0	31.0	31.0
	7U	29.0		12	31					CL						Q	1.03	1.0	25.0	24.0
																Q	0.43	2.0	37.0	36.0
	9U	34.0		20	29	50	35	22		CL						Q	1.10	0.5	35.0	35.0
																Q	1.13	1.0	28.0	27.0
	10U	36.0		22	21					CL						Q	1.77	2.0	13.0	12.0
68U	3U	11.1		7	13	21	8	12		CL			3.86	12	6	Q	1.57	0.5	14.0	14.0
																Q	2.49	1.0	15.0	15.0
																Q	1.87	2.0	16.0	16.0
	9U	36.1		10	29				2.79	CL						Q	0.99	0.5	30.0	30.0
																Q	0.94	1.0	26.0	26.0
																Q	1.15	2.0	26.0	25.0
71	4S	16.0		10	18	35	16	18		CL			1.40	19	6					
													1.80	18	6					
73U	4U	15.9		22	23					CH			2.74	23	13	Q	1.34	0.5	23.0	23.0
																Q	1.84	1.0		23.0
																Q	1.81	2.0	24.0	24.0
	5U	21.0		22	22	88	72	21		CH						Q	1.54	0.5	23.0	23.0
																Q	3.00	1.0	20.0	20.0
																Q	2.32	2.0	23.0	23.0
	6U	26.0		22	21	76	64	20		CH						Q	3.75	1.0	21.0	20.0
																Q	3.02	2.0	18.0	18.0
	7U	31.0		22	15					CH						Q	5.35	0.5	16.0	16.0
																Q	4.89	1.0	14.0	14.0
																Q	4.25	2.0	16.0	16.0
	10U	46.0		27	24	82	66	24		CH										
74	9S	40.8		5	23	77	63	22		CH			2.90	22	10					
													2.54	24	20					
	10S	45.7		5	17					CH			5.55	17	6					
													6.27	17	6					
													6.17	18	6					
76	6S	26.0		64	20	70	52	21		CH			5.00	20	10					
													4.61	20	8					
	7S	31.0		45	30					CL			1.80	24	13					
													0.84	35	7					
77	7S	31.0		42	21	51	31	21		CH			2.26	20	18					
													1.23	21	4					
	8S	36.0		42	23					CH			1.76	23	2					
													1.61	24	4					
													3.66	23	4					
	9S	41.0		42	23	59	35	22		CH			2.92	23	11					

SAMPLE IDENTIFICATION					CLASSIFICATION PROPERTIES								PHYSICAL PROPERTIES							
													Unconfined Compression			Triaxial Compression				
Boring Number	Sample No.	Sample Depth	Stratum Designation	N-Value	Natural Water Content for Entire Sample	Liquid Limit	Plasticity Index	Natural Water Content of Limit Sample	Specific Gravity (G)	Soil Type	% Sand	%Finer Than 200 Sieve	Compressive Strength (TSF)	Water Content at End of Test (%)	Strain at Failure (%)	Type of Test	Deviator Stress	Confining Pressure	Natural Water Content	Water Content at End of Test
													2.98	23	10					
	11S	51.0		42	21					CH			1.24	21	8					
													1.79	21	13					
													0.66	21	6					
79	15U	66.0		55	20	64	48	18	2.74	CH			3.66	19	6	Q	2.49	1.0	21.0	20.0
																Q	2.43	2.0	21.0	21.0
	16U	70.7		62	13					CH						Q	6.96	0.5	13.0	13.0
	17U	75.7		62	19					CH			4.82	18	10					
81	5S	21.0		15	38	62	38	32		CH			2.40	36	2					
													2.74	32	2					
													1.66	38	2					
	6S	26.0		20	43					CL			0.43	43	3					
													0.15	43	3					
													0.46	41	2					
	7S	30.7		40	39					SM+CL			0.85	26	3					
													2.46	40	4					
	11S	50.7		60	24	50	21	21		ML	81	19	1.66	28	6					
													5.40	24	4					
82U	5U	21.0		8	39	60	29	35	2.66	CH						Q	0.96	0.5	40.0	40.0
																Q	1.19	1.0	40.0	39.0
																Q	1.34	2.0	38.0	37.0
	6U	26.0		8	35					CH			1.37	32	3					
														37	3					
	7U	28.0		8	36	76	45	36		CH						Q	1.43	1.0	37.0	36.0
	9U	33.0		8	34					CH			1.40	33	8	Q	1.32	0.5	33.0	33.0
																Q	1.38	2.0	34.0	33.0
	10U	36.0		22	34	63	38	31	2.66	CH			1.10	34	7	Q	1.12	1.0	30.0	29.0
	11U	39.0		25	28					CH			0.68	34	8	Q	1.34	0.5	34.0	33.0
																Q	1.29	2.0	34.0	32.0
	12U	41.0		35	32					CH			1.70	29	2	Q	1.94	0.5	34.0	33.0
																Q	3.26	2.0	32.0	31.0
	13U	44.0		52	32	58	29	31	2.78	CH			1.20	33	4	Q	1.58	1.0	33.0	32.0
	14U	46.0		52	33					CH						Q	1.33	0.5	34.0	33.0
																Q	1.50	1.0	32.0	32.0
																Q	2.10	2.0	34.0	33.0
	15U	51.0		53	33					CH+ML			2.01	32	5	Q	1.51	2.0	34.0	33.0
	17U	60.7		30	38		37	37	2.62	CH						Q	5.53	0.5	38.0	38.0
																Q	3.90	1.0	38.0	37.0
	21U	80.6		24	26					ML			2.16	25	3					
83	5S	21.0		13	38	65	43	38		CH			0.97	39	6					
													1.19	38	5					
													1.08	37	8					
	7S	31.0		20	32					CH			1.42	31	4					
													2.21	32	4					
													4.76	32	6					
	9S	41.0		33	25	57	29	34		CH			1.55	26	3					
													5.30	21	3					
													4.45	24	2					
84U	4U	21.0		15	35					CH			0.37	31	4	Q	1.17	0.5	36.0	35.0
																Q	1.18	1.0	36.0	36.0

SAMPLE IDENTIFICATION					CLASSIFICATION PROPERTIES								PHYSICAL PROPERTIES: Unconfined Compression			PHYSICAL PROPERTIES: Triaxial Compression				
Boring Number	Sample No.	Sample Depth	Stratum Designation	N-Value	Natural Water Content for Entire Sample	Liquid Limit	Plasticity Index	Natural Water Content of Limit Sample	Specific Gravity (G)	Soil Type	% Sand	%Finer Than 200 Sieve	Compressive Strength (TSF)	Water Content at End of Test (%)	Strain at Failure (%)	Type of Test	Deviator Stress	Confining Pressure	Natural Water Content	Water Content at End of Test
																Q	1.37	2.0	45.0	38.0
	5U	26.0		25	33	68	43	32	2.76	CH						Q	1.39	0.5	34.0	34.0
																Q	1.31	1.0	34.0	34.0
																Q	1.01	2.0	35.0	35.0
	6U	31.0		42	33					CH			2.70	32	4	Q	2.26	5.0	38.0	38.0
																Q	2.61	1.0	31.0	31.0
																Q	2.66	2.0	30.0	30.0
	8U	40.7		47	35	61	34	36	2.75	CH			2.38	33	3					
	12U	61.0		28	55	85	58	48	2.75	CH						Q	2.84	0.5	64.0	64.0
																Q	3.84	1.0	59.0	59.0
																Q	2.79	2.0	49.0	49.0
	14U	70.5		31	21	43	10	20		ML	57	43	5.09	20	4					
85	6S	26.0		17	38	55	23	36		CH			0.80	39	4					
													1.38	42	3					
													1.39	40	5					
	8S	36.0		14	69	117	74	69		CH			2.91	68	5					
													2.94	67	5					
													3.40	59	4					
90U	3U	11.0		4	19	35	17	19		CL										
92	6S	26.0		33	20	27	8	19		CL			1.33	21	7					
													1.59	21	6					
													2.49	21	8					
	7S	31.0		30	20					CH			2.16	20	3					
													5.66	18	5					
													4.56	19	5					
	8S	36.0		25	21	34	12	20		CL			3.95	22	10					
													4.29	20	6					
93U	9U	35.9		50	22	53	29	23		CH			7.45	20	5	Q	4.29	0.5	22.0	22.0
	10U	41.0		70	22					CH			3.50	21	6	Q	4.00	1.0	21.0	21.0
	11U	45.7		90	16	26	10	16		CL						Q	2.46	2.0	22.0	22.0
	12U	50.8		120	17	48	31	17		CH			1.09	17	4					
													3.87	17						
													5.40	16	6					
96U	4U	18.0		22	23	31	40	24		CH						Q	1.29	0.5	22.0	22.0
																Q	1.57	1.0	21.0	21.0
																Q	1.32	2.0	22.0	22.0
	7U	36.1		43	22	50	30	22		CH			3.11	21	6					
98U	6U	25.7		33	21					CH						Q	3.04	0.5	20.0	20.0
																Q	3.18	1.0	21.0	21.0
																Q	3.14	2.0	21.0	21.0
	7U	30.6		33	24	54	27	25		CH			4.64	23	5	Q	3.74	1.0	25.0	25.0
																Q	3.00	2.0	24.0	23.0
	8U	37.7		41	16					CH						Q	3.05	0.5	15.0	15.0
																Q	4.05	0.5	17.0	17.0
99	8S	36.0		29	21	42	21	22		CL			3.96	20	9					
													4.46	20	9					
													4.05	21	12					
	9S	41.0		20	27					CH			2.33	28	8					

