Step 2 – Soil Mechanics

Introduction

Webster defines the term mechanics as a branch of physical science that deals with energy and forces and their effect on bodies. *Soil mechanics* is the branch of mechanics that deals with the action of forces on soil masses. The soil that occurs at or near the surface of the earth is one of the most widely encountered materials in civil, structural and architectural engineering. Soil ranks high in degree of importance when compared to the numerous other materials (i.e. steel, concrete, masonry, etc.) used in engineering.

Soil is a construction material used in many structures, such as retaining walls, dams, and levees. Soil is also a foundation material upon which structures rest. All structures, regardless of the material from which they are constructed, ultimately rest upon soil or rock. Hence, the load capacity and settlement behavior of foundations depend on the character of the underlying soils, and on their action under the stress imposed by the foundation. Based on this, it is appropriate to consider soil as a structural material, but it differs from other structural materials in several important aspects.

Steel is a manufactured material whose physical and chemical properties can be very accurately controlled during the manufacturing process. Soil is a natural material, which occurs in infinite variety and whose engineering properties can vary widely from place to place – even within the confines of a single construction project. *Geotechnical engineering* practice is devoted to the location of various soils encountered on a project, the determination of their engineering properties, correlating those properties to the project requirements, and the selection of the best available soils for use with the various structural elements of the project.

Likewise, steel is a material whose properties generally remain unchanged during the life of a structure. The properties of soils can change as the amount of moisture fluctuates and other environmental influences vary. Soils can change significantly under load. For example, loading can increase soil density and strength if pore pressure is allowed to dissipate. If pore water cannot escape, loading may drastically weaken the soil because the pore water, which has no shear strength, bears the load. These possible changes require the geotechnical engineer to predict soil behavior under anticipated load – whether the load is applied gradually or instantaneously.

Dr. Karl Terzaghi stated in his 1943 book titled *Theoretical Soil Mechanics*:

> ". . . the theories of soil mechanics provide us only with a working hypothesis, because our knowledge of the average physical properties of the subsoil and of the orientation of the boundaries between the individual strata is always incomplete and often utterly inadequate. Nevertheless, from a practical point of view, the working hypothesis furnished by soil mechanics is as useful as the theory of structures in other branches of civil engineering."

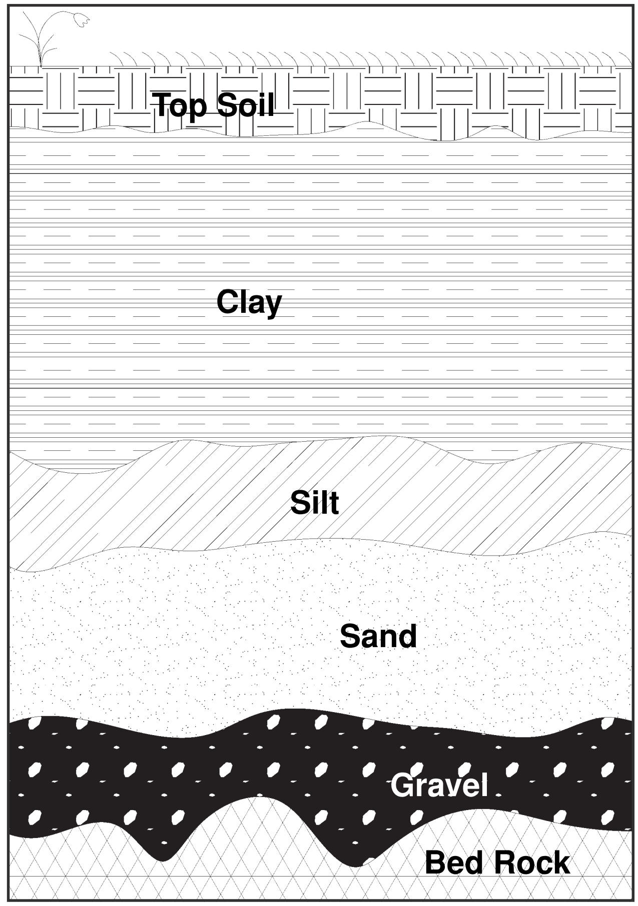
Advance planning and careful observation by the engineer during the construction process can help fill in the gaps between working hypothesis and fact. This section of the design manual intends to provide a basic understanding of soil mechanics so the engineer can develop a useful "working hypothesis" for the design and use of helical screw foundations.

The Soil Profile

Rock or soil material, derived by geologic processes, is subject to physical and chemical changes brought about by the climate and other factors prevalent at the location of the soil. Vegetation, rainfall, freeze/thaw, drought, erosion, leaching, and other natural processes result in profound changes gradually taking place in the character of the soil with the passage of time. This development brings about the soil profile.

The soil profile is a natural succession of zones or strata below the ground surface. It may extend to various depths, and each stratum may have various thicknesses. The upper layer of the profile is typically rich in organic plant and animal residues mixed with a given mineral-based soil. Soil layers below the topsoil can usually be distinguished by a contrast in color and degree of weathering. The physical properties of each layer usually differ from each other. Topsoil is seldom used for construction. Deeper layers will have varying suitability, depending on their properties and location. It is important to relate engineering properties to individual soil layers in order for the data to be meaningful. If data from several layers of varying strength are averaged, the result can be misleading and meaningless. Equally misleading is the practice of factoring a given soil's engineering properties for design. This can lead to overly conservative foundation design.

Figure 2.1
Residual Soil Profile



Definition of Soil

Soil is defined as sediments or other accumulation of mineral particles produced by the physical or chemical disintegration of rock, plus the air, water, organic matter, and other substances that may be included. Soil is typically a non-homogeneous, porous, earthen material whose engineering behavior is influenced by changes in moisture content and density.

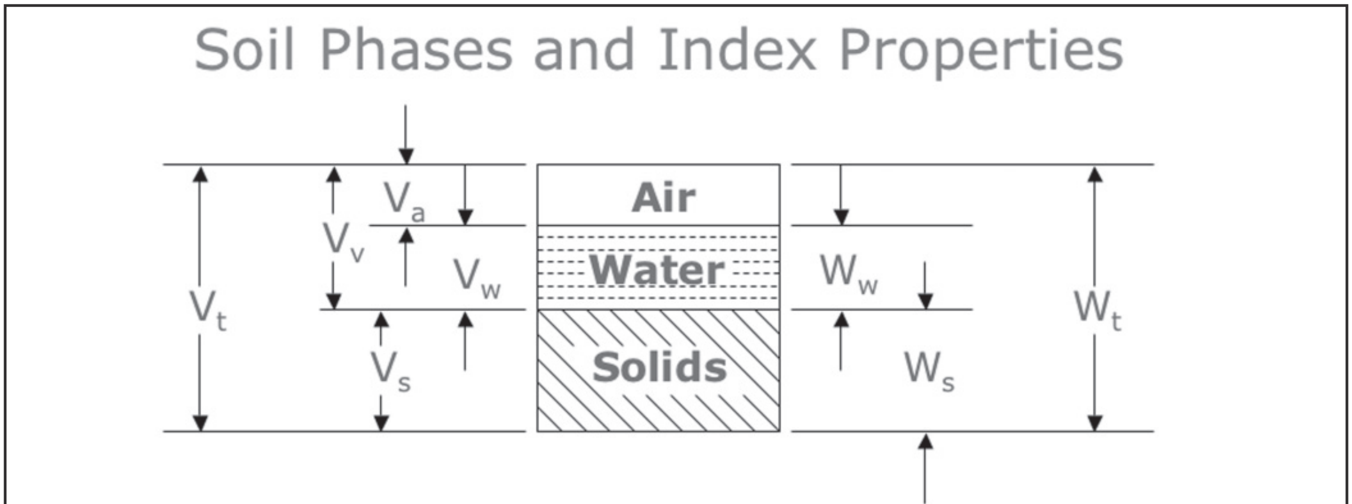
The origin of soil can be broken down to two basic types: *residual*, and *transported*. *Residual soil* is caused by the weathering (decomposition) of rock by chemical or physical action. Residual soils may be very thick in areas of intense weathering such as the tropics, or they may be thin or absent in areas of rapid erosion such as steep slopes. Residual soils are usually clayey, and their properties are related to climate and other factors prevalent at the location of the soil. Residual soils are usually preferred to support foundations, as they tend to have better, more predictable engineering properties.

