
# Soil Properties and Other Strange Things for Non-Geotechnical Engineers

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## Introduction

Geotechnical engineering is a fascinating subject. Unlike many engineering disciplines, it is not a pure science but rather it is an art form that requires both judgment and experience to arrive at a satisfactory solution. Unlike steel or concrete for instance, soil is quite different. For instance, geotechnical engineers can arrive at different but equally satisfactory recommendations for bearing capacity or settlement even when given the same information. Acceptable solutions are dependent upon many soil variables and the methods were used to predict the result.

The purpose of this course then, is to acquaint primarily the non-geotechnical engineer with some common properties, correlations and other interesting information about soil. The topics discussed herein have been simplified and are not exhaustive but they serve to demonstrate several principles. The interested reader should consult one of many textbooks on the topics discussed herein.

## Arriving at Acceptable Solutions

For a moment, consider that the deformation of a steel member under a compressive load is equivalent to the settlement of a foundation under its design load. Calculating the deformation of short sections of steel under an applied load is relatively straightforward and depends upon the applied load, area of the section, original length of the member and the elastic modulus of steel. All of these variables are easily acquired and require no interpretation.

On the other hand, calculating the theoretical settlement of foundations requires a great deal of interpretation and judgment. Factors such as the complexity of the soil profile, the engineering properties of the soil itself, the previous load history of the soil and the variation in groundwater level all play an important role in the outcome.

Is it strange then that geotechnical engineers can arrive at a different set of equally correct solutions to a problem even if they are given identical information? Since judgment is required along virtually every step of geotechnical design, differences in experience, judgment and methods of analysis can affect the conclusion. Soil properties are not specified and the engineer must develop the soil properties by explorations, testing and using the engineer's own experience and judgment. Since it is unlikely that anyone would have all of the information associated with a site, the engineer is faced with choosing simple models based on the limited data that is economically feasible to retrieve in order to predict the outcome.

An interesting study was undertaken in 1988 by Thomas F. Wolff to explore how judgment plays a role in geotechnical engineering design. In this study, a group of experienced practitioners and students were asked to design a shallow foundation (i.e. specify the size of footing required to carry the specified load but not exceed tolerable limits of settlement). Each participant was given identical information regarding loads and subsurface conditions. It was up to each participant to study the data, select appropriate values and methods to derive their conclusion.

The results of the study showed wide variability in the geotechnical values selected. In addition, interesting information was revealed about how the participants formulated their conclusions regarding values that were derived from the same set of subsurface information. Among the findings:

1. N-values, derived from the soil test boring logs, resulted in a range of values used in design. Participants selected values that ranged from 14 blows per foot to 26 blows per foot.

2. The soil friction angle selected by the participants ranged from 30 degrees to 35 degrees. However, no designer used a friction angle greater than 35 degrees even when correlations suggested a greater value.

3. Practitioners tend to be more conservative than students.

The study also showed that the participants recommended a wide range of footing sizes for design. The recommended footings ranged from 5 feet to 9.75 feet wide to support the same given load. Although each conclusion is correct, it reflects the results of many factors such as interpretation and selection of soil values, methods of analysis and the participant's experience.

**How Does the Bearing Capacity Solution Work?**

The discussion above revealed that there are many *correct* solutions to an assignment and the solution selected depends upon the practitioner. The method of analysis and selected values will change the results. Bearing capacity selection is a two phase approach. First the practitioner must select a safe bearing pressure to avoid catastrophic soil failure and second, the practitioner must select a bearing pressure that will not cause the foundation element to settle more than a tolerable amount. For shallow spread footings, total settlement is commonly limited to 1 inch although this depends upon the structure.

Bearing capacity is predicted based on commonly used bearing capacity equations. These methods vary from simple to complex. However, each are dependent upon the practitioner selecting soil values such as soil friction angle, soil cohesive strength and soil unit weight. The selection of soil values is also dependent upon the practitioners experience and how well the selected value represents actual conditions at the site.

