

SOIL MECHANICS

Soil Mechanics Soil mechanics is the science of understanding and predicting how soil will respond to externally applied forces or pressures.³

What is Soil? Soil is produced from rock by a process of weathering, i.e. the breaking down of rock into smaller and smaller pieces through mechanical and chemical processes.

Two Types of Soil: When soil remains over the rock from which it was produced; it is referred to as *Residual Soils*. This type of soil tends to have better properties for supporting foundation loads than *Transported Soils*, which is discussed below.

When soils are transported and deposited in other areas (away from the original rock), they are referred to as *Transported Soils*. The manner in which these soils are transported can be applied to further categorize this type of soil. See Table 6.1.

Table 6.1 Transported Soils

Types	Manner of Transport
Alluvial or Fluvial	running water
Glacial	glacier action
Aeolian	wind action

Soil Characterization: Most soil classification systems used in construction classify soils based upon two experimental characterizations of soil. These two measurements are (1) grain-size distribution curve (or gradation curve), and (2) soil consistency (Atterberg limits).³ These two methods of soil classification are discussed in the following paragraphs.

Grain-Size Distribution Curve: Grain-size analyses can be either mechanical or performed with a hydrometer. The mechanical method (Sieve Analysis) will be discussed here.

The soil is passed through a series of screens - sieves (largest opening on top). The sieve openings are progressively smaller from top to bottom. See Table 6.1 for U.S. standard sieve sizes. The mass (weight) of soil retained by each screen is carefully measured as well as the mass remaining in the bottom pan. This allows for the determination of the percent of the total weight of soil that passes through each of the different sieves. This information is then plotted on a semi-log chart. This chart is usually referred to as "The Grain Size Distribution Curve" or "Particle-Size Distribution Curve". This chart shows not only the range of particle sizes present in the soil but also the distribution of various size particles. Soils in which the particle sizes are distributed over a wide range (Figure 6.1) are termed *well graded*. Soils in which most of the grains are the same size are termed as *poorly graded*. Figure 6.1 shows a typical Grain-Size Distribution Curve for a cohesionless granular material.

Example Problem No. 6.1

The following measurements were obtained from a mechanical grain size analysis (Sieve Analysis).

Sieve Size (mm)	Weight Retained (g)
3/8" (9.5)	43
No. 4 (4.75)	195
No. 10 (2.00)	281
No. 40 (.425)	127
No. 100 (.150)	44
No. 200 (.075)	25
Pan (<.075)	135

Determine:

1. The average opening that allowed 50% of the total weight of the soil to pass through. D (50)
2. Coefficient of Uniformity (Cu)
3. Coefficient of Curvature (Cc)
4. The percent of total weight of the sample that is gravel, sand and clay.

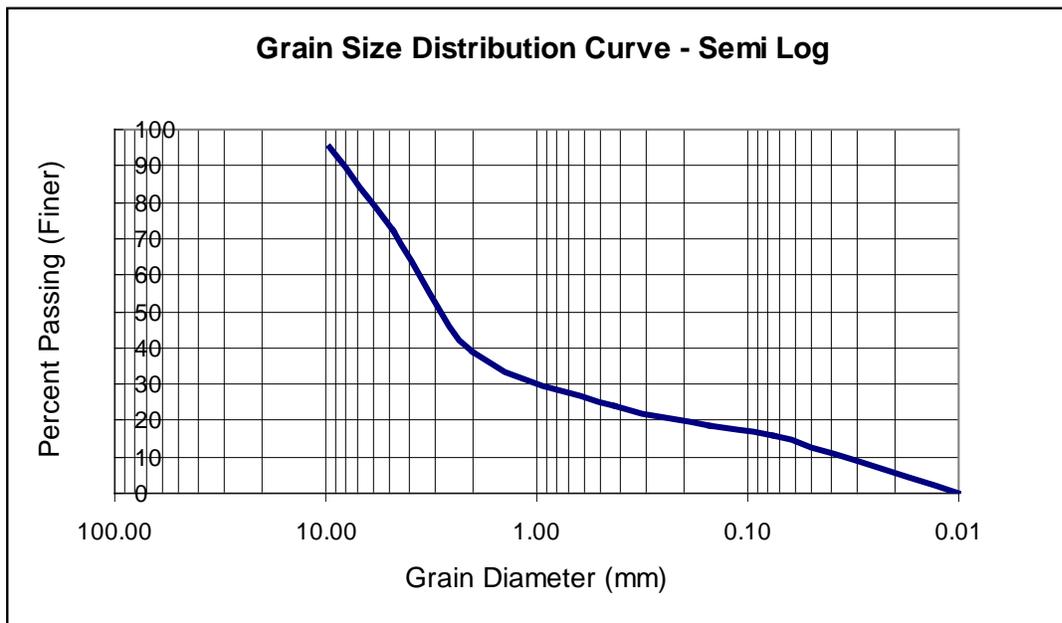
1 mm = .03937 inch g = grams = .002205 lbs