SAMPLE IDENTIFICATION					CLASSIFICATION PROPERTIES								PHYSICAL PROPERTIES							
													Unconfined Compression			Triaxial Compression				
Boring Number	Sample No.	Sample Depth	Stratum Designation	N-Value	Natural Water Content for Entire Sample	Liquid Limit	Plasticity Index	Natural Water Content of Limit Sample	Specific Gravity (G)	Soil Type	% Sand	%Finer Than 200 Sieve	Compressive Strength (TSF)	Water Content at End of Test (%)	Strain at Failure (%)	Type of Test	Deviator Stress	Confining Pressure	Natural Water Content	Water Content at End of Test
													2.56	26	11					
	10S	46.0		66	21	45	25	21		CL			5.44	21	5					
													4.21	20	6					
	12S	56.0		66	20					CH			6.26	19	3					
													4.14	21	5					
	13S	60.7		62	17	45	23	18		CL			####	16	2					
													3.96	18	4					
103	7S	31.0		58	22	48	26	22		CL			3.66	20	8					
													3.66	20	8					
													2.90	23	13					
	8S	36.0		60	22					CH			2.07	24	9					
													1.32	24	11					
													2.91	20	8					
	9S	40.8		65	20					CH			4.32	19	9					
													2.84	20	16					
104U	8U	35.6		90	19					CH						Q	3.60	2.0	19.0	19.0
	9U	40.6		90	19	42	25	20	2.80	CL						Q	3.58	1.0	19.0	18.0
	10U	45.6		90	18					CH						Q	8.45	0.3	17.0	17.0
																Q	8.65	1.0	16.0	15.0
	12U	54.7		54	16	33	15	17	2.73	CL						Q	8.95	0.5	14.0	14.0
111U	13U	60.0		16	45	64	36	45	2.76	CH						Q	3.71	0.5	45.0	44.0
																Q	2.89	1.0	47.0	47.0
																Q	2.83	2.0	43.0	43.0
	14U	62.5		16	51	77	44	52	2.78	CH						Q	2.20	0.5	51.0	50.0
																Q	3.55	1.0	53.0	52.0
																Q	3.56	2.0	50.0	50.0
	15U	66.0		14	49	59	34	48		CH						Q	3.70	0.5	50.0	49.0
																Q	3.04	1.0	47.0	47.0
																Q	2.34	2.0	49.0	48.0
112	14S	63.0		2	49	73	39	50		CH			2.21	49	6					
													2.71	47	6					
													2.73	49	6					
	15S	66.0		20	49					CH			3.28	48	4					
													3.42	47	4					
													2.90	49	3					
	16S	71.0		39	41	60	40	39		CH			3.09	43	7					
													4.22	42	4					
													3.11	40	3					
113U	16U	76.0		12	36	57	31	43	2.79	CH+SM			0.77	30	3	Q	2.54	0.5	37.0	37.0
																Q	2.42	1.0	38.0	38.0
																Q	2.64	2.0	29.0	29.0
	17U	80.1		12	47					CL			1.72	49	2	Q	2.04	0.5	50.0	50.0
																Q	4.04	1.0	46.0	46.0
																Q	3.96	2.0	47.0	47.0
	18U	84.1		15	48	73	44	43	2.63	CH			6.75	46	3	Q	3.05	0.5	46.0	46.0
																Q	3.70	1.0	51.0	51.0
																Q	4.01	2.0	51.0	50.0
115U	17U	80.0		8	36					ML			2.13	40	2					
	18U	85.0		8	33	70	35	43	2.77	CH						Q	4.08	0.5	47.0	48.0

SAMPLE IDENTIFICATION					CLASSIFICATION PROPERTIES								PHYSICAL PROPERTIES								
													Unconfined Compression			Triaxial Compression					
Boring Number	Sample No.	Sample Depth	Stratum Designation	N-Value	Natural Water Content for Entire Sample	Liquid Limit	Plasticity Index	Natural Water Content of Limit Sample	Specific Gravity (G)	Soil Type	% Sand	%Finer Than 200 Sieve	Compressive Strength (TSF)	Water Content at End of Test (%)	Strain at Failure (%)	Type of Test	Deviator Stress	Confining Pressure	Natural Water Content	Water Content at End of Test
																Q	2.86	1.0	39.0	39.0
																Q	3.67	2.0	27.0	27.0
	19U	91.0		24	42	60	34	42	2.64	CH						Q	4.03	0.5	42.0	41.0
																Q	3.97	1.0	41.0	41.0
																Q	4.11	2.0	42.0	42.0
118	13S	65.0		9	37	51	30	37		CL			0.52	38	6					
													2.37	35	6					
													1.71	36	5					
120U	6U	23.0		6	38					ML			1.92	37	2					
	7U	26.0		6	40	53	22	40		MH										
B&O ROUTE																				
12	5S	18.0		24	18.9	48.3	26.6	21.4		CL			4.44	19.7	9.1	Q	3.12	1.0	22.7	22.6
	7S	26.0		21	18.7	47.6	33.5	18.7		CL						Q	2.40	0.6	18.7	18.5
																Q	3.11	1.0	20.6	20.5
	8S	31.0		20	18.6	60.2	45.4	18.4		CH	30	70				Q	3.10	0.5	17.4	17.3
																Q	3.20	1.0	18.6	18.5
																Q	3.34	2.0	19.6	19.3
	9S	36.0		20	19.6	56.8	44.2	19.8		CH	27	73				Q	2.10	0.5	20.4	20.3
																Q	4.44	1.0	18.0	17.8
																Q	4.46	2.0	21.1	21.0
13U	7U	32.6		27	18.8	50.7	31.6	17	2.74	CH			6.88	19	3.9					
	8U	35.6		27	19.2	51.7	33.7	18.3	2.75	CH			2.89	19.2	2.1					
	9U	39.6		30	16.9	42.4	24.5	15.6	2.74	CL			6.85	17	5.2					
	10U	44.6		35	18.5	51.3	33.4	18.6	2.75	CH			6.87	18.8	6					
	12U	52.7		50	13.7					CL										
	15U	65.9		32	26.6	57	40.3	26	2.68	CH	52	48				Q	5.18	1.5	27.6	27.6
18	18S	83.0		47	29.4	65.5	32.6	29.7	2.74	CH	23	77				Q	3.01	0.5	27.9	27.7
																Q	3.52	1.0	30.4	30.2
																Q	4.38	2.0	30.8	30.6
	19S	89.8		122	15.7	42.8	25.8	16.1		CL						Q	5.84	0.5	17.1	16.9
																Q	9.50	1.0	15.0	14.9
19B	5S	21.0		15	19.8	27.3	11.9	20.1	2.73	CL	30	70				Q	1.81	0.5	20.4	20.2
																Q	1.55	1.0	20.4	20.3
																Q	1.92	2.0	19.4	19.3
	7S	30.8		7	21.3	28.4	12	21.5		CL						Q	1.45	0.9	21.5	21.2
	16S	75.8		30	17.3	38.4	24.2	18		CL						Q	3.29	0.5	15.2	15.1
																Q	3.76	1.0	20.7	20.5
																Q	6.20	2.0	15.4	15.2
20	7S	31.0		7	19.2	22.9	9.5	19.6		CL			1.70	18.4	6.4	Q	0.85	1.0	19.6	19.4
																Q	1.16	2.0	19.4	19.3
23UA	6U	22.1		7	23.6	35.7	17.2	20.8		CL			1.52	24.1	7.6	Q	1.74	1.0	23.6	23.7
																Q	1.68	2.0	23.3	22.5
	7U	26.0		7	23.4					CL						Q	2.18	0.5	24.0	23.9
																Q	2.41	1.0	23.6	23.6
																Q	2.58	2.0	22.6	22.4