Shallow footings can be described as square, rectangular, circular or continuous based on its shape. As an example of a simple equation, the ultimate bearing capacity (qu) of soil underlying a shallow strip footing can be calculated as:

$$qu = 1/2\gamma BN\gamma + cNc + \gamma DNq \qquad (1.0)$$

- $N\gamma$, Nc and Nq are bearing capacity factors that depend only upon the soil friction angle ($\phi$) as shown in Figure 1. The soil friction angle is commonly assigned by using charts or tables that correlate the penetration resistance obtained during the exploration program to the friction angle.

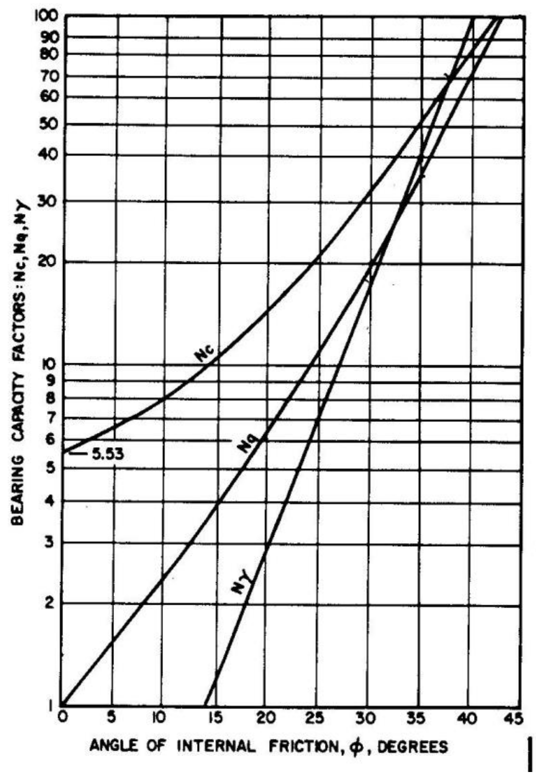


**Figure 1 – Bearing Capacity Factors**
[Ref: NAVFAC DM-7]

- The cohesion term "c" is obtained by laboratory or field-testing methods such as using a Torvane. Correlations using (Standard Penetration Test) SPT results are unreliable for assigning cohesion.

- The unit weight of the soil ($\gamma$) is commonly based on a published correlation with soil classification.

- The value "B" is the width of the footing and is the common symbol for the width.

- The value "D" is the depth of the footing below the lowest adjacent backfill. If the footing is backfilled equally on each side then D is the depth below grade. If the footing is backfilled unequally on each side as in a basement, then D is the lesser measurement.

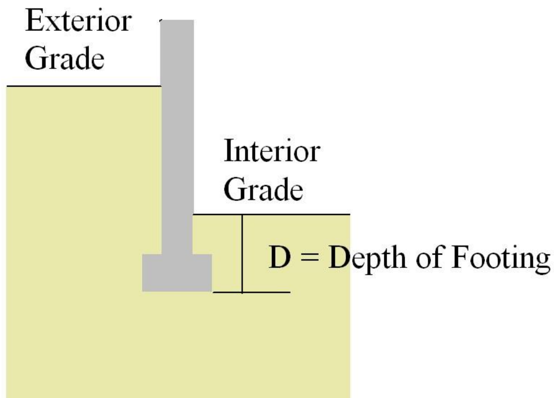


**Figure 2 – Depth of Footing**

The original bearing capacity equation shown in Expression (1.0) applies to continuous footings where the length L is very much greater than the width B. Since many footings however are square, rectangular or circular, the equation for a continuous footing was modified to account for the shape of the footing. Semi-empirical shape factors have been applied to each of the three components of the bearing capacity equation resulting in the following modifications:

- Square Footing: $qu = 0.4\gamma BN\gamma + 1.2cNc + \gamma DNq$
- Circular Footing: $qu = 0.3\gamma BN\gamma + 1.2cNc + \gamma DNq$
- Rectangular Footing: $qu = 1/2(1 - 0.2B/L)\gamma BN\gamma + 1.2cNc + \gamma DNq$

Later research improved the simple bearing capacity equations shown above by introducing a correction factor for shape of footing with load eccentricity, depth of footing, and inclination of load. Thus, the more complex General *Bearing Capacity Equation* has evolved as shown in Expression (2.0), which maintains the contribution from the three components identified earlier and incorporates appropriate correction factors for each term.

$$qu = 1/2\gamma BN\gamma\,(F\gamma sF\gamma d\,F\gamma i) + cNc(FcsFcdFci) + \gamma DNq(FqsFqd\,Fqi) \quad (2.0)$$

The factors beginning with "F" are the correction factors for depth (d), shape (s) and inclination of load (i) applied to the original terms proposed in Expression (1.0).

Further refinements include correction factors for sloping ground and tilting of the foundation base.

The ultimate bearing capacity obtained when using the General Bearing Capacity Expression (2.0) gives a bearing pressure that is too large for footings having widths (B) greater than approximately 6 feet. Accordingly a correction factor can also be applied to the first term of the General Bearing Capacity equation.

The ultimate bearing capacity derived above is divided by a factor of safety, commonly 3, to arrive at an allowable bearing capacity. This value might be further refined when considering settlement.

The reader should also remember that no matter how simple or complex the method, the solution is also dependent upon the soil values selected by the practitioner, which is also subject to interpretation.

*The calculation of bearing capacity and correction factors can become quite involved. Since there is no clearly defined universal set of values and equations used by all practitioners, it would not be unusual for the calculated results to vary among practitioners even when given the same set of subsurface conditions*.

**What's That Again?**

Water content (Wc), sometimes referred to as moisture content (Mc), can be greater than 100 percent. How is that? Does it seem unreal that water content could be greater than 100 percent especially when fully saturated soil is referred to as 100 percent saturated? Although it might be hard to visualize, you must first be aware that degree of saturation and water content are two completely different values and have different meaning. Thus, the correct answer is that it's all in the definition. Water content is defined as the <u>weight of water</u> divided by the <u>weight of the solid particles</u>. Percent saturation on the other hand is expressed as the <u>volume of water</u> divided by the <u>volume of voids</u>.

Since water content is not expressed in terms of <u>total volume</u> it is quite reasonable for the moisture content to be greater than 100 percent especially when considering an organic material such as peat. The weight of the vegetative particles (i.e. solids) in a unit volume of peat can be very much less than the weight of water included in that same unit volume. Peat can have a water content of several hundred percent.

While water content is related to the weight of solid particles, the degree of saturation is related to volume. The volume of water can not be greater than the total available volume to hold the water (i.e. volume of voids). Therefore, degree of saturation is limited to 100 percent. Values such as water content and degree of saturation are discussed below.

**Weight-Volume Relationship of Soil**

Soils are sediments and other unconsolidated material comprised of solid particles produced by disintegrations of rock and mixtures of such particles with organic substances. A volume of soil also contains liquid and gasses filling the void between the particles. Hence, a volume of soil is comprised of three phases: solid, liquid and gas.

Visualize for a moment a shovel full of soil. Likely, you will find solid particles such as sand of various sizes with voids between the particles. The voids are filled with air and quite possibly, some moisture. Imagine now that this sample is confined within a unit volume and all the solid particles are compressed together without any voids between the particles. Visualize that the water (moisture) contained in the sample collects on top of the solids and the air rides at the very top of the volume. This describes the three-phase diagram shown in Figure 3a. The diagram is presented in two dimensions rather than three.

In each of the definitions discussed, refer to the diagram shown in Figure 3a.