given

Solution to Problem 6.1

SIEVE NO.	SIEVE DIA mm	PERCENT PASSING OF TOTAL %	PERCENT RETAINED %	MASS OF SOIL RETAINED ON EACH SIEVE	MASS AMOUNT PASSING THRU EACH SIEVE
1	3/8	94.941	5.059	43	850.000
2	4	72.000	22.941	195	807.000
3	10	38.941	33.059	281	612.000
4	40	24.000	14.941	127	331.000
5	100	18.824	5.176	44	204.000
6	200	15.882	2.941	25	160.000
7	pan	0.000	15.882	135	135.000
TOTAL				850	

Figure 6.1



D (50) = The average sieve opening that allows 50% by weight to pass through
D (60) = The average sieve opening that allows 60% by weight to pass through
D (30) = The average sieve opening that allows 30% by weight to pass through
D (10) = The average sieve opening that allows 10% by weight to pass through
D (10) = The Effective Size

D (50) = 2.800 approx.
D (60) = 3.400 approx.
D (30) = 1.200 approx.
D (10) = 0.030 approx.

$$Cu = \text{Uniformity Coefficient} = D (60) / D (10) = 113.3333$$

Cc = Coefficient of Curvature (Graduation)

$$Cc = \frac{(D_{30})^2}{(D_{60} \times D_{10})} = 14.118$$

According to the Unified Soil Classification System: Percent

Gravel (>4.75 mm) =	100.000	72.000	28%
Sand (<4.75 mm > .075 mm) =	72.000	16.000	56%
Clay / Silt (< .075 mm) =			16%
			100%

Table 6.2
U.S. Standard Sieve Sizes

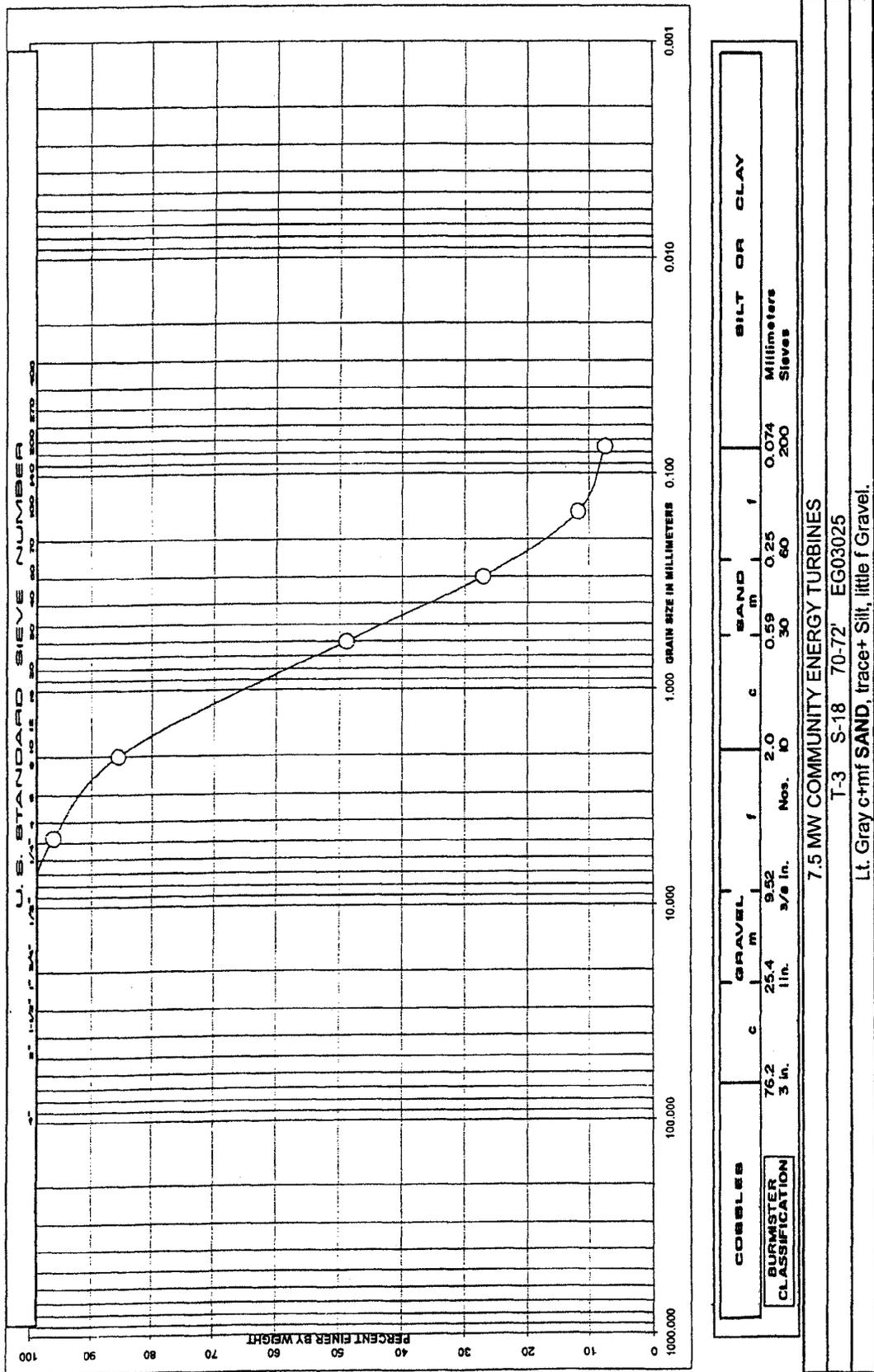
Sieve No.	Opening (mm)
4	4.750
6	3.350
8	2.360
10	2.000
16	1.180
20	.850
30	.600
40	.425
50	.300
60	.250
80	.180
100	.150
140	.106
170	.088
200	.075
270	.053