Transported or deposited soils are derived by the movement of soil from one location to the other by natural means. The means are generally wind, water, ice, and gravity. The character of the resulting deposit often reflects the modes of transportation and deposition and the source material. Deposits by water include alluvial floodplains, coastal plains, and beaches. Deposits by wind include sand dunes and loess. Deposits by melting ice include glacial till and outwash. Each of these materials has behavioral characteristics dependent on geological origin, and the geological name, such as loess, conveys much useful information. Transported soils – particularly by wind or water – are often of poor quality in terms of engineering properties.

Soil Volume & Density Relationships

A soil mass is a porous material containing solid particles interspersed with pores or voids. These voids may be filled with air, water, or both. Figure 2.2 shows a conceptual diagram of relative volumes of air, water, and soil solids in a given volume of soil. Pertinent volumes are indicated by symbols to the left while weights of these material volumes are indicated by symbols to the right. Figure 2.2 also provides several terms used to define the relative amounts of soil, air, and water in a soil mass. *Density* is the weight of a unit volume of soil. It is more correctly termed the *unit weight*. Density may be expressed either as a wet density (including both soil and water) or as a dry density (soil only). *Moisture Content* is the ratio of the weight of water to the weight of soil solids. *Porosity* is the ratio of the volume of voids to the total volume of the soil mass regardless of the amount of air or water contained in the voids. *Void ratio* is the ratio of the volume of voids to the volume of soil particles. The porosity and void ratio of a soil depend upon the degree of compaction or consolidation. For a particular soil in different conditions, the porosity and void ratio will vary and can be used to judge relative stability and load-carrying capacity – i.e. stability and load capacity increase as porosity and void ratio decrease. If water fills all the voids in a soil mass, the soil is said to be *saturated*.

Figure 2.2



Moisture Content-	ω	W_w/W_s
Degree of Saturation-	S	V_w/V_v
Void Ratio-	e	V_v/V_s
Porosity-	n	V_v/V_t
Dry Unit Weight (Dry Density)-	γ_d	W_s/V_t
Total Unit Weight-	γ_t	$(W_s + W_w)/V_t$
Saturated Unit Weight-	γ_s	$(W_s + V_v\gamma_w)V_t$
Effective (submerged) Unit Weight-	γ'	$\gamma_s - \gamma_w$

Permeability or *hydraulic conductivity* is the property of soil allowing it to transmit water. Its value depends largely on the size and number of the void spaces, which in turn depends on the size, shape, and state of packing of the soil grains. A clay soil can have the same void ratio and unit weight as a sand soil, but the clay will have a lower permeability because of the much smaller pores or flow channels in the soil structure. Water drains slowly from fine-grained soils like clays. As the pore water drains, clays creep, or consolidate slowly over time. Sand soils have high permeability, thus pore water will drain quickly. As a result, sands will creep, or consolidate when loaded until the water quickly drains. After drainage, the creep stops.

Basic Soil Types

As stated above, soil is typically a non-homogeneous material. The solid mineral particles in soils vary widely in size, shape, mineralogical composition, and surface-chemical characteristics. This solid portion of the soil mass is often referred to as the *soil skeleton*, and the pattern of arrangement of the individual particles is called the *soil structure*.

The sizes of soil particles and the distribution of sizes throughout the soil mass are important factors which influence soil properties and performance. There are two basic soil types that are defined by particle size. The first type is granular, or coarse-grained soils. *Granular* soils consist of particles that are large enough to be retained by the #200 sieve (0.074 mm). The #200 sieve has 200 openings per inch. Granular soils consist of cobbles, gravels, sands, and some silts. Granular, or coarse-grained soils are commonly referred to as *non-cohesive*, or *cohesionless* soils. The particles of cohesionless soils typically do not stick together except in the presence of moisture, in which surface tension tends to hold particles together. This is commonly referred to as *apparent cohesion*.

The second type is fine-grained soil. *Fine-grained* soils consist of particles that are small enough to pass through the #200 sieve. Typical fine-grained soils are silts and clays. Silt particles typically range from 0.074 to 0.002 mm. Clay particles are less than 0.002 mm. It is not uncommon for clay particles to be less than 0.001 mm (colloidal size). Fine-grained soils are commonly referred to as *cohesive* soils. The particles of cohesive soils tend to stick together due to molecular attraction.

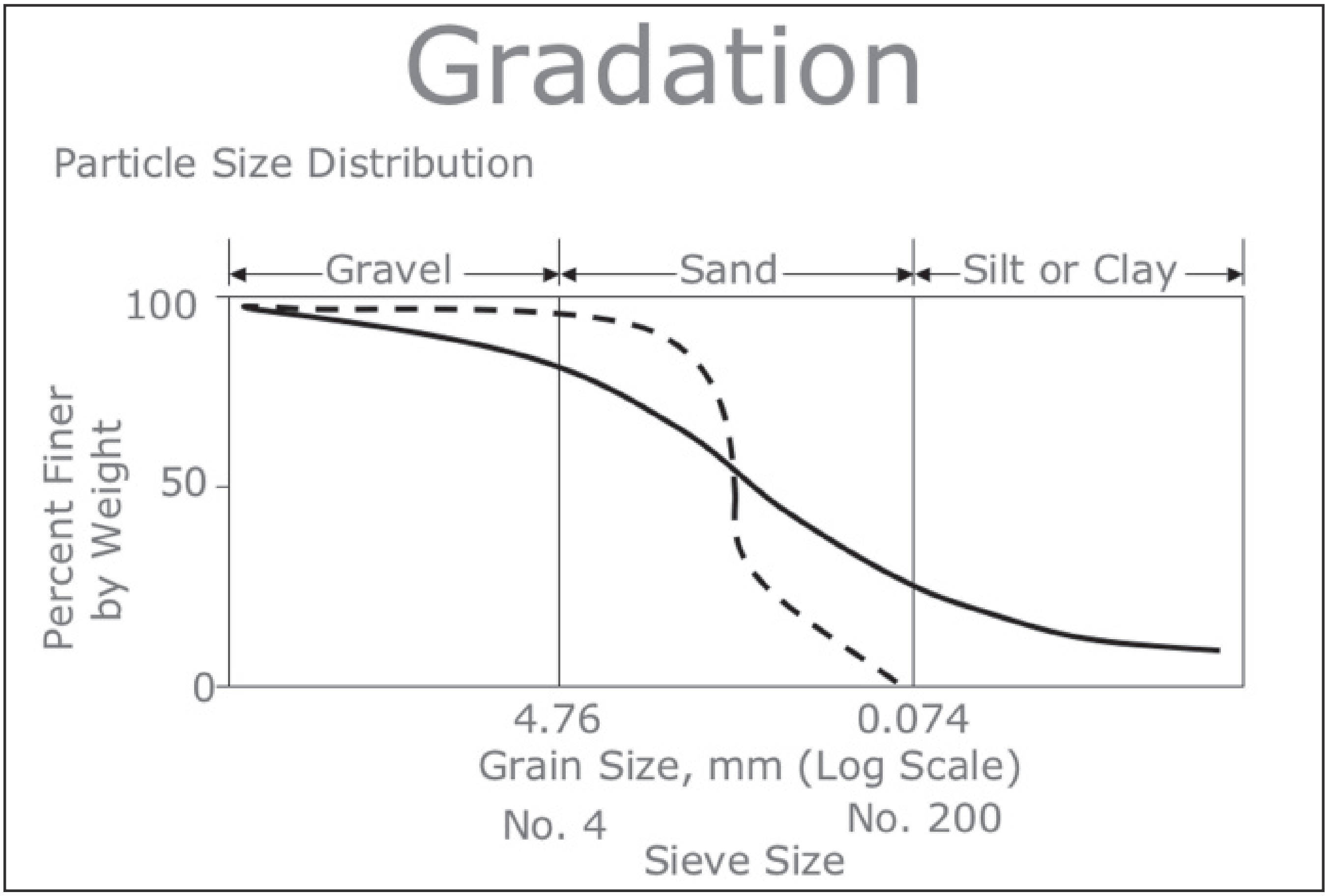
For convenience in expressing the size characteristics of the various soil fractions, a number of particle-size classifications have been proposed by different agencies. Table 2.1 shows the classification of soil particles as proposed by ASTM (Unified Soil Classification system), which has gained wide recognition.