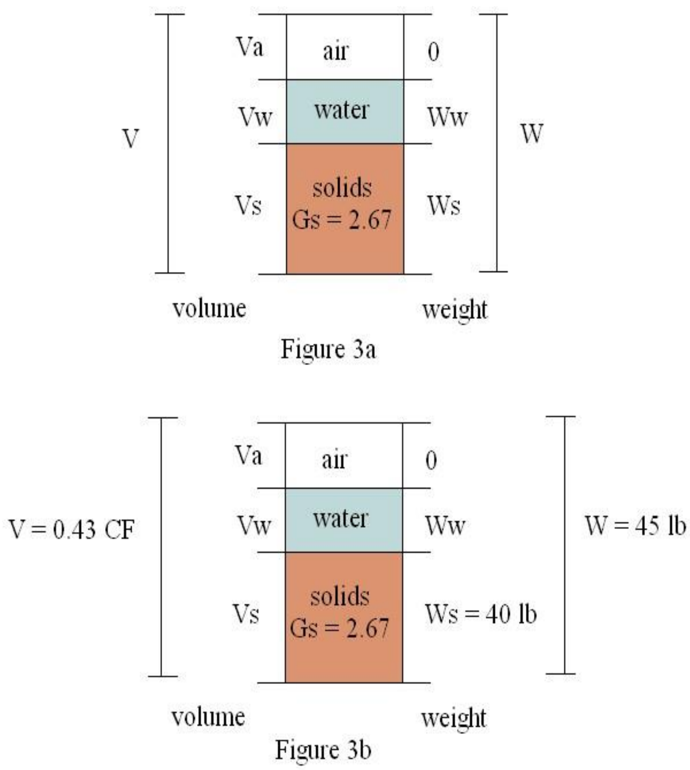


**Figure 3a, 3b – Three Phase Diagram**

The weight relationship of the phases is shown on the right hand side of the diagram while the volume relationship of the phases is shown on the left hand side of the diagram. It is important to note that each of the three phases, solid, liquid and gas has a volume but only solid and liquid have weight. Amongst geotechnical engineers, the gaseous state (i.e. air) has no weight. We're not picky about the molecular weight of air so in the grand scheme of things, the weight of air is zero.

From the diagram shown in Figure 3a, it is evident that:

- W = total weight of the mass while Ws = the weight of the solid phase, and Ww = the weight of the liquid (water) phase. Note that the total weight W is equal to Ws + Ww.

- V = total volume of the mass while Vs = the volume of the solid phase, Vw = the volume of the liquid (water) phase and Va = the volume of the gaseous (air) phase. Note that the total volume V is equal to Vs + Vw + Va. From now on, we'll refer to liquid as water and gas as air.

The volume of water and the volume of air comprise the volume of voids between the soil particles. The volume of the voids can be totally dry in which case there is no water or it can be totally full of water in which case there is no air. Both water and air can also be present in the volume of the voids. Note that the volume of voids Vv = Vw + Va.

The following definitions apply to soil.

**Void Ratio**

$$\text{Void Ratio (e)} = Vv / Vs \qquad (3.0)$$

Void ratio expresses the relationship between the volume of voids to the volume of solids in a unit volume of material. For a given sample of soil, a dense material has a lower void ratio than a loose material. When material is compacted in the field as part of constructing engineered fill, there is a void ratio reduction. The solid particles are forced closer together thus reducing the volume of the voids. For instance, when an 8 inch thick layer of soil is compacted, it becomes less that 8-inches thick. The volume of the voids is reduced by the compaction.

**Porosity**

$$\text{Porosity (n)} = (Vv / V) * 100 \qquad (4.0)$$

Porosity expresses the relationship between the volume of voids and the total volume. The higher the porosity of a material, the more porous the material becomes. Note that a soil with high porosity may not necessarily be highly pervious. Clay for instance has a high porosity but low permeability. Porosity is expressed as a percent.

**Degree of Saturation**

$$\text{Degree of Saturation (S)} = (Vw / Vv) * 100 \quad (5.0)$$

Degree of saturation expresses the relationship between the volume of water and the volume of the voids. Saturation is expressed as a percent. As shown earlier, if all the voids were filled with water, then Vw = Vv and S = 100%. The material would be fully saturated.

