# Part 650 Engineering Field Handbook
# National Engineering Handbook

## Chapter 4 Elementary Soils Engineering

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# Part 650 – Engineering Field Handbook

## Chapter 4 – Elementary Soils Engineering

### 650.0400 Introduction

A. All structural measures and many land treatment measures for which NRCS provides technical assistance involve the use of soil as a building material or a supporting medium. Soil engineering is the application of physical, chemical, and mechanical properties of soil for its use as a construction material and as a foundation for structures.

B. This chapter addresses soil engineering. It includes the following major sections: (1) an explanation of basic soil concepts that relate to engineering; (2) an engineering classification and description system for soil; and (3) guidance for estimating soil strength, permeability, erosion resistance, and other performance characteristics. This chapter also discusses site investigation and presents a procedure for the preliminary design of earthen embankment dams and stream bank projects.

### 650.0401 Basic Concepts

A. Soil

Soil is defined as sediments or other unconsolidated accumulations of solid particles produced by the physical disintegration and or chemical decomposition of rocks. It may or may not contain organic matter. Soil consists of the solid particles and void space' called "soil voids" or simply "voids" within the soil matrix.

B. Soil Solids

(1) The soil solids are made up of mineral particles resulting from physical disintegration and or chemical decomposition of the parent rock or the minerals making up the parent rock. For example, granite often breaks down into individual particles of its minerals: feldspar, quartz, and mica. Chemical action on rocks causes decomposition of the rocks. Water, air, and certain acids or salts will, when associated with rock minerals under favorable conditions, combine to form new minerals by chemical reaction that may not be present in the parent rock.

(2) Decomposition usually follows disintegration. The mineral content of the parent rock and the extent of disintegration and decomposition determine the kind of soil and the engineering properties of that soil.

(3) The ease with which the parent rock is disintegrated and decomposed and the length of time these processes have acted determine the size of the granular soil particles. Angularity of sand and gravel particles are described as angular, subangular, subrounded, or rounded. Detailed definitions for these terms are given later in this chapter.

(4) The shape of soil particles may be spherical, flat, elongated, or flat and elongated. The size and shape of clay particles are determined by the chemical composition and molecular structure of each kind of clay.

C. Soil Voids

(1) Soil voids are that portion of a soil volume not occupied by solids. In a natural state, the voids may be filled by liquid, gas, or both.

(2) Gas in the voids is usually air. In soil engineering, air in voids is treated as being weightless. Liquid in the voids is usually water, which is considered incompressible.

D. Origin

(1) Soil at a location may have been formed in place from rock or transported to the present site by gravity, water, wind, ice, or some combination of these. Soil formed in place is called residual. Transported soils are described by their method of transportation.

(i) Colluvium is soil that has been transported by gravity. This transportation may have been so slow that it is difficult to measure, or it can be nearly instantaneous. Examples are loosened particles that roll or slide down a steep slope, large masses of soil that slowly 'creep' downhill, and landslides.

(ii) Alluvium is soil that has been transported to its present location by water. Aeolian or eolian soil has been transported to its present location by wind. This term is derived from Aeolus, Roman god of the wind. The transportation of soil by wind and water is characterized by three processes: (1) soil particles are transported by being rolled or slid along the ground surface or by being picked up and carried in suspension; (2) particles tend to separate and deposit according to their weight (size); and (3) certain particle sizes may be in a loose arrangement and subject to compaction as they settle.

(iii) Glacial till is soil that has been gouged out of the earth's crust by ice (glaciers) at one location and transported to another. In this process, there is usually little or no separation of particles by weight.

(2) Transportation processes are important agents of disintegration, particularly in producing finer size particles by mechanical degradation of larger particles. Engineering properties are influenced on how the soil was transported and deposited.

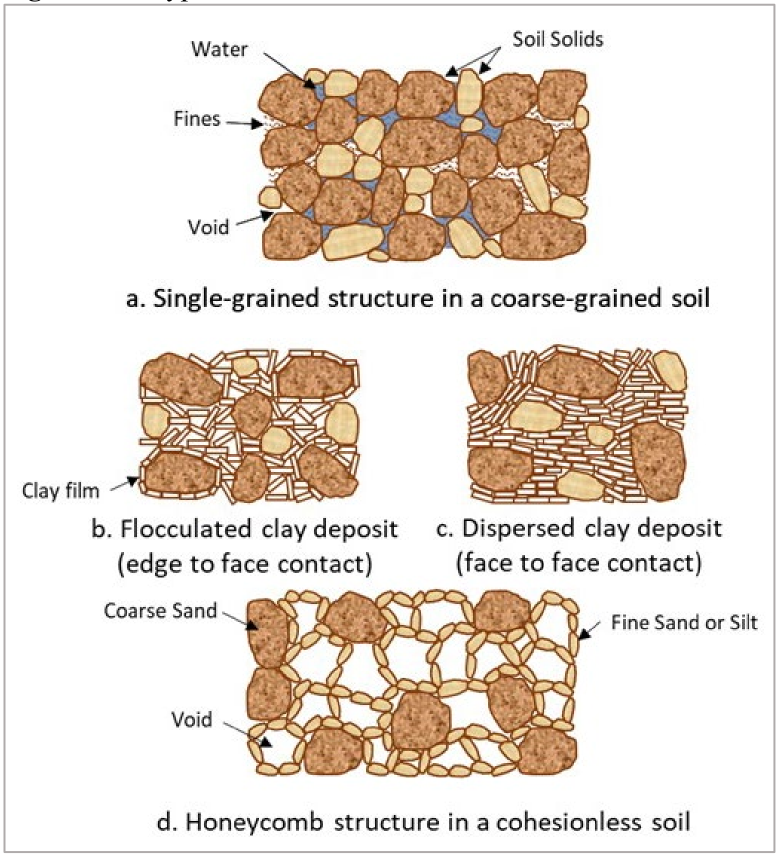
E. Structure

(1) Soil structure is defined as the arrangement and grouping of soil particles in a soil mass. Soils that have a “single-grain” structure are characteristic of coarse-grained or granular soils (figure 4-1a).

(2) A “Flocculent structure" describes a structure of elongated particles (usually clay particles) held together in groups or clusters of individual soil particles called "flocs," (figure 4-1b). When not flocculated, soil particles are said to be dispersed (deflocculated). Such soil has a "single-grain" structure, characteristic of coarse-grained or granular soils. Most clay particles are attracted to each other by electro-chemical bonding, which causes them to flocculate and adhere or cling together. Some soils contain salts or other compounds in the pore water, creating a condition in which the soil particles are not attracted to each other. Clays in which soil particles tend to separate or repel each other are called dispersive clays. When clays have deflocculated in water, they are called dispersed clays. These clays are formed by settlement of individual clay particles that are generally parallel in orientation (figure 4-1c)

(3) "Honeycomb structure" describes an arching arrangement of soil particles somewhat resembling a honeycomb in appearance (figure 4-1d). Particles are usually silt-sized and are relatively loose (open) but stable in its natural state. Soils with this type of structure are usually highly compressible and may collapse when the applied load becomes large enough to destroy the structural bonding or arching at the contact between particles.

**Figure 4-1:** Types of Soil Structure




F. Soil Water

(1) Most soils contain water. Water can exist in soil in several different forms. The water in soil voids may be influenced by external or internal pressures and can have positive, neutral, or negative pressure. Some definitions pertaining to water in soil are as follows:

(i) Hygroscopic water. The water adsorbed on the surface of soil particles as a thin film that (1) has properties substantially different from ordinary water, and (2) is removed by oven drying but not by air drying.

(ii) Capillary water. Water that is under tension in a soil due to stresses produced by menisci forming in the soil pores as water recedes into the voids from evaporation or is lost by other means. It is also the water which has been moved from one point to another through interconnected voids because of a change in capillary stress or tension.

(iii) Gravitational water. Water that is free to move through a saturated soil mass under the influence of gravity.

(2) Water content (w) is the percentage of the weight of water to the weight of the dry solids. The term "moisture content" is sometimes used instead of "water content."

(3) A saturated soil has its voids completely filled with water, and its water content is denoted by $w_{sat}$. Soil below the water table is usually considered to be saturated.

(4) Dry soil contains only air in the voids. Drying a soil to a constant weight in an oven at a temperature of 110 degrees Centigrade (C) +/- 5° C is the standard commonly used to determine the "dry weight," or in the SI units the "dry mass" of a soil.

(5) The optimum water or moisture content is the percentage of water in a soil, based on its dry weight, at which the maximum unit weight or density is obtained under a given compactive effort and is denoted by $w_{opt}$. The common procedure for determining water or moisture content is to dry the soil in an oven or by other means. Most clay soils that have drained to field capacity after wetting have a water content near optimum. Dry soils require the addition of considerable water to reach optimum water content. Soils with water content between dry and saturated are termed "wet." Saturated and dry conditions represent definite water contents, whereas wet makes up the range between these two limits.

G. Volume-Weight Relationships

(1) All soils are composed of solids, air, and water. In this section, the term "soil" includes all three components.

(2) Figure 4-2 (Sketch a) represents a volume of soil composed of solids and voids. The three-phase block diagram shown as Sketch b represents the bulk soil with the solids and void volumes separated into their respective proportions. The total volume is always equal to the sum of the volumes of the solids and voids.

$$V = V_s + V_v$$

(3) The volume of the voids is equal to the sum of the volumes of water and air as shown in sketch 4-2 c.

$$V_v = V_w + V_a$$

(4) The total volume is equal to the sum of the volumes of the solids, water, and air.

$$V = V_s + V_w + V_a$$

(5) The following volume relationships are useful in soil engineering.

(i) Void ratio. Void ratio "e" is the ratio of the volume of the voids to the volume of the solids. The void ratio can be equal to, greater than, or less than 1.0 and is usually expressed as a decimal.

$$e = \frac{V_v}{V_s} \text{ (dimensionless)}$$

(ii) Porosity. Porosity "n" is the percentage of the total volume that is void. Numerically, porosity can never be greater than 100 percent.

$$n = \frac{V_v}{V} \times 100\ (dimensionless)$$

(iii) Degree of saturation. Degree of saturation, "S", is the ratio of the volume of water in the voids to the volume of the voids, expressed as a percentage. When the voids are completely filled with water ($V_w$=$V_v$), the degree of saturation equals 100 percent.

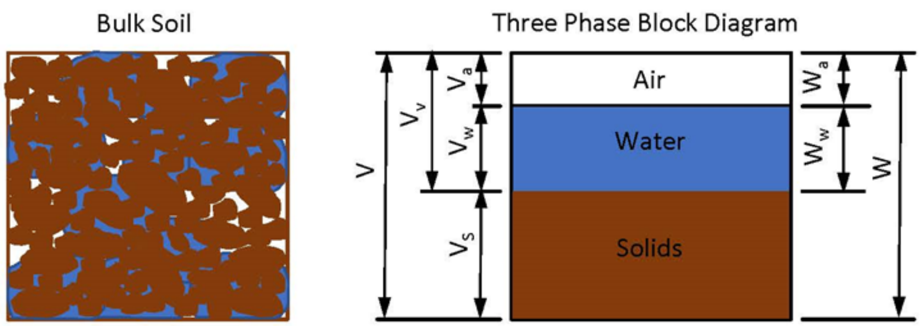
$$S = \frac{V_w}{V_v} \times 100\ (dimensionless)$$

(iv) Figure 4-2 (Sketch c) points out that the total weight is equal to the sum of the weights of the solids, water, and air.

$$W = W_s + W_w + W_a$$

Or $W = W_s + W_w$ since the weight of the air is considered to be zero.

**Figure 4-2:** Types of Soil Structure




Bulk Soil = Solids + Voids = Solids + Water + Air

**Volumes**

$$V = V_s + V_v$$
$$V = V_s + V_w + V_a$$
$$e = \frac{V_v}{V_s} \qquad n = \frac{V_v}{V} \text{ x } 100$$
$$n = \frac{e}{1+e} \text{ x } 100$$
$$e = \frac{n}{100 - n}$$

**Weights**

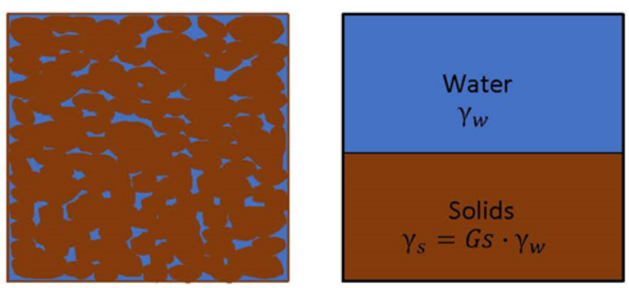
$$W = W_s + W_w + W_a$$
$$W = W_s + W_w$$
assuming $W_a = 0$

Moisture Content: $w = \frac{W_w}{W_s} \text{ x } 100$

Degree of Saturation: $S = \frac{V_w}{V_v} \text{ x } 100$

**Unit Weights**

Moist unit weight:
$$\gamma_m = \frac{W_s + W_w}{V}$$
Dry unit weight:
$$\gamma_d = \frac{W_s}{V} \text{ or } \frac{\gamma_m}{1+w}$$
Unit weight of water:
$$\gamma_w = 62.4 \frac{lb}{ft^3} \text{ or } 1.0 \text{ g/cm}^3$$




Saturated Soil (all voids filled with water)

Saturated unit weight:
$$\gamma_{sat} = \frac{W_s + W_w}{V}$$
$$\gamma_{sat} = \frac{W_s + V_v \cdot \gamma_w}{V}$$
where $W_w = V_v \cdot \gamma_w$

Submerged unit weight:
$$\gamma_{sub} = \gamma_{sat} - \gamma_w$$

Commonly used terms for saturation:
$$w_{sat} = \frac{\gamma_w}{\gamma_d} - \frac{1}{Gs}$$
$$\gamma_{sat} = \gamma_d \cdot (1 + w_{sat})$$
$$S = \frac{w}{w_{sat}} \text{x} 100$$

Specific Gravity:
$$Gs = \frac{W_s}{V_s \cdot \gamma_w} = \frac{\gamma_d \cdot (1+e)}{\gamma_w} = \frac{\gamma_d}{\gamma_w \cdot (100 - n)}$$

H. Water Content

An important weight ratio in soils engineering is the ratio of the weight of the water in the soil to the weight of the solids. This ratio multiplied by 100 is the percentage of water content or commonly referred to as moisture content.

$$W = \frac{Ww}{Ws} \times 100 = \text{percentage of water content}$$

I. Specific gravity

(1) The specific gravity ($G_s$) of soil solids is the ratio of the weight of a given volume of the soil solids compared to the weight of an equal volume of pure water. To be precise, if tested in a laboratory the lab report should state the temperature of the water. Often, this refinement is ignored in soil engineering, even though a standard set of conditions is used in determining specific gravity.

(2) The volume used to determine specific gravity of solids does not include any voids. This type of specific gravity, designated Gs, is commonly reported for sands and fines. Values usually fall between 2.5 and 2.8 for normal soils. A specific gravity of 2.65 is often used as an average value in qualitative evaluations. For highly plastic clays a $G_s$ of 2.70 can be considered. For soils with a high organic content will lead to a lower value, whereas some of the heavy minerals will give larger values.

J. Unit weight and density

Unit weight, ϒ (gamma), is defined as weight per unit volume. The total unit weight includes the weight of soil solids and water divided by the total volume, V. The equation is:

$$\gamma = \frac{\mathrm{w}}{\mathrm{v}}$$

(i) Units of weight and volume must be consistent.

- Imperial units

$$\gamma = \frac{\mathrm{w\ (lbs)}}{\mathrm{v\ (cu\ ft)}} = pcf$$

- SI units

$$\gamma = \frac{\mathrm{w\ (g)}}{\mathrm{v\ (cm^3)}} = \mathrm{g/cm^3}$$

(ii) Although not technically correct, density and unit weight are often used interchangeably in soil engineering. Unit weight is represented by $\gamma$ and density by ρ.

(iii) The unit weight of 1.0 cubic centimeter of distilled water at 4° C is 1.0 gram.

- Therefore, the unit weight of distilled water at 4° C in the SI system of units is:

$$\gamma_\mathrm{w} = \frac{1\ \mathrm{(g)}}{1\ \mathrm{(cm^3)}} = 1.0\ {}^{\mathrm{g}}/_{\mathrm{cm^3}}$$

- Since 1.0g = 0.002205 lbs and1.0 $cm^3$ = 0.00003531 cu ft

$$\gamma_w = \frac{1.0 \times 0.002205}{1.0 \times 0.00003531} = 62.47\ {}^{lbs}/_{cu\ ft}$$

- Therefore, in soil engineering the unit weight of water ($\gamma_w$) = 62.4 lbs/cu ft in foot-pound units

(iv) The unit weight of soil has a standard subscript and is as follows:

- Dry Unit Weight

$$\gamma_d = \frac{W_s}{V};\ \text{where } W_w = 0$$

- Moist Unit Weight

$$\gamma_m = \frac{W_s + W_w + W_a}{V};\ \text{where } W_a = 0$$

- Saturated Unit Weight

$$\gamma_{sat} = \frac{W_s + W_w}{V};\ \text{where } V_a = 0$$

- Submerged unit weight

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

(v) Submerged unit weight is sometimes referred to as the buoyant unit weight $\gamma_b$

(vi) A complete listing of volume-weight relationships is provided in figure 4-3.