1 mm = .03937 inch

Figure 6.2
Laboratory Sieve Set-up

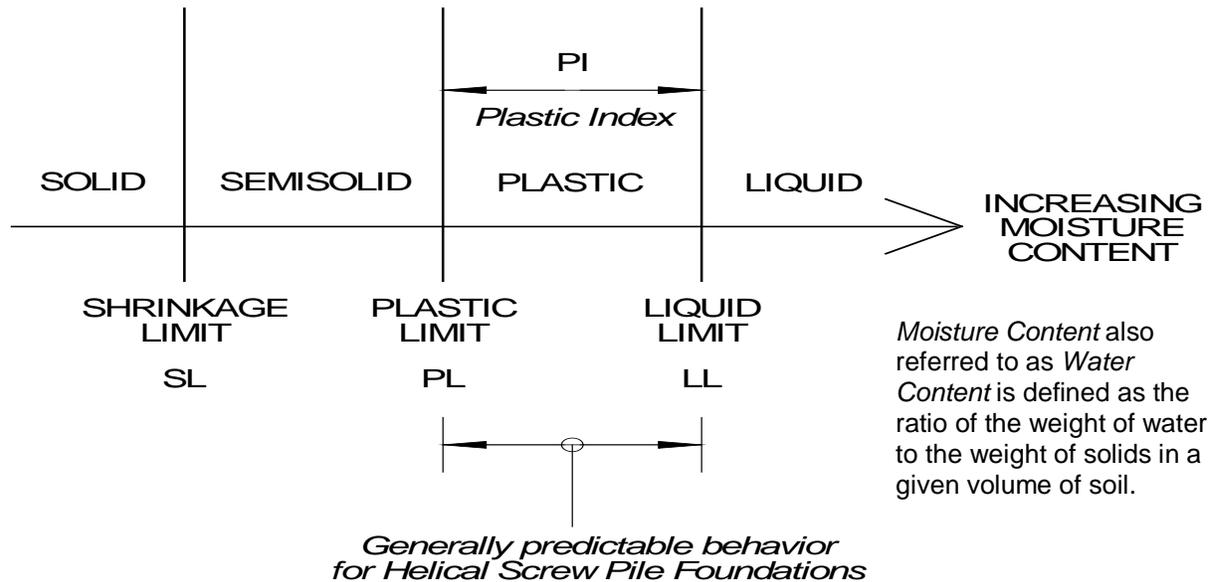


Figure 6.3 Typical Grain Size Distribution (Graduation Curve)



Soil Consistency (Atterberg Limits): When clay is present in fine-grained soils, it can be remolded in the presence of some moisture without crumbling. This cohesive nature is because of the adsorbed water surrounding the clay particles. In the early 1900's, Albert Atterberg, a Swedish scientist, developed a method to describe the consistence of fine-grained soils with varying moisture contents. At very low moisture content, soil behaves more like a brittle solid. When the moisture is very high, the soil may flow like a liquid with the properties of a heavy fluid. Based on water content, Atterberg defined four basic states characterizing the behavior of cohesive soils and the three limits (Atterberg Limits) separating the basic states.¹ See Figure 6.4.