TABLE 2.1

Soil Particle Sizes	Fraction	Sieve Size	Diameter
Boulders		12" Plus	300 mm Plus
Cobbles		3"– 12"	75 – 300 mm
Gravels	Coarse	0.75"– 3"	19 – 75 mm
	Fine	No. 4 – 0.75"	4.76 – 19 mm
Sand	Coarse	No. 10 – No. 4	2 – 4.76 mm
	Medium	No. 40 – No. 10	0.42 – 2 mm
	Fine	No. 200 – No. 40	0.074 – 0.42 mm
Fines (silts and clays)		Passing No. 200	0.074 mm

Another effective way to present particle size data is to use grain-size distribution curves such as that shown in Figure 2.3. Such a curve is drawn on a semi-logarithmic scale, with the percentages finer than the grain size shown as the ordinate on the arithmetic scale. The shape of such curves shows at a glance the general grading characteristics of soil. For example, the dark line on Figure 2.3 represents a well-graded soil – approximately 75% of the particles are sand and clays finer than 4.76 mm (#4 sieve) and about 25% of the particles are clays and silts finer than 0.074 mm (#200 sieve). *Well-graded* soils consist of particles that fall into each size class, i.e. gravel, sand, silt-size, clay-size, and colloidal-size. The dashed line on Figure 2.3 represents a poorly graded soil – almost all the particles are sands finer than 4.76 mm (#4 sieve) and none of the particles are finer than 0.074 mm (#200 sieve). *Poorly graded, or uniform* soils typically consist primarily of particles that fall into only one class size, such as only gravels or only sands.

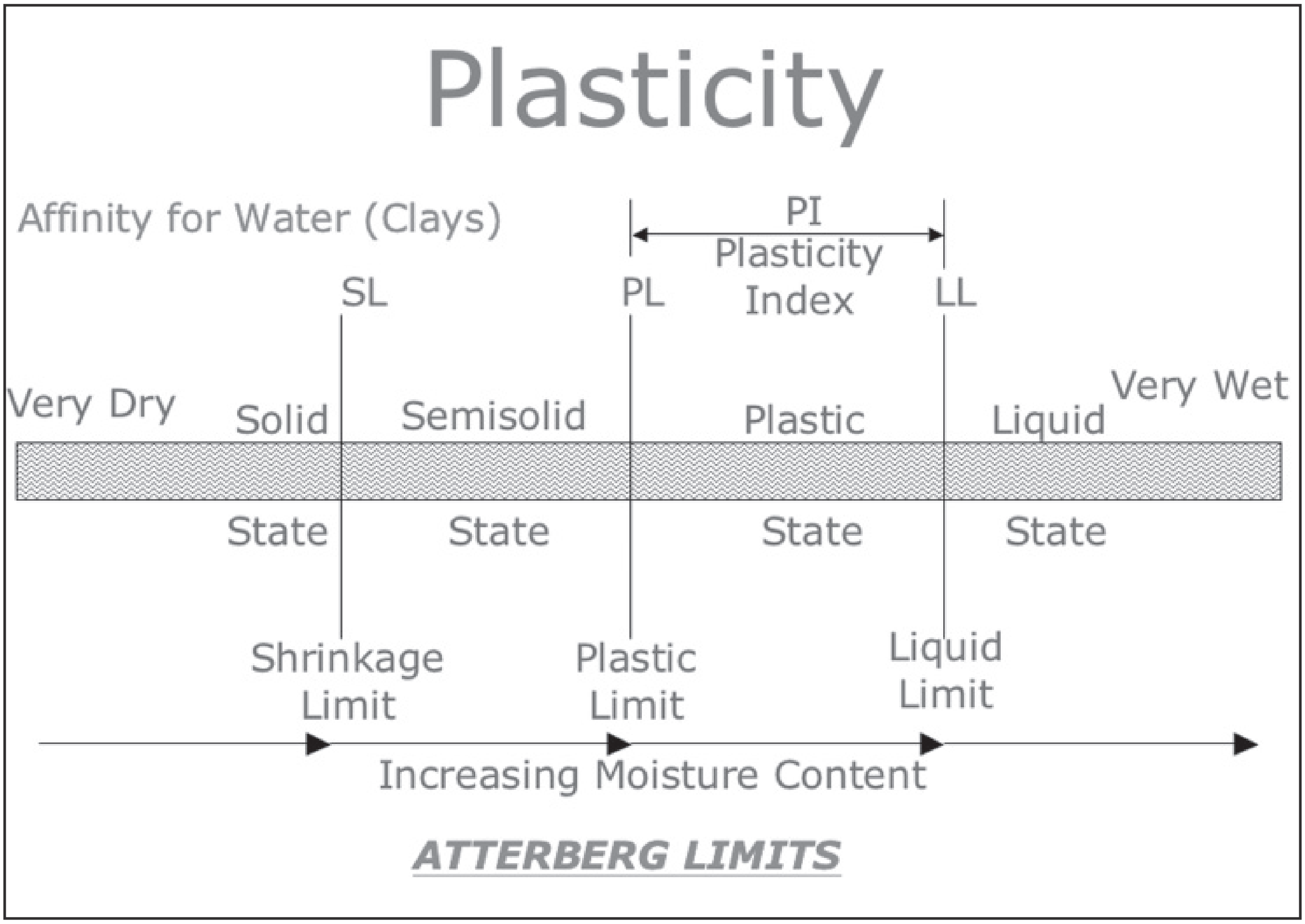
Figure 2.3



Soil Limit States & Index Properties

Most soils include a fine fraction of silt, clay or a combination. The consistency of these soils can range from a dry solid condition to a liquid form with successive addition of water and mixing as necessary to expand pore space for acceptance of water. The consistency passes from solid to semi-solid to plastic solid to viscous liquid as shown in Figure 2.4. A. Atterberg, a Swedish scientist, defined moisture contents representing limits dividing the states of consistency. These limits are known as *Atterberg Limits*. The *shrinkage limit* (SL) separates solid from semisolid, the *plastic limit* (PL) separates semisolid from plastic state, and the *liquid limit* (LL) separates plastic from liquid state. The width of the plastic state (LL-PL), in terms of moisture content, is the *plasticity index* (PI). The PI is an important indicator of the plastic behavior a soil will exhibit. The softness of a saturated clay can be expressed numerically by the *liquidity index* (I_L). Values of I_L greater than or equal to one are indicative of a liquefaction or "quick" potential. In other words, the soil structure may be converted into a viscous fluid when disturbed or remolded by pile driving, caisson drilling, or helical screw foundation installation.