**Water Content**

$$\text{Water Content (Wc)} = (Ww / Ws) * 100 \quad (6.0)$$

Water content expresses the relationship between the weight of water in a given volume of material to the weight of the solids contained in that same volume. Water content is expressed as a percent.

**Total Unit Weight**

$$\text{Unit Weight } (\gamma) = W / V \quad (7.0)$$

The unit weight relationship is the <u>total</u> unit weight of soil because it relates total weight (Ws + Ww) with total volume (Vs + Vw + Va). Note, if the material is moist or saturated, this would be the moist or saturated unit weight of soil.

**Dry Unit Weight**

Sometimes, it is important to know the dry unit weight of soil ($\gamma d$) especially when calculating the degree of compaction. The dry unit weight of soil is expressed as:

$$\text{Dry Unit Weight } (\gamma d) = Ws / W \quad (8.0)$$

During field control of compaction it is necessary to know the "degree of compaction" attained. The inspector or engineer will calculate the in-place <u>dry</u> unit weight of soil retrieved from the compacted fill and compare it to the theoretical maximum <u>dry</u> unit weight of the material determined by laboratory testing.

**Specific Gravity**

Specific gravity is another value used in calculations.

$$\text{The mass or apparent specific gravity (Gm)} = W / (V * \gamma w) \quad (9.0)$$

$$\text{The specific gravity of the solids (Gs)} = Ws / (Vs * \gamma w) \quad (10.0)$$

Note that $\gamma w$ is the unit weight of fresh water (i.e. 62.4 lb pcf).

From these relationships it is possible to make other engineering calculations. There are other relationships that are not shown but they are all based on the fundamental relationships discussed herein. Other relationships can be found in publications such as DM-7 (see references).

**Example 1 – Weight-Volume Relationship**

Refer to Figure 3b for an example calculation using the relationships expressed above. The diagram shows that 45 pounds of material was retrieved from a hole that had a volume of 0.43 cubic feet. The material was dried and reweighed. The dry weight (weight of the solids Ws) is 40 pounds. From this information and using the relationships expressed above or derived from the phase diagram calculate:

Total Unit Weight ($\gamma$) = W / V = 45 / 0.43 = 104.6 pcf

Dry Unit Weight ($\gamma d$) = Ws / V = 40 / 0.43 = 93.0 pcf

Volume of Solids (Vs) = Ws / ($\gamma w$ * Gs) = 40 / (62.4 * 2.67) = 0.24 cf (from Eq. 10)

Weight of water (Ww) = W – Ws = 45 – 40 = 5 lb

Volume of Water (Vw) = Ww / $\delta w$ = 5 / 62.4 = 0.08 cf

Volume of air (Va) = V – Vs – Vw = 0.43 – 0.24 – 0.08 = 0.11 cf

Volume of voids (Vv) = Vw + Va = 0.08 + 0.11 = 0.19 cf

Void ratio (e) = Vv / Vs = 0.19 / 0.24 = 0.79

Porosity (n) = Vv / V = 0.19 / 0.43 = 44%

Degree of Saturation (S) = Vw / Vv = .08 / 0.19 = 42%

Water content (Wc) = Ww / Ws = 5 / 40 = 12.5%

**Example 2 – Weight Volume Relationship**

Refer to Figure 3a for the relationship guidelines. In this example, assume that the total unit weight of a sample of soil is 117 pcf. The material is 100 percent saturated and the water content is 41 percent. Calculate the void ratio (e).

If the material is 100 percent saturated than all of the voids are filled with water and the volume of air (Va) equals zero. Since the total unit weight is 117 pcf, the total weight (W) = 117 pounds and the total volume (V) equals 1 cubic foot.