Figure 6.4 Atterberg Limits



$$\text{Liquidity Index} = (\text{Moisture Content} - \text{PL}) / (\text{LL} - \text{PL}) = (\text{Moisture Content} - \text{PL}) / \text{Plastic Index}$$

Shrinkage Limit (SL): When moisture is gradually removed from clay, shrinkage occurs until the moisture content has been reduced sufficiently. Inversely, as moisture content is increased, soils may expand. The moisture content that coincides with no further shrinkage is defined as its *Shrinkage Limit*. (I.e. If additional moisture is removed beyond the SL, further shrinkage will not continue.)

Plastic Limit (PL): The Plastic Limit is the lower limit of the Plastic Stage. When the soil is rolled into threads of 3.2 mm (1/8"), crumbling will not occur if the moisture content is greater than PL. In this case, the moisture content coinciding with crumbling would be defined as the soils Plastic Limit (PL). If the natural water content is less than the plastic limit (liquidity index negative), the soil cannot be remolded.²

Liquid Limit (LL): If the water content of a natural soil stratum is greater than the liquid limit (liquidity index greater than 1.0), remolding transforms the soil into a thick viscous slurry.²

Sensitivity: The strength of some types of soils may be significantly reduced when disturbed. These types of soils are referred to as *Sensitive*. Sensitivity (S_t) of a soil can be defined as the ratio of the unconfined compressive strength of an undisturbed specimen to the unconfined compressive strength of the specimen at the same water content but in a remolded state. The values of (S_t) for the vast majority of clays range between 2 and 4 and are considered *Insensitive*. For *sensitive clays*, (S_t) can range from 4 to 8. For *extra sensitivity clays*, (S_t) can range between 8 and 16. Clays with (S_t) greater than 16 are described as *quick clays*. Highly sensitive soils should be avoided when installing any type of pile. When installing Helical Screw Piles, this can be accomplished by increasing the depth of the pile beyond the sensitive stratum as indicated by the geotechnical report and boring log. If this cannot be accomplished, full scale load testing will be required to establish the proper torque factor (Kt) for the pile. Marine and lake clays and organic silts with high water content can be highly sensitive. In the U.S., highly sensitive clays are seldom encountered, but the possibility of their occurrence should not be overlooked.

Soil Classification: The most widely used system for classifying soils in the U.S. is the Unified Classification System. This method of classification categorizes soils into 15 groups and is shown here in Table 6.5.

Table 6.5 Soil Identification: Unified Soil Classification System

GW	GP	GM	GC	SW	SP	SM	SC	ML	CL	MH	CH	OL	OH	Pt		
Substantial amounts of all grain particle sizes	Predominantly one size or range of sizes with some intermediate sizes missing	Non-plastic fines (to identify, see ML below)	Plastic fines (to identify, see CL below)	Wide range in grain size and substantial amounts of all grain particle sizes	Predominantly one size or range of sizes with some intermediate sizes missing.	Non-plastic fines (to identify, see ML below)	Plastic fines (to identify, see CL below)	Stickiness	None	Medium	Low	Very High	Readily identified by color, odor, spongy feel and frequently by fibrous texture			
CLEAN GRAVELS Will not leave a stain on a wet palm.	DIRTY GRAVELS Will leave a stain on a wet palm.	CLEAN SANDS Will not leave a stain on a wet palm	DIRTY SANDS Will leave stain on wet palm	Liquid Limit	Dilatancy	Toughness	Stickiness	None	Med. to High	Medium	High	None				
GRAVELLY SOILS More than half of the coarse fraction is larger than 4.75 mm. 4.75 mm = 3/16 inch	SANDY SOILS More than half of the coarse fraction is smaller than 4.75 mm	Ribbon	None	Weak	Strong	Very Strong										
COARSE-GRAIN SOILS More than half the material (by weight) is individual grains visible to the naked eye.	FINE-GRAINED SOILS (CLAY) More than half the material (by weight) is individual grains not visible to the naked eye. (<.074 mm or <.003 in.)	HIGHLY ORGANIC SOILS														