**Figure 4-3:** Volume-Weight Relationship

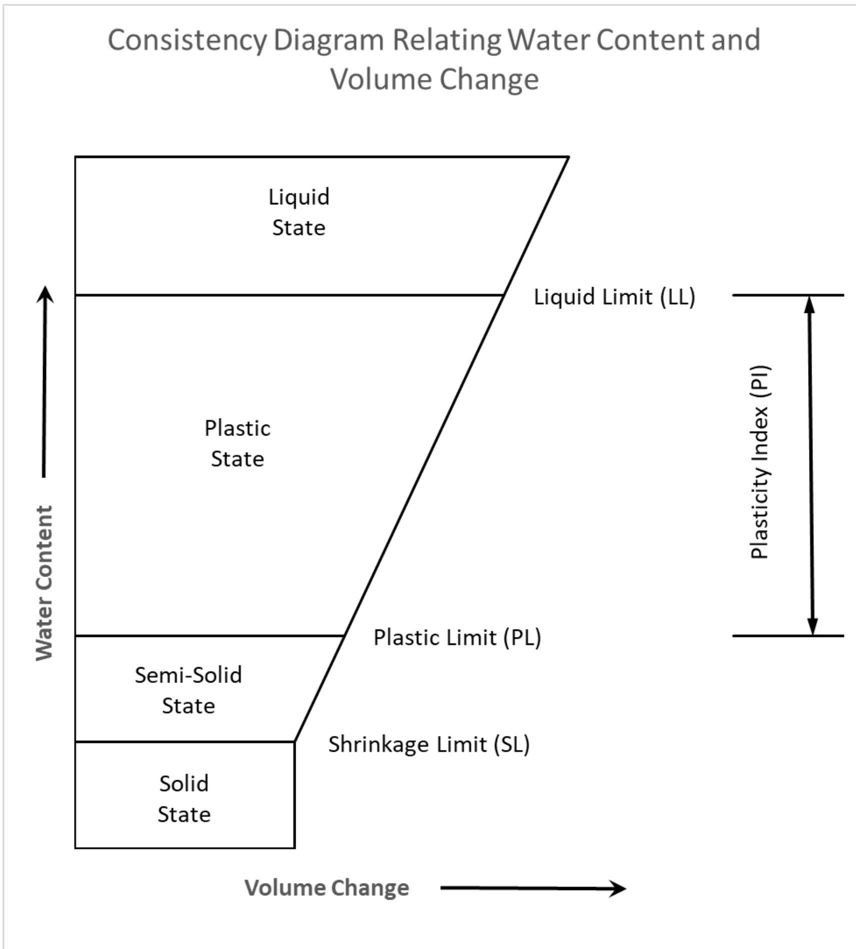
| | Property | | Saturated Sample | Unsaturated Sample | Other Useful Relationships | | | |
|---|---|---|---|---|---|---|---|---|
| Volume Components | $V_s$ | Volume of Solids | $\frac{W_s}{G_s \cdot \gamma_w}$ | | $V - (V_a + V_w)$ | $V \cdot (1 - n)$ | $\frac{V}{1+e}$ | $\frac{V_v}{e}$ |
| | $V_w$ | Volume of Water | $\frac{W_w}{\gamma_w}$ | | $V_v - V_a$ | $S \cdot V_v$ | $\frac{eSV}{1+e}$ | $eSV_s$ |
| | $V_a$ | Volume of Air | Zero | $V - (V_s + V_w)$ | $V_v - V_w$ | $(1 - S) \cdot V_v$ | $\frac{(1-S)eV}{1+e}$ | $(1 - S)eV_s$ |
| | $V_v$ | Volume of Voids | $\frac{W_w}{\gamma_w}$ | $V - \frac{W_s}{G_s \cdot \gamma_w}$ | $V - V_s$ | $\frac{nV_s}{1-n}$ | $\frac{eV}{1+e}$ | $eV_s$ |
| | $V$ | Total Volume of Sample | $V_s + V_w$ | $V_s + V_w + V_a$ | $\frac{W}{\gamma_d \cdot (1+w)}$ | $\frac{V_s}{1-n}$ | $V_s \cdot (1 + e)$ | $\frac{V_v \cdot (1+e)}{e}$ |
| | $n$ | Porosity | $\frac{V_v}{V}$ | | $1 - \frac{\gamma_d}{G_s \cdot \gamma_w}$ | $\frac{e}{1+e}$ | $\frac{\gamma_d \cdot w_{sat}}{\gamma_w}$ | $1 - \frac{V_s}{V}$ |
| | $e$ | Void Ratio | $\frac{V_v}{V_s}$ | | $\frac{G_s \cdot \gamma_w}{\gamma_d} - 1$ | $\frac{n}{1-n}$ | $\frac{\gamma_d \cdot w_{sat}}{\gamma_w - \gamma_s \cdot w_{sat}}$ | $\frac{V}{V_s} - 1$ |
| Weight of Sample | $W_s$ | Weight of Solids | $W - W_w$ | | $\frac{W}{1+w}$ | $G_s \cdot V \cdot \gamma_w \cdot (1 - n)$ | $\frac{W_w \cdot G_s}{e \cdot S}$ | $V \cdot \gamma_d$ |
| | $W_w$ | Weight of Water | $W - W_s$ | | $w \cdot W_s$ | $S \cdot \gamma_w \cdot V_v$ | $\frac{e \cdot W_s \cdot S}{G_s}$ | $V_w \cdot \gamma_w$ |
| | $W$ | Total Weight of Sample | $W_s + W_w$ | | $W_s \cdot (1 + w)$ | $V \cdot \gamma_d \cdot (1 + w)$ | | |
| | $W_{sat}$ | Saturated Weight of Sample | $W_s + V_v \cdot \gamma_w$ | N/A | $W_s \cdot (1 + w_{sat})$ | $V \cdot \gamma_d \cdot (1 + w_{sat})$ | | |
| | $W_{sub}$ | Submerged Weight of Saturated Sample | $W_s - V_s \cdot \gamma_w$ | N/A | $W_s \cdot \left[\frac{G_s - 1}{G_s}\right]$ | $V \cdot \gamma_d \cdot \left[\frac{G_s - 1}{G_s}\right]$ | $w_{sat} - V \cdot \gamma_w$ | |
| Unit Weights | $\gamma_d$ | Dry Unit Weight | $\frac{W_s}{V_s + V_v}$ | $\frac{W_s}{V_s + V_w + V_a}$ | $\frac{W}{V \cdot (1+w)}$ | $\frac{\gamma_m}{1+w}$ | $\frac{n \cdot \gamma_w}{w_{sat}}$ | $\frac{G_s \cdot \gamma_w}{1+e}$ |
| | | | | | $G_s \cdot \gamma_w \cdot (1 - n)$ | $\frac{e \cdot \gamma_w}{(1+e) \cdot w_{sat}}$ | $\frac{G_s \cdot \gamma_w}{1 + \left(\frac{w \cdot G_s}{S}\right)}$ | $\frac{G_s \cdot \gamma_w}{1 + G_s \cdot w_{sat}}$ |
| | $\gamma_m$ | Moist Unit Weight | N/A | $\frac{W_s + W_w}{V}$ | $\frac{W_s}{V} \cdot (1 + w)$ | $\gamma_d \cdot (1 + w)$ | $\frac{(1+w) \cdot G_s \cdot \gamma_w}{1+e}$ | $\frac{(1+w) \cdot G_s \cdot \gamma_w}{1 + \frac{w \cdot G_s}{S}}$ |
| | $\gamma_{sat}$ | Saturated Unit Weight | $\frac{W_s + V_v \cdot \gamma_w}{V}$ | N/A | $\left[\frac{G_s - 1}{G_s \cdot w_{sat} + 1}\right] \cdot \gamma_w + \gamma_w$ | $\gamma_d \cdot (1 + w_{sat})$ | $\gamma_d + n \cdot \gamma_w$ | $\frac{(G_s + e) \cdot \gamma_w}{1+e}$ |
| | $\gamma_{sub}$ | Submerged Unit Weight | $\gamma_{sat} - \gamma_w$ | | $\left[\frac{G_s - 1}{G_s \cdot w_{sat} + 1}\right] \cdot \gamma_w$ | $\gamma_d - (1 - n) \cdot \gamma_w$ | $\gamma_d - \frac{\gamma_d}{G_s}$ | $\left[\frac{G_s - 1}{1+e}\right] \cdot \gamma_w$ |
| Common Terms | $w$ | Moisture Content | N/A | $\frac{W_w}{W_s}$ | $\frac{W}{W_s} - 1$ | $\frac{e \cdot S}{G_s}$ | $\frac{n \cdot S}{G_s \cdot (1-n)}$ | $S \cdot \left[\frac{\gamma_w}{\gamma_d} - \frac{1}{G_s}\right]$ |
| | $w_{sat}$ | Saturated Moisture Content | $\frac{W_{sat} - W_s}{W_s}$ | N/A | $\frac{n \cdot \gamma_w}{\gamma_d}$ | $\frac{e \cdot \gamma_w}{\gamma_d \cdot (1+e)}$ | $\frac{\gamma_w}{\gamma_d} - \frac{1}{G_s}$ | $\frac{\gamma_{sat} - \gamma_d}{\gamma_d}$ |
| | $S$ | Degree of Saturation | $\frac{V_w}{V_v} = 1.0$ | $\frac{V_w}{V_v} < 1.0$ | $\frac{W_w}{V_v \cdot \gamma_w}$ | $\frac{w}{w_{sat}}$ | $\frac{w \cdot G_s}{e}$ | $\frac{w \cdot G_s \cdot \gamma_d}{G_s \cdot \gamma_w - \gamma_d}$ |
| | $G_s$ | Specific Gravity | $\frac{W_s}{V_s \cdot \gamma_w}$ | | $\frac{\gamma_d \cdot (1+e)}{\gamma_w}$ | $\frac{\gamma_d}{\gamma_w \cdot (1-n)}$ | $\frac{\gamma_d}{\gamma_w - \frac{w \cdot \gamma_d}{S}}$ | $\frac{W_s}{W_s - W_{sub}}$ |

Notes:
1. Weight of air is assumed to be zero.
2. Values of w, $w_{sat}$, S and n are expressed as decimals.
3. $\gamma_w$ is the unit weight of water equal to 62.4 pcf (1.0 g/cc) for fresh water and approximately 64 pcf (1.025 g/cc) for salt water.
4. $\gamma_d$ is commonly used in soils mechanics and the conversion to moist unit weight ($\gamma_m$) knowing the moisture content (w) is $\gamma_m = \gamma_d \times (1+w)$
   Example: $\gamma_d$ = 100.0 pcf and w = 15% $\gamma_m$ = 100.0 x (1+0.15) = 100.0 x 1.15 = 115.0 pcf
   Conversely if $\gamma_m$ = 115.0 pcf $\gamma_d$ = 115.0 / 1.15 = 100.0 pcf
5. A commonly used term is the saturated moisture content ($w_{sat}$) which is equal to ($\gamma_w/\gamma_d$) - (1/Gs)
   Example: Knowing $\gamma_d$ = 100.0 pcf and Gs = 2.65 Find the Saturated Moisture Content ($w_{sat}$) and the Saturated Unit Weight ($\gamma_{sat}$)
   Imperial Units ($\gamma_w$ = 62.4 pcf): $w_{sat}$ = (62.4/100) - (1/2.65) = 0.247 or 24.7% $\gamma_{sat}$ = 100.0 x (1+ 0.247) = 124.7 pcf
   SI Units ($\gamma_w$ = 1.0 g/cc) ($\gamma_d$ = 100.0/62.4 = 1.602 g/cc): $w_{sat}$ = (1.0/1.602) - (1/2.65) = 0.247 $\gamma_{sat}$ = 1.602 x (1.247) = 2.0 g/cc
6. The Degree of Saturation (5) for the sample with a $\gamma_d$ = 100 pcf from Notes 4 and 5 is equal to: $w/w_{sat}$ = 15.0/24.7 = 0.607 or 60.7%
   The submerged unit weight ($\gamma_{sub}$) if the sample were to be saturated (S = 100%) is equal to $\gamma_{sat} - \gamma_w$ = 124.7 - 62.4 = 62.3 pcf

K. Atterberg Limits

(1) As water is added to a dry plastic soil, the remolded mixture will eventually have the characteristics of a liquid. In changing from a solid to a liquid, the material first becomes a semisolid and then plastic. The Swedish scientist, Albert Atterberg developed tests to determine the water content at which these changes take place. The points at which the changes occur are known as the Atterberg limits. The system uses standardized testing procedures to establish four states of consistency - solid, semisolid, plastic, and liquid. Each is based on a range of water content.

**Figure 4-4:** Atterberg Limits




(2) Soils increase in volume as their water content increases above the shrinkage limit. This is illustrated in figure 4-4.

(3) The Atterberg limits are very useful in defining the behavior of fine-grained soil or the fine-grained components of coarse-grained soils with fines. Fines are soil particles that pass the No. 200 sieve (0.075 mm), Atterberg limits are determined on materials that pass the No. 40 sieve (0.425 mm).

(4) Refer to figure 4-4. In the determination of Atterberg limits, the water content of minus 40 material is measured at various moistures determined by prescribed test procedures. The plastic limit (PL) is the water content, by percent, at which the soil changes from a semisolid to a plastic state. The liquid limit (LL) is the water content, by percent, at which the soil water mixture changes from a plastic state to the liquid state. The difference between these two values is the range in water content at which the soil is plastic and is called the plasticity index (PI). The test procedures for liquid limit, plastic limit, and plasticity index of soils are given in ASTM designation D4318. The test procedure for shrinkage limit is given in ASTM D-427.

(5) In practice, the percent sign is dropped when referring to numerical values of the Atterberg limits. A material with a liquid limit of 50 percent is referred to as having a LL of 50.

(6) The Atterberg limits are defined as:

(i) Shrinkage limit (SL). The shrinkage limit is the water content at which a further reduction in water does not cause a decrease in the volume of the soil mass. This defines the limit between the solid and semisolid states of consistency.

(ii) Plastic limit (PL). The plastic limit is the water content corresponding to an arbitrary limit between the plastic and semisolid states of a soil's consistency. This is the water content at which a soil will just begin to crumble when rolled into a thread approximately 3mm (1/8 in) in diameter.

(iii) Liquid limit (LL). The liquid limit is the water content corresponding to the arbitrary limit between the liquid and plastic states of a soil's consistency. This is the water content at which a pat of soil, cut by a groove 2mm wide (5/64 in), will flow together for a distance of 13mm (1/2 in) under the impact of 25 blows in a standard liquid limit apparatus.

(iv) Plasticity index (PI). The plasticity index is the numerical difference between the liquid limit and plastic limit.

$$\mathrm{PI} = \mathrm{LL} - \mathrm{PL}$$

(v) Nonplastic (NP). When the liquid limit or plastic limit cannot be determined, or if the plastic limit is equal to or greater than the liquid limit, the soil is termed "nonplastic."

## 650.0402 The Unified Soil Classification System

A. Classification Using Laboratory Data

(1) NRCS uses the Unified Soil Classification System (USCS), ASTM D-2487 to classify soils for engineering purposes. This system is a 2-letter designation resulted in 15 soil subdivisions. Soils having similar engineering properties are placed into groups. The USCS system is based on the identification of soils according to their particle-size, gradation, plasticity index, liquid limit, and organic matter content. ASTM D-2487 describes the USCS. Particle-size of the sands and gravels are determined by sieve analyses. A hydrometer analysis determines the silt and clay sizes. Plastic and liquid limits are determined by Atterberg Limit tests.

(2) This system is for use on naturally occurring soils. The group names and symbols may be used to describe such materials as shale, clay stone, glacial till, alluvium, residuum soils, and other geologic formations. General engineering and hydrologic properties of soils can be estimated from physical characteristics allowing rapid preliminary assessment of site conditions during a field investigation program when little time is available for laboratory analysis.

B. Names and Symbols

(1) The USCS uses both group symbols and names to describe a soil's properties. Group symbols are based on laboratory tests performed on the minus 75mm (3 in) fraction of the material. The soil name description may describe larger material. Cobbles are rocks that are 75mm (3 in) to 300mm (12 in) and boulders are greater than 12 in.

(2) Soils having similar engineering properties are placed in soil groups. Each group is designated by a name and a two-letter symbol. The most important engineering characteristic of the group is described by the first letter in the symbol, and the second describes the engineering characteristic. Some soils are given dual symbols.

(3) The letters used as symbols and the soil properties they represent are listed below:

(i) G = gravel
(ii) S = sand
(iii) M = silt
(iv) C = clay
(v) O = organic
(vi) PT = peat
(vii) W = well-graded
(viii) P = poorly-graded
(ix) H = high liquid limit
(x) L = low liquid limit

(4) Two- or three-word names are used that have modifiers when needed. The names, with their two-letter symbols are as followed:

(i) Coarse-grained soils

- GW = well-graded gravel
- GP = poorly-graded gravel
- GM = silty gravel
- GC = clayey gravel
- SW = well-graded sand
- SP = poorly -graded sand
- SM = silty sand
- SC = clayey sand

(ii) Coarse grained soils may be modified by adding "with silt," "with clay," "with organic fines," "with sand," and/or "with gravel" Depending on the percentage of fines and sand or gravel contents.

(iii) Fine-grained soils

- CL = lean clay
- CH = fat clay
- CL = silty clay
- ML = silt
- MH = elastic silt
- OL or OH = organic silt
- OL or OH = organic clay

(iv) Fine grained soils may be modified by adding: “sandy,” “gravelly,” “with sand,” or “with gravel” depending on the percentages of sand or gravel contents.

(v) Highly organic soils

- PT = peat

C. Sieve Sizes

U.S. sieve sizes are used in describing soil classes. Commonly used sieves used in the descriptions and the sizes of their openings are shown in figure 4-5.

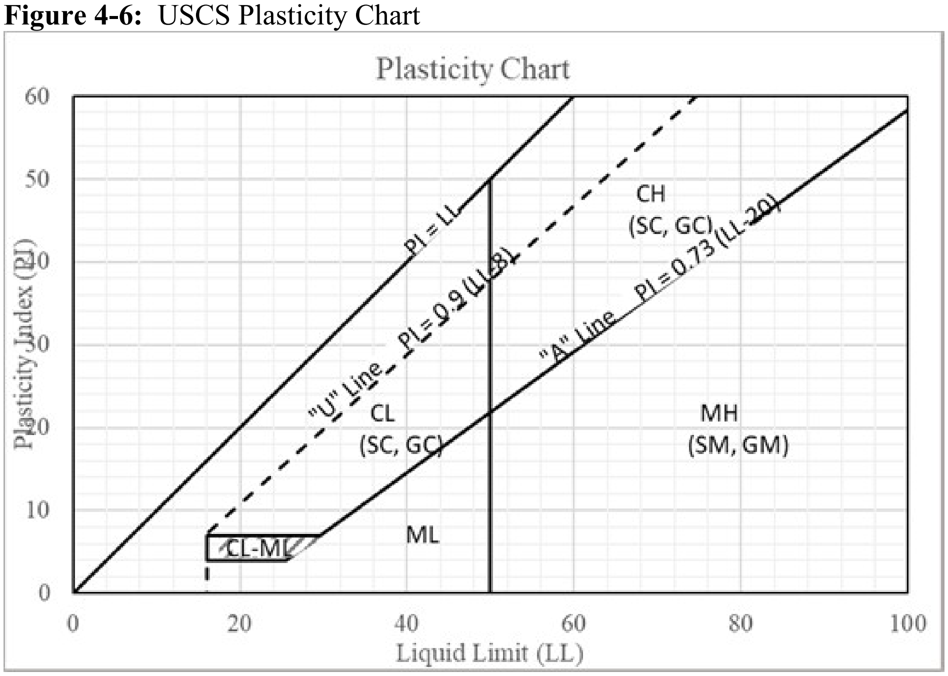
**Figure 4-5:** Sieve Designation and Size of Openings

| US Standard Sieve Sizes | Size of Opening in mm | Size of Opening in Inches |
|---|---|---|
| 3" | 75.0 | 3" |
| 3/4" | 19.0 | 3/4" |
| #4 | 4.75 | 3/16" |
| #10 | 2.00 | 0.0787 in. |
| #40 | 0.425 | 0.0167 in. |
| #200 | 0.075 | 0.00295 in. |

D. The Plasticity Chart

The plasticity chart, figure 4-6, is used to classify soil fines. It is constructed with the liquid limit (LL) as abscissa and the plasticity index (PI) as ordinate. Two lines are plotted on the chart, the A-line and the U-line. The A-line separates clayey soils from silty soils and lies approximately parallel to the plot of many basic geologic materials. The A-line is horizontal at PI = 4 then is equal to the plasticity index as defined by PI = 0.73 (LL-20). The U-line is the approximate "upper limit" for the plot of natural soils as defined by PI = 0.9 (LL-8). It is used to check for erroneous data. Any test data plotting to the left or above the U-line is probably in error and should be retested for verification. The formula for the U-line is vertical from LL = 16 to PI = 7, then is equal to the line defined by PI = 0.9 (LL-8). The plasticity chart also has a crosshatched area above the A-line between Pl's of 4 to 7 that defines an area of silty clays (CL-ML). The A-line and U-line can be extended for plotting soils with LL > 100 and PI > 60 by using the defined equations.

**Figure 4-6:** USCS Plasticity Chart



E. Classifying soils using the USCS

(1) Soils consist of a mixture of different sizes of particles. Mechanical analysis or field estimates are used to determine these sizes. The graphical representation is referred to as cumulative particle-size distribution curve, grain-size distribution or simply as gradation. Sieve analysis is used for particles larger than the No. 200 sieve and a hydrometer for the finer silt and clay sized particles

(2) In the USCS soils are classified as fine-grained or coarse-grained by the percentage of soil that passes the No. 200 sieve. If more than 50 percent of the soil, by dry weight, is retained on the No. 200 sieve, it is a coarse-grained soil. If 50 percent or more passes the No. 200 sieve, it is a fine-grained soil. The fine-grained soils are classified based on the plasticity chart (figure 4-6); whereas coarse-grained soils are based on their sand and gravel percentages. The name and verbal description include reference to the coarse fragments in the soil when applicable.

(3) In the USCS gravels are between 75mm (3 in) and 4.76mm (No. 4 sieve) sizes. Sands are between the No. 4 and No. 200 sieve sizes and fines are finer than the No. 200 sieve.

(4) Classifying Fine-Grained Soils:

(i) Plasticity is one of the most important index properties of a fine-grained soil. Plasticity is the property of soil which allows it to be deformed beyond the point of recovery without cracking or appreciable volume change. The name associated with the more plastic soils is clay (C), and the name for the less plastic or non-plastic soils is silt (M).

(ii) A soil whose plot of the liquid limit (LL) and plasticity index (PI) on the plasticity chart (figure 4-6) plots on or above the “A” line and has a PI of 4 or greater is classified as a fine-grained clayey soil (C). If it plots below the “A” line or has a PI less than 4, the material is classified as a fine-grained silty soil (M). Silty clays (CL-ML) have PI's of 4 to 7 (inclusive) and plot above the “A” line in the crosshatched area on the plasticity chart.

(iii) Soil that has a liquid limit of 50 or greater has a high (H) liquid limit and may be either a clay or silt. Clays with a high liquid limit are called fat clays or high plastic clays and have the symbol CH. Silts with a high liquid limit are called elastic silts and have the symbol MH. Soil that has a liquid limit less than 50 has a low (L) liquid limit and may be either a clay or a silt. Clays with a low liquid limit are called lean clays and have the symbol CL. Silts having a low liquid limit are called silts and have the symbol ML.

(iv) Fine-grained soils with sand and/or gravel. If the soil has 15 percent or more but less than 30 percent sand and/or gravel, then the word descriptors “'with sand" or "with gravel" are added to the group name. If the coarse-grained portion is one-half or more sand use “with sand”. Use "with gravel" if the coarse-grained portion is more than one-half gravel. Examples: silt with sand, ML; fat clay with gravel, CH.

(v) If the soil has 30 percent or more sand or gravel, add the words "sandy" or "gravelly" to the group name. Examples: sandy elastic silt, MH; gravelly lean clay, CL.

(5) Classifying Organic soils:

(i) Silts and clays that contain enough organic material to affect their engineering behavior significantly are classified as organic soils. They can be identified in the laboratory as being organic by performing liquid limit tests on both air dried and oven dried samples. If the liquid limit of the oven dried sample is less than 75 percent of the liquid limit of the air-dried sample, classify the soil as organic (O). It is either a low liquid limit organic silt or clay (OL) or a high liquid limit organic silt or clay (OH) based on its air-dry liquid limit and plotted position on the plasticity chart. Soils that fall on the crosshatched area of the chart are classified as organic clay (OL).

(ii) A soil composed primarily of plant tissue in various stages of decomposition is a highly organic soil and should be classified as peat (PT). These soils have a strong organic odor and are generally dark brown to black in color. Peat has a spongy consistency and a fibrous amorphous texture.

(6) Summary of fine-grained soils. There are eight soil classifications of fine-grained soils: clayey soil with low liquid limit, or lean clay (CL); clayey soil with high liquid limit, or fat clay (CH); silty soil with low liquid limit, or silt (ML); silty soil with high liquid limit, or elastic silt (MH); organic soil with low liquid limit, or organic clay (OL) and silt (OL); organic soil with high liquid limit, or organic clay and silt (OH); silty clay (CL-ML); and highly organic soil peat (PT).

(7) Classifying Coarse-Grained Soils.