SAMPLE IDENTIFICATION					CLASSIFICATION PROPERTIES								PHYSICAL PROPERTIES							
													Unconfined Compression			Triaxial Compression				
Boring Number	Sample No.	Sample Depth	Stratum Designation	N-Value	Natural Water Content for Entire Sample	Liquid Limit	Plasticity Index	Natural Water Content of Limit Sample	Specific Gravity (G)	Soil Type	% Sand	%Finer Than 200 Sieve	Compressive Strength (TSF)	Water Content at End of Test (%)	Strain at Failure (%)	Type of Test	Deviator Stress	Confining Pressure	Natural Water Content	Water Content at End of Test
	8U	29.8		7	21.7	43.2	22.8	21.1		CL			2.67	22.6	7.7	Q	2.39	1.0	20.6	20.4
																Q	3.00	2.0	21.8	21.3
	9U	34.0		11	21.6					CL			2.48	21	13.3	Q	1.70	1.0	22.5	22.2
																Q	2.60	2.0	21.0	20.8
24U	8U	25.8		10	22.7	25.4	11.4	21.2	2.69	CL			1.15	22.2	8.5					
	9U	30.0		19	18.7	29.8	15.3	18	2.70	CL	23	77				Q	2.32	1.0	18.6	18.4
																Q	1.84	2.0	18.9	18.6
	13U	45.8		27	24.3	41.2	24.2	25.6	2.66	CL	11	89				Q	0.96	1.0	24.1	24.6
27U	3U	8.0		14	20.8	36.1	20.9	24.6	2.68	CL										
	6U	15.8		14	17.1					SM						Q	1.69	0.5	15.3	15.6
	7U	21.0		13	19.1	26.2	10.6	17.9	2.70	CL						Q	2.12	0.7	18.7	18.2
																Q	1.41	1.3	20.3	19.5
	8U	28.0		12	20.1	24.2	9.6	20	2.68	CL	30	70				Q	1.44	0.7	20.1	19.8
																Q	0.93	1.4	19.9	19.1
	17U	55.0		10	29.2	39.1	20.6	30.6	2.70	CL	14	86				Q	1.62	1.2	29.0	28.8
																Q	1.54	2.3	29.0	28.6
	18U	60.0		18	27.6	33	16	25.7	2.68	CL						Q	1.77	1.6	26.5	26.2
																Q	0.60	3.2	28.3	27.4
28	4S	16.5		17	17.4															
	5S	21.0		16	20	20.6	11.9	20.4	2.70	CL	45	55	0.47	18.4	3.2	Q	0.87	0.7	20.5	20.0
																Q	0.90	1.3	20.7	20.6
	6S	26.0		16	18.9	24.6	8.3	19.4		CL			1.02	17	5	Q	1.97	0.9	20.0	19.6
																Q	2.16	1.7	18.9	18.6
29D	6S	26.0		22	21	27.8	13.3	17		CL						Q	1.49	0.5	15.8	15.6
																Q	1.45	1.0	18.5	18.4
																Q	3.74	2.0	17.3	17.2
30U	11U	29.0		4	22.4	44.6	22.5	22.6	2.74	CL			3.50	22.4	4.3					
33U	2U	5.6			19.8					CL			0.40	19.5	10					
	7U	34.3			22					CH			2.14	22.1	9					
34U	6U	25.8			23.3	52.1	29.2	23.1	2.81	CH						Q	0.88	0.5	22.7	22.9
																Q	0.93	1.5	23.8	23.8
																Q	1.60	3.0	22.9	23.8
	7U	30.8			23.8	62.5	39	23.5		CH			1.67	25.1	4					
35U	2U	6.0			27.2					CH			0.56	26.9	11					
	3U	10.8			28.1					CH			0.41	27.5	7	Q	0.52	1.0	29.4	30.8
																Q	0.51	3.0	27.4	28.8
	4U	15.7			25.6					CH			1.02	23.8	2	Q	1.27	1.0	24.4	24.8
																Q	0.98	3.0	28.4	28.4
	5U	20.8			24.8	63.4	41.3	23.9	2.79	CH			1.01	24.9	4	Q	0.81	1.0	24.7	26.4
																Q	0.93	3.0	24.9	24.9
	6U	25.7			23.7	75	55	22.3	2.77	CH			1.44	23.2	6					
36U	2U	5.5			28.3					CH			0.74	28	10					
	3U	10.8			26.5					CH			0.59	26.5	18	Q	0.59	1.0	27.1	27.6
																Q	0.46	3.0	25.2	25.3
	4U	15.7			24.4					CH			1.21	20.4	12	Q	0.37	1.0	28.6	30.3

SAMPLE IDENTIFICATION					CLASSIFICATION PROPERTIES								PHYSICAL PROPERTIES							
													Unconfined Compression			Triaxial Compression				
Boring Number	Sample No.	Sample Depth	Stratum Designation	N-Value	Natural Water Content for Entire Sample	Liquid Limit	Plasticity Index	Natural Water Content of Limit Sample	Specific Gravity (G)	Soil Type	% Sand	%Finer Than 200 Sieve	Compressive Strength (TSF)	Water Content at End of Test (%)	Strain at Failure (%)	Type of Test	Deviator Stress	Confining Pressure	Natural Water Content	Water Content at End of Test
																Q	0.56	3.0	23.9	24.4
	5U	20.7			22.6	54	37.8	21.3	2.79	CH						Q	0.69	1.0	21.1	21.4
																Q	1.28	3.0	23.9	24.9
	6U	25.6			26.3	73.9	53.4	24.6	2.84	CH			0.84	25.1	6					
	7U	30.6			17.7															
	8U	35.5			24.9	38.3	7.8	22.7	2.68	ML			2.20	24.3	15					
VII-1U	3U	7.4		11	20					CL			0.59	19	11					
VII-1UA	10U	27.7		11	26.8	60.8	39.3	25.2	2.72	CH			1.84	26	10	Q	1.53	3.0	27.7	27.8
	12U	32.6		9	27					CH			1.32	25.1	7	Q	1.31	3.0	27.3	27.7
	14U	37.9		9	25.9	51.2	31.1	25.8	2.80	CH			2.44	24	11	Q	1.02	1.0	28.1	28.9
																Q	1.46	3.0	25.6	26.4
	16U	42.9		20	36.1	62.3	40.6	38.5		CH	0	100	1.02	37.1	7					
	17U	48.0		20	36.4	54.2	32.7	32.6	2.74	CH			1.12	37.1	7					
	18U	53.0		20	50.9					CH			0.96	46.3	5					
													1.33	55.3	6					
	19U	58.2		20	41.3	59.2	35	38	2.68	CH			1.27	42.3	3					
	20U	62.8		20	28.2	42.3	22.3	28.2		CL					12	Q	1.18	1.0	27.3	27.9
																Q	0.72	3.0	29.9	30.8
VII-2	9S	31.0		22	25.9	43.6	24.5	23.4		CL	7	93	0.61	26.5	6					
													0.76	24.2	10					
	11S	40.9		8	21.3					CL			0.81	20.4	15					
VII-3U	7U	22.7		10	22.2	27.9	11.3	23.1	2.70	CL			1.40	22.6	8					
	14U	48.0		8	34	47	23	34.8	2.73	CL			0.46	32.8	10					
	17U	57.8		3	17.4					CL			0.81	20.4	15					
VII-4	8S	25.7		19	20.6					CL			2.24	20.7	4					
													2.54	20	6					
	10S	32.7		37	21.1					CL			2.76	20.8	4					
VII-6U	5U	21.7		17	16.4					CL						Q	1.30	1.0	16.8	17.6
																Q	2.42	3.0	13.9	14.0
	7U	30.8		17	21.9					CL			1.83	21.8	5	Q	1.68	1.0	20.5	21.4
																Q	1.55	3.0	23.2	23.6
VII-10U	9U	42.6		35	23.3					CH			2.82	23.2	7					
VII-11U	8S	33.5		26	27.5					CH			1.49	27.2	4					
VII-14U	3S	7.8		33	24.4	64.9	45.1	31.4		CH			2.75	28.6	7					
													0.55	18	4					
VII-15	6S	22.5		8	17.9					CL			2.29	19.7	4					
													1.65	15.8	9					
VII-23U	4U	17.9		10	22.7	32.8	14.5	25.6	2.71	CL						Q	2.08	1.0	22.4	26.8
																Q	0.43	3.0	26.6	26.4
	5U	20.7		10	31.9	48	30.6	21.4	2.75	CL			1.67	35	9					
	8U	33.0		26	29	60.9	43.8	23.8	2.74	CH						Q	0.33	1.0	28.5	27.7
																Q	1.90	3.0	22.2	22.6
	10U	37.9		18	26.3					CH			1.54	23.2	6	Q	1.48	1.0	25.1	25.7
																Q	0.68	3.0	30.6	31.3