The water content is 41 percent; therefore Ww / Ws = 0.41 which becomes Ww = (Ws)(0.41)

The total weight (W) = Ww + Ws and by substitution for Ww, W = (Ws)(0.41) + Ws and W = Ws(1 + 0.41)

Solving for Ws where W = 117 pounds, Ws = 117 / 1.41 or 83 pounds. Therefore, Ww = 117 – 83 = 34 pounds

The soil is 100 percent saturated, Vv = Vw and Vw = Ww / ($\gamma$w) = 34 / 62.4 = 0.54 cf

Since V = Vs + Vw, then Vs = V – Vw. Therefore, Vs = 1 – 0.54 = 0.46 cf

By definition, Void Ratio (e) = Vv / Vs = 0.54 / 0.46 = 1.17

There types of calculations are used to derive soil values.

**Notations**

There are a plethora of symbols and notation contained in textbooks on soil mechanics or foundation engineering. Every property or soil value is expressed by a symbol: unit weights of soil, void ratio, overburden stress, active pressure coefficient are examples. Different symbols might be used to express the same term or value, which can be confusing.

Often the notation that one uses in practice is a matter of preference and how one was “brought up” by education. In the final analyses however, it doesn’t matter what each symbol stands for as long as one understands the meaning intended. If one understands the meaning, then the specific notation is irrelevant.

Several examples are summarized in Table 1. Some notations show minor variations but others can be confusing especially when the reader consults several sources.

**Table 1 – Common Notations**

| Value | Nomenclature | Textbook Author |
|---|---|---|
| Maximum past pressure | $\sigma'_c$ | Sowers |
| | $p_c$ | Das |
| | $p_o'$ | PHT |
| Active earth pressure | $P_A$ | Sowers |
| | $P_a$ | Das |
| | $P_A$ | PHT |
| Ultimate bearing capacity of soil | $q_o$ | Sowers |
| | $q_u$ | Das |
| | $q_d'$ | PHT |

Sowers = G. B. and G. F. Sowers
PHT = Peck, Hanson and Thornburn
Das = Braja M. Das

**Percent Compaction**

Often structures are supported on compacted fill. Typically, unsuitable material can be removed from below a structure and replaced with compacted fill to support the structure. The structure can also be built to a new grade using compacted fill. In order for the structure to perform in a satisfactory manner, the fill must be placed and compacted to a specified standard. A standard commonly used is the degree or percent compaction although relative density is also used.

Percent compaction is a measure of the density (unit weight) of soil in place after it has been compacted to a standardized theoretical maximum density determined by laboratory methods. There are several ways of measuring the in-place density of soil in the field as defined by the American Society for Testing and Materials (ASTM). Methods such as the nuclear density gage (ASTM D2922), sand cone (ASTM D1556) and balloon (ASTM D2167) are available. The nuclear density gage is quite common today followed by the sand cone. The balloon method is seldom used. Using one of the specified methods, it is possible to determine the in-place density of the compacted soil.

Since the soil is almost always moist, the in-place density (unit weight), which has moisture in the voids is converted to dry density, which eliminates the weight variable related to moisture and expresses the density in terms of the weight of solids (Ws). The dry unity weight is calculated by determining the weight of solids that came out of a hole having a volume (V). The in-place density expressed in terms of dry unit weight becomes the numerator in the calculation for percent compaction.

The denominator is derived in the laboratory using a sample of the same soil that was placed as fill. Methods such as Standard Proctor (ASTM D698) or Modified Proctor (ASTM D1557) are used to determine the "theoretical" maximum dry density of the soil material. Note that the dry density is used to avoid the variability in unit weight resulting from moisture. The term "theoretical" is used because the density is determined in the laboratory based on using a specific amount of energy to compact the material.

The Standard Proctor Test was developed to duplicate in the laboratory as nearly as possible the results that could be obtained by compaction equipment working in the 1930s. Since then, compaction equipment has improved and it was possible to achieve higher dry unit weights. For this reason, the modified test was developed in response to the higher compactive efforts being achieved.