Chemical Composition: Besides the difference in grain size, the chemical composition of soil can also be helpful in distinguishing between various types of soils. Sand and gravel usually consist of the same minerals as the original rock from which they were created by the erosion process. This can be quartz, feldspar or glimmer. Sand mostly consists of quartz. The chemical formula of this mineral is SiO_2 . Fine-grained soils may contain the same minerals, but they also contain the so-called clay minerals, which have been created by chemical erosion. The main clay minerals are kaolinite, montmorillonite and illite. These minerals consist of compounds of aluminum with hydrogen, oxygen and silicates. They differ from each other in chemical composition, but also in geometrical structure, at the microscopic level. The microstructure of clay usually resembles thin plates. On the microscale there are forces between these very small elements, and ions of water may be bonded. Because of the small magnitude of the elements and their distances, these forces include electrical forces and the Van der Waals forces. Although the interaction of clay particles is of a different nature than the interaction between the much larger grains of sand or gravel, there are many similarities in the global behavior of these soils. There are some essential differences, however. The deformations of clay are time dependent, for instance. When a sandy soil is loaded it will deform immediately, and then remain at rest if the load remains constant. Under such conditions a clay soil will continue to deform, however. This is called creep. It is very much dependent upon the actual chemical and mineralogical constitution of the clay. Also, some clays, especially clays containing large amounts of montmorillonite, may show a considerable swelling when they are getting wetter. Peat contains the remains of decayed trees and plants. Chemically it therefore consists partly of carbon compounds. It may even be combustible, or it may produce gas. As a foundation material peat and other organic soils (organic clay) are usually not very suitable, because they tend to be very compressible.³

Soil Shear Strength: The structural strength of soil is primarily a function of its shear strength, where shear strength refers to the soil's ability or resist sliding along internal, 3-dimensional surfaces within a mass of soil.³ The capacity of a Helical Screw Pile Foundation is dependent upon the effective bearing capacity of the soils surrounding its helices. Soil bearing capacity is directly related to the soil's shear strength and because of this; shearing strength is one of the most important engineering properties of soil to consider when designing foundations. The shearing strength of a material is the maximum shear stress that it can withstand prior to shear failure. As discussed below, soil shear stress can be categorized as either occurring in cohesionless, cohesive or mixed soils. This manual addresses cohesionless and cohesive soils.

Soil Shear Strength – Cohesionless (Granular) Soils: Cohesionless soils, such as sands and gravels, derive their shear strength from the mechanical sliding friction and interlocking forces developed between the individual grains. As in the case for all friction calculations, normal force (i.e. over-burden pressure) acting on these grains will increase the soil's shear strength capacity accordingly. For a granular soil, the angle of internal friction (FA) includes both the soil's sliding friction and the interlocking forces. The internal friction angle (FA) is governed by properties such as unit weight, grain size and shape. As discussed in Section 8, the bearing capacity of a cohesionless soil is equal to the over-burden pressure times the Bearing Capacity Factor (Nq). Nq is determined solely by the angle of internal friction (FA) of the cohesionless soil.

Before designing foundations, in cohesionless soils, the soil's angle of internal friction (FA) has to be determined.

Several methods may be applied to determine the angle of internal friction (FA). A geotechnical engineering firm usually performs the types of tests shown below during or immediately after their sub-surface exploration. The results of the tests are included in their geotechnical report and sub-surface exploration report.

1. **Shear Box Method:** Straightforward and relatively portable - results may not be as accurate as those obtained with a Triaxial Shear Test.
2. **Triaxial Shear Test:** Most accurate method.
3. **Determine the approximate angle of internal friction (FA) based on SPT N values:** Should be used only when other information is not made available. Soil-boring logs often do not include tested values for the angle of internal friction (FA), but do include SPT N values of the soil. See Section 8.

For foundation design, the SPT or Standard Penetration Test is the most widely used method in the U.S. for soil exploration and testing. The SPT method will be discussed in detail later in this section.

Soil Shear Strength – Cohesive Soils (Fine Grained Soils): Clay is the finest textured of all the soil classes. Clay usually forms extremely hard clods or lumps when dry but is extremely sticky and plastic when wet. When containing the proper amount of moisture (Figure 6.4, Plastic Index region), it can be “ribboned out” to a remarkable degree by squeezing between thumb and forefinger, and may be rolled into a long, very thin wire.⁶

In its natural state, except in areas near the surface, clay is usually saturated (i.e. all the voids between solids are filled with water). The following assumes saturated clay of medium plasticity.

Cohesion: Due to the extremely small grain size ($< .074$ mm or $< .003$ ”), the friction and interlocking forces that occur between the fine-grained soils (clay) are a great deal less than that with granular soils (sand). But clay does have shear strength. When moist, clay soils have particles that because of molecular and mechanical bonding “stick together”. This “*bonding together of the particles*” is not well understood, but appears to be influenced by moisture content, grain structure, and unit weight. The shear strength that is provided to the soil by this “*bonding or attraction between particles*” is referred to as “Cohesion”.