(i) Coarse-grained soil particles are divided into sand and gravel at the No. 4 sieve. Particles retained on the No. 4 sieve are gravel-size. Particles passing the No. 4 and retained on the No. 200 are sand size. If more than 50 percent by dry weight of the coarse portion of a coarse-grained soil is predominately gravel-size, the soil is classified as gravel. If 50 percent or more of the coarse fraction of a coarse-grained soil is predominately sand size, it is classified as sand.

(ii) **Soils with less than 5 percent fines**. If less than 5 percent of the total sample by dry weight passes the No. 200 sieve, these small number of fines generally do not affect the soil's engineering properties. These soils are referred to as "clean sand or gravel." Only the characteristics of the coarse portion are important. The classification as sand or gravel is the material's primary characteristic.

(iii) The second most important characteristic of soil with less than 5 percent fines is its gradation (range of particle sizes). The soil may consist predominantly of one size; a mixture of coarse and fine materials with the intermediate sizes missing; or a mixture of relatively equal portions of all particle sizes. Soils in the first group are classified as poorly-graded and those in the last group are well-graded. The range of particle sizes are drawn on the cumulative particle-size plot from a sieve analysis. An example is plotted on a cumulative particle-size distribution curve (figure 4-7) and analyzed to determine if they meet the following criteria.

(iv) Sands and Gravels are well-graded when:

- For Gravel

$$C_u = \frac{D_{60}}{D_{10}} \textit{ is greater than } 4, \textit{and}$$

$$C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} \textit{ is at least } 1, \textit{but nor more than } 3$$

- For Sand

$$C_u = \frac{D_{60}}{D_{10}} \textit{ is greater than } 6, \textit{and}$$

$$C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}} \textit{ is at least } 1, \textit{but nor more than } 3$$

where: $C_u$ = coefficient of uniformity
$C_c$ = coefficient of curvature
D = diameter of particles determined by a sieve or hydrometer analysis

(v) $D_{60}$, $D_{10}$, $D_{30}$ = Diameter of particles such that 60 percent, 10 percent, and 30 percent of the sample is smaller than that diameter. The D sizes can be determined by reading from a grain-size distribution graph for 60, 10, and 30 percent fines by dry weight.

(vi) Both conditions ($C_u$ and $C_c$) for soils with less than 5 percent fines must be met in order to have a well-graded soil. If one or both conditions is not met, the soil is poorly-graded.

(vii) Some soils can be classified by visual inspection of the cumulative particle-size plot. When the curve is smooth and is concave upwards between the 10 percent and 60 percent lines they are well-graded. The gravel plotted on figure 4-7 is well-graded. Soils that are not well-graded are poorly-graded.

(viii) The gravel plotted on figure 4-7 is a well-graded gravel with a Cu = 5.8 and Cc = 1.1, which meets both criteria and is classified as a GW with sand. The “with sand” descriptor was added because the example sample contained 55 percent gravel, 43 percent sand and 2 percent fines.

(ix) There are four classifications of coarse-grained soils where the fines content is less than 5 percent. They are well-graded gravel (GW), poorly-graded gravel (GP), well-graded sand (SW), and poorly-graded sand (SP). If a gravel contains more than 15 percent sand, add "with sand" to the soil name. If a sand contains more than 15 percent gravel, add "with gravel" to the soil name.

(x) Classifications of the coarse soils described above are based on sieve analysis only.

(xi) **Soils with 12 to 50 percent fines.** If a soil is coarse-grained and has a fines content of more than 12 percent but less than 50 percent, the primary behavior characteristic is represented by both symbols, whether the soil is a sand or gravel. They are often referred to as dirty coarse-grained soils. The second behavior characteristic is based on that portion of the material passing the No. 40 sieve. If this portion of the material is clayey (has a PI > 7 and plots on or above the "A" line), the material is a coarse-grained soil with clayey fines (SC or GC). If it has a PI < 4 or plots below the "A" line, it is a coarse-grained soil with silty fines (SM or GM). If the PI is at least 4 and not greater than 7 and plots on or above the "A" line, the soil is clayey sand (SC-SM) or clayey gravel (GC-GM). If a gravel contains 15 percent or more sand, add "with sand" to the group name. If a sand contains 15 percent or more gravel, add "with gravel" to the group name.

(xii) There are six classifications of coarse-grained soils where the content of fines are greater than 12 percent. They are clayey gravel (GC) or (GC-GM), silty gravel (GM), clayey sand (SC) or (SC-SM), and silty sand (SM).

(xiii) **Soils with 5 to 12 percent fines.** Soils containing at least 5 percent, but not more than 12 percent fines are given a dual classification, which consists of the symbol for the soil with less than 5 percent fines first and the symbol for soils with more than 12 percent fines second. The dual symbol means that the soil has significant engineering characteristics represented by both symbols. The classifications are: SW-SM, SW-SC, SP-SM, SP-SC, GW-GM, GW-GC, GP-GM, and GP-GC. The first symbol is obtained by classifying the soil as though it had fewer than 5 percent fines. The second symbol is obtained by classifying the soil as though it had more than 12 percent fines. The group name corresponds to the name associated with the first symbol plus "with clay" or "with silt." If a gravel contains 15 percent or more sand, add "and sand" to the group name. If a sand contains 15 percent or more gravel, add "and gravel" to the group name. Examples: well-graded sand with silt (SW-SM), poorly-graded gravel with clay (GP-GC). Figure 4-7 graphically represents the classification of coarse-grained soils.

(8) Use of the Unified Soil Classification figure 4-8 is the soil classification chart for the USCS. The USCS has several outstanding features:

(i) It is logical and concise. Technicians and engineers normally make use of the five fine-grained soils (ML, CL, CL-ML, MH, CH), four clean coarse-grained soils (SP, SW, GP, GW), and six dirty coarse-grained soils (SM, SC, SC-SM, GM, GC, GC-GM) In addition, there are two organic soils (OL and OH), one highly organic soil (PT), and eight dual classified coarse-grained soils with 5 to 12 percent fines.

(ii) It provides information on important physical characteristics, such as particle size, grain size distribution, and plasticity. These index tests can be correlated with strength, consolidation, permeability parameters, and shrink-swell potential.

**Figure 4-7:** Grain-sized Distribution

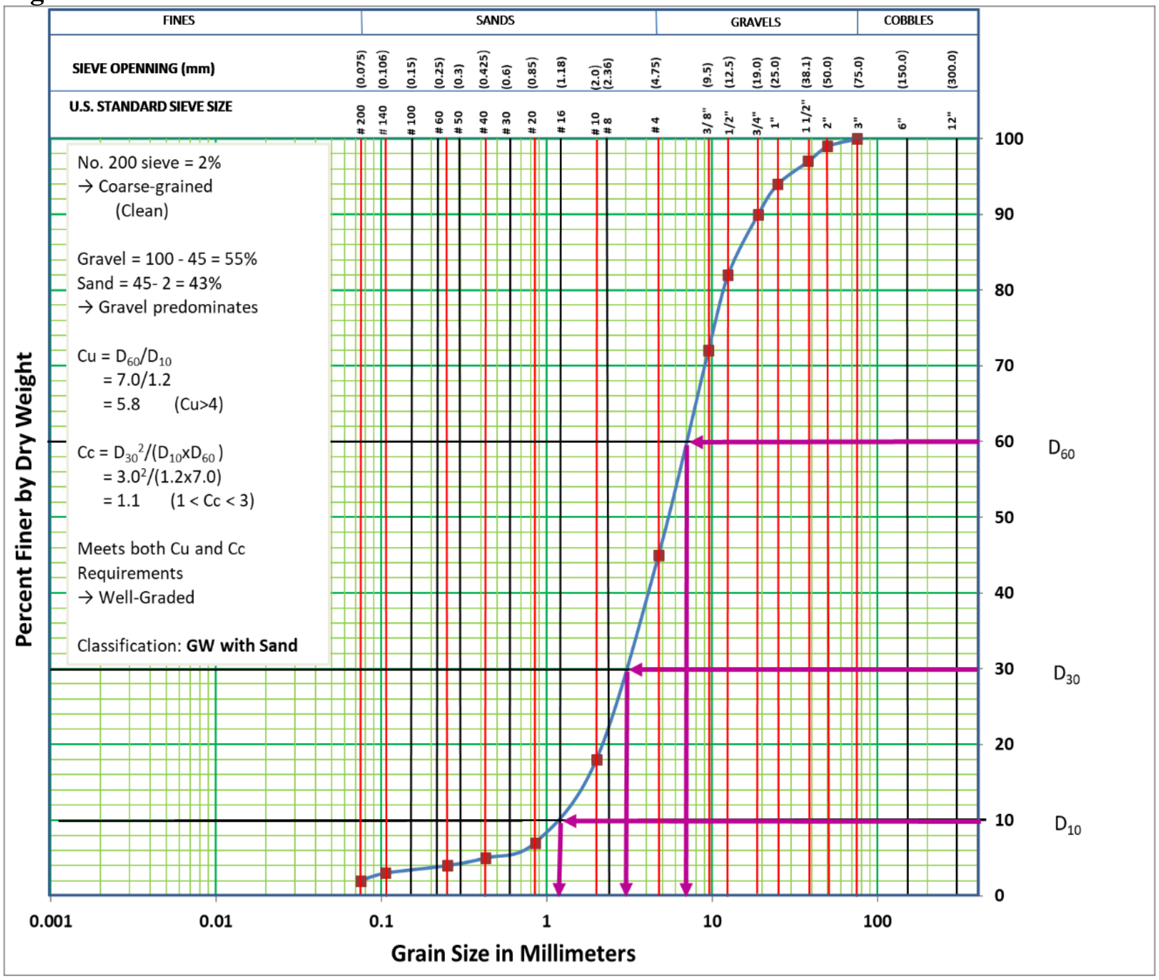


**Figure 4-8:** Soil Classification Chart

**Soil Classification Chart**

| Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[A] | | | | Soil Classification: Group Symbol | Soil Classification: Group Name[B] |
|---|---|---|---|---|---|
| COARSE-GRAINED SOILS | Gravels (More than 50 % of coarse fraction retained on No. 4 sieve) | Clean Gravels (Less than 5 % fines[C]) | Cu $ 4 and 1 #Cc # 3[D] | GW | Well-graded gravel[E] |
| | | | Cu < 4 and/or [Cc<1 or Cc> 3][D] | GP | Poorly graded gravel[E] |
| | | Gravels with Fines (More than 12 % fines[C]) | Fines classify as ML or MH | GM | Silty gravel[E,F,G] |
| More than 50 % retained on No. 200 sieve | | | Fines classify as CL or CH | GC | Clayey gravel[E,F,G] |
| | Sands (50 % or more of coarse fraction passes No. 4 sieve) | Clean Sands (Less than 5 % fines[H]) | Cu $ 6 and 1 #Cc # 3[D] | SW | Well-graded sand[I] |
| | | | Cu < 6 and/or [Cc<1 or Cc> 3][D] | SP | Poorly graded sand[I] |
| | | Sands with Fines (More than 12 % fines[H]) | Fines classify as ML or MH | SM | Silty sand[F,G,I] |
| | | | Fines classify as CL or CH | SC | Clayey sand[F,G,I] |
| FINE-GRAINED SOILS | Silts and Clays | inorganic | PI > 7 and plots on or above "A" line[J] | CL | Lean clay[K,L,M] |
| | Liquid limit less than 50 | | PI < 4 or plots below "A" line[J] | ML | Silt[K,L,M] |
| 50 % or more passes the No. 200 sieve | | organic | $\frac{\text{Liquid limit – oven dried}}{\text{Liquid limit – not dried}} < 0.75$ | OL | Organic clay[K,L,M,N] Organic silt[K,L,M,O] |
| | Silts and Clays | inorganic | PI plots on or above "A" line | CH | Fat clay[K,L,M] |
| | Liquid limit 50 or more | | PI plots below "A" line | MH | Elastic silt[K,L,M] |
| | | organic | $\frac{\text{Liquid limit – oven dried}}{\text{Liquid limit – not dried}} < 0.75$ | OH | Organic clay[K,L,M,P] Organic silt[K,L,M,Q] |
| HIGHLY ORGANIC SOILS | | Primarily organic matter, dark in color, and organic odor | | PT | Peat |

[A] Based on the material passing the 3-in. (75-mm) sieve.
[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[C] Gravels with 5 to 12 % fines require dual symbols:
GW-GM well-graded gravel with silt
GW-GC well-graded gravel with clay
GP-GM poorly graded gravel with silt
GP-GC poorly graded gravel with clay

[D] $Cu5D_{60}/D_{10}$ $Cc5 \frac{\$D_{30}d^2}{D_{10}3D_{60}}$

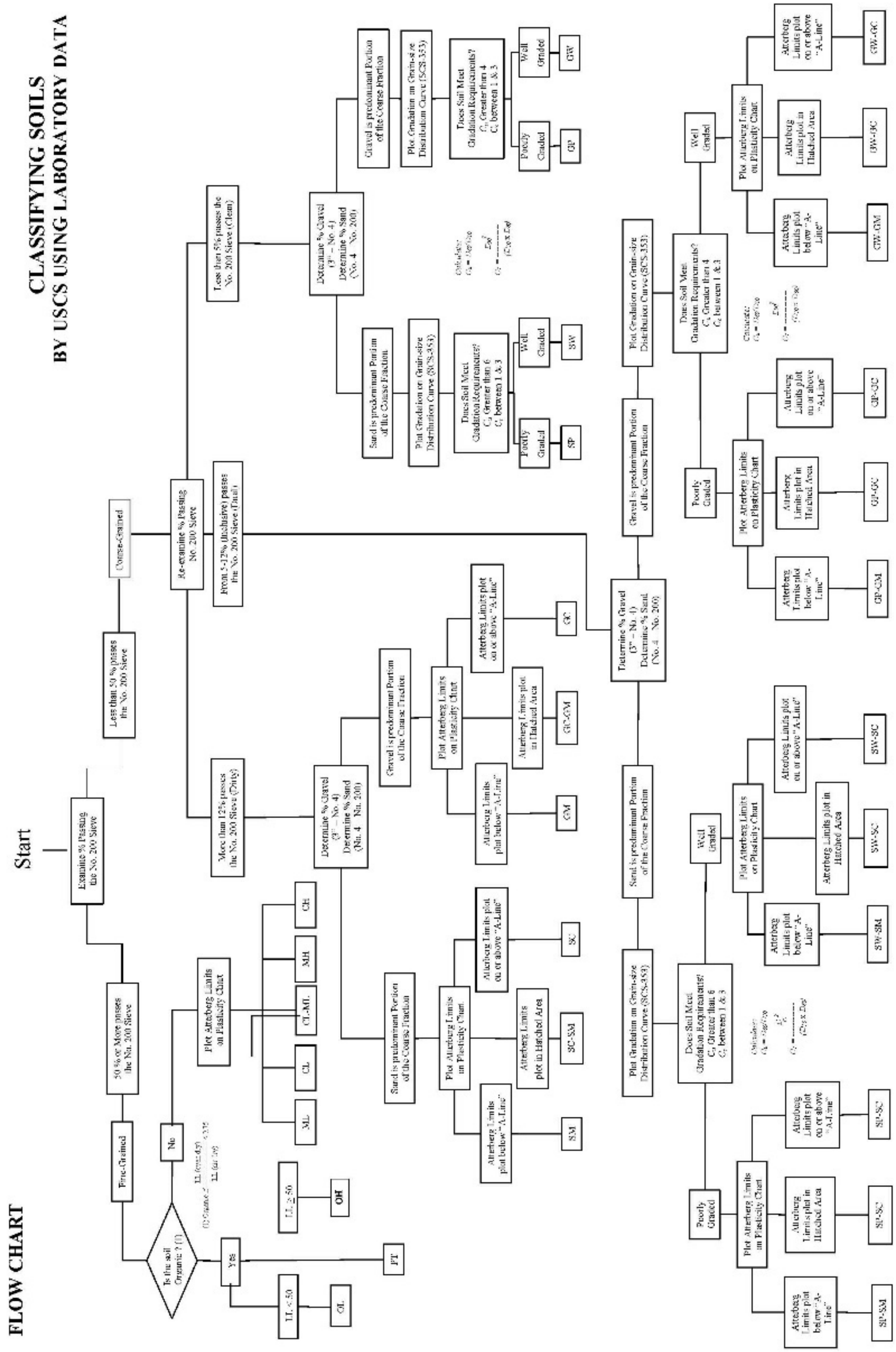
[E] If soil contains $15 % sand, add "with sand" to group name.
[F] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
[G] If fines are organic, add "with organic fines" to group name.
[H] Sands with 5 to 12 % fines require dual symbols:
SW-SM well-graded sand with silt
SW-SC well-graded sand with clay
SP-SM poorly graded sand with silt
SP-SC poorly graded sand with clay
[I] If soil contains $15 % gravel, add "with gravel" to group name.
[J] If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay.
[K] If soil contains 15 to <30 % plus No. 200, add "with sand" or "with gravel," whichever is predominant.
[L] If soil contains $30 % plus No. 200, predominantly sand, add "sand" to group name.
[M] If soil contains $30 % plus No. 200, predominantly gravel, add "gravelly" to group name.
[N] PI $ 4 and plots on or above "A" line.
[O] PI < 4 or plots below "A" line.
[P] PI plots on or above "A" line.
[Q] PI plots below "A" line.

(9) Interpretations based on classification must be made with diligence. A single classification represents a range of materials and may approach the classification of materials next to it. An example is a silt (ML) soil with LL = 42 and PI = 15. This plot is close to the "A" line on the plasticity chart. It has characteristics closer to a CL than to a nonplastic ML (PI = 0). An ML soil with 49 percent sand and gravel approaches an SM or GM classification, A well-graded nonplastic SM with 15 percent fines may have a high resistance to piping, whereas a poorly-graded SM with 45 percent nonplastic fines and fine sands would have very low resistance to piping.

(10) Figure 4-9 is a flow chart used to classify soils in the USCS using laboratory data.

(11) Detailed discussion and problems on the USCS are given in the NRCS Soil Mechanics Training Series Level 1.

**Figure 4-9:** Classifying Soils by USCS Using Laboratory Data



(12) Figure 4-10 compares the relationship of particle size between the USDA textural soil classification, the USCS, and the American Association of State Highway and Transportation Officials (AASHTO) soil classes. Classifications in the different systems are based on differing soil characteristics, and any attempt to translate between systems will be approximate at best. Design decisions should not be based on translations between systems.

(i) The USDA textural classes are based exclusively on particle-size distribution. Soil texture has agricultural applications such as determining crop suitability and to predict the response of the soil to environmental and management conditions. For engineering purposes clay mineralogy and the effects of plasticity and water-holding characteristics are of significance. The two classifications used for engineering purposes in the United States are the USCS and the AASHTO classification systems.

(ii) Because the USDA textural classes and uses are based on different soil characteristics, there is little direct correlation between systems. One type of soil can be identified by two or more classes. For instance, a silt loam in the USDA textural class can be either a ML or CL soil. When evaluating the engineering properties related to the two uses symbols, the general behavior characteristics are distinctly different. The USDA classification system does not explicitly consider plasticity, which is the essential characteristic of the uses.

(iii) In the Army Corps of Engineers publication TR-15-4 (March 2015) data from diverse databases were analyzed. Each of the 8 sources were first presented separately, and then combined. A summary of the most probable and possible classifications is summarized in figure 4-11. The consensus was determined to be the most frequent UDSA classifications that had occurred. There was good consensus between data sources for most of the soil types. The correlation should only be used for preliminary classification if only USDA textural classification is available. For high quality control, laboratory analysis based on uses classification should be used.

(iv) The AASHTO system was developed in 1929 as the Public Road Administration classification system. The soils are classified into seven major groups (A-1 to A7). There is no significance to the letter "A" other than serve as a rating of the soil materials as subgrade for pavements. A-1, A-2, and A-3 are granular material with 35 percent passing the No. 200 sieve. Those soils with > 35 percent passing the No. 200 sieve are classified as A-4, A-5, A-6, and A-7 and contain mostly silt and clay-type particles. In the AASHTO system silty applies to the fine fraction with a plasticity index (Pl) of 10 or less. Clayey is applied to the fine fraction as having a Pl of 11 or more. Figure 4-12 compares the AASHTO system and the USCS and was published by Liu (1967). Each of the soil groups are sited as being most probable, possible, and possible but improbable.