The energy used to compact the soil in the laboratory is based on dropping a 5.5-pound weight 12-inches (ASTM D698) or 10-pound weight 18-inches (ASTM D1557) a specified number of times on each layer of soil placed in a standard mold. For instance the maximum dry density can be determined using ASTM D1557 Method C when 5 equal height layers of soil are placed in a 6-inch diameter mold and each layer is compacted by making 56 hammer drops of a 10-pound weight falling 18-inches from the surface of the specimen. *The result is that the soil has been compacted using a specific amount of energy from the falling weight*.

In the field, the energy is applied by the compactor. If the contractor is so inclined, he can vary lift thicknesses and the number of passes made over a lift of soil. Depending upon circumstances, this can result in more energy being applied to compacting the soil in the field than applied in the laboratory to determine the "theoretical" maximum dry density. In this case, when the soil is over-compacted in the field, the resulting degree or percent compaction can be greater than 100 percent. Therefore, it is possible to achieve greater than 100 percent compaction although the results might seem strange.

**Consolidation vs. Compaction**

Consolidation and compaction are not the same although they both achieve the same end result. Consolidation and compaction both result in a void ratio reduction resulting in settlement. During consolidation, water is forced out or drained from the voids between the solid particles. This occurs when fully saturated clay settles below a building load.

During compaction however, air is forced out of the void space. This occurs in the field during compaction of an engineered fill.

**Soil Classification by Grain Size**

There are several published classification systems used to describe soil. These classification systems are based primarily on the size and distribution of various solid particles as well as the degree of plasticity. Although these systems are close, they are not necessarily identical. Examples include classification systems described by ASTM and AASHTO. The Unified Soil Classification System (USCS) is also a popular system. Some engineers will use the Burmister Soil Classification System and in this author's opinion the words used to describe the material flow easily in reports and are more descriptive. For instance, coarse to fine sand, little gravel trace silt could be used to describe a particular granular soil.

Soil descriptions shown on logs or in reports are often based on a visual classification by the person preparing the log without benefit of a mechanical analysis to verify the description. This leads to variances that are subjective. What one person describes as a coarse to fine sand could be described as a medium to fine sand by another because of visual interpretation. Terms such as "trace" meaning 10 percent and less, or "little" meaning 10 percent to 20 percent are sometimes difficult to apply when describing a material that contains particle sizes that are at or close to the borderline between the two descriptions. The author finds that it can be very difficult to *visually* distinguish between the percent of fine sand size material and silt size material. The days of the "taste test" are long since over.

Classification systems also vary in the particle size assigned to a specific soil description. As an example, Table 2 shows descriptive categories for two classification systems based on grain size. A review of these two systems shows the differences. The AASHTO system breaks fines into silt, clay and colloids, which differs from the USCS. The AASHTO system does not recognize medium sand or cobble size material as a specific classification while the USCS recognized these particle sizes. The complete classification of material also depends upon consistency as measured by the liquid limit and plasticity index of the material as derived from the Atterberg limits test.

**Table 2 - Comparison of Descriptive Grain Sizes**

<table>
<tr><td rowspan="2">USCS</td><td rowspan="2" colspan="3">fines (silt, clay)</td><td colspan="3">sand</td><td colspan="2">gravel</td><td rowspan="2">cobbles</td><td rowspan="2">boulders</td></tr>
<tr><td>fine</td><td>medium</td><td>coarse</td><td>fine</td><td>coarse</td></tr>
<tr><td colspan="11"></td></tr>
<tr><td rowspan="2">AASHTO</td><td rowspan="2">colloids</td><td rowspan="2">clay</td><td rowspan="2">silt</td><td colspan="2">sand</td><td rowspan="2" colspan="3">gravel</td><td rowspan="2" colspan="2">boulders</td></tr>
<tr><td>fine</td><td>coarse</td></tr>
</table>

[Based on Carter and Bentley, p. 4]

## Atterberg Limits

Atterberg Limits, most commonly Liquid Limit and Plastic Limit, are an integral part of several engineering classification systems to characterize fine grained soil. Fine grained soil such as silt and clay are finer the No. 200 sieve (finer than 0.002 mm grain size). These limits along with Plasticity Index can be used with other engineering properties to correlate with engineering behavior such as compressibility and permeability.