Figure 2.4



Atterberg limits can be used as an indicator of stress history of a given soil. For example, if the moisture content (w_n) of a saturated clay is approximately the same as the LL, the soil is probably normally consolidated. This typically results in an empirical torque multiplier for helical screw foundations (K_t) = 10. See Step 9 for a detailed description of K_t. If the w_n of a saturated clay is greater than the LL, the soil is on the verge of being a viscous liquid and K_t will be less than 10. If the w_n of a saturated clay is close to the PL, the soil is some-to-heavily *overconsolidated* and K_t typically ranges between 12 and 14. If the w_n of a saturated clay is intermediate (between the PL and LL), the soil is probably overconsolidated and K_t will be above 10. Most soils are overconsolidated, or have a history of having been loaded to a pressure higher than exists today. Some common causes are desiccation, the removal of overburden through geological erosion, or melting of overriding glacial ice.

Clays lying at shallow depth and above the water table often exhibit a preconsolidation behavior known as *desiccation*. They appear to be overconsolidated, but the overburden pressure required has never existed in the soil. Desiccated clays are caused by an equivalent internal tension resulting from moisture evaporation. This is sometimes referred to as *negative pore pressure*. The problem with desiccated or partly dry expansive clay is predicting the amount of potential expansion and the expansion or swell pressure – so that preventive measures can be taken.

As water content decreases, clay particles tend to interact with one another through water dipoles to form a microstructure. The water dipole acts to link together clay particles like weak magnets. These bonds, along with the soil microstructure, greatly influence the shear strength of the clay. Many clays experience a temporary partial dispersion when they are re-molded or reworked by hand or machine. These are termed *sensitive clays*. *Sensitivity* is defined as the ratio of the strength of the soil in the undisturbed state to that of the soil in the remolded state. All clays are sensitive to some degree, but highly sensitive soils cannot be counted on for shear strength after a helical screw foundation, drilled shaft, driven pile, etc. has been passed through it. Highly sensitive soils include marine and lacustrine (lake) deposited clays. Typical values of sensitivity are shown in Table 2.2.

Table 2.2
Sensitivity of Clays

Clay Type	Descriptive Term	Sensitivity (S_t)
Overconsolidated, Low to Medium Plastic Clays	Insensitive	1 – 3
Normally consolidated, Medium Plastic Clays	Medium Sensitivity	4 – 8
Marine Clays	Highly Sensitive	10 – 80

Engineering Soil Classification

The engineering soil classification that has gained common use is the Unified Classification. This system is an outgrowth of the Airfield Classification developed by Dr. A. Casagrande and has over the years been adopted by various agencies and associations. The Unified System incorporated the textural characteristics of the soil into engineering classification and utilizes the grain-size classification shown in Table 2.1. The basics of the system are shown in Table 2.4. All soils are classified into 15 groups, each group being designated by two letters. These letters are abbreviations of certain soil characteristics as follows:

Table 2.3
Soil Characteristics

G	Gravel	**O**	Organic
S	Sand	**W**	Well graded
M	Nonplastic or low plasticity fines	**P**	Poorly graded
C	Plastic fines	**L**	Low liquid limit
Pt	Peat, humus, swamp soils	**H**	High liquid limit

Table 2.4
ASTM (Unified) Soil Classification System

Major Divisions			Group Symbols	Typical Descriptions
Coarse-Grained Soils – More than 50% retained on #200 sieve*	Gravels – 50% or more of coarse fraction retained on #4 sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravels with Fines	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands – 50% or more of coarse fraction passes #4 sieve	Clean Sands	SW	Well-graded sands, and gravelly sands, little or no fines
			SP	Poorly graded sands and gravelly sands, little or no fines
		Sand with Fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
Fine-Grained Soils – 50% or more passes #200 sieve*	Silts and Clays – Liquid Limit less than 50		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays – Liquid Limit 50 or more		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			Pt	Peat, muck, and other highly organic soils

*Based on the material passing the 3" (76 mm) sieve.

GW and SW groups comprise well-graded gravely and sandy soils that contain less than 5% of non-plastic fines passing the #200 sieve. GP and SP groups comprise poorly graded gravels and sands containing less than 5% of nonplastic fines. GM and SM groups generally include gravels or sands that contain more than 12% of fines having little or no plasticity. GC and SC groups comprise gravelly or sandy soils with more than 12% of fines, which exhibit either low or high plasticity. ML and MH groups include the predominately silty materials and micaceous or diatomaceous soils. An arbitrary division between the two groups is where the liquid limit is 50. CL and CH groups comprise clays with low and high liquid limits, respectively. They are primarily inorganic clays. Low plasticity clays are classified as CL and are usually lean clays, sandy clays, or silty clays. Medium-plasticity and high plasticity clays are classified as CH. OL and OH groups are characterized by the presence of organic matter, including organic silts and clays. The Pt group is highly organic soils that are very compressible and have undesirable construction characteristics. Peat, humus, and swamp soils with a highly organic texture are typical.

Soil fractions are typically associated with keywords to help describe the percentage of soil components. Table 2.5 relates descriptive keywords with percentage ranges.

Table 2.5
Soil Fractions

Description	Percentage
Trace	1 - 10
Little	10 – 20
Some	20 – 35
And	35 - 50

Final classification of a soil in the Unified System will require laboratory tests to determine the critical properties, but a tentative field classification can often be used, but considerable skill and experience are required. Soil boring logs often include the engineering classification of soils as described herein.

Effective Stress and Pore Water Pressure

The *total* stress within a mass of soil at any point below a water table is equal to the sum of two components, which are known as *effective stress* and *pore water pressure*. Effective stress is defined as the total force on a cross section of a soil mass which is transmitted from grain to grain of the soil, divided by the area of the cross section, including both solid particles and void spaces. It sometimes is referred to as intergranular stress. Pore water pressure is defined as the unit stress carried by the water in the soil pores in a cross section. Effective stress governs soil behavior and can be expressed as:

$$\sigma' = \sigma - u \qquad \text{(Equation 2.1)}$$

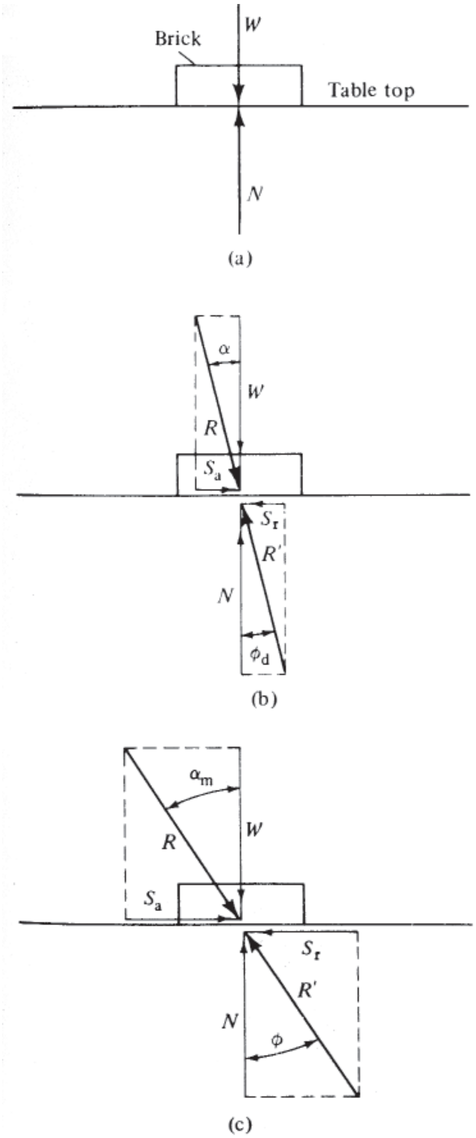
Where: σ' = the effective stress in the soil
σ = total (or applied) stress
u = pore water pressure

Soil Strength

One of the most important engineering properties of soil is its shearing strength, or its ability to resist sliding along internal surfaces within a given mass. Shear strength is the property that materially influences the bearing capacity of a foundation soil. The basic principle is similar in many respects to an object resting on a table. For example, imagine a brick resting on a table top as shown in Figure 2.5. The brick is in equilibrium under its own weight and the equal and opposite reaction force is provided by the table. Now imagine a horizontal force is applied to the brick near the tabletop. If this horizontal force is small, the brick will remain at rest, and the applied horizontal force will be balanced by an equal and opposite force. This resisting force is developed as a result of the roughness characteristics of the bottom of the brick and the table surface. If the applied horizontal force is gradually increased, the resisting force will also increase, always being equal in magnitude to the applied force. When the applied force equals or exceeds the resistance, the brick will slide across the tabletop. The slippage is a shear failure. The applied horizontal force is a shearing force and the developed force is shear resistance.