SAMPLE IDENTIFICATION					CLASSIFICATION PROPERTIES								PHYSICAL PROPERTIES							
													Unconfined Compression			Triaxial Compression				
Boring Number	Sample No.	Sample Depth	Stratum Designation	N-Value	Natural Water Content for Entire Sample	Liquid Limit	Plasticity Index	Natural Water Content of Limit Sample	Specific Gravity (G)	Soil Type	% Sand	%Finer Than 200 Sieve	Compressive Strength (TSF)	Water Content at End of Test (%)	Strain at Failure (%)	Type of Test	Deviator Stress	Confining Pressure	Natural Water Content	Water Content at End of Test
VII-26	14S	61.7		121	19.3					CL			5.15	18.9	2					
													4.13	19.3	2					
VII-29U	3U	7.5		15	22.6	57	34.8	24.8	2.68	CH			1.67	22.5	6					
	10S	31.6		33	19.7	49.4	27.7	18.8		CL			4.80	19.7	11					
	11S	34.8		48	19.4					CL			2.09	18.2	20					
													1.52	20.1	6					
VII-30	8S	30.6		9	20.6					CH			5.48	20.6	7					
VII-31	7S	23.8		39	22.3					CH			5.16	22	12					
	9S	28.7		39	17					CL			4.87	17.2	12					
VII-34	7S	34.0		17	20.9					ML			1.64	20.4	3					
	10S	37.1		47	21.4	72.7	52.4	21.3		CH			1.59	21.6	1					
													1.13	21.1	1					
VII-35	8S	28.5		30	18.3	52.5	34.6	20.9		CH			3.32	18.1	6					
													2.89		7					
VII-49	7S	19.9		10	26.5	62.1	38.2	24.6		CH			1.56	26.5	5					
	8S	23.6		10	23.6	76.2	52.7	23.2		CH	1	99	2.40	23.2	7					
VII-50	8S	22.1		35	23.4	66	39.3	20.9		CH			3.96	23.4	3					
	10S	26.3		23	17.2	37.9	17.5	17		CL	19	81	6.02	16.8	7					
VII-53	8S	26.0		15	17	36.1	18.8	16		CL			2.12	16.6	11					
VII-56	9U	25.6		7	16.4					CL	42	58								
LE'FANT-PENTAGON ROUTE																				
L-1	9S	45.5		9	50					CL			2.06	70	7.4					
													2.63	41.9	5.4					
L-2	9S	39.0		26	23.1					CL										
L-5	4S	14.5		3	67.6	101	60.7	83.8		OH			0.34	66	8.1					
													0.53	60.9	10.9					
													0.44	74	6.2					
	5S	19.5		2	84.7					OH			0.51	85.5	6.1					
													0.59	83.7	6.7					
	10S	40.0		4	69					OH			0.64	68.5	6.2					
	11S	45.0		2	71.6					CH			0.59	69	8.6					
					221					OH			0.48	220	9.9					
L-6	3S	11.0		2	76.8					OH			0.43	76.9	7.4					
													0.43	74.2	8.5					
													0.42	78.1	8.7					
	5S	21.0		2	80.8	96.5	52.4	83.8		CH			0.44	77.2	6.2					
													0.58	84	6.5					
	6S	26.0		3	83.7					OH			0.52	85.1	6.9					
													0.48	78.5	4.9					
													0.49	84.5	6.2					

SAMPLE IDENTIFICATION					CLASSIFICATION PROPERTIES								PHYSICAL PROPERTIES							
													Unconfined Compression			Triaxial Compression				
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	7S	31.0		5	68					OH			0.53	70	5.9					
													0.59	64.9	7.4					
	8S	36.0		3	69.7	81.3	47.6	67		OH			0.63	71.1	8.6					
													0.39	68	7.4					
													0.71	69.7	9.9					
	9S	41.0		2	65.1					OH			0.73	66.4	7.5					
													0.79	63.2	9.2					
	10S	46.0		2	68.2					OH			0.65	58	8.7					
													0.82	70	9.9					
													0.61	68	6.7					
L-7U	5U	21.1		3	76	72.2	22.7	77	2.57	OH			0.51	71.4	6.2	Q	0.57	2.0	79.2	78.5
	6U	26.1		2	86.5					OH						Q	0.48	0.5	92.0	91.9
																Q	0.49	1.0	82.8	82.3
	7U	31.1		2	77					OH			0.56	69.7	9.7	Q	0.48	0.5	75.6	75.5
																Q	0.62	1.0	81.8	81.5
	8U	36.1		3	73.3	69.6	25.4	71.4		OH			0.65	72.6	8	Q	0.65	2.0	73.8	73.4
	9U	41.1		4	72.7					OH			0.45	73.1	6.2					
													0.46	76.5	3.5					
	11U	51.1		3	63.4	77.4	29.3	69.5	2.64	OH			0.56	65.1	5.3	Q	0.59	1.0	66.8	66.6
	12U	56.1		2	56.8					OH			0.82	62.6	6.4	Q	0.50	0.5	57.2	56.4
																Q	0.74	2.0	57.4	57.0
	13U	61.1		3	65.8	71.8	25.3	60.9	2.52	OH			0.33	69.6	12.1					
													0.81	71.9	6.3					
L-9U	4U	16.0		3	54					OH			0.48	51.1	8.9	Q	0.47	0.5	58.1	57.7
																Q	0.55	2.0	52.5	52.2
	5U	21.0		4	73.3	62.6	38.2	61.3		OH			0.60	70.3	7.2	Q	0.56	1.0	77.7	77.5
	6U	26.0		5	80.7					OH			0.59	87	5.4	Q	0.60	0.5	80.6	80.1
																Q	0.58	1.0	74.5	73.0
	7U	31.0		3	87.4	82.3	26.2	88.3	2.55	OH			0.60	90.2	8	Q	0.78	2.0	83.5	83.0
	8U	36.0		5	64.5					OH			0.77	46.7	7.1	Q	0.73	0.5	72.0	72.0
																Q	0.78	1.0	70.0	
	9U	41.0		6	72.1	76.6	27.6	81		OH			0.83	66.1	7.1	Q	0.88	2.0	76.3	76.2
	10U	46.0		6	71					OH			0.86	80.2	6.3	Q	0.78	0.5	64.5	63.7
																Q	0.86	1.0	68.5	67.6
	11U	51.0		6	70.7	73	27.5	63.7	2.55	OH			0.74	72.5	7.6	Q	0.78	2.0	71.1	70.7
	14U	66.0		6	56.7	70.7	26.5	64.8		OH						Q	0.88	0.5	42.0	42.0
																Q	0.76	1.0	54.9	54.7
																Q	0.89	2.0	72.0	71.5
L-10	3S	12.0		3	65.3	83.4	42	70.1		OH			0.55	69.1	7.3					
													0.51	58.6	8.5					
													0.60	53.8	9.8					
													0.63	47	8.6					
													0.49	46.7	8.7					
													0.66	63.1	9.4					
													0.65	60.6	13.5					
													0.79	84.1	8.6					
													0.98	79.7	9.6					
													0.68	76.4	9.7					
L-12	7S	31.0		10	65.7					OH			0.90	65	7.9					
													1.06	73.1	7.9					
													0.91	58.6	7.3					