As a clayey soil is mixed with excessive water, it flows like a semi-liquid. As the material dries, it passes through a plastic, semisolid and then solid state. There is a reduction in the water content and also the void ratio as the material shrinks. The water content at which the soil changes from a liquid to plastic state is the Liquid Limit (LL) and the water content at which the soil changes from a plastic to semisolid state is the Plastic Limit (PL). Although these limits represent water content, they are expressed without the percent designation.

The Plasticity Index (PI) is the difference between the Liquid Limit and the Plastic Limit and is a measure of plasticity.

$$PI = LL - PL \quad (11.0)$$

A high Plasticity Index indicates that the material has significant clay content, while a low Plasticity Index near 0 indicates that the material is non-plastic such as silt.

Methods for determining the Liquid Limit, Plastic Limit and Plasticity Index of soils are described in ASTM D4318.

**Shear Strength**

Shear Strength is a fundamental engineering property of soil and it is usually expressed as:

$$S = c + \sigma \tan(\phi) \qquad (12.0)$$

Where S = shear strength
c = cohesion (property of cohesive soil)
$\sigma$ = normal stress on shear plane (usually the effective weight of the soil overburden above the shear plane)
$\phi$ = angle of internal friction of soil

For clay (cohesive soil) in undrained conditions, f = 0, thus S = c.

For sand (cohesionless soil), c = 0, thus $S = \sigma \tan(\phi)$.

For a soil that exhibits both cohesion and friction, equation 12.0 expresses the shear strength.

**Sensitivity**

Most clay loses some of its strength and stiffness when remolded or disturbed. The main cause may be a reorientation of the individual clay particles to a less favorable orientation. Sensitivity is determined in the laboratory as the quotient of the undisturbed strength to the remolded strength. Commonly the unconfined compression test with a value of the unconfined compressive strength (qu) is used to determine strength.

$$\text{Sensitivity} = \text{qu (undisturbed sample)} / \text{qu (remolded sample)} \qquad (13.0)$$

Insensitive clay that does not lose significant strength when disturbed has sensitivity less than 2. On the other hand, "quick" clay loses significant strength and has a sensitivity that exceeds 16. A common classification is shown in Table 3.

**Table 3 – Sensitivity Classification**

| Sensitivity | Classification |
|---|---|
| <2 | Insensitive |
| 2-4 | Moderately sensitive |
| 4-8 | Sensitive |
| 8-16 | Very sensitive |
| 16-32 | Slightly quick |
| 32-64 | Medium quick |
| >64 | Quick |

[Foundation Engineering Handbook, p. 136]

Sensitivity can also be described by the type of clay as shown in Table 4.

**Table 4 – Soil Description vs. Sensitivity**

| Sensitivity | Soil Description |
|---|---|
| 2-8 | Clay of medium plasticity, normally consolidated |
| 10-80 | Highly flocculent, marine clay |
| 1-4 | Clay of low to medium plasticity, overconsolidated |
| 0.5-2 | Fissured clay, clay with sand seams |

[Sowers and Sowers, p. 148]

**Engineering Properties of the Mass**

Solutions to engineering problems requiring an assessment of engineering properties of soil and rock involve determining quantitative information on the mass involved. When a site is explored for instance, a finite number of samples are retrieved from which engineering values are derived for the entire study area. This leads to two questions that must be answered by the engineer undertaking the assignment.

1. Are the samples tested representatives of the mass?

2. What are the combined effects of stratification, cracks, planes of weakness and other geometric and structural aspects of the mass?

In most cases, experience and judgment are required to interpret the results so that they can be used to develop a satisfactory engineering solution.

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