Several methods are often used to determine Cohesion (c). A geotechnical engineering firm usually performs the types of tests shown below during or immediately after their sub-surface exploration. The results of the tests are included in their geotechnical report and sub-surface exploration report.

1. Unconfined Compression Test: Widely used method. The most direct quantitative measure of consistency is the load per unit area at which unconfined prismatic or cylindrical samples of the soil fail in a simple compression test. This quantity is known as the *unconfined compressive strength* (q_u) of the soil.² An axial load is placed onto a sample (with no lateral support), the load is increased until the soil fails. The compressive stress that causes failure is defined as the *unconfined compressive strength* (q_u) of the soil. The height of the sample will be twice its width or diameter. Assuming ($FA = 0.00$), Cohesion (c) is equal to approximately $q_u / 2$.

2. Shear (or Torvane): A small hand held device with vanes extending in a radial direction from the center of the end surface of a cylinder (1” diameter across vanes). The vanes are pressed into the clay to their full depth whereupon a torque is applied through a calibrated spring until the clay fails along the cylindrical surface circumscribing the vanes. Cohesion (c) is read directly from the indicator on the calibrated spring. This is often part of the procedure when conducting Standard Penetration Test.

3. Pocket Penetrometer: Generally not considered as accurate as the above methods. The Pocket Penetrometer measures the unconfined compressive strength (q_u) [$c = q_u / 2$]. This is often part of the procedure when conducting Standard Penetration Tests. The plunger is pushed into a split spoon sample to a depth of 1/4”. The compressive strength of the soil is read directly from a calibrated spring. This reading is approximately equal to the *unconfined compressive strength* q_u of the soil. The results of this test should be used with caution. Depending on the type of clay, engineers often discount the values of a Pocket Penetrometer by as much as 30% to 50%. This test is not recommended for sandy or silty clays.

4. Determine the approximate cohesion of the soil by the results of the Standard Penetration Test: Not as accurate as items 1 and 2 above. Should be used only if no other information is made available. The following equation is based on data abstracted from Terzaghi and Peck² and is based on saturated clay of medium plasticity.

Cohesion (kps) = SPT N value / 8

[Equation 6.1]

Note: The design method discussed in Section 8 is based on the assumption that the *angle of internal friction* (FA) of clay is 0.00. In this case, *cohesion* will equal the shear strength of the clay. Likewise, the method discussed in Section 8 is based on the assumption that the *cohesion* of granular soils such as sand is 0.00. These assumptions may not truly reflect the properties of all soils, but do seem to provide adequate estimates of pile capacities.

Expansion and Shrinkage: Most clay will experience volume changes coinciding with changes in moisture content. Soil that is particularly susceptible to significant volume changes is said to be in the soil's *active zone*. Both Expansion and Shrinkage can cause serious damage to structures and foundations. The helical section of a pile should always be installed well below the active zone. The shaft passing through the active zone should be as small as possible so as to minimize skin friction between the soil (active zone) and pile. Battered piles should be avoided.

Negative Skin Friction: Deep foundation elements installed through compressible material can experience "downdrag" forces or negative skin friction along the shaft that results from downward movement of adjacent soil relative to the pile. This Negative skin friction results primarily from consolidation of a soft deposit caused by dewatering (moisture removal) or the placement of fill. Negative skin friction is particularly severe on batter pile installations because the force of subsiding soil is large on the outer side of the batter pile and soil settles away from the inner side of the pile. This can result in bending of the pile. Batter pile installations should be avoided where negative skin friction [and soil expansion or shrinkage] is expected to develop.⁵

Loads transferred into the pile from either soil *expansion or shrinkage or negative skin friction* can be significant, especially with large diameter wood or steel piles. The large surface area of these types of piles can result in very large friction forces (skin friction). This can adversely affect the performance of the pile, and cause the pile to fail. In these types of soils, it is recommended to use MACLEAN-DIXIE Helical Screw Piles Foundations. The small surface area of these piles minimizes skin friction. When using 1.25" sq. to 2.00 sq. helical anchors, skin friction forces can be disregarded with little error.

Sub-surface Investigation: Prior to designing a Helical Pile Foundation, the following information will be required.

1. The Maximum Working Load that can be expected.
2. A Geotechnical Report: This report is prepared by geotechnical engineers and provides pertinent information concerning the project. The Geotechnical Report should include any information that could affect the present or future behavior of the foundation. This would include, seasonal water tables, sub-surface conditions, and any type of natural occurrence, which could affect the future integrity of the foundation. The geotechnical report should address the corrosion potential of the soil. A major part of the geotechnical report will be the attached "soil boring logs". Boring Logs show the results of sub-surface investigations. These are usually performed by a Standard Penetration Test (SPT) or a Cone Penetration Test (CPT). The Standard Penetration Test, developed in 1927, is currently the most popular and economical means to obtain subsurface information. Virtually all of the information required to design a Helical Screw Pile Foundation can be extracted from the Geotechnical Report and the attached Boring Logs. The Cone Penetration Test has certain advantages and although not as popular as the SPT is now more widely used. The Standard Penetration Test method will be discussed here.