The *shear strength* is the maximum shear resistance that the materials are capable of developing. Shear strength consists of two parts. The first part is the friction between particles (physical property). The second part is called *cohesion*, or no-load shear strength due to a chemical bond between particles.

Figure 2.5
Friction on Horizontal Surface
Spangler (1982)



Angle of Internal Friction
The shear strength of a granular soil, such as sands, gravels and some silts, is closely analogous to the frictional resistance of solids in contact, as described above and shown in Figure 2.5. The relationship between the normal stress acting on a plane in the soil and its shearing strength can be expressed by the following equation, in terms of stress:

$$\tau_f = \sigma \tan\phi \qquad \text{(Equation 2.2)}$$

Where: τ_f = the shearing stress at failure, or the shear strength
σ = normal stress acting on the failure plane
ϕ = friction angle

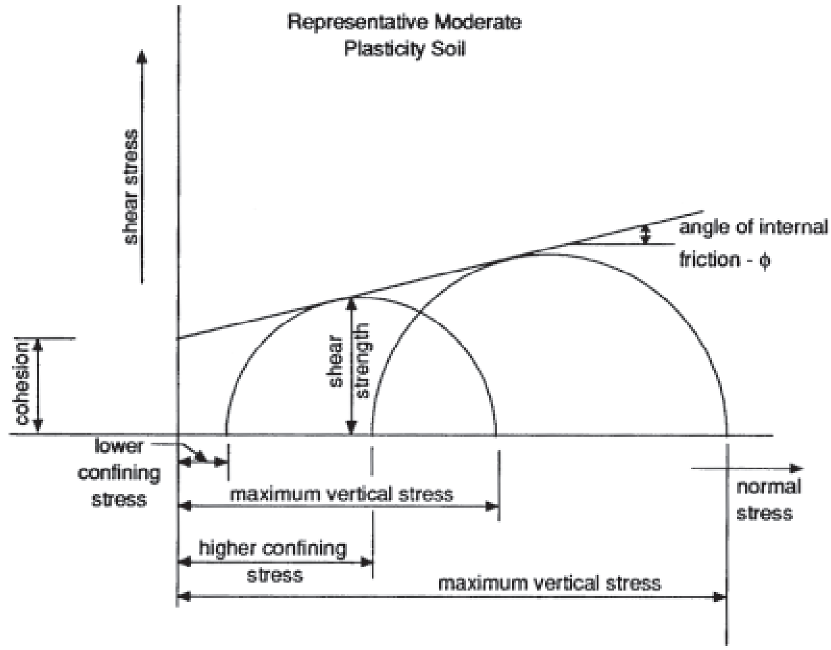
The internal friction of a given soil mass is related to the sliding friction between individual soil grains and the interlocking of soil particles. Shear strength attributable to friction requires a normal force (σ), and the soil material must exhibit friction characteristics, such as multiple contact areas. In dense soils, the individual soil grains can interlock, much like the teeth of two highly irregular gears. For sliding to occur, the individual grains must be lifted over one another against the normal stress (σ). Therefore, the force required to overcome particle interlock is proportional to the normal stress, just the same as sliding friction is proportional to normal stress. In soil mechanics, ϕ is designated the angle of internal friction, because it represents the sum of sliding friction plus interlocking. The angle of internal friction (ϕ) is a function of density, roundness or angularity, and particle size.

Cohesion
When a saturated clay is consolidated, that is, when the volume of voids decreases as a result of water being squeezed out of the pores, the shear strength increases with normal stress. If the shear strength of clays which have a previous history of consolidation (i.e. preconsolidated) is measured, the relationship between shear strength and normal stress is no longer a line intersecting the ordinate at zero. The clays exhibit a memory, or cohesive shear strength. In other words, the clays remember the preconsolidation pressure they were previously subjected to. This means considerable shear strength is retained by the soil. Figure 2.6 is an example of the relationship between shear strength

and normal stress for a preconsolidated plastic clay as derived from a triaxial shear test. The intersection of the line at the ordinate is called the cohesion.

Cohesion is analogous to two sheets of flypaper with their sticky sides in contact. Considerable force is required to slide one over the other, even though no normal stress is applied. Cohesion is the molecular bonding or attraction between soil particles. It is a function of clay mineralogy, moisture content, particle orientation (soil structure), and density. Cohesion is associated with fine grain materials such as clays and some silts.

Figure 2.6
Mohr's diagram for moderately plastic soil
Portland Cement Association (1992)



Coulomb Equation for Shear Strength

The equation for shear strength as a linear function of total stress is called the Coulomb equation because it was first proposed by C.A. Coulomb in 1773.

$$\tau_f = c + \sigma \tan\phi \qquad \text{(Equation 2.3)}$$

In terms of effective stress:

$$\tau_f = c + (\sigma - u)\tan\phi \qquad \text{(Equation 2.4)}$$

where: τ_f = shear strength
c = cohesion
σ = total stress acting on the failure plane
ϕ = friction angle
u = pore water pressure

Equations 2.3 and 2.4 are two of the most widely used equations in geotechnical engineering, since they approximately describe the shear strength of any soil under drained conditions. They are the basis for the bearing capacity equations (4.1 and 4.3) presented in Step 4.

Determination of Soil Parameters

To this point, various definitions, identification properties, limit states, engineering classifications, and soil strength properties have been discussed. This section details some of the more common soil exploration methods used to determine these various soil parameters. The extent to which a soil exploration program should reach depends on the magnitude of the project. If the proposed construction program involves only a small expenditure, the designer cannot afford to include more in the investigation than a small number of exploratory borings or test pits and a few classification tests on representative soil samples. The lack of information about subsoil conditions must be compensated for by using a liberal factor of safety. However, if a large-scale construction operation is to be carried out under similar soil conditions, the cost of a thorough and elaborate subsoil investigation is usually small compared to the savings that can be realized by utilizing the results in design and construction, or compared to the expense that would arise from a failure due to erroneous design assumptions. The designer must be familiar with the tools and processes available for exploring the soil, and with the methods for analyzing the results of laboratory and field tests.

The first step in any subsoil exploration should be an investigation of the general geological character of the site. The more clearly the geology is understood, the more efficiently can the soil exploration be carried out. The second step is to make exploratory drill holes or test pits that furnish more specific information regarding the general character and thickness of the individual soil strata. These two steps are recommended minimums. Other steps depend on the size of the project and the character of the soil profile.