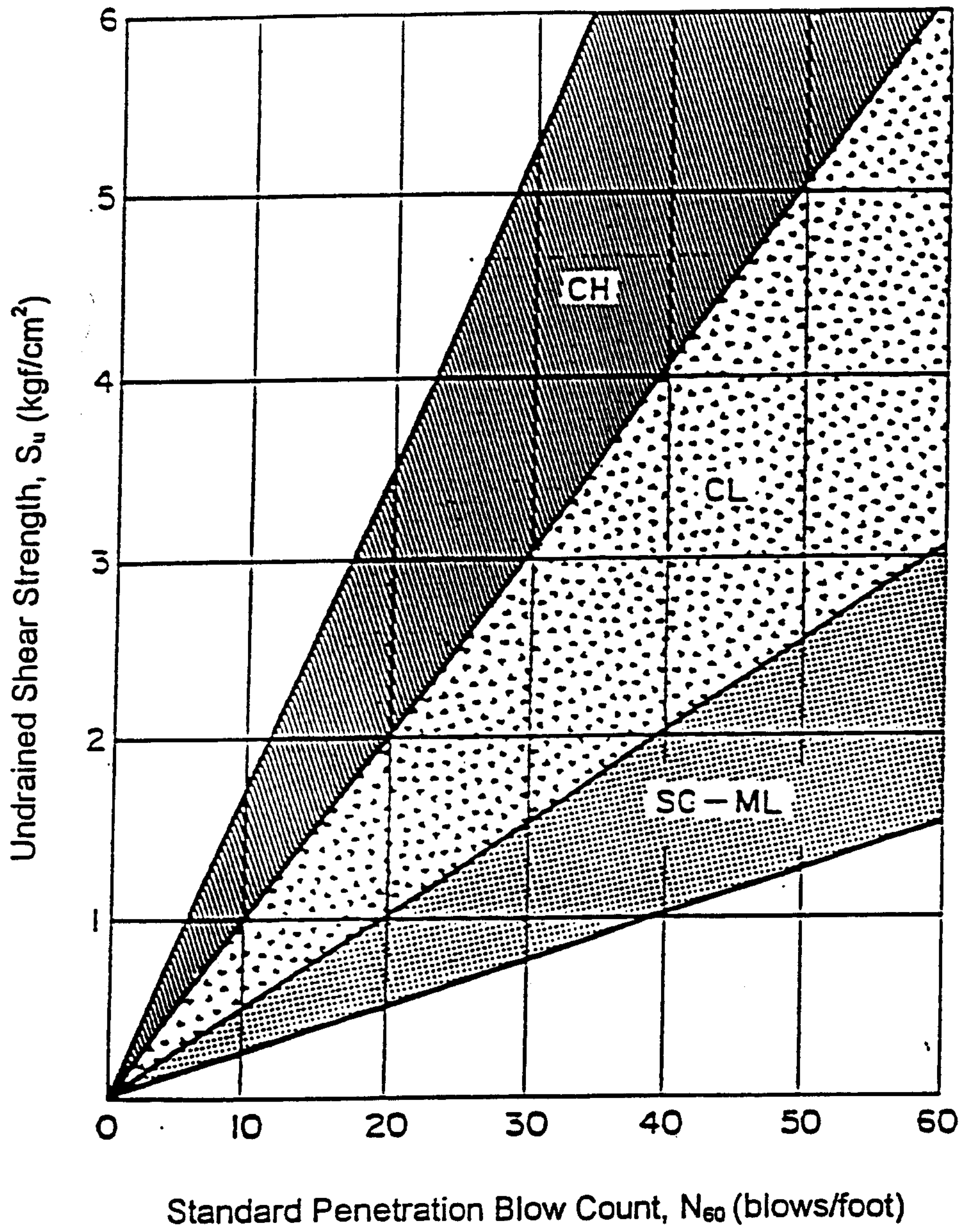
SAMPLE IDENTIFICATION					CLASSIFICATION PROPERTIES								PHYSICAL PROPERTIES							
													Unconfined Compression			Triaxial Compression				
Boring Number	Sample No.	Sample Depth	Stratum Designation	N-Value	Natural Water Content for Entire Sample	Liquid Limit	Plasticity Index	Natural Water Content of Limit Sample	Specific Gravity (G)	Soil Type	% Sand	%Finer Than 200 Sieve	Compressive Strength (TSF)	Water Content at End of Test (%)	Strain at Failure (%)	Type of Test	Deviator Stress	Confining Pressure	Natural Water Content	Water Content at End of Test
L-13	13S	64.0		7	64.6					OH			0.78	62.6	7.8					
	14S	66.1		11	62.4					OH			1.28	65.2	6.7					
													1.26	59.3	6.7					
	15S	68.4		11	59.1	77.6	43.5	63.3		OH			1.20	57.9	6.7					
													1.22	58	7.9					
	16S	71.0		11	64.5					OH			1.23	61.5	9					
													1.00	66.9	10.4					
	18S	78.1		14	70.9					OH			1.44	71	7.4					
													1.25	70.1	8.9					
	20S	88.6		18	61					OH			0.94	69.4	10.1					
													1.41	54.9	7.5					
L-14U	4U	16.0		3	60.8	57.7	25.2	45.9	2.64	OH			0.79	61.4	6.3					
	14U	73.0		24	56.3	78.1	31.6	62.2	2.55	OH			1.64	54.9	5.4	Q	1.40	1.0	56.3	56.1
	15U	76.0		24	57.1					OH			1.15	53.9	9.5	Q	1.40	0.5	64.4	64.1
																Q	1.51	2.0	52.9	52.4
	16U	79.0		25	65.4					OH			1.85	56.4	6.3	Q	1.81	0.5	69.4	68.7
																Q	1.82	2.0	70.2	69.9
	17U	82.0		28	58.9	57.8	27.9	52.8	2.75	OH			2.06	60.8	6.8	Q	1.68	1.0	61.2	61.2
	18U	84.0		28	62.3					OH			1.97	66	7.7	Q	1.97	0.5	55.7	55.6
																Q	2.01	2.0	64.3	64.2
	19U	88.0		30	60.5					OH			1.63	71.8	6.7	Q	1.38	1.0	48.5	48.2
													1.07	59	7.4					
	20U	92.0		42	56.3					OH			1.41	55.8	8.9					
													1.76	57	8.1					
L-15	6S	24.0		8	45.7					OH			0.61	43.8	7.9					
													0.65	47.6	8.5					
L-16	6S	24.0		7	49	61	30.8	50.3		OH			0.75	52	10					
													0.81	48.7	9					
													0.86	46	10					
	10S	40.9		8	45					OL			0.90	39.4	7					
													0.75	51.5	7					
	12S	51.0		6	42					OH			0.19	33.2	3					
													0.49	49.5	6					
	13S	56.0		7	59	50.7	18.8	56		OH			1.60	63	8					
													1.58	61.2	6					
													1.24	50.6	9					
	15S	63.0		7	58	76.7	24.7	65.5		OH			1.20	57.1	6					
													1.21	54.9	7					
													1.30	60.2	8					
L-17U	4U	12.9		2	51	68.6	37	51.9	2.71	OH			0.59	56.5	8	Q	0.50	0.5	44.1	43.6
																Q	0.68	1.0	50.5	49.6
	5U	16.0		5	46	58.6	27.9	47		OH			0.69	42.5	11	Q	0.67	2.0	49.5	49.0
	11U	37.9		8	42	64.7	47.3	62.7	2.59	OH			0.93	40.2	5	Q	0.65	0.5		37.9
																	1.25	1.0	40.4	39.8
	17U	55.0		13	52	60.9	23.3	53.6		OH			1.17	53	7	Q	0.90	0.5	57.0	56.6
																Q	1.24	1.0	41.2	40.5
	18U	58.0		13	54					OH						Q	1.01	0.5	47.0	46.8
																Q	1.26	1.0		57.7
																Q	1.25	2.0	56.3	55.6
	19U	61.0		32	61	89.2	47.9	62	2.60	OH			0.44	61.6	6	Q	1.59	2.0	55.1	54.5