**Figure 4-10:** Relationship between Particle Size when Comparing the USDA Textural Soil Classes, the USCS, and the AASHTO Soil Classes

| USDA | FINE EARTH | | | | | | | | | ROCK FRAGMENTS (spherical or cube-like) | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Clay | | Silt | | Sand | | | | | Gravel | | | Cobbles | Stones | Boulders |
| | fine | coarse | fine | coarse | very fine | fine | medium | coarse | very coarse | fine | medium | coarse | | | |
| millimeters | 0.0002 | 0.002 | 0.02 | 0.05 | 0.1 | 0.25 | 0.5 | 1 | 2 | 5 | 20 | 76 | 250 | 600 | |
| U.S. Standard Sieve Number (opening) | | | | 800 | 140 | 60 | 35 | 18 | 10 | 4 | 3/4" | 3" | 10" | 25" | |

| | FLAT FRAGMENTS | | | |
|---|---|---|---|---|
| | Channers | Flagstone | Stones | Boulders |
| millimeters | 150 | 380 | 600 | |
| inches | 6" | 15" | 24" | |

| USCS | FINE-GRAINED SOILS (50% or more by dry weight passing the No. 200 (75µm) sieve | COARSE-GRAINED SOILS (more than 50% retained on the No .200 | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Silt or Clay (Fines) | Sand | | | Gravel | | Cobbles | Boulders |
| | | fine | medium | coarse | fine | coarse | | |
| millimeters | 0.075 | 0.425 | 2 | 4.75 | 19 | 75 | 300 | |
| U.S. Standard Sieve Number (opening) | 200 | 40 | 10 | 4 | 3/4" | 3" | 12" | |

**Figure 4-11:** Comparison of the USDA Texture Classification System and the USCS

| Soil Group in USDA System | Most Probable | Possible | Consensus between Data Sources |
|---|---|---|---|
| Sand | SP, SM, SW | SP-SM | SP |
| Loamy Sand | SM | CL, SC | SM |
| Sandy Loam | SM, SC | ML, CL | SM |
| Sandy Clay Loam | SC, CL | SC | SC |
| Sandy Clay | SC, CL | CL | SC |
| Loam | CL, ML | ML | CL |
| Silt Loam | ML, CL, SM | ML | ML |
| Silt | ML | -- | ML |
| Clay Loam | CL, MH | -- | CL |
| Silty Clay Loam | CL, MH | ML, CH | CL |
| Clay | CH, CL | CL, GC | CH |
| Silty Clay | CH, CL, MH | -- | CH |

**Figure 4-12:** Comparison of the AASHTO System and the USCS (Liu, 1967)

| Soil Group in AASHTO System | Most Probable | Possible | Possible but Improbable |
|---|---|---|---|
| A-1-a | GW, GP | SW, SP | GM, SM |
| A-1-b | SW, SP, GM, SM | GP | -- |
| A-3 | SP | -- | SW, GP |
| A-2-4 | GM, SM | GC, SC | GW, GP, SW, SP |
| A-2-5 | GM, SM | -- | GW, GP, SW, SP |
| A-2-6 | GC, SC | GM, SM | GW, GP, SW, SP |
| A-2-7 | GM, GC, SM, SC | -- | GW, GP, SW, SP |
| A-4 | ML, OL | CL, SM, SC | GM, GC |
| A-5 | OH, MH, ML, OL | -- | SM, GM |
| A-6 | CL | ML, OL, SC | GC, GM, SM |
| A-7-5 | OH, MH | ML, OL, CH | GM, SM, GC, SC |
| A-7-6 | CH, CL | ML, OL, SC | OH, MH, GC, GM, SM |

## 650.0403 Field Identification and Description of Soils

A. This section explains how to classify and describe soils by visual-manual methods in ASTM D-2488. The methods are similar to those in ASTM D-2487 used in laboratory classification. The classification is basically the same as described in the previous section. It does contain some modifications to adapt it to the less precise visual-manual procedure as compared to laboratory methods. It must be clearly stated in the report that classification is based on visual-manual methods.

**Figure 4-13:** A photograph of augured samples from the site are good documentation.

B. Dual and Borderline Classes

(1) Care should be taken not to confuse the dual classifications described in the preceding section and the borderline classifications sometimes used in the field procedure. Dual symbols in ASTM D-2487 are used in laboratory classification where soils fall into the dual categories described in the preceding section. These are fine-grained soils or the fine-grained portion of coarse-grained soils falling in the crosshatched area on the plasticity chart (CL-ML), and coarse-grained soils that have 5 to 12 percent fines (SP-SM, SP-SC, SW-SM, SW-SC, GP-GM, GP-GC, GW-GM, GW-GC).

(2) Field classification includes a borderline classification category not to be used when classifying with laboratory data. When a soil has properties that do not distinctly place it in a specific group, it can be classified using a borderline symbol indicating the soil may fall into one or the other of the two groups. A borderline classification consists of two group symbols separated by a slash, for example, CL/CH, GM/SM. The slash stands for "or" (CL/CH means CL or CH). The first group symbol represents the most likely classification, or it is the classification of similar samples from the adjacent area. The group name is the name for the first symbol except for the following borderline classes and names.

(i) CL/CH = lean to fat clay
(ii) ML/CL = clayey silt
(iii) CL/ML = silty clay

**Figure 4-14:** Field assessments should not take the place of laboratory analysis for high risk sites.

(3) Borderline symbols should not be used indiscriminately. Every effort should be made to place the soil in a single group.
(4) When a soil is field classified the percentages of sand, gravel, and fines are estimated to the nearest 5 percent (5%, l0%, 15%, etc.). If some sand, gravel, or fines are present but are estimated to be less than 5 percent, the quantity is listed as a trace. Estimating in multiples of 5 percent results in a small change in the definition of clean sands and gravels (SW, SP, GW, GP), The field classification lists clean coarse-grained soil as having 5 percent or less fines, rather than less than 5 percent, as listed in laboratory classifications.
(5) Apparatus needed for the tests
(i) A small supply of water.
(ii) Pocket knife or small spatula.
(6) Useful auxiliary apparatus:
(i) A jar with a lid or a test tube with stopper.
(ii) Small bottle of diluted hydrochloric acid (one-part 10N HCL to 3 parts of water).
(iii) Hand lens.
(7) Sample size. In order to classify a soil accurately, the sample must be large enough to contain a representative percentage of each particle size. Recommended minimum sample size based on maximum particle size is given in figure 4-15.

**Figure 4-15:** Minimum Sample Size

| Maximum particle size (sieve opening) | Minimum sample size |
|---|---|
| 4.75mm (No. 4 ) | 110g (0.25 lb) |
| 9.5mm (3/8 in) | 220g (0.5 lb) |
| 19mm (3/4 in) | 1 kg (2.2 lb) |
| 38mm (1-1/2 in) | 8kg (18 lb) |
| 75mm (3 in) | 60kg (132 lb) |

C. Field Classification

(1) Soil should be classified using several different field tests rather than a single test. Screening and weighing of samples are not intended. Estimates of percentages of materials by dry weight may be made using visual-manual techniques. With the exception of grain-size and gradation, these tests are performed on that portion of the sample smaller than the No. 40 sieve. When coarse materials that can be separated by hand are removed, the remaining material is roughly that portion passing the No. 40 sieve. Practice with materials of known percentages will be helpful in perfecting these techniques.

(2) Highly organic soil. First determine if a soil classifies as highly organic peat (PT). Peat is composed primarily of vegetable tissue at various stages of decomposition. It has an organic odor, a dark brown to black color, a spongy consistency and a texture ranging from fibrous to formless. A soil classified as peat needs no further classification procedure. The significant items in the description still need to be completed.

**Figure 4-16:** Visual and manual techniques are important tools for classifying soils in the field.


(3) Percentages of sand, gravel, and fines. If the soil is not peat, the next step in classification is to estimate the percentages of sand, gravel, and fines in the sample. The percentage is by dry weight, which differs from volume. A sample that is one-half gravel and one-half fines by volume would be approximately 60 percent gravel and 40 percent fines by weight because the fines have more voids which results in a lower density than the gravel.

(4) Estimates should be to the nearest 5 percent, visual-manual techniques and the accumulative percentages must total 100 percent. If some component is present but constitutes less than 5 percent of the sample, then list it as a trace. A trace does not constitute part of the 100 percent. For example: 65 percent fines, 35 percent sand with a trace of gravel.

(5) Some suggested procedures for estimating percentages of sand, gravel, and fines are:

(i) Jar Method. Thoroughly shake a mixture of soil and water in a straight-sided jar or test tube, then allow the mixture to settle. Sand sizes will fall out first, in 20 to 30 seconds, and successively finer particles will follow. The proportions of sand and fines can then be estimated from their relative volumes.

(ii) Mental Sacking. Mentally visualize the gravel-size particles placed in a sack or other container and the sand and fines in a different sack or sacks, then mentally compare the number of sacks or containers with gravel, sand, and fines.

(iii) Inspection. Spread the sample on a flat surface and examine the particles to determine the approximate grain size. If more than 50 percent of the sample by weight has individual grains that are visible to the naked eye, the material is coarse-grained. If less than 50 percent, it is a fine-grained material.

(6) Aggregated dry particles may appear to be sand-site grains. Saturate the sample and break these aggregates down by rubbing the wetted soil between the thumb and forefinger. Sand-size grains can be detected, as they will feel rough and gritty.

(7) The soil is a **clean gravel** or **clean sand** if the percentage of fines is estimated to be 5% or less. When wetted a clean coarse-grained soil will not leave a stain on your palm. A dirty soil will leave a stain. Practicing with samples that have known percentages of sand, gravel, and fines will help in learning to estimate the percentages.

D. Classifying Fine-grained Soils

(1) A soil is fine-grained if it contains 50 percent or more fines. The soil may be given a borderline classification if the estimated proportion of fines is 45 to 55 percent. Select a representative sample of material and remove the particles larger than the No. 40 sieve. This is about the smallest size particle that can be removed by hand. About a handful of material will be needed. Use this material to perform the dilatancy, toughness, and strength tests. These tests are illustrated in figure 4-22.

(2) **Dilatancy.** Select enough material to mold into a ball about 15mm (0.6 in) in diameter. Add water, if needed, until it has a soft but not sticky consistency. Smooth the soil in the palm of one hand with the blade of a knife or spatula. Shake horizontally, striking the side of the hand against the other several times. Note the appearance of water on the surface. Squeeze the sample and note the disappearance of water. Describe the reaction as:

(i) None. No visible change. This is a characteristic of a CH.

(ii) Slow Reaction. Water appears slowly on the surface during shaking and does not disappear or disappears slowly when squeezed. This is a characteristic of a CL and MH.

(iii) Rapid Reaction. Water appears quickly on the surface during shaking and disappears quickly when squeezed. This is a characteristic of a ML.

(3) **Toughness.** Take the specimen from the dilatancy test, shape it into an elongated pat and roll it on a hard surface or between your hands into a thread about 3mm (1/8 in) in diameter. If it is too wet to roll, spread it out and let it dry. Fold the thread and reroll repeatedly until the thread crumbles at a diameter of 3mm (1/8 in). The soil has then reached its plastic limit. Note the pressure required to roll the thread and the strength of the thread. Circumferential breaks in the thread indicate a CH or CL material. Longitudinal cracks and diagonal breaks indicate a MH material. After the thread crumbles, lump the pieces together and knead until the lump crumbles. Note the toughness of the material during kneading. Describe the toughness of the thread as:

(i) Low. Only slight pressure is required to roll the thread near the plastic limit. The thread and lump are weak and soft. This would be a characteristic of a ML.

(ii) Medium Tough. Medium pressure is required to roll the thread near the plastic limit. The thread and the lump have medium stiffness. Such plastic is a characteristic of a MH.

(iii) High or Tough. Considerable pressure is required to roll the thread near the plastic limit. The thread and lump are very stiff. Such plastic is a characteristic of a CH or CL.

(iv) If a thread cannot be rolled, the soil is non-plastic (figure 4-17).

**Figure 4-17:** Rolling threads to determine a soil's toughness and plasticity.


(4) **Dry strength.** Take enough material to mold into a ball about 12mm (1/2 in) in diameter. Add water, if necessary, and mold the material until it has the consistency of putty. From this material make at least three test specimens about 5mm (0.2 in) in diameter and allow them to air dry. Natural dry lumps of about the same size may be used. The natural lumps will usually have a lower strength than molded material. Do not use natural lumps that contain medium or coarse sand.

Crumble the soil specimens in with your fingers (figure 4-18) and describe their strength as:

- None. The specimen crumbles into powder with the pressure of handling.
- Low. The specimen crumbles into powder with finger pressure.
- Medium. The specimen breaks into pieces or crumbles into powder with considerable finger pressure.
- High. The specimen cannot be broken with finger pressure but can be broken between the thumb and a hard surface.
- Very High. The specimen cannot be broken between the thumb and a hard surface.

**Figure 4-18:** Crumbling a dried soil specimen to determine a soil's dry strength.

(5) Determine if the soil is organic or inorganic. Most soils for engineering purposes are inorganic. An organic soil can be identified by its odor and its dark brown or black color. Classify inorganic soils using the criteria in figure 4-19, Classify organic soils using the criteria in figure 4-20.

**Figure 4-19:** Classification of Inorganic Fine-Grained Soil

| Dilatancy | Toughness | Dry strength | Group name | USCS Symbol |
|---|---|---|---|---|
| Slow to rapid | NP or low | None to low | Silt | ML |
| None to slow | Low to med | Low to medium | Elastic silt | MH |
| None to slow | Medium | Medium to high | Lean clay | CL |
| None | High | High to very high | Fat clay | CH |

**Figure 4-20:** Classification of Organic Fine-Grained Soil

| Dilatancy | Toughness | Dry strength | Group name | USCS Symbol |
|---|---|---|---|---|
| Slow to Rapid | None | None to Low | Organic Silt | OL |
| None to Slow | Low | Low to Medium | Organic Clay | OL |
| None to Slow | None to Low | None to Medium | Organic Silt | OH |
| None | Low to Medium | Medium to High | Organic Clay | OH |

(6) Additional tests that may be useful in classifying fine-grained soils are as follows:

(i) Ribbon Test (figure 4-21). Use a sample that has a moisture content at or slightly below the plastic limit. Form a ribbon by squeezing and working the sample between the thumb and forefinger. A weak ribbon (0 to 2.5 cm long) that breaks easily indicates an ML soil. A hard ribbon which breaks fairly readily indicates an MH soil. A flexible ribbon (2.5 to 5.0 cm long) with medium strength indicates a CL soil. A strong, flexible ribbon (greater than 5 cm long) indicates a CH soil.

**Figure 4-21:** The ribbon test is useful for classifying fine-grained soils.

(ii) Adhesion Test. Saturate a pat of soil and let it dry on your hands. An ML soil will brush off with little effort. A CL or MH soil rubs off with moderate effort. A CH soil requires rewetting to remove it completely.

(iii) Shine Test. Mold a sample into a pat of moist soil that is near the plastic limit. With a knife blade or smooth object such as a fingernail create a smooth surface. A reflective and shiny surface indicates high plasticity. Soils with high plasticity generally have a shiny appearance when moved in the direction of light, whereas soils with low plasticity have a dull appearance. Do not mistake shininess of a soil that contains mica for shininess created by the colloidal content of clays.

(7) Estimating liquid limit. Take a pat of moist soil and mold into a ball about 12mm (1/2 cu in) and add enough water to make the soil soft but not sticky. Rapidly add enough water to cover the outer surface. Break the pat open immediately. A positive reaction has occurred when the water has penetrated through the surface layer. If the water has B penetrated, the LL is low. If the water has penetrated, the LL is high. Visual observation of this phenomenon is much easier in direct sunlight.

(8) Modifiers for sand and gravel. Modifiers can be added to the soil group name of fine-grained soils to indicate the presence of sand and/or gravel.

(i) If the soil has 15 to 25 percent sand and/or gravel, the words “with sand" or "with gravel" is be added. If sand predominates, use "with sand." If gravel predominates, use 'with gravel." Examples are organic silt with sand, OL; and lean clay with gravel, CL. Notice when using visual-manual methods the percentages are estimated to the nearest 5 percent; whereas the division between soils may be stated differently than when using laboratory data for classifying. In the preceding paragraph, instead of saying 15 percent but less than 30 percent, it says "15 to 25 percent." (25 percent is the first percentage less than 30 percent.)

(ii) If 30 percent or more of the soil is sand or gravel, the words “sandy" or “gravelly" is added. Add "sandy" if there is as much or more sand than gravel. Add “gravelly" if there is more gravel than sand. Examples are gravelly fat clay, CH; and sandy organic clay, OH.

**Figure 4-22:** Dilatancy, Strength, and Toughness Tests

**Dilatancy Test:**

(a) (b) (c)

Strick and Shake your hand horizontally as shown in photo (a) until the appearance of water on the surface as shown on photo (b). Record no visible change, slow, or rapid. Note when squeezed, dilatant soil becomes dull as shown on photo (c).

**Dry Strength Test:**

Test the strength of the dry balls or lumps by crushing between the fingers.

**Toughness Test:**

(a) (b) (c)

(a) Method of rolling thread.
(b) Thread of soil above plastic limit.
(c) Crumbling thread as plastic limit is reached.

D. Classifying Coarse-grained Soils

(1) If a soil has more than 50 percent fine visible to the naked eyes, it is classed as a coarse-grained soil.
(2) The soil is a gravel if the percentage of gravel is greater than the percentage of sand.
(3) The soil is a sand if the estimated percentage of sand is equal to or greater than the percentage of gravel.
(4) The soil is well-graded if it has a wide range of particle sizes and a substantial quantity of all intermediate sizes.
(5) A soil is poorly-graded if it has a narrow range of particle sites or if it has a wide range of sizes with some intermediate sizes missing (gap graded).
(6) The soil is a **clean gravel** or **clean sand** if the percentage of fines is estimated to be 5% or less. When wetted a clean coarse-grained soil will not leave a stain on your palm. A dirty soil will leave a stain.

**Figure 4-23:** This Photograph represents a POOR (gap/uniformed)-graded sample has a range of particle sizes missing. This sample has coarse and fine but no medium size particles.


**Figure 4-24:** A poorly graded sample is predominately one size of particle.

**Figure 4-25:** This Photograph represents a WELL graded sample has a wide range of particle sizes that are about equally distributed by weight.

(7) If the soil has 5 percent or less fines, it is a clean coarse-grained soil, and is classified as follows:
- (i) Poorly-graded sand, SP
- (ii) Well-graded sand, SW
- (iii) Poorly-graded gravel, GP
- (iv) Well-graded gravel, GW

(8) If a coarse-grained soil has 15 percent or more fines, then it is a sand with fines or a gravel with fines. Classify the fines as clay or silt as described under fine-grained soils. The soil D will be classified as one of the four coarse-grained soils with fines:
- (i) Silty sand, SM
- (ii) Clayey sand, SC
- (iii) Silty gravel, GM
- (iv) Clayey gravel, GC

(9) If a coarse-grained soil has 10 percent fines, it is given a dual classification. The first symbol is to correspond to the symbol for clean sand or gravel (SW, SP, GW, GP) and the second to the symbol for sand or gravel with fines (SM, SC, GM, GC). The group name will correspond to the group name for the first symbol plus "with silt" or "with clay" to indicate the plasticity of the fines. Examples are well-graded sand with silt, SW-SM; and poorly-graded gravel with clay, GP-GC.

(10) If the coarse-grained soil is predominantly sand or gravel but contains 15 percent or more of the other, then the words "with sand" or "with gravel" is be added to the group name. For example: well-graded sand with gravel, SW-SM; and poorly-graded gravel with sand and silt, GP-GM.

(11) The soil is either a **gravel with fines** or a **sand with fines** if the percentage of fines is estimated to be between 5 and 12%. If a soil sample is dropped in a beaker of water, a cloud remaining after about 30 seconds indicates more than 12% fines (figure 4-26).

**Figure 4-26:** Estimating percentage of fines in a soil sample by dropping the sample in a beaker of water.

(12) The visual-manual classifications are summarized in figure 4-27.

**Figure 4-27:** Field Identification by Visual-Manual Methods

| | | | | | | |
|---|---|---|---|---|---|---|
| Coarse-Grained Soil 55% to 100% Sand and Gravels[3] | Gravel (Percent Gravel > Percent Sand) | 0, Trace or 5% Fines | Well-Graded | | GW | Well-Graded Gravel |
| | | | Poorly Graded | | GP | Poorly Graded Gravel |
| | | 10% Fines | Clayey Fines | Well-Graded | GW-GC | Well-Graded Gravel with Clay |
| | | | | Poorly Graded | GP-GC | Poorly Graded Gravel with Clay |
| | | | Silty Fines | Well-Graded | GW-GM | Well-Graded Gravel with Silt |
| | | | | Poorly Graded | GP-GM | Poorly Graded Gravel with Silt |
| | | 15 to 45% Fines | Clayey Fines | | GC | Clayey Gravel |
| | | | Silty Fines | | GM | Silty Gravel |
| | Sand (Percent Sand > Percent Gravel) | 0, Trace or 5% Fines | Well-Graded | | SW | Well-Graded Sand |
| | | | Poorly Graded | | SP | Poorly Graded Sand |
| | | 10% Fines | Clayey Fines | Well-Graded | SW-SC | Well-Graded Sand with Clay |
| | | | | Poorly Graded | SPS-SC | Poorly Graded Sand with Clay |
| | | | Silty Fines | Well-Graded | SW-SM | Well-Graded Sand with Silt |
| | | | | Poorly Graded | SP-SM | Poorly Graded Sand with Silt |
| | | 15 to 45% Fines | Clayey Fines | | SC | Clayey Sand |
| | | | Silty Fines | | SM | Silty Sand |
| Fine-Grained Soil 50% to 100% Fines[4,5] | Clays & Silts with Low Liquid Limits | Plastic Fines | | | CL | Lean Clay |
| | | Low Plastic Fines with Very Low Liquid Limit | | | CL-ML | Silty Clay |
| | | Non Plastic or Low Plastic Fines | | | ML | Silt |
| | | Plastic Fines with Significant Organics | | | OL | Organic Clay |
| | | Non or Low Plastic Fines with Significant Organics | | | OL | Organic Silt |
| | Clays & Silts with High Liquid Limits | Plastic Fines | | | CH | Fat Clay |
| | | Non Plastic or Low Plastic Fines | | | MH | Elastic Silt |
| | | Plastic Fines with Significant Organics | | | OH | Organic Clay |
| | | Non or Low Plastic Fines with Significant Organics | | | OH | Organic Silt |
| Highly Organic Soil | | Primary Organic Matter that is usually Dark Brown to Black in Color and has an Organic Odor | | | PT | Peat |

Notes
1. Percentages are based on estimating amounts of fines, sand and gravel to the closest 5%.
2. A borderline symbol is two symbols separated by a slash and should be used to indicate that the soil has been identified as having properties that do not distinctly place the soil into a specific group.
   - A borderline symbol may be used with soil having 45 and 55 % fines. One for coarse-grained the other for fine-grair
     For example: GM/ML or CL/SC
   - A borderline symbol may be used when the percentage of sand and gravel area about the same.
     For example: GP/SP, SC/GC, GM/SM
   - A borderline symbol may be used when the soil could be either a silt or a clay.
     For example: CL/ML, CH/MH, SC/SM
   - A borderline symbol may be used when soils within a borrow area shows similarity.
     For example: A sample is identified as CH and the other soils is considered to be either CL or CH. Classify as CH/CL
3. If a coarse-grained sample is predominantly sand or gravel, but contains 15% or more of the other coarse-grained m add the words "with gravel" or "with sand".
   For example: Poorly Graded Gravel with sand, GP; Clayey Sand with gravel, SC.
4. If a fine-grained sample is estimated to have 15 to 25% sand or gravel, or both add the words "with sand" or "with gr whichever is more predominant. If equal percentages of sand and gravel, or both use "with sand".
   For example: Lean Clay with sand ,CL; Silt with gravel, ML.
5. If a fine-grained sample is estimated to have 30% or more sand or gravel, or both add the words "sandy" or "gravelly If equal percentages of sand and gravel, or both use "sandy".
   For example: sandy Lean Clay, CL; gravelly Fat Clay, CH; sandy Silt, ML.