Drilling

Drilling is typically the cheapest and most expedient procedure for making borings. Types of borings are *wash borings*, *rotary drilling* and *auger drilling*. Any one of the three can be used, but auger drilling is the most common – particularly for shallow borings. Wash borings and rotary drilling are similar in that they both use casing to hold the borehole open and a fluid medium to bring cuttings to the surface. The casing is either driven with a hammer or rotated mechanically while the hole is being advanced. The cutting bit and drill rods are inserted inside the casing and are rotated manually or mechanically. The cuttings allow the driller to visually classify the soil as to its type and condition and record the data on a log sheet at the depth of the cutting bit. Wash borings typically use water whereas rotary drilling typically uses drilling mud such as bentonite slurry. In some soil profiles, drilling mud prevents caving making full-length casing unnecessary. While drilling proceeds, the driller observes the color and appearance of the mixture of soil and water/mud. This enables the driller to establish the vertical sequence of the soil profile. At 5 ft (1.5 m) intervals, or when a change in strata is noticed, the cutting bit is removed and a spoon sample is taken.

Auger drilling typically uses a segmented continuous flight auger rotated mechanically while the hole is being advanced. The continuous flight auger often includes a hollow stem, which acts as a casing to hold the borehole open. Water or drilling mud is typically not used. Cuttings are carried to the surface by the auger flights, which allow visual classification of the soil. The advantage of the hollow stem auger is to permit the sampler and rod to be inserted down through the auger rather than having to remove the auger stems each time a sampler is inserted. Samplers are inserted inside the auger casing to retrieve disturbed and undisturbed soil samples typically at 5 ft (1.5 m) intervals. Figure 2.7 demonstrates an auger drilling operation.

Figure 2.7

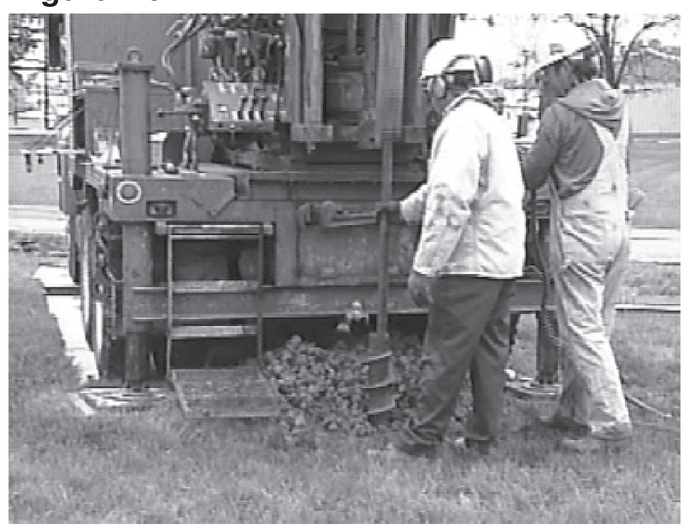
The *groundwater table (GWT)*, or *phreatic surface* is defined as the elevation at which the pressure in the water is equal to that of the atmosphere. Information regarding the location of the groundwater table is very important to the design and construction of deep foundations – especially in granular soils. Careful observations should always be made and recorded, if circumstances permit, during exploratory drilling. It is customary to note the water level on completion of the hole and after allowing the hole to stand overnight or for 24 hours before backfilling. The use of drilling mud to stabilize the walls of the hole may preclude obtaining this information.

Sampling

The cuttings or washings from exploratory drill holes are inadequate to furnish a satisfactory conception of the engineering characteristics of the soils encountered, or even the thickness and depths of the various strata. To obtain soil samples from exploratory drill holes, a *sampling spoon* is attached to the drill rod and lowered to the bottom of the hole. It is forced or driven into the soil to obtain a sample and is then removed from the hole. The soil is extracted from the spoon and inspected and classified by the driller. A small portion of the sample is placed in a glass jar and sealed for later visual inspection and laboratory determination of index properties.

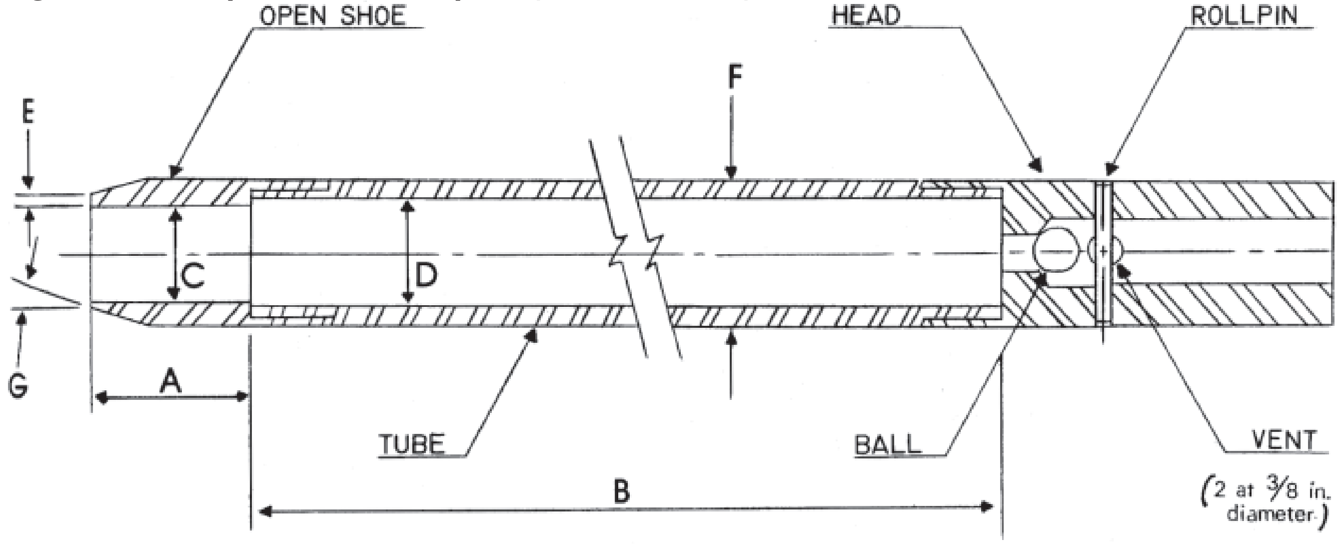
The most common method of obtaining some information concerning relative density or the stiffness of in-situ soil consists of counting the number of blows of a drop weight required to drive the sampling spoon a specified distance into the ground. This dynamic sounding procedure is called the *standard penetration test* (SPT). The essential features include a drop hammer weighing 140 lb (63.5 kg) falling through a height of 30 in. (0.76 m) onto an anvil at the top of the drill rods, and a split spoon having an external diameter of 2 in. (50.8 mm) and a length of 30 in. (0.76 m). The spoon is attached to the drill rods and lowered to the bottom of the drill hole. After the spoon reaches the bottom, the number of blows of the hammer is counted to achieve three successive penetrations of 6 in. (0.15 m). The number of blows for the first 6 in. is disregarded because of the disturbance that exists at the bottom of the drill hole. The number of blows for the second and third 6 in. increments are added and designated the *standard penetration resistance* (SPT), *N value*, or *blow counts*. The data obtained from SPT tests are commonly recorded on soil boring logs relative to the sounding depth where the sample was taken. SPT values are widely used to correlate the shearing strength of soil for the design of deep foundations – including helical screw foundations. Details of the equipment and procedures are specified in ASTM D 1586. Figure 2.8 illustrates a drill crew conducting a Standard Penetration Test. The split spoon sampler is shown in Figure 2.9.

Figure 2.8



Static sounding methods such as the cone penetration test (CPT) are widely used to give quantitative data in soil exploration. The cone penetrometer is pushed into the soil and the pressure exerted on the rod is measured. Friction cones allow both the tip resistance of the cone and the friction resistance along a jacket or sleeve to be measured. Some cone penetrometers are equipped with porous filters to permit the direct measurement of the porewater pressure. Such a devise is known as a piezocone. Cone penetrometers cannot

Figure 2.9 – 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 ± 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.

penetrate more than a few meters in dense sand, but they have been used to depths up to 40 m or more in soft soils. The friction ratio defined by the friction resistance divided by the tip resistance can be correlated with the type of soil encountered by the penetrometer. Since no samples are obtained by use of penetrometers, borings and sampling are needed for definitive information about the type of soil being investigated.