The Standard Penetration Test (SPT) ASTM D 1586:

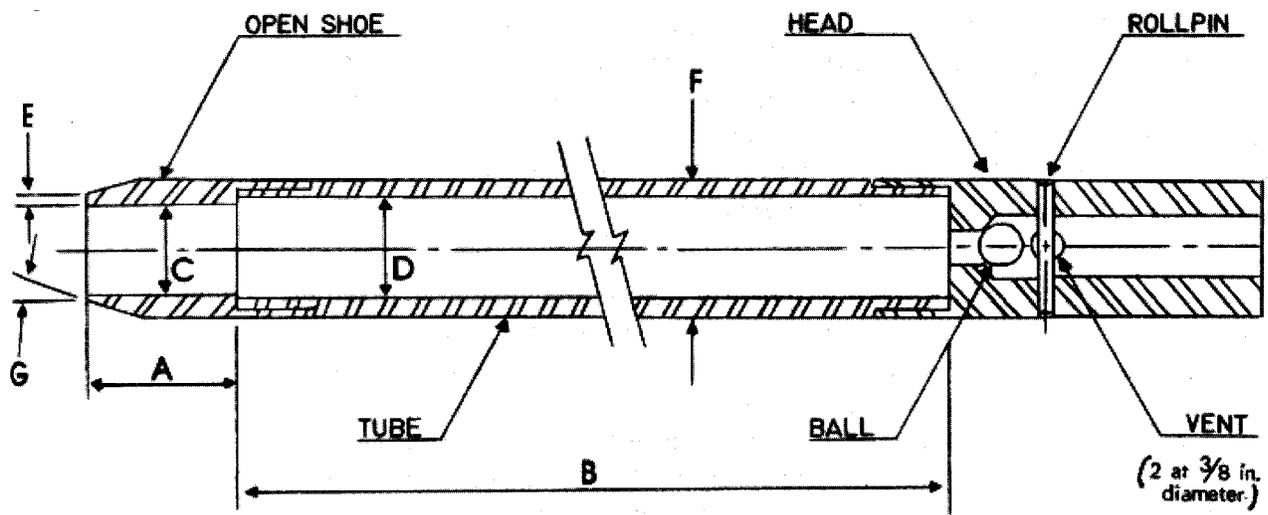
The number of blows required to drive a split spoon sampler a distance of 12 inches after an initial penetration of 6 inches is referred to as an "N" value or SPT "N" value. This SPT test is covered under ASTM Standard D1586 (latest revision), which requires the use of a standard 2" (O.D.) split barrel sampler (Figure 6.5), driven by a 140-pound hammer dropping 30 inches in free fall. This procedure is generalized as follows⁵:

- a. Clean the boring of all loose material, and material disturbed by drilling.
- b. Insert sampler, verifying the sampler reaches the same depth that was drilled.
- c. Obtain a consistent 30-inch free-fall drop of the hammer.
- d. Using the standard hammer and drop, drive the sampler 18 inches, or until refusal is reached. Refusal is defined as a penetration of less than 6 inches for 100 hammer blows.
- e. Count and record the number of blows required to drive the split spoon for each 6 inches of penetration. The sum of the last 12 inches of penetration is referred to as "SPT N value" at the sounding depth.

Along with additional information, the number of blows (from 6" to 18") will be recorded onto the Boring Logs. The boring logs should show the true adjusted ASTM D 1586 SPT N-values. Throughout this test, the split spoon will be opened and its contents examined and tested as discussed above. The results of these tests will be recorded onto the bore logs and/or the geotechnical report. Unfortunately, there is not a standard format for either the Geotechnical Report or the Boring Log. It is highly recommended that the engineer carefully review and understand these documents before designing a Helical Screw Pile Foundation. Always ask questions. See Figure 6.6 for an example of a typical boring log.

In gravel deposits (with particles 1" and greater), the SPT values may be artificially large. This can be the result of gravel particles being lodged in the open end of the Split Spoon Sampler. The boring log often includes "recovery (inches)". A small recovery in gravelly soils often indicates a plugged sampler. SPT N-values in these soils should be adjusted or discounted.