E. Soil Description

(1) When a soil boring is made, a test pit dug, or a natural soil exposure examined, a log should document the observations. The log should contain all pertinent information.
(2) Numbering test holes and samples. Use figure 4-28 for numbering test holes.

**Figure 4-28**: Test Hole Numbering System

| Location | Hole numbers |
|---|---|
| Centerline of dam | 1-99 |
| Borrow area | 101-199 |
| Emergency spillway | 201-299 |
| Centerline of principal spillway | 301-399 |
| Stream channel | 401-499 |
| Relief wells | 501-599 |
| Other | 601-699 |
| Other | 701-700. etc. |

F. Soil Description Checklist

(1) Project and location
  (i) Test hole number and location (figure 4-28)
  (ii) Sample number and depth of sampling
(2) Group name and group symbol (figure 4-27)
(3) Maximum particle size or dimension.
(4) Percentage of cobbles and/or boulders (by volume)
(5) Percentage gravel, sand, fines (by dry weight)
(6) Particle-size range of coarse material (figure 4-29)
  (i) Gravel-fine, coarse
  (ii) Sand-fine, medium, coarse

**Figure 4-29:** USCS Sand and Gravel Sizes

| Term | Particle size | Example |
|---|---|---|
| Gravel | 75 to 4.75mm (3 to 3/16 in) | Orange to pea |
| 1. Course | 75 to 19mm (3 to 3/4 in) | Orange to grape |
| 2. Fine | 19 to 4.75mm (3/4 to 3/16 in) | Grape to pea |
| Sand | 4.75 to 0.075mm (#4 to 200) | Pea to powdered sugar |
| 1. Course | 4.75 to 2.00mm (#4 to #lo) | Pea to rock salt |
| 2. Medium | 2.00 to 0.425mm (#I 0 to #40 | Rock salt to table salt |
| 3. Fine | 0.425 to 0.075mm (#40 to #200) | Table salt to powdered sugar |
| Fines | Less than 0.075mm (#200) | |

(7) Particle shape and angularity (figures 4-30 and 4-31)
  (i) Particle shape - flat, elongated, flat and elongated
  (ii) Angularity - angular, subangular, sub-rounded, rounded
(8) Dilatancy (figure 4-22) None, slow, rapid
(9) Toughness (figure 4-22) Low, medium, high
(10) Dry strength of fines (figure 4-22) None, low, medium, high, very high
(11) Plasticity of fines Non-plastic, low, medium, high
(12) Odor - only if organic or unusual
(13) Color (in moist condition)
(14) Natural moisture content. Dry, moist, wet
(15) Reaction with HCI none, weak, strong
(16) Consistency for fine-grained soils (figure 4-32) Very soft, soft, firm, hard, very hard

(17) Structure Stratified, laminated, fissured, slickensided, lensed, homogeneous
(18) Cementation Weak, moderate, strong
(19) Geologic name, soil series, or local name
(20) Additional comments

**Figure 4-30:** Particle Shapes

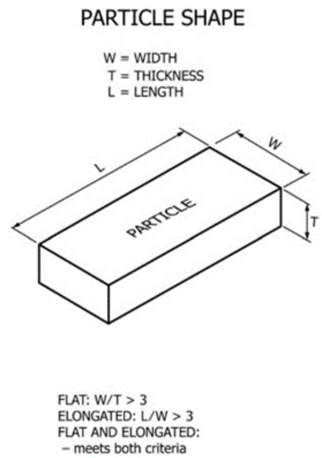



**Figure 4-31:** Particle Angularity


**Figure 4-32:** Field Determination of Consistency of Fine-Grained Soil

| **Consistency** | **Identification procedure** | **Shear strength Kg/cm$^2$ (tons/ft$^2$)** | **Blow count (blows/ft)** |
|---|---|---|---|
| Very soft | Thumb will penetrate soil more than 25mm (1 in) | Less than 0.25 | 0-4 |
| Soft | Thumb will penetrate soil about 25mm (1 in) | 0.25 to 0.5 | 5-8 |
| Firm | Thumb will indent soil about 6mm (1/4 in) | 0.5 to 1.0 | 9-15 |
| Hard | Thumb will not indent soil but soil is readily indented with thumbnail | 1.0 to 2.0 | 15-30 |
| Very hard | Thumbnail will not indent soil | Over 2.0 | Over 30 |

G. Test Holes and Soil Sampling

(1) Principal spillway, channel, and emergency spillway holes that are on the centerline of the dam should be given principal spillway, channel, and emergency spillway numbers rather than centerline of dam numbers (see figure 4-28). Number foundation holes in the area of the base of the dam, but not in the immediate vicinity of the centerline of the dam or appurtenances, as "other."

(2) Samples are numbered by using the test hole number followed by a period and the sample number. Samples from test hole number 104 would be numbered 104.1, 104.2, 104.3 from the top down in elevation.
(3) Maximum particle size. If the largest size particles are sand-size, then describe them as fine, medium, or coarse sand. If the largest particles are gravel-size, list the smallest size sieve that will pass all particles. If the largest particles are cobbles or boulders, list the maximum dimension of the largest cobble or boulder.
(4) Percentages of cobbles and boulders. Cobbles and boulders are defined as:
    (i) Cobbles. Particles that will pass a 12-inch square opening and are retained on a 3-inch sieve.
    (ii) Boulders. Particles that will not pass a 12-inch square opening.
(5) Estimate the percentage of cobbles and boulders by volume. The report should state "estimated by volume," since other percentages are by weight. Example: "estimated 10 percent cobbles (by volume), maximum size 7 inches."
(6) Particle-size range of coarse materials. Figure 4-29 gives particle-size ranges and guidelines for estimating particle sires.
(7) Particle shape. In particle shape, the length, width, and thickness refer to the greatest, intermediate, and least dimension as shown in figure 4-30 and defined as follows:
    (i) Flat: W/T >3
    (ii) Elongated: L/W > 3
    (iii) Flat and elongated particles meet criteria for both flat and elongated.
(8) Particle Angularity. This describes the angularity of the larger size sand particles, gravel, cobbles and boulders as illustrated in figure 4-31.
    (i) Angular. Particles have sharp edges and relatively plane sides with unpolished surfaces.
    (ii) Subangular. Particles are similar to angular but have rounded edges.
    (iii) Subrounded. Particles have nearly plane sides but have well-rounded corners and edges.
    (iv) Rounded. Particles have smoothly curved sides and no edges.
(9) Plasticity of Fines. The plasticity of fines can be estimated with the same test as the toughness test, according to the following criteria.
    (i) Nonplastic. A 3mm thread cannot be rolled at any water content.
    (ii) Low. The thread can barely be rolled, and the lump cannot be formed when drier than the plastic limit.
    (iii) Medium. The thread is easy to roll, and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
    (iv) High. Considerable time rolling and kneading is required to reach the plastic limit. The thread can be rerolled several times after it reaches the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.
(10) Color. Soil colors should be determined from moist samples. If the soil contains layers or patches of varying colors, they should be recorded along with their colors.
(11) Natural water content. Describes the moisture content at the time of sampling.
    (i) Dry. Absence of water, dusty, dry to the touch.
    (ii) Moist. Damp but no visible water.
    (iii) Wet. Visible free water: usually soil is from below the water table.

(12) Reaction with HCL. The presence of calcium carbonate usually correlates to its reaction with hydrochloric (HCL) acid. An HCL solution is comprised of one-part concentrated HCL (10N) to three parts distilled water.
   (i) None. No visible reaction.
   (ii) Weak. Some reaction with bubbles forming slowly.
   (iii) Strong. Violent reaction with bubbles forming immediately.

(13) Consistency. Defined as the relative ease with which a soil can be deformed, either in the undisturbed or molded state. Degrees of consistency for fine-grained soils are described by the terms very soft, soft, firm, hard, and very hard. Figure 4-32 gives a field identification method and estimated strength for soil consistency classes.

(14) Structure. Describes the geologic formation of intact soils.
   (i) Stratified. Alternating layers of varying material or color; note thickness.
   (ii) Laminated. Alternating layers of varying material or color with the layers less than 6mm thick; note thickness.
   (iii) Fissured. Breaks along definite planes of fracture with little resistance.
   (iv) Slickensided. Fracture planes appear polished or glossy, sometimes striated.
   (v) Blocky. Cohesive soil that can be broken down into small angular lumps which resist further breakdown.
   (vi) Lensed. Inclusion of small pockets of different soils such as small lenses of sand scattered through a mass of clay; note thickness.
   (vii) Homogeneous. Same color and appearance throughout.

(15) Cementation. Describes the geologic formation of intact soils.
   (i) Weak. Crumbles or breaks with handling or little finger pressure.
   (ii) Moderate. Crumbles or breaks with considerable finger pressure.
   (iii) Strong. Will not crumble or break with finger pressure.

H. Site Investigation

(1) All construction sites require a geologic investigation. Levels of geologic investigations include reconnaissance, preliminary, and detailed as defined in the NRCS Title 210, National Engineering Handbook, Part 631, “Geology” (210-NEH-631). The scale of investigation depends on design requirements, complexity of the site, and class of the structure. Detailed investigations may not be necessary for small, low-hazard structures such as farm ponds, drop structures, or chutes built in areas of generally homogeneous soil materials. For such structures, the relevant engineering characteristics of site materials may only need to be recognized and evaluated based on experience in the area and appropriately documented. 210-NEH-631, Chapter 2 “Engineering Geology Investigations” (210-NEH-631-2) should be reviewed to help determine when to seek support from a NRCS Geologist.

(2) Sites for terraces, diversions, and waterways should be checked for soil classification, thickness of topsoil, suitability of the subsoil for the intended crops, stoniness of soil, depth to bedrock, permeability, elevation and slope of the water table, and artesian pressure. In open ditch design, erosion resistance and stability of the side slopes depend on soil type and ground water conditions. Construction costs are affected by soil type, stoniness, depth to rock, and depth to water table.

(3) Foundations for dams must be checked for strength, permeability, compressibility, dispersive clays (piping), water table elevation, and depth to bedrock. The erosion resistance of the emergency spillway materials must be estimated. An adequate quantity of suitable borrow soils must also be located.

(4) Waste storage structure sites must be investigated to ensure the integrity of the structure and to avoid pollution of ground water. A site should be checked for soil classification, permeability, depth to bedrock, and elevation of the water table. An adequate quantity of suitable borrow soils must also be located for the embankment and liner system if proposed. All existing and proposed wells and springs that may be affected by the structure should be located on a map.
(5) Preliminary data. First gather and record all useful soil and geologic information that is available from soils and geologic maps. This information should be recorded on a site map. Next, walk over the site, observe the surface geologic features and record them on the site map. Features that may be visible on the surface are, but not limited to springs, seeps, rock outcrops, boulders, slides, sinkholes, and manmade openings. Excavations, road cuts, and ditch and stream banks give opportunities to see soil and geologic profiles.
(6) Field Investigation Program. After this work is completed, take a close look at the information that has been collected and decide what geologic questions have been answered and what questions remain. Then ask yourself what additional information is needed to answer these questions. A few borings with a hand auger may suffice, backhoe pits may be required, or a full boring and testing program with power equipment may be required to supply the needed information. Other investigation methods (e.g., geophysical surveys, CPTs, test pits, etc.) can be included if more appropriate for the project. The investigation must continue until all questions are answered with reasonable assurance. One should not hesitate to ask for assistance from an experienced geologist or engineer.
(7) Geologic observations should continue throughout construction and the geologic records updated as new information is disclosed. Occasionally, new geologic information discovered during construction may require design changes.
(8) Detailed requirements for geological investigations are given in 210-NEH-631-2.
(9) Generally, borings at dam sites should be made on the centerline of the embankment, one on each of the abutments, and on the centerline of the principal spillway. Usually, borings are carried through all compressible and permeable strata to a relatively incompressible, impermeable base, but it is usually not necessary to go deeper than the dam is high. The number of borings required will depend on the length of the dam and the uniformity or variability of the foundation. The minimum number of borings on the principal spillway centerline usually consists of one on the intersection of the centerline of fill and the centerline of the spillway plus one each at the inlet and outlet. Other borings may be needed at a proposed drain location or to identify unusual foundation conditions. Enough borings need to be made at the emergency spillway location to identify the materials to be excavated and to determine the erosion resistance of the soils at the proposed spillway grade and below down to elevation of the outlet.
(10) A recommended numbering system for soil borings is given in the section on soil description (figure 4-28).

(11) Ground Water

(i) Soil borings and test pits should be kept open until the ground water level has stabilized and the ground water surface elevation is measured and recorded. If a pit needs to be backfilled or a drill hole is caving, a perforated pipe should be installed, and the ground water elevation measured in the pipe. It often requires 24 hours for water elevations to stabilize, and it may take weeks to months to stabilize in soils with low permeability. Where artesian pressures are encountered, a piezometer would be required to measure the head.

(ii) It is NRCS policy to backfill and tightly seal all drill holes and pits completely to prevent ground water contamination by surface water or to prevent loss of artesian pressure. Filled holes also remove hazards to livestock and equipment.

(iii) Seasonal variation in the water table should be determined by direct observation and measurement at various times during the year, or it should be estimated. Sometimes in a soil profile, the upper and lower limits of mottling are good indications of the seasonal water table fluctuation. The depth below which the soil is entirely grey usually indicates the elevation of the seasonal low water table. The depth above which the soil is primarily red, orange, or yellow usually indicates the seasonal high-water table.

## 650.0404 Estimation of Soil Performance and Performance Requirements

A. Soil Performance and Engineering Properties

Estimates of soil behavior, such as its shear strength, compressibility, or permeability, can be made from the soil description and classification. These estimates based on index tests are adequate to design most low hazard, low cost structures. The reasonableness of soil test results should be evaluated by comparison with estimated soil parameters.

B. Strength

(1) Shear strength of soil is made up of two components: interparticle friction and cohesion. Shear strength also depends on other factors such as material composition, stress history, temperature, strain, strain rate, and structure.

(2) In its simplest form, friction is the resistance to sliding of one block of non-plastic soil against another. It is similar but not exactly the same action as the resistance of a block of wood sliding across a wooden floor. The greater the weight placed on the sliding block, the greater the force necessary to slide the block. Likewise, as load is applied to the soil or the intergranular pressure is increased, the frictional resistance or frictional shear strength increases. The friction in soil is different from simple friction because of such factors as the irregularity of the planes, interlocking of particles, and size and shape of the soil grains. The shear strength of clean sands and gravels and non-plastic silts can be attributed to friction.

(3) Cohesion is due to the physical and chemical properties of soils which cause the attraction of particles. It is principally related to the plasticity and stickiness of soil fines content. Cohesive strength does not increase with increasing load or intergranular pressure in the soil. Highly plastic clays have mostly cohesive shear strength. Silty and sandy clays can have a combination of both cohesive and frictional strength.

(4) Friction and cohesion increase as soil density increases. Friction increases as particle size increases and is less for a poorly-graded soil as compared to a well-graded soil.

(5) Over a long time period, clay soil that has mostly cohesive strength may creep like a highly viscous liquid. Therefore a retaining wall for clay may tip over after 20 years even though the soil supported itself vertically while the wall was being built.
(6) Silt-size particles are larger than clay particles and generally do not have significant cohesive attraction, as does clay. Silt particles act more like sand, having frictional strength; but being smaller, have less strength than sand.
(7) Most soils are a mixture of sand, gravel, silt, and clay and exhibit characteristics of both cohesive and non-cohesive soil. Soil usually needs more than 35 percent sand and gravel before the coarse materials significantly affect the strength characteristics.
(8) Stable slopes H:V (horizontal to vertical) for moist soils of medium or greater density can be estimated as:
   (i) Rock 1:1 to 1-1/2:1
   (ii) Gravel 1-1/2:1 to 2:1
   (iii) Sand 2:l to 2-1/2:l
   (iv) Lean Clay 2-1/2:1 to 4:l
   (v) Fat Clay 2-1/2:1 to 5:1
   (vi) Silt 3:1 to 4:1
(9) Silts and sands with water seeping from them and soft clays present difficult stability problems and should be analyzed using refined techniques on a case-by-case basis.

C. Permeability

(1) Permeability is the property that allows water to flow through a porous medium. The porous mediums of interest are soil and rock.
(2) Water moves through the voids between soil particles and with few exceptions through joints, fractures, and solution cavities in rock. In some rock, such as certain sandstones, water moves through the rock mass. The symbol for permeability is K, and it is given as a velocity through the total soil mass at a gradient of 1. The most common dimensions used for K are cm/sec and ft/day. (2,830 cm/sec = 1 ft/day.)
(3) Permeability of coarse-grained soils and fine-grained soils must be considered separately. In order to treat a soil as a coarse-grained soil when estimating permeability, it must have less than 5 percent fines, and the fines must be non-plastic. Often, even as little as 5 percent fines will reduce the permeability of a coarse graded soil. The permeability of clean sand and gravel is controlled by the finest 10 to 20 percent of the soil's gradation.
(4) Permeability of concrete sand will range from about $3.5 \times 10^{-3}$ cm/sec to $3.5 \times 10^{-2}$ cm/sec (10 to 100 ft per day). Gravel with a minimum size of 5mm (3/16 inch), will have a permeability of at least 15 cm/sec (40,000 ft per day). A gravel of 9.5mm (3/8 inch) has a permeability around 25 cm/sec (70,000 ft per day).
(5) The permeability of fine-grained soils that have a significant content of fines is dependent on the characteristics of the fines. Generally, soils with 15 percent or more fines are enough for permeability to be controlled by the fines.
(6) Naturally occurring soils usually have a structure comprised of aggregates or peds. Water flows between the aggregates and through channels in the soil created by roots and burrowing animals and insects. A naturally occurring CL may range from being essentially impermeable to having as much permeability as $3.5 \times l0^{-4}$ cm/sec (1 ft per day), depending on the soil structure and number and size of channels.

(7) The permeability of a compacted soil with a significant content of fines depends on the soil gradation and plasticity, the moisture content at compaction, and the degree of compaction. When the soil is compacted dry, the aggregates remain intact, and water flows between them in a manner similar to the way it flows in sand. When the soil is compacted wet, the soil aggregates are malleable and are more easily molded together, creating a solid mass of much lower permeability. The optimum moisture content from the standard Proctor compaction test can be used as the estimated division between dry and wet compaction. Soils that have a significant amount of fines and are compacted 2 percent dry of optimum can be expected to have a permeability of 10 to 1,000 times that of the same soil compacted 2 percent wet of optimum.

(8) With all things being equal, the greater the soil density, the lower its permeability. Moreover, the greatest influence on permeability of soils having a significant content of fines is the moisture content during compaction.

D. Compressibility

(1) Compressibility is the property of a soil that pertains to its susceptibility to decrease in volume when subjected to load. Volume change is primarily the result of changes in pore volume and is time related. Soils with high amounts of organic matter are often highly compressible.

(2) Other commonly used terms related to compressibility are consolidation, settlement, the degree of compaction and collapse.

(3) Consolidation is the gradual reduction in soil volume resulting from an increase in compressive stress. It consists of *initial* consolidation, which is a comparatively sudden reduction in volume resulting from the expulsion and compression of gas; *primary* consolidation, which results principally from a squeezing out of water and is accompanied by a transfer of load from the soil water to the soil solids; and *secondary* consolidation (i.e., creep), resulting from the plastic deformation of the internal structure of the soil mass under a constant load over time.

(4) Settlement is the displacement of a structure due to the compression of the underlying soil.

(5) Compaction is the densification of a soil by means of mechanical manipulation.

(6) Collapsible soils are low density soils that have considerable strength when dry or moist. They lose strength and undergo sudden compression when they are saturated. Some will collapse under their own weight when saturated; others, only when loaded.