In general, soil samples taken from split spoon samplers are disturbed to some degree. For soil samples to be used for laboratory analysis, the degree of disturbance of the samples must be reduced to a minimum. Reasonably satisfactory samples can be obtained in 50 mm samplers made of steel tubing about 1.5 mm thick. The lower ends are beveled to a cutting edge to give a slight inside clearance. This type of sampler is commonly referred to as a Shelby tube. The Shelby tube is attached to the end of the drill rod and pushed vertically down into the soil to obtain an undisturbed sample. 2"-diameter ring enclosed samples may be taken with 3" O.D. sampler with segmented ring liner. The rings fit directly into the direct shear device or consolidometer. Hand samples or grab samples are sometimes taken from cuttings or test pits and are useful for soil classification and determining index properties.

Measuring Shear Strength

Shear strength is measured both in the field and in the laboratory. One of the most versatile devices for investigating undrained shear strength and sensitivity of soft clays is the *vane shear test*. It generally consists of a multi-bladed rectangular vane fastened to the bottom of a vertical rod. The blades are pressed their full depth into the clay surface and then rotated at a constant rate by a crank handle. The shear resistance of the soil can be computed from the torque and dimensions of the vane. One such type of the vane shear test is the *torvane*. It is a convenient hand-held device useful for investigating the strength of clays in the walls of test pits in the field or for rapid scanning of the strength of Shelby tubes or split spoon samples. A calibrated spring allows undrained shear strength (cohesion) to be read directly from the indicator.

Another device used in the laboratory or the field is the *pocket penetrometer*. As with the vane shear test, the pocket penetrometer is commonly used on Shelby tube and split spoon samples, and freshly cut test pits to evaluate the consistency and approximate *unconfined compressive strength* (q_u) of clay soils. The penetrometer's plunger is pushed into the soil ¼" and a reading taken on the sliding scale on the side. The scale is a direct reading of shear strength. Pocket penetrometer values should be used with caution for unsaturated or overconsolidated clays. It is not recommended for use in sandy, silty, or rocky soils

The *unconfined compression* (UC) test is used to determine the consistency of saturated clays and other cohesive soils. A cylindrical specimen is set up between end plates. A vertical load is applied incrementally at such a rate as to produce a vertical strain of about 1 to 2% per minute – which is rapid enough to prevent a volume change in the sample due to drainage. The *unconfined compressive strength* (q_u) is considered to be equal to the load at which failure occurs divided by the cross-sectional area of the sample at the time of failure. In clay soils where undrained conditions are expected to be the lower design limit (i.e. the minimum factor of safety), the undrained shear strength (i.e. cohesion) governs the behavior of the clay. This undrained shear strength is approximately equal to ½" the unconfined compressive strength of undisturbed samples.

The consistency of clays and other cohesive soils is usually described as *soft*, *medium*, *stiff*, or *hard*. Tables 2.6 and 2.7 can be found in various textbooks and are reproduced from Bowles, 1988. Values of consistency, overconsolidation ratio (OCR), and undrained shear strength (cohesion) empirically correlated to SPT N-values per ASTM D 1586 are given in Table 2.6. It should be noted that consistency correlations can be misleading because of the many variables inherent in the sampling method and the soil deposits sampled. As such, Table 2.6 should be used as a guide. Current practice often makes the use of Table 2.6 necessary because of the lack of reliable shear strength data. N_{70} indicates the raw "N" value from a normal sampling system, typically having a 70% efficiency in the hammer and rods.

Table 2.6
Consistency of Saturated Cohesive Soils

Consistency	Consolidation History	Blows/ft (N_{70})	Cohesion, ksf (kPa)	Comments
Very Soft	Normally Consolidated	0 – 2	<0.25 (12)	Runs through fingers
Soft	Normally Consolidated	3 – 5	0.38 (18.2) to 0.63 (30.2)	Squeezes easily in fingers
Medium	Normally Consolidated	6 – 9	0.75 (36) to 1.13 (54.1)	Can be formed into a ball
Stiff	Normally Consolidated to Overconsolidation Ratio of 2 - 3	10 – 16	1.25 (59.9) to 2 (95.8)	Hard to deform by hand squeezing
Very Stiff	Overconsolidated	17 – 30	2.13 (102) to 3.75 (179.6)	Very hard to deform by hand
Hard	Highly Overconsolidated	>30	>3.75 (179.6)	Nearly impossible to deform by hand

The relative density of sands, gravels, and other granular soils is usually described as very loose, loose, medium dense, dense, very dense, or extremely dense. The standard penetration test is a good measure of granular soil density. Empirical values for relative density, friction angle and unit weight as correlated to SPT N-values per ASTM D 1586 are given in Table 2.7. It should be noted that SPT values can be amplified in gravel because a 1"+ gravel particle may get lodged in the opening of the sampler. This can be checked by noting the length of sample recovery on the soil boring log (Table 2.8). A short recovery in gravelly soils may indicate a plugged sampler.

Table 2.7
Empirical Values for D_r, Friction Angle, & Unit Weight vs. SPT*

Description		Very Loose	Loose	Medium	Dense	Very Dense
Relative Density (Dr) (%)		0	15	35	65	85
SPT (N_{70})	Fine	1-2	3-6	7-15	16-30	?
	Medium	2-3	4-6	8-20	21-40	40+
	Coarse	3-6	5-9	10-25	26-45	45+
FrictionAngle (ϕ)	Fine	26-28	28-30	30-33	33-38	38+
	Medium	27-29	29-32	32-36	36-42	50+
	Coarse	28-30	30-34	34-40	40-50	50+
Total Unit Weight (γ_{wet}) (PCF)		70-100	90-115	110-130	110-140	130-150

*Assuming a 20 ft (6 m) depth of overburden and 70% rod efficiency on hammer

Shear strength also can be estimated by installing a helical screw foundation "probe" and logging installation torque vs. depth. The torque values can be used to infer shear strength based on the torque-to-capacity relationship discussed in Step 9.

Measuring Other Properties
Moisture or water content is measured by weighing a soil sample taken from the field on a laboratory scale. The soil sample is then placed in a standard oven for a sufficient time to allow all the moisture to evaporate. After being removed from the oven, soil sample is weighed again. The dried weight is subtracted from the original weight to determine the water weight of the sample. These methods are also used to determine the total (wet) unit weight and the dry unit weight.

Index properties (liquid limit, plastic limit, shrinkage limit, etc) are determined using specially developed apparatus and procedures for performing these tests. The equipment, specifications and procedures are closely followed in ASTM D 4318.

Special Conditions

All natural materials, such as soil, will exhibit conditions of variability that may make a single solution inadequate for inevitable problems that arise. It is wise to remember Dr. Terzaghi's emphasis to have a secondary solution ready when dealing with the variability of soils.

Deep Fill, Organic and Collapsible Soils

The existence of deep fills, organic and collapsible soils on a given project site are typically known before the start of the project. This is usually determined during the subsurface investigation by means of drilling or sounding. However, on large projects like an underground pipeline or transmission line that covers many miles, these soils may occur in undetected pockets and hence present a potential problem. The best solution is to be aware of the possibility of their existence and be prepared to install helical screw foundations deeper to penetrate through this material into better bearing soil. *It is not recommended to locate the helical bearing plates in these soils.*

Expansive Soils

Expansive soils exist all over the earth's surface, in nearly every region. The natural in-place weathering of rock produces sand, then silt, and finally clay particles – hence the fact that clay is a common soil type. Most clay soils exhibit volume change potential depending on moisture content, mineralogy, and soil structure. The upward forces (swell pressure) of expansive clay may far exceed the adfreeze forces generated by seasonally frozen ground, yet foundations continue to be founded routinely in expansive soil with no allowance for the potential expansion. Foundations should be designed to penetrate below the expansive soil's active zone, or be designed to withstand the forces applied the foundation, e.g. to prevent "slab dishing or doming." The *active zone* is defined as the depth of expansive soil that is affected by seasonal moisture variation. Another method used to design foundations on expansive soil is to prevent the soil's moisture content from changing. Theoretically, if the moisture content does not change, the volume of the clay soil will not change. This is typically difficult to control.