SAMPLE IDENTIFICATION					CLASSIFICATION PROPERTIES								PHYSICAL PROPERTIES							
													Unconfined Compression			Triaxial Compression				
Boring Number	Sample No.	Sample Depth	Stratum Designation	N-Value	Natural Water Content for Entire Sample	Liquid Limit	Plasticity Index	Natural Water Content of Limit Sample	Specific Gravity (G)	Soil Type	% Sand	%Finer Than 200 Sieve	Compressive Strength (TSF)	Water Content at End of Test (%)	Strain at Failure (%)	Type of Test	Deviator Stress	Confining Pressure	Natural Water Content	Water Content at End of Test
																Q	1.11	2.0	64.0	62.6
L-18	7S	26.0		11	34					SM			0.82	40.5	5					
													0.63	33.2	5					
													0.39	23	4					
	11S	40.9		9	76					OH			1.70	62	7					
													1.91	70	6					
													1.98	70.4	6					
	12S	46.0		9	61	80.9	30.4	61.5	2.65	OH			1.46	60.3	8					
													1.13	62.2	8					
													1.60	59.4	8					
	13S	51.0		10	59	105	39.2	75		OH			1.38	55.5	8					
													1.09	50	4					
													1.42	68.8	7					
	14S	56.0		10	24					OL			0.74	33	4					
													1.16	33.5	4					
													0.78	35.8	4					
L-20U	4U	11.6		2	61	92.8	55.2	76.6		CH						Q	0.25	0.5	57.2	56.8
	5U	16.0		2	69	91.2	40.1	70.2		OH			0.38	43.6	11	Q	0.54	1.0	88.5	88.1
																Q	0.45	2.0	74.5	74.1
	6U	20.0		3	85	85	48.5	60		CH			0.51	93	13	Q	0.32	0.5	92.5	91.2
																Q	0.51	1.0	78.0	77.4
	7U	24.0		4	88	90.8	52.6	79		OH			0.43	97	12	Q	0.43	0.5	93.5	92.2
																Q	0.58	2.0	76.0	74.8
	8U	27.9		10	68					OL						Q	0.49	1.0		85.0
																Q	0.45	2.0	51.2	50.0
	12U	47.0		16	43	52.5	19.7	41.2		OH			1.59	48.5	7	Q	1.81	0.5	35.5	35.5
																Q	1.56	1.0	43.9	43.6
	13U	50.9		20	48					OH			1.43	45.5	15	Q	2.10	2.0	49.0	48.2
L-26	14S	64.5		53	26					CH			2.24	27.1	1					
L-33U	8U	32.4		16	19	28	10.4	19.4		CL			1.09	19.3	4					
L-37U	5U	23.8		5	119	228	66.5	132.4	2.10	OH										
	6U	27.5		4	24					ML						Q	0.46	0.5	22.6	22.4
																Q	0.42	1.0	24.4	23.1
																Q	0.59	2.0	24.3	23.4

APPENDIX C

Shear Strength vs. N-Value Graph

- Shear Strength vs. N-value (after Sowers, 1979)



Note: As originally proposed, this correlation used the uncorrected SPT blowcount, N. However, hammers delivering 60% of the theoretical energy have been the most commonly used hammers for SPT tests, and it seems likely that the data on which the correlation was based was obtained primarily from tests with such hammers. It therefore seems logical to use N_{60} with this correlation, and it is the recommendation of this report that this be done.

Figure 40. Relationship between standard penetration blow count, N and undrained shear strength, S_u (after Sowers, 1979)

APPENDIX D

- ASTM D 1586 – Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils.

Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils[1]

This standard is issued under the fixed designation D 1586: the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

This standard has been approved for use by agencies of the Department of Defense. Consult the DOD Index of Specifications and Standards for the specific year of issue which has been adopted by the Department of Defense.

$^{\epsilon 1}$ NOTE—Editorial changes were made throughout October 1992.

1. Scope

1.1 This test method describes the procedure, generally known as the Standard Penetration Test (SPT), for driving a split-barrel sampler to obtain a representative soil sample and a measure of the resistance of the soil to penetration of the sampler.

1.2 *This standard does not purport to address all of the safety problems, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.* For a specific precautionary statement, see 5.4.1.

1.3 The values stated in inch-pound units are to be regarded as the standard.

2. Referenced Documents

2.1 *ASTM Standards:*

D 2487 Test Method for Classification of Soils for Engineering Purposes[2]

D 2488 Practice for Description and Identification of Soils (Visual-Manual Procedure)[2]

D 4220 Practices for Preserving and Transporting Soil Samples[2]

D 4633 Test Method for Stress Wave Energy Measurement for Dynamic Penetrometer Testing Systems[2]

3. Terminology

3.1 *Descriptions of Terms Specific to This Standard*

3.1.1 *anvil*—that portion of the drive-weight assembly which the hammer strikes and through which the hammer energy passes into the drill rods.

3.1.2 *cathead*—the rotating drum or windlass in the rope-cathead lift system around which the operator wraps a rope to lift and drop the hammer by successively tightening and loosening the rope turns around the drum.

3.1.3 *drill rods*—rods used to transmit downward force and torque to the drill bit while drilling a borehole.

3.1.4 *drive-weight assembly*—a device consisting of the hammer, hammer fall guide, the anvil, and any hammer drop system.

3.1.5 *hammer*—that portion of the drive-weight assembly consisting of the 140 ± 2 lb (63.5 ± 1 kg) impact weight which is successively lifted and dropped to provide the energy that accomplishes the sampling and penetration.

3.1.6 *hammer drop system*—that portion of the drive-weight assembly by which the operator accomplishes the lifting and dropping of the hammer to produce the blow.

3.1.7 *hammer fall guide*—that part of the drive-weight assembly used to guide the fall of the hammer.

3.1.8 *N-value*—the blowcount representation of the penetration resistance of the soil. The *N*-value, reported in blows per foot, equals the sum of the number of blows required to drive the sampler over the depth interval of 6 to 18 in. (150 to 450 mm) (see 7.3).

3.1.9 ΔN—the number of blows obtained from each of the 6-in. (150-mm) intervals of sampler penetration (see 7.3).

3.1.10 *number of rope turns*—the total contact angle between the rope and the cathead at the beginning of the operator's rope slackening to drop the hammer, divided by 360° (see Fig. 1).

3.1.11 *sampling rods*—rods that connect the drive-weight assembly to the sampler. Drill rods are often used for this purpose.

3.1.12 *SPT*—abbreviation for Standard Penetration Test, a term by which engineers commonly refer to this method.

4. Significance and Use

4.1 This test method provides a soil sample for identification purposes and for laboratory tests appropriate for soil obtained from a sampler that may produce large shear strain disturbance in the sample.

4.2 This test method is used extensively in a great variety of geotechnical exploration projects. Many local correlations and widely published correlations which relate SPT blowcount, or *N*-value, and the engineering behavior of earthworks and foundations are available.

5. Apparatus

5.1 *Drilling Equipment*—Any drilling equipment that provides at the time of sampling a suitably clean open hole before insertion of the sampler and ensures that the penetration test is performed on undisturbed soil shall be acceptable. The following pieces of equipment have proven to be

[1] This method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.02 on Sampling and Related Field Testing for Soil Investigations.

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[2] *Annual Book of ASTM Standards*, Vol 04.08.