(7) Collapsible soil foundations cause problems when they are saturated after being loaded by a structure. Examples are settlement of a dam foundation resulting in cracking of the dam when stored water saturates the foundation; settlement and cracking of an irrigation ditch dike or lining when seepage from the ditch saturates the foundation; or settlement of a building when the foundation soil is saturated with water from a downspout or leaks in water or sewer pipes.

(8) Collapsible soils usually occur in geologic regions such as alluvial fans in arid and semiarid areas, debris flow sediments, and aeolian windblown loess soils. Low density soils above the water table are also suspected of being collapsible. They can be identified by consolidation testing of undisturbed samples. In testing, samples are loaded in a moist condition, then saturated. Collapsible soils will show sudden settlement upon wetting.

E. Bearing Capacity

(1) The allowable bearing capacity of a soil is the maximum average load per unit area of a foundation that will not produce shear failure by rupture of the supporting soil or produce excessive settlement. Bearing capacity depends not only on the soil, but on the footing size, shape, and depth at which maximum settlement the structure can endure without excessive damage. Bearing capacity is most often controlled by reducing the settlement potential.

(2) Bearing capacity of the soil is seldom a problem for light farm structures except on soft, low density or low strength soils. When settlement problems occur, it can be the result of differential settlement which is due to building on two soils with different settlement characteristics, such as undisturbed soil and loose backfill or structure with non-uniform loads.

(3) Relatively heavy, rigid structures, such as concrete manure tanks, must be set on strong uniform foundations to avoid differential settlements that can cause cracking.

(4) Many local building codes contain values of maximum allowable soil pressure for foundation design. They are based on experience with other similar structures. Values are also found in engineering and building construction handbooks. These maximum allowable values are called presumptive bearing pressures. They should be used only for small, low cost structures with low potential consequences.

F. Compaction

(1) One method of soil improvement is by compaction. Compacted soil should be thought of as a different material from the soil in its natural state. Increasing a soil's density by compaction increases soil strength and decreases compressibility. Other things being equal, increasing soil density by compaction will decrease permeability. However, trying to achieve low permeability in a fine-grained soil without moisture control by compacting it to a high density can be a mistake. High densities require soil moisture near or below optimum. Low permeability requires compaction moisture above optimum. The moisture content at the time of compaction is often more important than density in determining low permeability. This is discussed further in the section on permeability.

(2) Some adverse effects of compacting soil are as follows: compaction creates a potential for increased swelling in some fine-grained soils; compacting backfill against a retaining wall increases soil pressures against that wall; compaction usually makes a soil less suitable for plant growth; and too high of compaction increases the potential for soil cracking as a result of differential movement.

(3) When samples of a dry soil are moistened and then compacted with increasing moisture content, the soil will increase in density with increasing moisture until a "maximum density" at an "optimum moisture" is reached. After optimum moisture is reached, increases in moisture will result in a decrease in dry density. The optimum moisture content and maximum density content varies for each soil.

(4) The type of test described above is known as a Proctor density test and is referred to as a moisture-density test. NRCS most often uses the moisture-density test described in ASTM D-698. The test outlines standard test procedures to determine the relationship between water content and dry unit weight in establishing a compaction curve. From this compaction curve the maximum dry density and optimum water (moisture) content can be determined for that soil.

(5) The most important items to control when compacting soil are soil moisture, lift thickness, compaction equipment type and weight, and number of equipment passes per lift.

(6) Proctor density tests are the standard test for the placement moisture content for fine-grained soils. Soils that must flex without cracking should be placed at moisture contents above optimum. At locations where cracking is not important, soil can be placed from about 2 percent below to 2 percent above optimum. Embankments in arid or semi-arid areas that may crack from deep drying need special consideration.

(7) Some guidelines that can be used in the field to estimate correct moisture where testing is not feasible are:

(i) If the embankment becomes firm and hard, the moisture content is probably too low. Equipment should sink into the soil a few inches.

(ii) The soil should have enough moisture so that, when formed into a ball, it will not break if struck sharply with a pencil.

(iii) For most plastic soils, optimum moisture is a little below the plastic limit.

(8) The moisture content of clean gravel and coarse sand have very little effect on their compaction characteristics. Moist fine sand has a very low density when dumped into place. A moisture content near saturation will make compaction easier. Dirty sands act much like fine-grained soils with similar plasticity.

(9) Maximum lift thickness in order to obtain satisfactory compaction depends primarily on the equipment used for compaction, assuming the equipment is suitable for the soil. Figure 4-33 can be used as a guide for lift thickness.

**Figure 4-33:** Maximum Lift Thickness for Compaction Equipment

| Equipment | Maximum loose inches | Lift thickness cm |
|---|---|---|
| Hand compactor (mechanical) | 4 | 10 |
| Track tractor | 6 | 15 |
| Small plate vibrator | 8 | 20 |
| Sheepsfoot roller | 9 | 25 |
| Rubber-tired roller | 12 | 30 |
| Rubber-tired equipment | 18 | 45 |
| Vibratory roller | 24 | 60 |

(10) Rock site should be limited to 2/3 of the loose lift thickness so that the rock will fit within the compacted lift thickness.

(11) For efficient compaction, the equipment must be suitable for the soil type. Clean sands and gravels need vibratory compactors. Small, hand-guided plate vibrators are available for confined areas. Track tractors are also effective in compacting clean coarse-grained soils. They should be operated as fast as practical so they will produce vibrations.

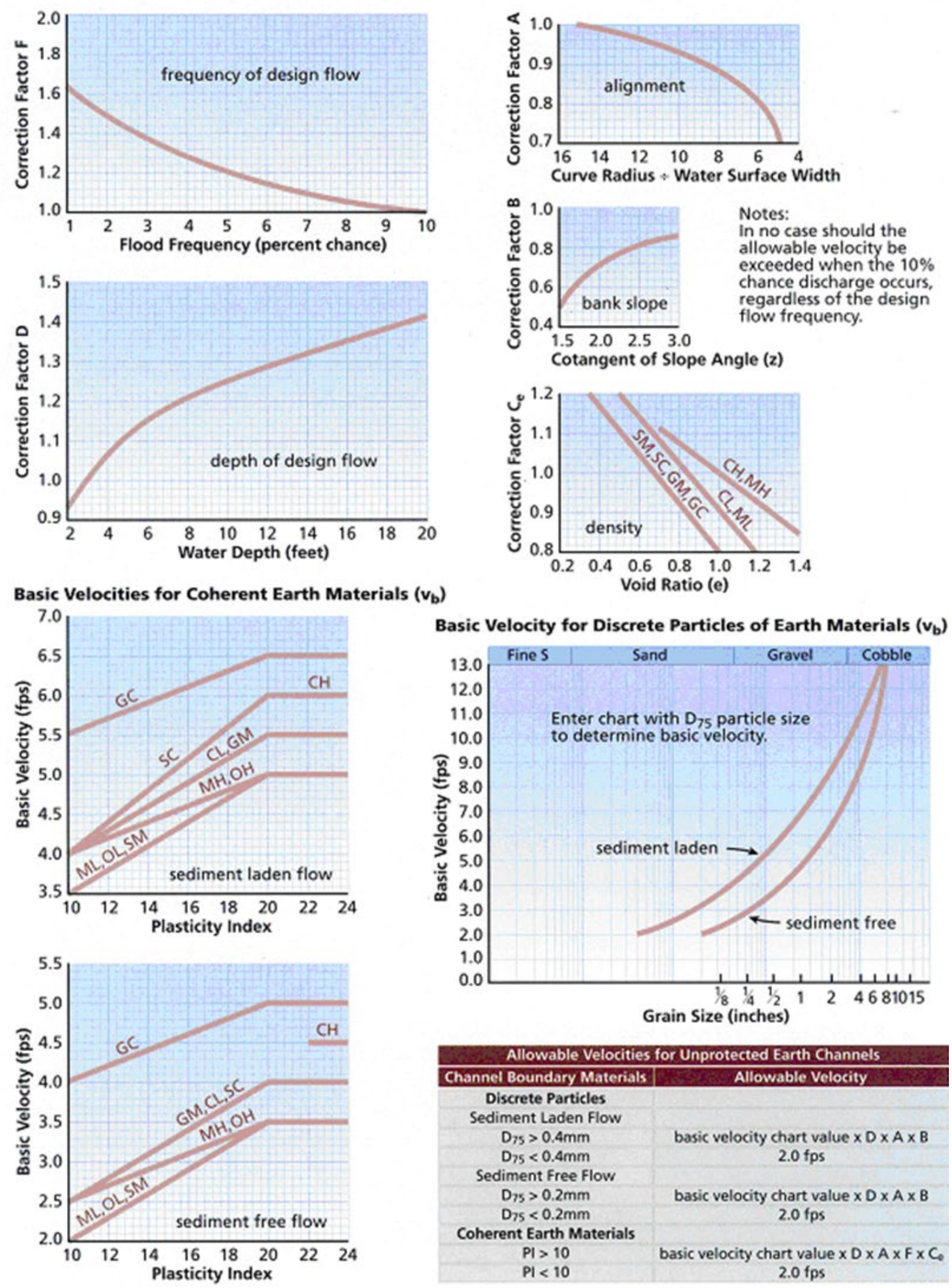
(12) Plastic fine-grained soils require sheepsfoot or tamping rollers. Rubber-tired rollers are good for wet plastic soils, low plastic, and non-plastic fine-grained soil.

G. Erosion Resistance

(1) Resistance of soil to sheet erosion has been evaluated in the development of the Universal Soil Loss Equation. The soil erodibility factor was given the symbol K. The erodibility factor K should not be used to evaluate a soil's erosion resistance to flow in a channel (diversions, waterways, drainage or irrigation channels, floodways, emergency spillways).

(2) Procedures to evaluate a soil's resistance to water flow are given in Technical Release No. 25, Design of Open Channels, and Technical Release No. 52, A Guide for Design and Layout of Earth Emergency Spillways as well as 210-NEH-654-8. Figure 8-1 from 210-NEH-654-8 is reproduced in figure 4-34 for clarity of this section.

**Figure 4-34:** Allowable velocities for unprotected channels (from figure 8-1, 210-NEH-654-8)




**Allowable Velocities for Unprotected Earth Channels**

| Channel Boundary Materials | Allowable Velocity |
|---|---|
| **Discrete Particles** | |
| Sediment Laden Flow | |
| $D_{75} > 0.4$mm | basic velocity chart value x D x A x B |
| $D_{75} < 0.4$mm | 2.0 fps |
| Sediment Free Flow | |
| $D_{75} > 0.2$mm | basic velocity chart value x D x A x B |
| $D_{75} < 0.2$mm | 2.0 fps |
| **Coherent Earth Materials** | |
| PI > 10 | basic velocity chart value x D x A x F x $C_e$ |
| PI < 10 | 2.0 fps |

(3) Soils resist erosion during channel flow in two ways:
(i) Coarse-grained soils, such as sand and gravel, resist movement by the weight of individual particles. Other things being equal, the heavier a particle, the greater its resistance to movement. Platy, elongated, and low-density particles are moved more easily than equidimensional and denser particles. In figure 4-24, a clean coarse-grained soil's resistance to erosion is evaluated on its 0.75 size, the size at which 75 percent by weight of the soil particles are smaller.
(ii) Fine-grained soils resist erosion as a mass held together by the cohesion between particles. TR-25 and 210-NEH-654-8, rates erosion resistance on the basis of a soil's Plasticity Index (PI) and Unified Classification. As shown in figure 4-24, the higher the PI, up to 20, and the more clayey the soil, the greater its erosion resistance. A CL, with PI = 20, has greater erosion resistance than a CL with PI = 15. A CL with PI = 15 has greater erosion resistance than an ML with PI = 15. The erosion resistance of fine-grained soils also increases with increased density. Density is indicated by void ratio as shown in figure 4-24. A decreasing void ratio indicates increasing density.

(4) Coarse-grained soils that have cohesive fines resist erosion by both the weight of the sand or gravel particles and the cohesion of the fines. Coarse-grained soils with noncohesive fines have about the same erosion resistance as clean coarse-grained soil. Some very dense soils, such as dense glacial tills, have a high erosion resistance even though their particle-size and cohesiveness would not indicate so. A qualitative assessment of the relative erosion resistance of soils in the project area can be gained by close observation of the performance of waterways, diversions, streams, and other channels.
(5) Figure 4-35 presents a generalized listing of soils by USDA Soil Texture and USCS group symbol with the more erosion resistant soils listed first and the least resistant last.

**Figure 4-35:** Comparative Erosion Resistance of Soils From Most Resistant to Least Resistant

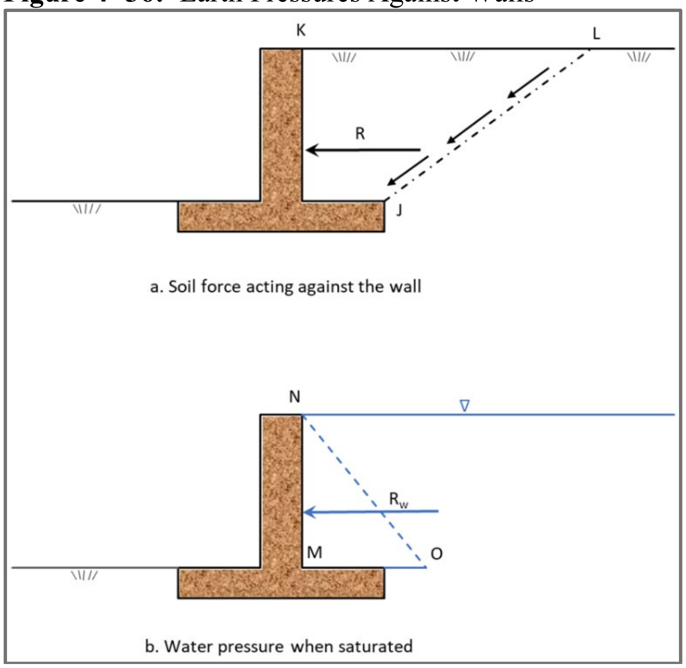
| ***Most Resistant*** | | |
|---|---|---|
| ↓ | Poorly-Graded Gravel, Clayey Gravel | GW, GC |
| | Clay, High Plasticity Clay, Silty Gravel, Well-graded Gravel | CL, CH, GP, GM |
| | Silt | MH, ML |
| | Sand, Loamy Sand, Sandy Clay Loam | SW, SP, SM, SC |
| | Organic Silts | OL |
| ***Least Resistant*** | | |

G. Earth Pressures

(1) It is important that retaining walls be backfilled in the manner that were assumed in the design in accordance to the height and type of materials used. What might seem like a small change in the backfill could double the load on a wall. The following discussion explains some of the factors that determine earth loads on walls.

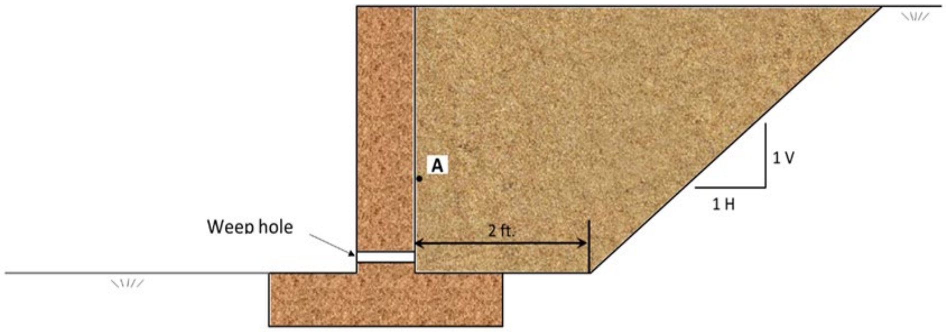
(2) Soil resting against the side of a wall exerts a force on the wall in a nearly horizontal direction. The horizontal loading is referred to as lateral earth pressure. The size of the force depends on the wall height and length, type of soil, its moisture content, the wall movement, and time since backfilling.
(3) The resultant force exerted by the soil backfill against a wall (figure 4-36(a)) results in a tendency of the wedge of soil JKL to slide down the slope JL. The wedge of soil is restrained by both the wall and shear strength along JL. If the soil is saturated, then a water load (figure 4-36(b)) represented by the triangle MNO is added to the soil load. Since water has no shear strength, the total water load must be resisted by the wall. However, there is a reduction in the soil portion of the load under saturated conditions because only the buoyant weight of the soil is acting against the wall.

**Figure 4- 36:** Earth Pressures Against Walls




(4) The combined soil-water load is large, and only specially designed reinforced concrete walls can withstand the pressure. Walls should have a drainage system designed to prevent buildup of water pressures where possible.
(5) Lowest forces on a wall can be obtained by backfilling with uncompacted gravel. Forces from uncompacted sand are only slightly larger. The wall must be able to move slightly or deflect slightly; and the sand or gravel section must be free draining and be of a minimum size or larger, as shown in figure 4-37. Outlets must be provided for water collected in the sand and/or gravel backfill.
(6) The horizontal pressure at any point A is 0.3 to 0.4 times the weight of soil above point A for uncompacted placed sand or gravel.
(7) Compaction of backfill will increase pressures because compaction forces are "locked into" the soil. Also, stresses from backfilling and compaction operations can add additional pressures. Drainage can reduce soil pressure if properly sized for drainage.

**Figure 4-37:** Minimum Sand or Gravel Section




Sand or Gravel backfill

(8) Soil with clayey fines may stand vertically at the time of excavation. This would indicate that clayey soils have very low horizontal pressures against walls. However, when wet, clayey soils will 'creep" over a long time period and gradually build up high soil pressures. Pressures from clay soils will reach from 0.8 to 1.0 times the weight of overlying soil. Pressures from compacted clay backfill also could be high and may be greater than 1, which can lead to problems with stability.

(9) Retaining walls must be designed using professional engineering criteria. More detailed guidance in determining pressures and loads for structural wall design can be found in the Engineering Technical Release, TR-210-74 Lateral Earth Pressures.

H. Cracking

(1) The primary cause of failure of embankments is cracking and piping. A crack in an embankment becomes a flow path for water. Flow through the crack may erode the sides, creating a void through the dam. Cracking may be caused by either or both of two these physical actions.

(i) Cracking may be caused by soil shrinkage that results from drying. The finest-grained materials have the highest cracking potential. Plasticity is an indirect measurement of cracking potential because higher plasticity is associated with finer materials and clay minerals. Fine-grained non-plastic materials will shrink, but shrinkage will be less than in plastic materials.

(ii) Foundation materials subject to shrinkage cracking should be protected from drying while they are exposed. This may be done with sprinkling, covering, or delaying final grading until backfill can start. The fill surface must also be protected from drying between lifts.

(2) Cracking may be caused by embankment movement as a result of either foundation settlement or differential embankment settlement caused by steep changes in the foundation grade. Usually, 2H:1V abutment or trench side slopes are flat enough to prevent excessive differential settlement.

(3) Failure due to cracking also depends on deformability of the soil and its ability to heal itself. Deformability without cracking is a result of both a soil's plasticity and the water content. Compaction with moisture above optimum helps in controlling cracking caused by settlement.

I. Shrink-Swell

(1) Highly plastic soils under low load have a high swell potential when water content increases. Swelling potential may be estimated from the plasticity index of a soil. Soils with a plasticity index greater than 20 usually have a medium to high swell potential; soils with a PI greater than 35 usually have a very high swell potential. Swelling reduces the strength of soils significantly.

(2) Figure 4-38 may be used to estimate volume change based on the shrinkage limit (SL) and plasticity index (PI).

**Figure 4-38:** Volume Change Potential (Adapted from Holtz and Gibbs 2:9)

| Volume Change | Shrinkage Limit | Plasticity Index |
|---|---|---|
| Probably low | 12 or more | 0-15 |
| Probably moderate | 10-12 | 15-30 |
| Probably high | 0-10 | 30 or more |

(3) If possible, soils that have a high shrink-swell potential should be placed in the lower and interior zones of an embankment where moisture change will be restricted and where loads will restrain swelling.

J. Frost Heave

(1) Soil movement caused by frost action is responsible for much damage each winter to roads, streets, buildings, and other improvements. Damage is caused by movement as the soil freezes and by loss of bearing capacity when it thaws.

(2) As soil freezes, water moves from unfrozen soil to the freezing area, building ice lenses causing the soil to expand. Three conditions are necessary for ice lenses to form: (1) freezing soil, (2) a source of water, and (3) a permeable, fine-grained soil that will transmit water by capillary action. Frost heave can be controlled by controlling any one of the three conditions.

(3) The water source is usually the water table. Sometimes the water source is infiltrating surface water. Ideally the water table, a common control measure is drainage to lower the water table. The water table should be lowered to 6 to 8 ft below the ground surface.

(4) A second method of control is to place a soil barrier between the ground water and the freezing soil. This may be a zone of clean gravel that will not transmit water by capillarity or a zone of compacted clay that has such low permeability that it will not transmit enough water during the winter to create a problem. In some cases, ridged foam insulation is used under paving or around foundations to prevent ground freezing. Good drainage under paving will reduce damage during thawing.