The tensile strength of deep foundations must be sufficient to resist the high tensile forces applied to the foundation by expansive soil via skin friction. This is typically known as *negative skin friction*, because the expansive soil can exert enough pressure to "jack" a foundation out of the ground. Helical screw foundations are a good choice in expansive soils due to their relatively small shaft size – which results in less surface area subjected to swell pressures and jacking forces. Isolating footings, slabs, and grade beams from sub-grade soils by using void form is a typical detail used in areas like Denver, CO where expansive soil is present. The void form isolates the structure from contact with the expansive soil, thereby eliminating the destructive effects of swell pressures.

A Plasticity Index (PI) greater than 30 is a red flag to the geotechnical engineer. A PI $\geq$ 30 indicates the soil has significant volume change potential and should be investigated further. There are fairly simple tests (Atterberg, soil suction test, swell potential) that can be conducted but should be practiced by the informed designer.

Shrinkage

Shrinking is the opposite of swelling soils. A volume change soil swells with increasing moisture content, but it will shrink with decreasing moisture content. Soil shrinkage can cause serious distress to a foundation/structure. The mechanism is the same as the expansive, but in the opposite direction.

One other aspect of shrinking soils is that strong desiccation forces in the past geologic history may have preconsolidated the soil to a significant level. Overconsolidation ratio's (OCR's) of 4 to 8 are not uncommon on the Gulf Coast. This can be helpful in founding structures in this denser soil, rather than penetrating it and then going very deep.

Lateral Earth and Water Pressure

This is not a special condition as much as it is a general consideration when designing basement walls and slabs below the water table. Waterproofing, reinforcing and drainage (to daylight) is the usual remedy. The reinforcing may include the use of helical screw tiebacks.

Seasonally Frozen Ground

The most obvious soil in this category is the frost susceptible soils (typically, silt) as illustrated by the growth of frost needles and ice lenses in freezing weather. This leads to a commonly observed expansion phenomenon known as *frost heave*. Frost heave is typically observed on road-beds, under concrete slabs, and along freshly exposed cuts. Capillary breaks and vapor barriers in conjunction with proper drainage will do much to control this problem, before helical screw foundations are installed.

A subcategory of this condition is seasonal permafrost. If possible, these ice lenses should be penetrated and not relied on for bearing.

Practical Guidelines for Estimating Soil Strength Parameters

The strength parameters typically provided for a fine-grained cohesive soil is the unit weight (seldom greater than 120 pcf [1922.2 kg/m^3]), and the cohesion (c). The cohesion is often obtained from an empirical correlation as shown in Table 2.6. Drilling and sampling can be used to determine the blow counts of cohesive soils and hence the cohesion. Generally, if one can indent their thumb about 1⁄16" into the newly exposed clay surface, the cohesion may be assumed to be 2 ksf (95.8 kPa). If a sharp blow with the geologist hammer can indent the surface, it can be assumed that the cohesion is 4 ksf (191.5 kPa).

The strength parameters typically provided for granular soils are unit weight (Maximum of 140 pcf 2242.6 [kg/m^3]) and friction angle (ϕ). The friction angle is often obtained from an empirical correlation as shown in Table 2.7. Drilling and sampling can be used to determine the blow counts of granular soils and hence the friction angle. The friction angle can also be assumed (conservatively) to be equal to the *angle of repose*. The angle of repose is defined as the angle of a cone shaped heap of clean, dry sand poured out on a level surface.

Noting the natural drainage ditches visible from the roadway or aerial view will also give one an idea of soil type. Sand or granular soils typically have sharply angled ditch bottoms whereas cohesive clay soils have wide U-shaped bottoms. The sides of the ditches stand vertical in loess and even in weathered loess the sides tend to stand vertical for a short distance.

Natural vegetation can also provide a clue to soil type and water regime. Most notably, mesquite bushes belie a deep expansive soil. Pine trees typically don't like "wet feet," whereas Cypress trees do. Tamarack often indicate deep wet organic soil.

The Geotechnical Report

The geotechnical report provides a summary of the findings of the sub-surface investigation, and the results of the laboratory testing. Geotechnical reports usually include an introduction detailing the scope of work performed, site history including geology, subsurface conditions, soil profile, groundwater location, potential design constraints such as seismic parameters and corrosion potential, foundation options, allowable load capacities, and an appendix which includes *soil boring logs*. The soil boring logs provide a wealth of information that is useful in the design of helical screw foundations. Borings logs come in literally endless variety since there is no standard form, but they contain basically the same type of information – most of which has been discussed in this section. Items to expect on a soil boring are: total boring depth, soil profile, description and classification, sample number and type, standard penetration test N-values, moisture content, Atterberg limits, unconfined compression strength or undrained shear strength (cohesion), groundwater table location. An example boring log is shown in Table 2.8.

Table 2.8 – Sample Boring Log

LOG OF TEST BORING

Project Name: ABC Plant Expansion	Client: Hubbell Power Systems	Project Number:	Boring: B-1
Site Location: Anywhere, USA	Start Date: 01/15/03	Finish Date: 01/15/03	Sheet No. 1 of 1
Drilling Company: North American Drilling	Drilling Method: Cont. Flight Auger	Logged by: DEB	Checked by: WEB

Drilling Equipment: CME Model 55, 3-1/4-inch I.D. hollow-stem augers	Boring Dia. (in) 5	Total Depth (ft) 37	Ground Elev (ft) 6	Depth/Elev. Ground Water (ft) @ Finish: 25	@ 24 hr: 22
SPT: Blows per ft on 2" OD sampler with 140 lb hammer falling 30"	NOTES:				

Depth (ft)	Elev. (ft.)	Sample Type	Sample Number	SPT – N (blows/ft)	Recovery (in)	Dry Unit Weight (pcf)	Moisture Content (%)	Soil Profile	Description and Classification	Unconfined Compressive Strength q_u (psf)	Atterberg Limits LL, PL, PI
									(CH) CLAY: Reddish brown fine silty, soft to medium. Trace root fibers, moist		
		Bulk	G-1			85	30				51, 28, 23
5	2	SH	H-1			83	28			650	50, 29, 21
									(CL) Silty CLAY: Light brownish gray w/dark yellowish brown silt seams and partings, medium-stiff to stiff		
10	-3	SS	S-1	9	14	90	24				42, 20, 22
15	-8	SS	S-2	10	16	92	23				
									(ML) Sandy SILT: Intermixed gray & dark yellowish brown, trace black silt partings, non-plastic, medium-dense		
20	-13	SS	S-3	14	18	95	27				
									(SM) Silty SAND: Dark yellowish brown, trace gray with black silt seams, trace to medium coarse sand trending to fine gravel, medium-dense to dense		
25	-18	SS	S-4	21	12	101	29				
30	-23	SS	S-5	28	14	103	30				
									(GW) Sandy GRAVEL: Dark brown, trace yellowish brown sand lenses trending to coarse gravel with depth, dense		
35	-28	SS	S-6	35	8	110	25				
37	-29	SS	S-7	50 /3"	3				LIMESTONE: Highly weathered, brown to gray, AUGER REFUSAL		

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