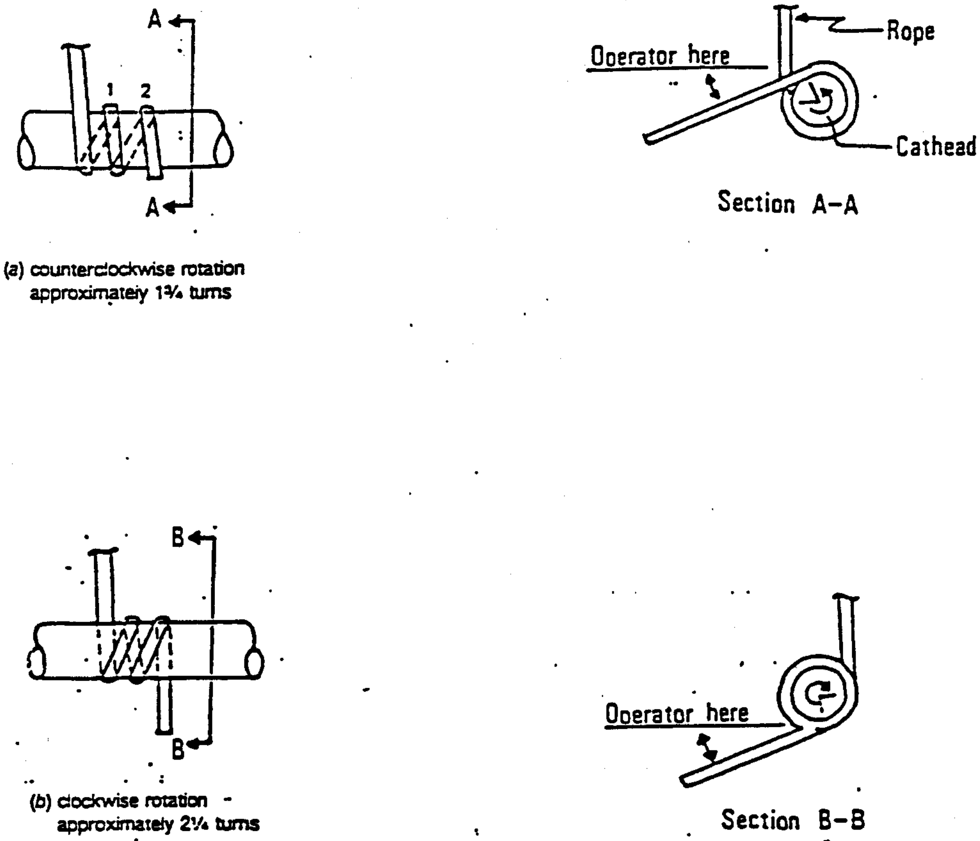


FIG. 1 Definitions of the Number of Rope Turns and the Angle for (*a*) Counterclockwise Rotation and (*b*) Clockwise Rotation of the Cathead

suitable for advancing a borehole in some subsurface conditions.

5.1.1 *Drag, Chopping, and Fishtail Bits*, less than 6.5 in. (162 mm) and greater than 2.2 in. (56 mm) in diameter may be used in conjuction with open-hole rotary drilling or casing-advancement drilling methods. To avoid disturbance of the underlying soil, bottom discharge bits are not permitted; only side discharge bits are permitted.

5.1.2 *Roller-Cone Bits*, less than 6.5 in. (162 mm) and greater than 2.2 in. (56 mm) in diameter may be used in conjunction with open-hole rotary drilling or casing-advancement drilling methods if the drilling fluid discharge is deflected.

5.1.3 *Hollow-Stem Continuous Flight Augers*, with or without a center bit assembly, may be used to drill the boring. The inside diameter of the hollow-stem augers shall be less than 6.5 in. (162 mm) and greater than 2.2 in. (56 mm).

5.1.4 *Solid, Continuous Flight, Bucket and Hand Augers*, less than 6.5 in. (162 mm) and greater than 2.2 in. (56 mm) in diameter may be used if the soil on the side of the boring does not cave onto the sampler or sampling rods during sampling.

5.2 *Sampling Rods*—Flush-joint steel drill rods shall be used to connect the split-barrel sampler to the drive-weight assembly. The sampling rod shall have a stiffness (moment of inertia) equal to or greater than that of parallel wall "A" rod (a steel rod which has an outside diameter of 1⅝ in. (41.2 mm) and an inside diameter of 1⅛ in. (28.5 mm).

NOTE 1—Recent research and comparative testing indicates the type rod used, with stiffness ranging from "A" size rod to "N" size rod, will usually have a negligible effect on the *N*-values to depths of at least 100 ft (30 m).

5.3 *Split-Barrel Sampler*—The sampler shall be constructed with the dimensions indicated in Fig. 2. The driving shoe shall be of hardened steel and shall be replaced or repaired when it becomes dented or distorted. The use of liners to produce a constant inside diameter of 1⅜ in. (35 mm) is permitted, but shall be noted on the penetration record if used. The use of a sample retainer basket is permitted, and should also be noted on the penetration record if used.

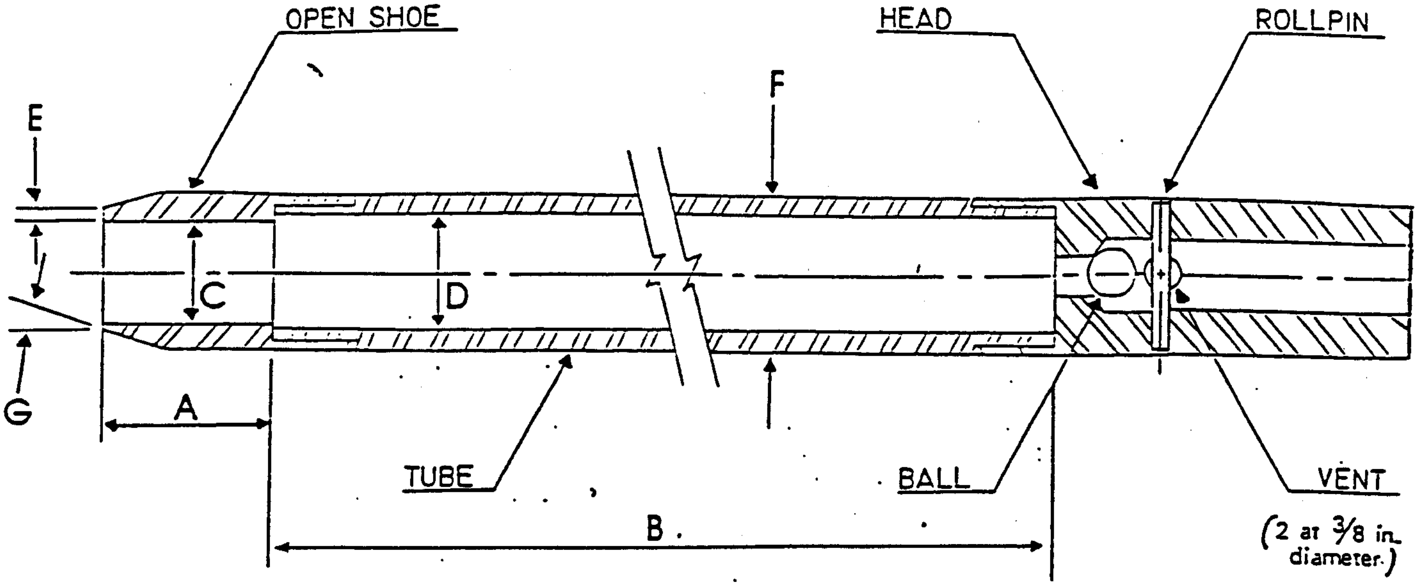
NOTE 2—Both theory and available test data suggest that *N*-values may increase between 10 to 30 % when liners are used.

5.4 *Drive-Weight Assembly:*

5.4.1 *Hammer and Anvil*—The hammer shall weigh 140 ± 2 lb (63.5 ± 1 kg) and shall be a solid rigid metallic mass. The hammer shall strike the anvil and make steel on steel contact when it is dropped. A hammer fall guide permitting a free fall shall be used. Hammers used with the cathead and rope method shall have an unimpeded overlift capacity of at least 4 in. (100 mm). For safety reasons, the use of a hammer assembly with an internal anvil is encouraged.

NOTE 3—It is suggested that the hammer fall guide be permanently marked to enable the operator or inspector to judge the hammer drop height.

5.4.2 *Hammer Drop System*—Rope-cathead, trip, semi-automatic, or automatic hammer drop systems may be used, providing the lifting apparatus will not cause penetration of



A = 1.0 to 2.0 in. (25 to 50 mm)
B = 18.0 to 30.0 in. (0.457 to 0.762 m)
C = 1.375 ± 0.005 in. (34.93 ± 0.13 mm)
D = 1.50 ± 0.05 − 0.00 in. (38.1 ± 1.3 − 0.0 mm)
E = 0.10 ± 0.02 in. (2.54 ± 0.25 mm)
F = 2.00 ± 0.05 − 0.00 in. (50.8 ± 1.3 − 0.0 mm)
G = 16.0° to 23.0°

The 1½ in. (38 mm) inside diameter split barrel may be used with a 16-gage wall thickness split liner. The penetrating end of the drive shoe may be slightly rounded. Metal or plastic retainers may be used to retain soil samples.

FIG. 2 Split-Barrel Sampler

the sampler while re-engaging and lifting the hammer.