K. Dispersive Clay Soils

(1) Dispersive clays are highly erodible soils that have contributed to the failures of numerous earthen embankments. With dispersive clays, individual clay particles are readily suspended and carried downgradient into the soil or downslope by surface runoff. Dispersive soils are high in exchangeable sodium. Dispersive clay erodes as the individual clay particles go into suspension and may do so even in quiet water, whereas considerable velocity is required to erode a clay that is not predominated by sodium. Because these individual dispersive clay particles have much less mass than sand or silt-sized particles, dispersive clay are much more erodible. Terraces, diversions, dikes, and dams constructed with dispersed clay soils will have severe surface erosion and are more suspectable to internally erode through cracks and discontinuities and relatively porous medium (e.g., suffusion) in the embankment or foundation. Rainfall and erosion of exposed surfaces by concentrated flow may be particularly severe in waterways and channels built in dispersed clays.

(2) Non-plastic and low plastic (PI of 4 or less) silts and fine sands are highly erosive by nature and should not be confused with dispersed clays.

(3) Dispersive soils have been identified in many parts of the world and soils with a high preponderance of sodium cations and are not rare in nature. Dispersed clays have long been associated with soils formed in arid and semi-arid climates in areas of alkaline soils but can also be found in humid climates as well.

(4) Dispersed clays can sometimes, but not always, be recognized in cuts where construction activity has removed the surface soil or where the underlying soil has been used in fills. They have a characteristic erosion pattern consisting of narrow, deep, parallel gullies and sometimes results in subsurface erosional features referred to as vertical and horizontal tunnels or jugholes. Suspect soils can be tested for dispersion at a soil mechanics laboratory. Soil samples collected for dispersion testing should be sealed in moisture-proof bags to preserve their natural moisture. The crumb test, a simple field test, may be used to identify some dispersive clays and to select samples for laboratory testing.

(5) Dispersed clays should not be used in construction if an economical alternative borrow source can be found.

(6) If dispersed clays are used, they should be protected from drying and cracking with a covering of erosion resistant soil. Erosion of dispersed clay has been successfully controlled by treatment with hydrated lime. Treatment technique and rate should be developed by testing in a soil mechanics laboratory. Internal erosion can be controlled by a properly designed sand filter.

(7) Detailed identification and requirements of dispersive soils are given in 210-NEH-633-13, “Dispersive Clays”.

L. Behavior characteristics of soils.

(1) Figures 4-39 (a), (b), and (c) contain soil performance interpretations based on their USCS symbol. These are generalizations and should not be used as a basis for design.

**Figure 4-39:** Behavior Characteristics of Soil (a)

| Behavior Characteristics of Soil | | | | | | | |
|---|---|---|---|---|---|---|---|
| **USCS Symbol** | **Typical Names** | **Shear Strength when Compacted and Saturated** | **Compressibility when Compacted and Saturated** | **Workability as Construction Material** | **Permeability** | | |
| | | | | | **When Compacted** | **K cm per second** | **K feet per day** |
| GW | Well-graded gravels, gravel-sand mixtures, little or no fines | Excellent | Negligible | Excellent | Pervious | K > $10^{-2}$ | K > 30 |
| GP | Poorly graded gravels, gravel-sand mixtures, little or no fines | Good | Negligible | Good | Very Pervious | K > $10^{-2}$ | K > 30 |
| GM | Silty gravels, gravel-sand-silt mixtures | Good to Fair | Negligible | Good | Semi-Pervious to Impervious | K > $10^{-3}$ to $10^{-6}$ | K = 3 to $3x10^{-3}$ |
| GC | Clayey gravels, gravel-sand-clay mixtures | Good | Very Low | Good | Impervious | K > $10^{-6}$ to $10^{-8}$ | K = $3x10^{-3}$ to $3x10^{-5}$ |
| SW | Well-graded sands, gravelly sands, little or no fines | Excellent | Negligible | Excellent | Pervious | K > $10^{-3}$ | K > 3 |
| SP | Poorly graded sands, gravelly sands, little or no fines | Good | Very Low | Fair | Pervious | K > $10^{-3}$ | K > 3 |
| SM | Silty sands, sand-silt mixtures | Good to Fair | Low | Fair | Semi-Pervious to Impervious | K > $10^{-3}$ to $10^{-6}$ | K = 3 to $3x10^{-3}$ |
| SC | Clayey sands, sand-clay mixtures | Good to Fair | Low | Good | Impervious | K > $10^{-6}$ to $10^{-8}$ | K = $3x10^{-3}$ to $3x10^{-5}$ |
| ML | Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity | Fair | Medium to High | Fair | Semi-Pervious to Impervious | K > $10^{-3}$ to $10^{-6}$ | K = 3 to $3x10^{-3}$ |
| CL | Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays | Fair | Medium | Good to Fair | Impervious | K > $10^{-6}$ to $10^{-8}$ | K = $3x10^{-3}$ to $3x10^{-5}$ |
| OL | Organic silts and organic silty clays of low plasticity | Poor | Medium | Fair | Semi-Pervious to Impervious | K > $10^{-4}$ to $10^{-6}$ | K = $3x10^{-1}$ to $3x10^{-3}$ |

**Figure 4-39:** Behavior Characteristics of Soil (a) – continued

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| MH | Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts | Fair to Poor | High | Poor | Semi-Pervious to Impervious | $K > 10^{-4}$ to $10^{-6}$ | $K = 3x10^{-1}$ to $3x10^{-3}$ |
| CH | Inorganic clays of high plasticity, fat clays | Poor | High to Very High | Poor | Impervious | $K > 10^{-6}$ to $10^{-8}$ | $K = 3x10^{-3}$ to $3x10^{-5}$ |
| OH | Organic clays of medium to high plasticity, organic silts | Poor | High | Poor | Impervious | $K > 10^{-6}$ to $10^{-8}$ | $K = 3x10^{-3}$ to $3x10^{-5}$ |
| PT | Peat and other highly organic soils | Not suitable for construction | | | | | |

**Figure 4-39:** Behavior Characteristics of Soil (b)

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| **Behavior Characteristics of Soil for Embankments** | | | | | | | | |
| **USCS Symbol** | **Compaction Characteristics** | **Standard Proctor Unit Density pcf** | **Type of Roller Preferred** | **General Characteristics** | | **Resistance to Piping** | **Plastic Deformation Under Load Without Sharing** | **General Description and Use** |
| | | | | **Permeability** | **Compressibility** | | | |
| GW | Good | 115 - 135 | Crawler tractor or steel wheeled & vibratory | High | Very Slight | Good | None | Very stable, pervious shells of dikes and dams |
| GP | Good | 100 - 130 | Crawler tractor or steel wheeled & vibratory | High | Very Slight | Good | None | Reasonable stable, pervious shells of dikes and dams |
| GM | Good with close control | 95 - 130 | Rubber-tired or sheepsfoot | Medium | Slight | Poor | Poor | Reasonable stable, not well suited to shells but may be used for impervious cores or blankets |
| GC | Good | 95 - 130 | Sheepsfoot or rubber-tired | Low | Slight | Good | Fair | Fairly stable, may be used for impervious core |
| SW | Good | 110 - 135 | Crawler tractor & vibratory or steel wheeled | High | Very Slight | Fair | None | Very stable, pervious sections, slope protection required |
| SP | Good | 100 - 125 | Crawler tractor & vibratory or steel wheeled | High | Very Slight | Fair to Poor | None | Reasonably stable, may be used in dike with flat slopes |
| SM | Good with close control | 80 - 135 | Rubber-tired or sheepsfoot | Medium | Slight | Poor to Very Poor | Poor | Fairly stable, not well suited for shells, but may be used for impervious cores or dikes |
| SC | Good | 85 - 130 | Sheepsfoot or padfoot | Low | Slight | Good | Fair | Fairly stable, use for impervious core for flood control structures |
| ML | Good to Poor with close control essential | 75 - 110 | Padfoot | Medium | Medium | Poor to Very Poor | Very Poor | Poor stability, may be used for embankments with proper control (varies with water content) |

**Figure 4-39:** Behavior Characteristics of Soil (b) - continued

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| CL | Fair to Good | 90 - 120 | Sheepsfoot or padfoot | Medium | Medium | Good to Fair | Good to Poor | Stable, impervious cores and blankets |
| OL | Fair to Poor | 80 -100 | Padfoot | Medium to Low | Medium to High | Good to Poor | Fair | Not suitable for embankments |
| MH | Poor to Very Poor | 75 - 110 | Sheepsfoot | Medium to Low | Very High | Good to Poor | Good | Poor stability, core of hydraulic fill dam, not desirable in rolled fill construction |
| CH | Fair to Poor | 80 - 110 | Sheepsfoot | Low | High | Excellent | Excellent | Fair stability with flat slopes, thin cores, blanket and dike sections |
| OH | Poor to Very Poor | 65 - 100 | Padfoot | Medium to Low | Very High | Good to Poor | Good | Not suitable for embankments |
| PT | Do not use for embankment construction | | | | | | | |

**Figure 4-39:** Behavior Characteristics of Soil (c)

**Behavior Characteristics of Soil for Channels and Foundations**

| USCS Symbol | CHANNELS | | FOUNDATIONS | | | | |
|---|---|---|---|---|---|---|---|
| | Long duration under constant flows | | Foundation soils are influenced to a great degree by their geologic origin and their mode of deposition. Judgment and testing must be used in addition to these generalizations. | | | | |
| | Relative Desirability | | Relative Desirability | | | | |
| | Erosion Resistance | Compacted Earth Lining | Sites where Seepage Is Important | Sites where Seepage Is Not Important | Bearing Value Under Load | Requirements for Seepage Control: Permanent Reservoir | Requirements for Seepage Control: Floodwater Reservoir |
| GW | 1 | N/A | N/A | 1 | Good | Positive cutoff or blanket | Control only within volume acceptable plus pressure relief if required |
| GP | 2 | N/A | N/A | 3 | Good | Positive cutoff or blanket | Control only within volume acceptable plus pressure relief if required |
| GM | 4 | 4 | 1 | 4 | Good | Seepage control based on site analysis | Must be filter compatible |
| GC | 3 | 1 | 2 | 6 | Good | Seepage control based on site analysis | Must be filter compatible |
| SW | 6 | N/A | N/A | 2 | Good | Positive cutoff or upstream blanket & toe drains or wells | Control only within volume acceptable plus pressure relief if required |
| SP | 7 (if gravelly) | N/A | N/A | 5 | Good to poor depending on density | Positive cutoff or upstream blanket & toe drains or wells | Control only within volume acceptable plus pressure relief if required |

**Figure 4-39:** Behavior Characteristics of Soil (c) - continued

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| SM | 8 (if gravelly) | 5 (erosion critical) | 3 | 7 | Good to poor depending on density | Upstream blanket & toe drains or wells | Sufficient control to prevent dangerous seepage from piping |
| SC | 5 | 2 | 4 | 8 | Good to poor | Seepage control based on site analysis | Must be filter compatible |
| ML | Critical | 6 (erosion critical) | 6 (if saturated or pre-wetted) | 9 | Very poor, may be susceptible to liquefaction | Positive cutoff or upstream blanket & toe drains or wells | Sufficient control to prevent dangerous seepage from piping |
| CL | 9 | 3 | 5 | 10 | Good to poor | Seepage control based on site analysis | Seepage control based on site analysis |
| OL | Critical | 7 (erosion critical) | 7 | 11 | Fair to poor, may have excessive settlement | N/A | N/A |
| MH | N/A | N/A | 8 | 12 | Poor | Seepage control based on site analysis | Seepage control based on site analysis |
| CH | 10 | 8 (volume change critical) | 9 | 13 | Fair to poor | Seepage control based on site analysis | Seepage control based on site analysis |
| OH | N/A | N/A | 10 | 14 | Very poor | N/A | N/A |
| PT | N/A | N/A | Remove from foundation | | | | |

## 650.0405 Preliminary Embankment Design for Earth Dams

A. Scope

This section presents a method for selecting a structurally sound preliminary embankment design by means of a qualitative evaluation of soil characteristics, site topography, and foundation conditions. It presents an orderly approach to the concurrent evaluation of the many factors, that influence embankment stability. The validity of conclusions reached by this method depends on the competence of the site investigation, the adequacy of the testing program, and the soundness of the designer's judgment. Design practices based on quantitative assessment’s, such as analyses of slope stability, seepage, and settlement, are not discussed herein; however, they are necessary to the design of large dams for preliminary design. In the case of small earth dams that have low hazard potential and whose magnitude or importance do not justify extensive investigation or testing programs, the conclusions reached by this method may constitute final design. For these small dams, the method presented herein should be regarded as the minimum analysis required for adequate design.

B. General

An earth dam embankment must be designed to be stable for any force condition or combination of force conditions that may reasonably be expected to develop during the life of the structure. Other than overtopping caused by inadequate spillway capacity, the three most critical conditions that may cause failure of the embankment are as follows:

(i) Differential settlement within the embankment or its foundation caused by variation of materials, variation of height of embankment, or compression of foundation strata. This may cause the formation of cracks through the embankment that are roughly parallel to the abutments. These cracks could concentrate seepage through the dam and lead to failure by internal erosion.

(ii) The development of seepage through the embankment and foundation. This condition may cause piping within the embankment or foundation or both.

(iii) The development of shearing stresses within the embankment and foundation due to the weight of the fill and lowering of strength upon wetting. If the magnitude of the shearing stresses exceeds the strength of the materials, sliding of embankment or foundation may occur, resulting in the displacement of large portions of the embankment.

C. Embankment Stability

(1) The stability of an embankment depends on several factors. Each of these must be considered in the development of the embankment design. The finished design must include all design features necessary to overcome any detrimental influence which may be exerted on the foundation and embankment. The factors are as follows:

(i) Physical characteristics of the fill materials. The classification of soils for engineering by USCS symbol are summarized in figure 4-40. Relative characteristics of compacted fill materials are presented in figure 4-41.

(ii) Configuration of the site. The total settlement of any given section of the embankment will differ from that of the section immediately adjacent to it, since, the height of the embankment varies through the dam length. The width of the dam and slope of the abutments profoundly influence the degree of settlement. The narrower the dam and the steeper the abutments, the more critical is the problem of differential settlement. Differential settlement can also occur in an embankment adjacent to structures (e.g., concrete weir spillways, etc.) or on top of structures (e.g., conduits)

**Figure 4-40:** Working Classifications of Soils for Use as Fill Materials for Rolled Earth Dams

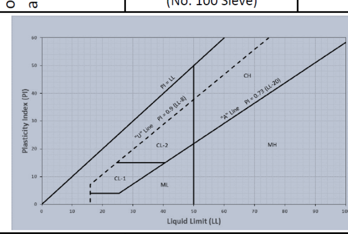
<table>
<tr><th colspan="3">Major Divisions</th><th>USCS Symbol</th><th>Typical Name</th><th colspan="3">Laboratory Classification Criteria</th><th>Behavior Group</th></tr>
<tr><td rowspan="9">Coarse-Grained Soils<br>$d_{50}$ > No. 200 Sieve (0.075 mm)</td><td rowspan="9">Sands and Gravels</td><td rowspan="4">Clean</td><td>GW</td><td>Well-graded gravels; sandy gravels</td><td rowspan="9">Determine percentages of gravel and sand. Depending on percentage of fines (No. 200), coarse-grained soils are classified as follows:<br>- Less than 5% - GW, GP, SW, SP<br>- More than 12% - GC, GM ,SC, SM<br>- 5% to 12% - Classification requires dual symbols</td><td>$C_U \geq 4$ and $1 \leq C_c \leq 3$</td><td rowspan="4">$C_u = \frac{d_{60}}{d_{10}}$<br>$C_c = \frac{{d_{30}}^2}{d_{10} x d_{60}}$</td><td rowspan="4">I</td></tr>
<tr><td>GP</td><td>Poorly graded gravels; sandy gravels</td><td>Not meeting all requirements for GW</td></tr>
<tr><td>SW</td><td>Well-graded sands; gravelly sands</td><td>$C_U \geq 6$ and $1 \leq C_c \leq 3$</td></tr>
<tr><td>SP</td><td>Poorly graded sands; gravelly sands</td><td>Not meeting all requirements for SW</td></tr>
<tr><td rowspan="2">With Clay Fines</td><td>GC</td><td>Clayey gravels; sandy, clayey gravels</td><td colspan="2" rowspan="2">Atterberg Limits above the "A" Line</td><td rowspan="2">II</td></tr>
<tr><td>SC</td><td>Clayey sands; gravelly, clayey sands</td></tr>
<tr><td rowspan="3">With Silt Fines</td><td>GM</td><td>Silty gravels; sandy, silty gravels</td><td colspan="2">Atterberg Limits below the "A" Line</td><td rowspan="2">III</td></tr>
<tr><td>SM-1</td><td>Coarse silty sands</td><td>$d_{50} \geq 0.15$ mm (No. 100 Sieve)</td><td rowspan="2">Atterberg Limits below the "A" Line</td></tr>
<tr><td>SM-2</td><td>Fine silty sands</td><td>$d_{50} < 0.15$ mm (No. 100 Sieve)</td><td rowspan="2">IV</td></tr>
<tr><td rowspan="7">Fine-Grained Soils<br>$d_{50} \leq$ No. 200 Sieve (0.075 mm)</td><td rowspan="7">Silts and Clays</td><td rowspan="3">LL < 50</td><td>ML</td><td>Inorganic silts ; rock flour; ash; very fine silty sands; sandy, clayey silts</td><td colspan="3" rowspan="5">
</td></tr>
<tr><td>CL-1</td><td>Inorganic clays of low plasticity; silty clays; sandy, gravelly clays (PI < 15)</td><td>V</td></tr>
<tr><td>CL-2</td><td>Inorganic clays of medium plasticity; silty, sandy or gravelly clays (PI ≥ 15)</td><td>VI</td></tr>
<tr><td rowspan="2">LL ≥ 50</td><td>CH</td><td>Inorganic clays of high plasticity; fat clay</td><td>VII</td></tr>
<tr><td>MH</td><td>Inorganic silts with high liquid limits; micaceous or diatomaceous silt, elastic silts</td><td>VIII</td></tr>
<tr><td rowspan="2">Organic</td><td>OL</td><td>Organic silts and clays of low plasticity</td><td>Same as ML</td><td colspan="2" rowspan="2">$\frac{LL\ (oven\ dry\ soil)}{LL\ (air\ dry\ soil)} < 0.75$</td><td rowspan="2">IX</td></tr>
<tr><td>OH</td><td>Organic silts and clays of high plasticity</td><td>Same as MH</td></tr>
</table>

**Figure 4-41:** Characteristics of Compacted Fill Materials

**Characteristics of Compacted Fill Materials**

| Behavior Group | USCS Symbols (Table 4-17) | Relative Resistance to Failure (1) Greatest to (6) Least | | | Relative Characteristics | | | Best Equipment to use |
|---|---|---|---|---|---|---|---|---|
| | | Shearing | Piping | Cracking | Permeability | Compressibility | Compaction | |
| I | GW, GP, SW, SP | 1 | N/A | N/A | High | Very Slight | Good | Crawler tractor; steel-wheeled roller |
| II | GC, SC | 3 | 3 | 4 | Low | Slight | Fair | Sheepsfoot roller; rubber-tired roller |
| III | GM, SM-1[1] | 2 | 5 | 3 | Medium | Slight | Good | Rubber-tired roller; sheepsfoot roller |
| IV | SM-2[2], ML | 3 | 6 | 6 | Medium | Slight to Medium | Good to Poor | Padfoot roller (close control essential) |
| V | CL-1 (PI < 15) | 4 | 4 | 5 | Low | Medium | Good to Fair | Padfoot or Sheepsfoot (close control essential) |
| VI | CL-2 (PI ≥ 15) | 5 | 2 | 2 | Low | Medium to High | Good to Fair | Sheepsfoot roller; padfoot roller |
| VII | CH | 6 | 1 | 1 | Low | High | Fair to Poor | Sheepsfoot roller |
| VIII | MH | 6 | Variable | Variable | Medium to Low | Very High | Poor to Very Poor | Sheepsfoot roller |
| IX | OL, OH | 6 | Variable | Variable | Medium to Low | Very High | Very Poor | Not suitable for embankments |

[1] SM-1 $d_{50} \geq 0.15$mm
[2] SM-2 $d_{50} < 0.15$mm

(2) Foundation materials.

(i) The character and distribution of foundation materials must be considered based on of shear strength, compressibility, and permeability. In some cases, the shear strength of the foundation may govern the choice of embankment slopes. Permeability and stratification of the foundation may govern the choice of a zoning plan and drainage features. In many cases, foundations contain compressible soils which settle under the weight of the embankment even though their shear strength is satisfactory. When such settlement occurs in the foundation, the embankment settles. This settlement is rarely uniform over the basal area of the embankment. Therefore, fill materials used on such sites must be sufficiently plastic to deform without cracking.

(ii) Relative characteristics of the various soil groups pertinent to foundation evaluation are presented in figure 4-32. This table is a general solution to foundation problems and may not address all foundation conditions. A foundation composed of homogeneous soil is simple to evaluate; however, such a condition is rarely found in natural soil deposits. The more usual condition is a stratified deposit composed of layers of several soil types. The number of possible combinations of materials and arrangements of materials within a deposit is so great as to render a general solution impossible. The geologic history of the site, the extent of stratification, and the order in which materials occur within the stratification are of great significance in determining the suitability of the foundation. A complex stratified foundation containing plastic or compressible soil should be investigated by an experienced engineer or geologist.