Figure 6.5 Split-Barrel Sampler – ASTM D 1586



- 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 = $1.50 \pm 0.05 - 0.00$ in. ($38.1 \pm 1.3 - 0.0$ mm)
- E = 0.10 ± 0.02 in. (2.54 ± 0.25 mm)
- F = $2.00 \pm 0.05 - 0.00$ in. ($50.8 \pm 1.3 - 0.0$ mm)
- G = 16.0° to 23.0°

Figure 6.6 Typical Soil Boring Log

SAMPLE		Sampler: 2 inch split spoon				Sheet: 1 of 2		
Project: 7.5MW Community Energy Jersey Atlantic Wind Project		Hammer/Fall: 140 lbs/30 inch				Boring No.: T-2		
		Rig Type: Truck				Location Plan: See Plan		
		Drilling Method: Mud Rotary				Drilling Company: Environmental Remediation		
Project#: EG03025		Ground Elevation: ++13.1'				Drilling Inspector: T. Antonetti		
Start Date: 7/17/03		Depth To Water: -7.0'				Soils Engineer: Paul Echaniz, P.E.		
End Date: 7/17/03		Ground Water Elev: ++6.1'				@ 8:35 AM		
Depth Below Surface (ft)	Blows				Sample		Identification of Soils/Remarks	Stratigraphy
	Casing per ft.	Sampler per 6 Inches			ID	Depth (ft)		
1		1	3	4	4			
2					S-1	0-2	S-1 Orange-Yellow cmf SAND, trace Silt, trace mf Gravel, trace roots. (moist)	
3		3	5	6	6			
4					S-2	2-4	S-2 Yellow-Brown cmf SAND, trace Silt, trace mf Gravel. (moist)	
5		5	6	6	7			
6					S-3	4-6	S-3 DK. Brown c'mf Sand, and Silt.	F
7		5	5	5	4			
8					S-4	6-8	S-4 Dk. Brown-Reddish m*f SAND, little* Silt. (moist)	
9		3	3	3	3			
10					S-5	8-10	S-5 Dk. Brown-Reddish m*f SAND, little Silt, trace* decayed vegetation. (wet)	
11		3	3	2	4			
12					S-6	10-12	S-6 Top: Brown-Reddish m*f SAND, trace* Silt. (wet) Bot: Yellow-Gray cmf SAND, trace Silt, trace* mf Gravel, trace* decayed wood.	13.0'
13								
14								
15								
16		2	2	2	2			MO1
17					S-7	15-17	S-7 Gray Organic SILT, little* Peat, trace* mf Sand. (med soft to firm) (wet)	18.5'
18								
19		P	U	SH	ED	UD-1	17-19 Recovery = 13" P.P.=0.25, TV.=2.0-1.0 tsf	
20						UD-1	Gray Organic SILT, little* Peat. Found	
21		6	9	11	9		mf* SAND, little* Silt @ bottom of tube. Rec = 23 3/4"	
22					S-8	20-22	S-8 Gray m*f SAND, trace* Silt. (organic odor)	SB2
23								24.0'
24								
25								
26		1	1	WH	WH			MO3
27					S-9	25-27	S-9 Gray Organic SILT, trace* mf Sand. (wet) Recovery=24" P.P.=0.25 tsf, TV=3.75-1.0 tsf	28.0'
28								
29								
30		P	U	SH	ED	UD-2	28-30 UD-2 Gray mf SAND, little* Organic Silt. Recovery 10"	
31		WH	WH	WH	WH			MO3
32					S-10	30-32	S-10 Gray mf Sand, and* Silt. (organic odor)	33.0'
33								
34								
35								
36		1	WH	1	1			
37					S-11	35-37	S-11 Gray mf SAND, little* Silt. (organic odor)	MO3
38								
39								
40								

VISUAL IDENTIFICATION TERMS USED								
	Clayey Soils	@ Ball Moist	Relative Density (Dr) Of Granular Soils		Consistency of Clayey Soils		Proportions Used	
Clayey SILT	slight PL	Thread 1/4"	Very Loose	0-15%	soft (S)	0.1-0.5 tsf	trace	1-10%
SILT & CLAY	low PL	Thread 1/8"	Loose	15-35%	firm (F)	0.5-1.0 tsf	little	10-20%
CLAY & SILT	med PL	Thread 1/16"	Medium	35-65%	med hard (MH)	1.0-2.0 tsf	some	20-35%
Silty CLAY	high PL	Thread 1/32"	Dense	65-85%	hard (H)	2.0-4.0 tsf	and	35-50%
CLAY	very high PL	Thread 1/64"	Very Dense	85-100%	very hard (VH)	over 4.0 tsf		

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