5.5 *Accessory Equipment*—Accessories such as labels, sample containers, data sheets, and groundwater level measuring devices shall be provided in accordance with the requirements of the project and other ASTM standards.

6. Drilling Procedure

6.1 The boring shall be advanced incrementally to permit intermittent or continuous sampling. Test intervals and locations are normally stipulated by the project engineer or geologist. Typically, the intervals selected are 5 ft (1.5 mm) or less in homogeneous strata with test and sampling locations at every change of strata.

6.2 Any drilling procedure that provides a suitably clean and stable hole before insertion of the sampler and assures that the penetration test is performed on essentially undisturbed soil shall be acceptable. Each of the following procedures have proven to be acceptable for some subsurface conditions. The subsurface conditions anticipated should be considered when selecting the drilling method to be used.

6.2.1 Open-hole rotary drilling method.

6.2.2 Continuous flight hollow-stem auger method.

6.2.3 Wash boring method.

6.2.4 Continuous flight solid auger method.

6.3 Several drilling methods produce unacceptable borings. The process of jetting through an open tube sampler and then sampling when the desired depth is reached shall not be permitted. The continuous flight solid auger method shall not be used for advancing the boring below a water table or below the upper confining bed of a confined non-cohesive stratum that is under artesian pressure. Casing may not be advanced below the sampling elevation prior to sampling. Advancing a boring with bottom discharge bits is not permissible. It is not permissible to advance the boring for subsequent insertion of the sampler solely by means of previous sampling with the SPT sampler.

6.4 The drilling fluid level within the boring or hollow-stem augers shall be maintained at or above the in situ groundwater level at all times during drilling, removal of drill rods, and sampling.

7. Sampling and Testing Procedure

7.1 After the boring has been advanced to the desired sampling elevation and excessive cuttings have been removed, prepare for the test with the following sequence of operations.

7.1.1 Attach the split-barrel sampler to the sampling rods and lower into the borehole. Do not allow the sampler to drop onto the soil to be sampled.

7.1.2 Position the hammer above and attach the anvil to the top of the sampling rods. This may be done before the sampling rods and sampler are lowered into the borehole.

7.1.3 Rest the dead weight of the sampler, rods, anvil, and drive weight on the bottom of the boring and apply a seating blow. If excessive cuttings are encountered at the bottom of the boring, remove the sampler and sampling rods from the boring and remove the cuttings.

7.1.4 Mark the drill rods in three successive 6-in. (0.15-m) increments so that the advance of the sampler under the impact of the hammer can be easily observed for each 6-in. (0.15-m) increment.

7.2 Drive the sampler with blows from the 140-lb (63.5-

kg) hammer and count the number of blows applied in each 6-in. (0.15-m) increment until one of the following occurs:

7.2.1 A total of 50 blows have been applied during any one of the three 6-in. (0.15-m) increments described in 7.1.4.

7.2.2 A total of 100 blows have been applied.

7.2.3 There is no observed advance of the sampler during the application of 10 successive blows of the hammer.

7.2.4 The sampler is advanced the complete 18 in. (0.45 m) without the limiting blow counts occurring as described in 7.2.1, 7.2.2, or 7.2.3.

7.3 Record the number of blows required to effect each 6 in. (0.15 m) of penetration or fraction thereof. The first 6 in. is considered to be a seating drive. The sum of the number of blows required for the second and third 6 in. of penetration is termed the "standard penetration resistance," or the "*N*-value." If the sampler is driven less than 18 in. (0.45 m), as permitted in 7.2.1, 7.2.2, or 7.2.3, the number of blows per each complete 6-in. (0.15-m) increment and per each partial increment shall be recorded on the boring log. For partial increments, the depth of penetration shall be reported to the nearest 1 in. (25 mm), in addition to the number of blows. If the sampler advances below the bottom of the boring under the static weight of the drill rods or the weight of the drill rods plus the static weight of the hammer, this information should be noted on the boring log.

7.4 The raising and dropping of the 140-lb (63.5-kg) hammer shall be accomplished using either of the following two methods:

7.4.1 By using a trip, automatic, or semi-automatic hammer drop system which lifts the 140-lb (63.5-kg) hammer and allows it to drop 30 ± 1.0 in. (0.76 m ± 25 mm) unimpeded.

7.4.2 By using a cathead to pull a rope attached to the hammer. When the cathead and rope method is used the system and operation shall conform to the following:

7.4.2.1 The cathead shall be essentially free of rust, oil, or grease and have a diameter in the range of 6 to 10 in. (150 to 250 mm).

7.4.2.2 The cathead should be operated at a minimum speed of rotation of 100 RPM, or the approximate speed of rotation shall be reported on the boring log.

7.4.2.3 No more than 2¼ rope turns on the cathead may be used during the performance of the penetration test, as shown in Fig. 1.

NOTE 4—The operator should generally use either 1¾ or 2¼ rope turns, depending upon whether or not the rope comes off the top (1¾ turns) or the bottom (2¼ turns) of the cathead. It is generally known and accepted that 2¾ or more rope turns considerably impedes the fall of the hammer and should not be used to perform the test. The cathead rope should be maintained in a relatively dry, clean, and unfrayed condition.

7.4.2.4 For each hammer blow, a 30-in. (0.76-m) lift and drop shall be employed by the operator. The operation of pulling and throwing the rope shall be performed rhythmically without holding the rope at the top of the stroke.

7.5 Bring the sampler to the surface and open. Record the percent recovery or the length of sample recovered. Describe the soil samples recovered as to composition, color, stratification, and condition, then place one or more representative portions of the sample into sealable moisture-proof containers (jars) without ramming or distorting any apparent stratification. Seal each container to prevent evaporation of soil moisture. Affix labels to the containers bearing job designation, boring number, sample depth, and the blow count per 6-in. (0.15-m) increment. Protect the samples against extreme temperature changes. If there is a soil change within the sampler, make a jar for each stratum and note its location in the sampler barrel.

8. Report

8.1 Drilling information shall be recorded in the field and shall include the following:

8.1.1 Name and location of job,

8.1.2 Names of crew,

8.1.3 Type and make of drilling machine,

8.1.4 Weather conditions,

8.1.5 Date and time of start and finish of boring,

8.1.6 Boring number and location (station and coordinates, if available and applicable),

8.1.7 Surface elevation, if available,

8.1.8 Method of advancing and cleaning the boring,

8.1.9 Method of keeping boring open,

8.1.10 Depth of water surface and drilling depth at the time of a noted loss of drilling fluid, and time and date when reading or notation was made,

8.1.11 Location of strata changes,

8.1.12 Size of casing, depth of cased portion of boring,

8.1.13 Equipment and method of driving sampler,

8.1.14 Type sampler and length and inside diameter of barrel (note use of liners),

8.1.15 Size, type, and section length of the sampling rods, and

8.1.16 Remarks.

8.2 Data obtained for each sample shall be recorded in the field and shall include the following:

8.2.1 Sample depth and, if utilized, the sample number,

8.2.2 Description of soil,

8.2.3 Strata changes within sample,

8.2.4 Sampler penetration and recovery lengths, and

8.2.5 Number of blows per 6-in. (0.15-m) or partial increment.

9. Precision and Bias

9.1 *Precision*—A valid estimate of test precision has not been determined because it is too costly to conduct the necessary inter-laboratory (field) tests. Subcommittee D18.02 welcomes proposals to allow development of a valid precision statement.

9.2 *Bias*—Because there is no reference material for this test method, there can be no bias statement.

9.3 Variations in *N*-values of 100 % or more have been observed when using different standard penetration test apparatus and drillers for adjacent borings in the same soil formation. Current opinion, based on field experience, indicates that when using the same apparatus and driller, *N*-values in the same soil can be reproduced with a coefficient of variation of about 10 %.

9.4 The use of faulty equipment, such as an extremely massive or damaged anvil, a rusty cathead, a low speed cathead, an old, oily rope, or massive or poorly lubricated rope sheaves can significantly contribute to differences in *N*-values obtained between operator-drill rig systems.

9.5 The variability in N-values produced by different drill rigs and operators may be reduced by measuring that part of the hammer energy delivered into the drill rods from the sampler and adjusting N on the basis of comparative energies. A method for energy measurement and N-value adjustment is given in Test Method D 4633.

10. Keywords

10.1 blow count; in-situ test; penetration resistance; split-barrel sampling; standard penetration test

APPENDIX E

- References

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