**Figure 4-42:** Characteristics of Foundation Materials

**Characteristics of Foundation Materials**

| USCS Symbol | Shear Strength | Sensitivity to Shock | Compressibility | Permeability | Seepage Control Requirements |
|---|---|---|---|---|---|
| GW | High | None | Very Slight | High | Positive Cutoff |
| GP | High | None | Very Slight | High | Positive Cutoff |
| GM | High | None | Very Slight | Medium to Low | Toe Trench or None |
| GC | High | None | Slight | Low | None |
| SW | High | None | Very Slight | High | Upstream Blanket & Toe Drain; Upstream Blanket & Cutoff; Cutoff, Upstream Blanket & Toe Drain; Cutoff |
| SP | Usually High | High for Loose Fine Sand | Very Slight | High | |
| SM | Usually High | High for Loose Fine Sand | Very Slight | Medium | |
| SC | High | None | Slight | Low | None |
| ML | Medium | High for Loose Silts | Medium | Medium | Toe Trench |
| CL | Medium | None | Medium | Medium to Low | None |
| OL | Low | None | High | Low | None |
| MH | Low | High for Loose Silts | Varies considerably | Low | None |
| CH | Medium to Low | None | Varies considerably | Low | None |
| OH | Low | None | Very High | Low | None |
| PT | Very Low | None | Very High | Very High | Remove from Foundation |

(3) Embankment zoning.

(i) The stability of embankment slopes and the seepage pattern are greatly influenced by the zoning of the embankment. The position of the saturation line within a homogeneous earth dam is theoretically independent of the type of soil used in the embankment. Unless drainage is provided, this saturation line intersects the downstream slope at a point above the toe. In many cases, the quantity of seepage is so slight that it does not affect the slope's stability; however, in some cases the saturation of the toe will cause sloughing or serious reduction of shear strength in the downstream section of the embankment. It is desirable to include a toe drain in the design of most homogeneous dams.

(ii) Dams founded on pervious foundations or constructed of materials that exhibit susceptibility to piping and cracking should always be protected by adequate toe drainage. Toe drains and seepage cutoff designs may be constructed of sand, gravel, or rock, depending on the nature of the fill material. Whenever a gravel or rock toe drain is installed, a graded filter should be placed between the fill and the drain. Filter design criteria is presented in 210-NEH-633-26, “Gradation Design of Sand and Gravel Filters”.

(iii) Where suitable materials are available it is desirable to design a zoned embankment in which the upstream and downstream thirds of the embankment are composed of coarse-grained pervious soils. The middle third or core section of the fill should be composed of more impervious fine-grained soils that would be protected and serve to prevent large amounts of seepage water and to impound the reservoir. The downstream pervious shell serves to lower the line of saturation that may get through or around the embankment core section in the same manner as does a toe drain and increases the total shear resistance of the embankment. The upstream pervious shell protects the embankment from failure due to stresses caused by rapid drawdown of the reservoir. Also, this type of zoning usually results in the placement of the more shear-resistant materials in the outer embankment zones.

(iv) Fill placement. The type of compaction control required to produce a fill of adequate density depends on the nature of the soil to be used in the embankment. Soils that indicate susceptibility to piping and cracking must be placed at a carefully controlled moisture content and compacted to a density determined by laboratory compaction tests. The most effective types of compaction equipment for various soil types are, indicated in figure 4-41.

D. Required Design Data

In order to develop a sound embankment design, the designer must have the following information:

(i) Topographic survey of the site (from preliminary investigation report)

(ii) Detailed report of the foundation investigation (usually included in geological report).

(iii) Report of available quantities of fill materials, by type (usually included in the geological report).

(iv) Report of the characteristics of fill materials and foundation materials (materials testing laboratory report).

E. Selection of Embankment Type

The tables and figures presented can be as aids to evaluation of the factors discussed herein. Figure 4-43 illustrates 10 conventional embankment types, each of which is best applicable to a specific combination of site, foundation, and soil conditions. Figure 4-44 presents a method by which an experienced engineer may evaluate site conditions to determine embankment design type by using the foundation type (figure 4-45) and behavior groups (figure 4-40). This evaluation combined with other information gathered will enable the designer to select the proper embankment design type.

**Figure 4-43:** Embankment Types

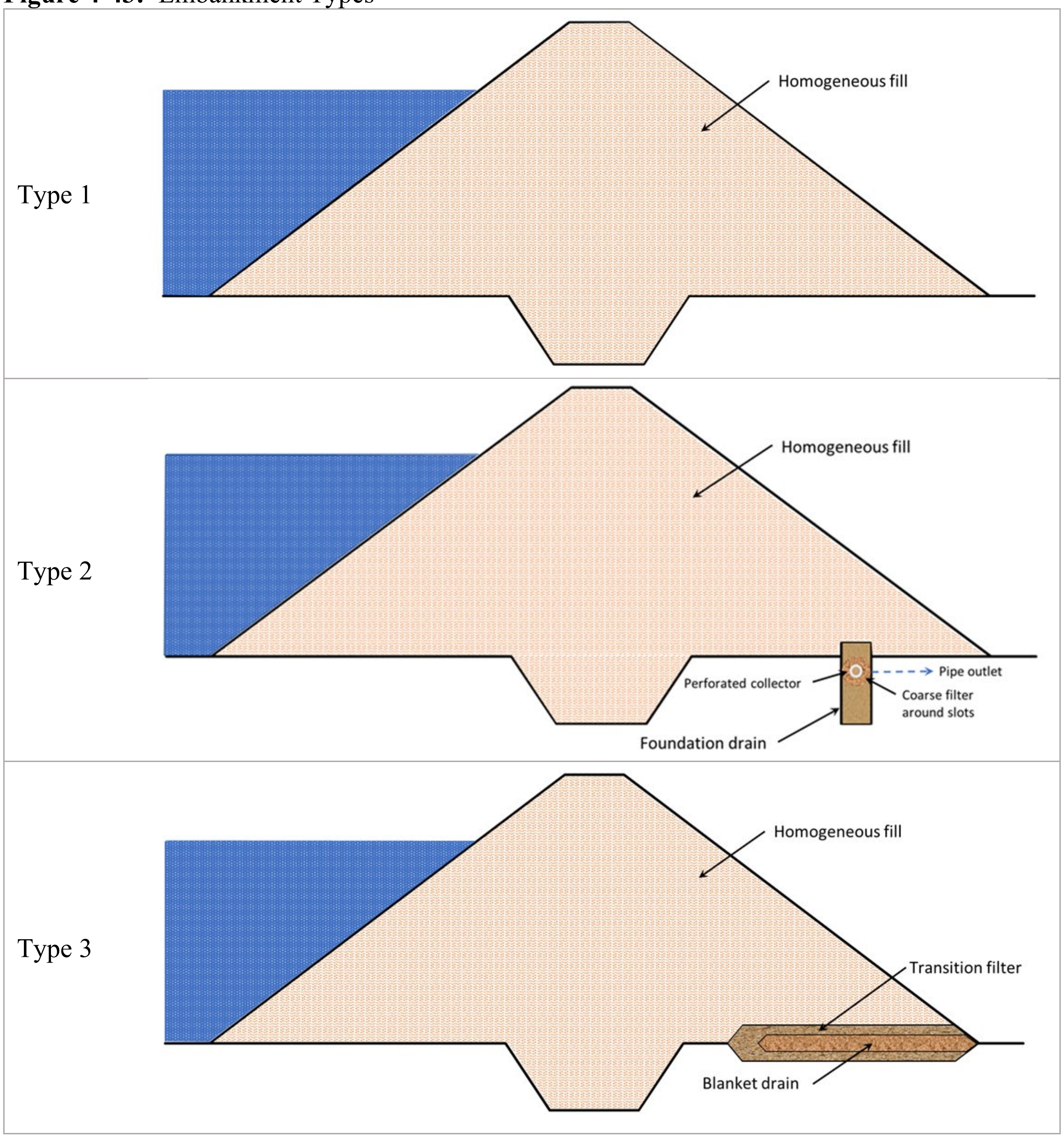


**Figure 4-43:** Embankment Types - continued

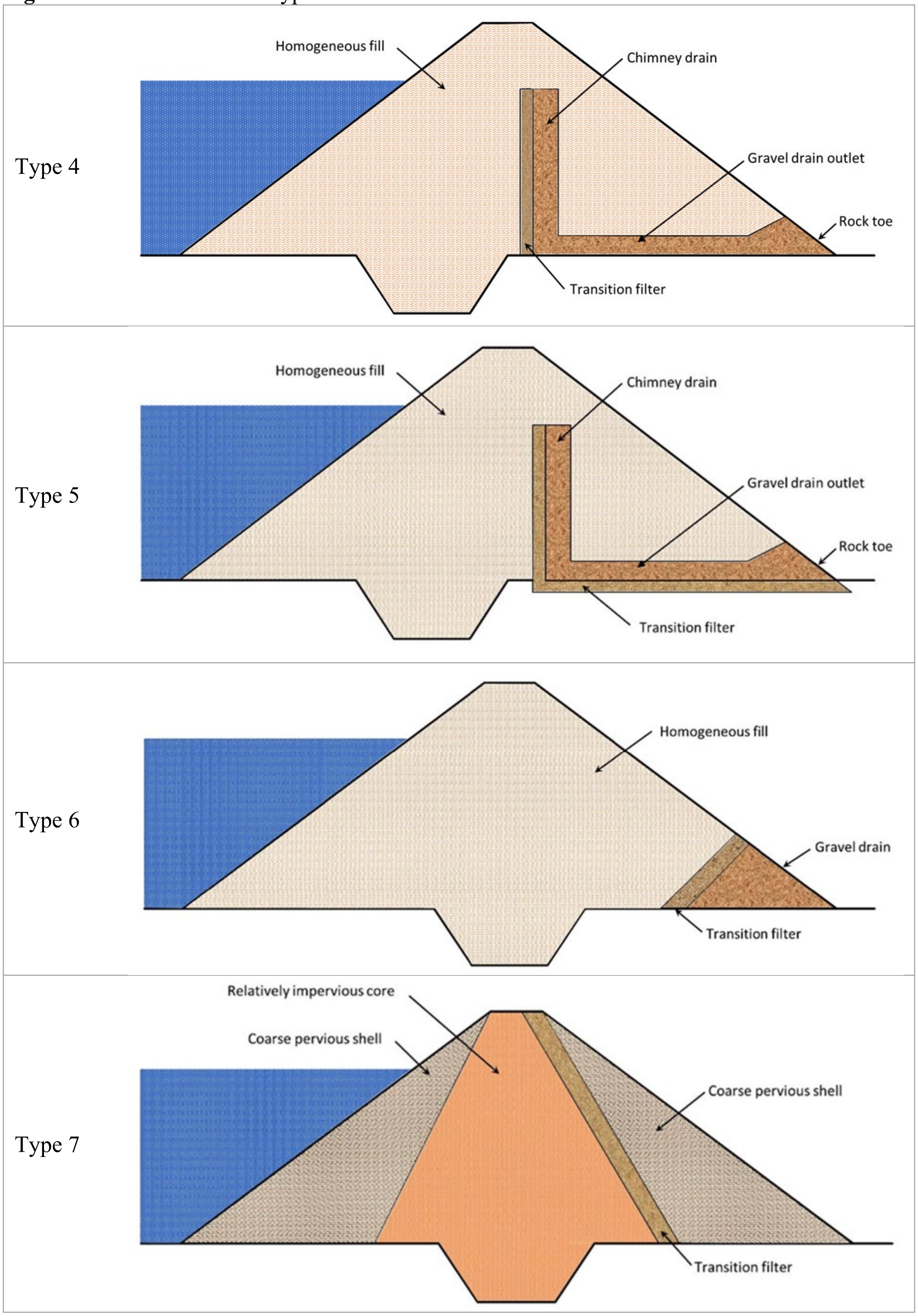


**Figure 4-43:** Embankment Types - continued

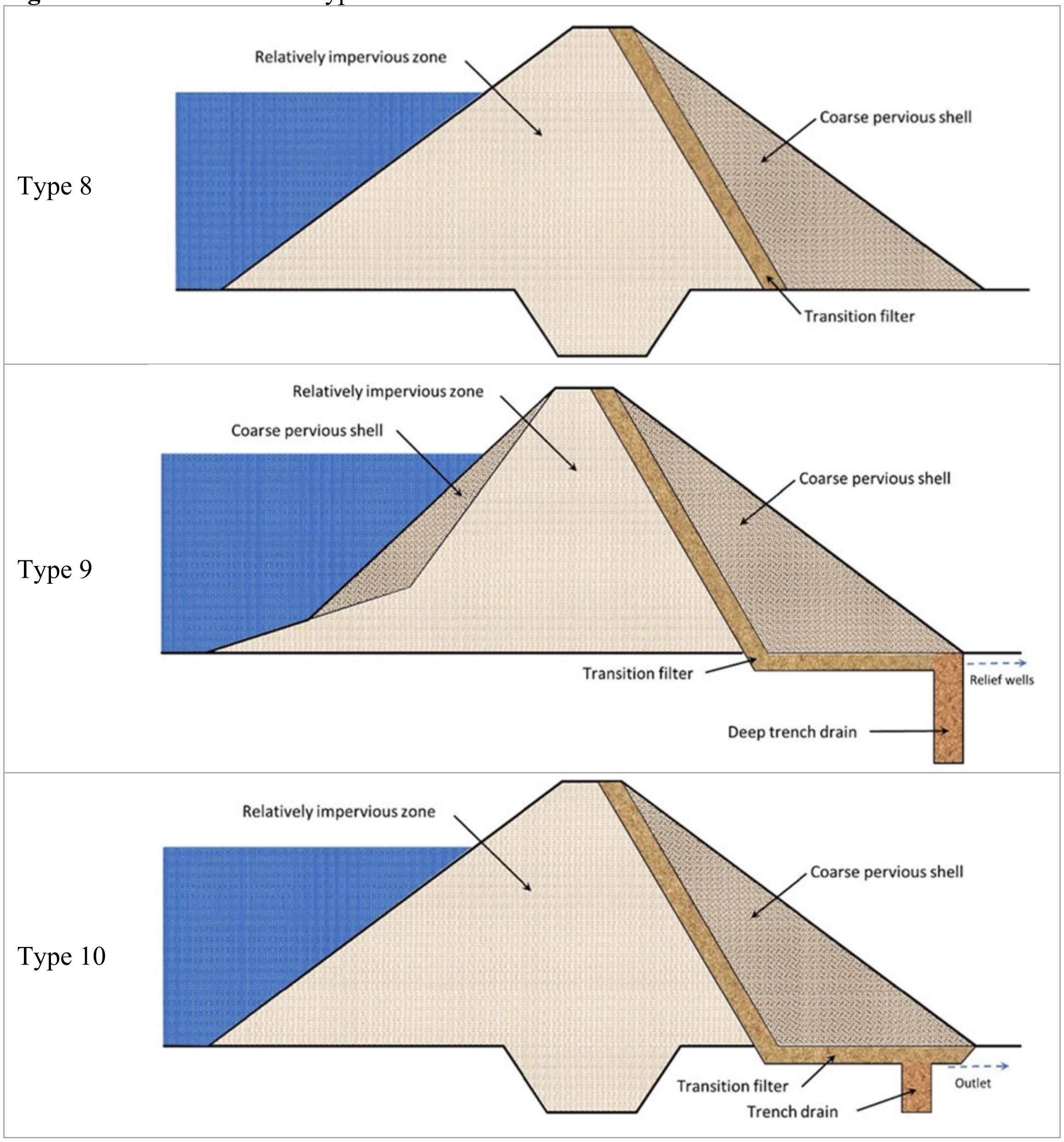
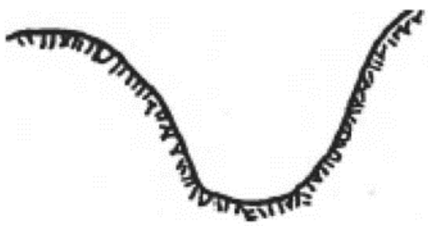


**Figure 4-44:** Evaluation of Site and Foundation Conditions

| Site Configuration[1] | Foundation Classification[2] | | Foundation Type |
|---|---|---|---|
| | Compressibility | Permeability[3] | |
| <br>Type A<br>1. Relatively Broad Gap.<br>2. Gently sloping abutments | None to Very Slight | Impervious | 1a |
| | | Pervious | 1b |
| | Slight | Impervious | 2a |
| | | Pervious | 2b |
| | Medium | Impervious | 3a |
| <br>Type B<br>1. Relatively Narrow gap.<br>2. Steep abutments. | | Pervious | 3b |
| | High | N/A | 4 |
| | Use Table 4-XX if compressibility is for homogeneous foundation for a certain soil type; otherwise use the overall relative reaction to compressibility. (Unconsolidated material will tend to have a higher potential for consolidation.) | | |

[1] Determined from topographic survey.
[2] Determined from the report of the geological investigation and laboratory testing.
[3] Relative to that of the embankment.

**Figure 4-45:** Type of Embankment Design Required

| **Type of Embankment Design Required** | | | | | | |
|---|---|---|---|---|---|---|
| Site Configuration (Table 4--20) | Foundation Type (Table 4-20) | Homogeneous Section | | Zoned Section | | |
| | | Behavior Group (Table 4-17) | Embankment Type (Figure 4-13) | Behavior Group | | Embankment Type (Figure 4-13) |
| | | | | Shell | Core | |
| A | 1a | II, V, VI, VII, VIII | 1 | I, III, IV | II, V, VI, VII, VIII | 7 or 8 |
| | 1a | III, IV | 4 or 6 | I | III, IV | 7 or 8 |
| | 1b | II, V, VI, VII, VIII | 2 or 3 | I, III, IV | II, V, VI, VII, VIII | 9 or 10 |
| | 1b | III, IV | 5 | I | III, IV | 10 |
| | 2a | II, V, VI, VIII | 1 | I, III, IV | II, V, VI, VII | 7 or 8 |
| | 2a | III, IV, VII | 4 or 6 | I | III, IV, VIII | 7 or 8 |
| | 2b | II, V, VI, VII | 2 or 3 | I, III, IV | II, V, VI, VII, VIII | 9 or 10 |
| | 2b | III, IV, VIII | 5 | I | III, IV | 10 |
| | 3a | V, VI, VII | 1, 4 or 6 | I, III, IV | V, VI, VII | 7 or 8 |
| | 3a | II, III, IV, VIII | 4 or 6 | I | II, III, IV, VIII | 7 or 8 |
| | 3b | V, VI, VII | 2 or 3 | I, III, IV | V, VI, VII | 9 or 10 |
| | 3b | II, III, IV, VIII | 5 | I | II, III, IV, VIII | 10 |
| | 4 | Evaluate from test data on undisturbed samples only. | | | | |
| B | 1a | II, V, VI, VII | 1 | I, III, IV | II, V, VI, VII | 7 or 8 |
| | 1a | III, IV, VIII | 4 or 6 | I | III, IV, VIII | 7 or 8 |
| | 1b | II, V, VI, VII | 2 or 3 | I, III, IV | II, V, VI, VII | 9 or 10 |
| | 1b | III, IV, VIII | 5 or 6 | I | III, IV, VIII | 10 |
| | 2a | II, VI, VII | 1 or 6 | I, III, IV, V | II, VI, VII | 7 or 8 |
| | 2a | III, IV, V, VIII | 4 or 6 | I | III, IV, V, VIII | 7 or 8 |
| | 2b | II, VI, VII | 2 or 3 | I, III, IV, V | II, VI, VII | 9 or 10 |
| | 2b | III, IV, V, VIII | 5 or 6 | I | III, IV, V, VIII | 10 |
| | 3a | VI, VII | 1, 4 or 6 | I, III, IV, V | VI, VII, VIII | 7 or 8 |
| | 3a | II, III, IV, V, VIII | 4 | I | II, III, IV, V | 7 or 8 |
| | 3b | VI, VII | 2 or 3 | I, III, IV, V | VI, VII, VIII | 9 or 10 |
| | 3b | II, III, IV, V, VIII | 5 | I | II, III, IV, V | 10 |
| | 4 | Evaluate from test data on undisturbed samples only. | | | | |

## 650.0406 References


A. Soil Conservation Service, Soil Mechanics Notes.

(1) SM Note No. I, Guide for determining the gradation of sand and gravel filters.
(2) SM Note No. 2, Light weight piston sampler for soft soils and loose sands.
(3) SM Note No. 3, Soil mechanics considerations for embankment drains.
(4) SM Note No. 4, Preparation and shipment of undisturbed core samples.
(5) SM Note No. 5, Flow net construction and use.
(6) SM Note No. 6, Glossary, symbols, abbreviations, and conversion factors.
(7) SM Note No. 7, The mechanics of seepage analysis.
(8) SM Note No. 8, Soil mechanics testing standards.
(9) SM Note No. 9, Permeability of selected clean sands and gravels.
(10) SM Note No. 10, The static cone penetrometer: the equipment and using the data. (Additional Soil Mechanics Notes are being prepared. January 1988.)

B. National Engineering Handbooks.

(1) NEH, Section 8, "Engineering Geology".
(2) NEH, Section 18, "Ground Water".

C. Technical Releases.

(1) TR 026, The use of soils containing more than 5 percent rock larger than the no. 4 sieve.
(2) D. TR 027, Laboratory and field test procedures for control of density and moisture of compacted earth embankments.
(3) TR 028, Clay minerals.
(4) TR 071, Rock materials field classification procedure.

D. ASTM D-2487, Classification of soils for engineering purposes.

E. ASTM D-2488, Description and identification of soils (visual-manual procedure).

F. Terzaghi and Peck. Soil mechanics in engineering practice.

G. Sowers, George F. Introductory soil mechanics and foundations: geotechnical engineering.