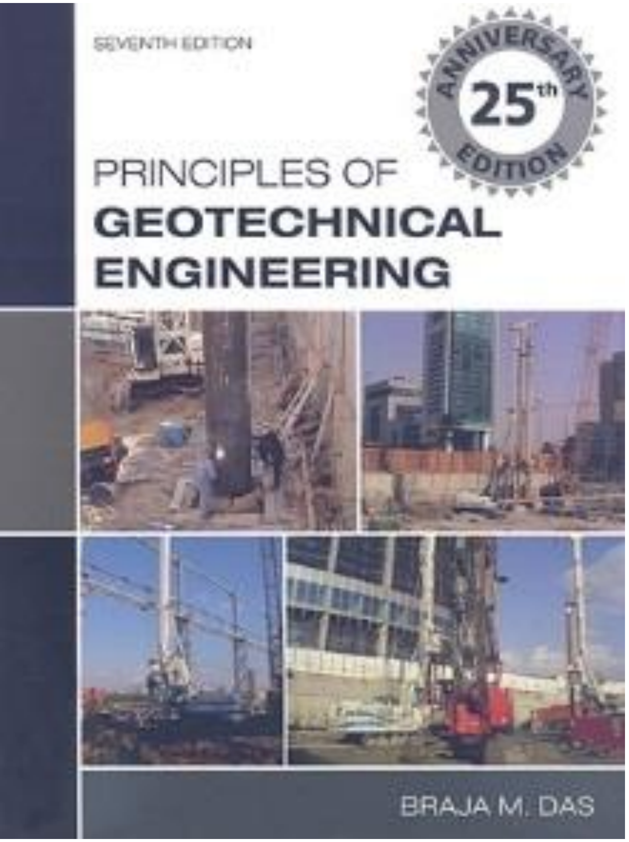
Soil Mechanics

Introduction of soil mechanics

Dr Jasim M Abbas Al Shamary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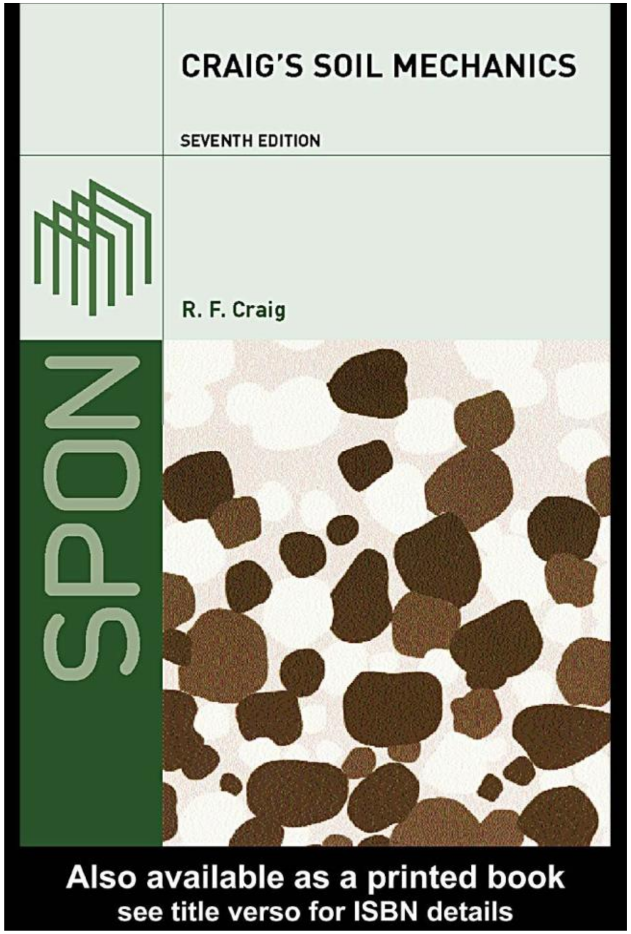
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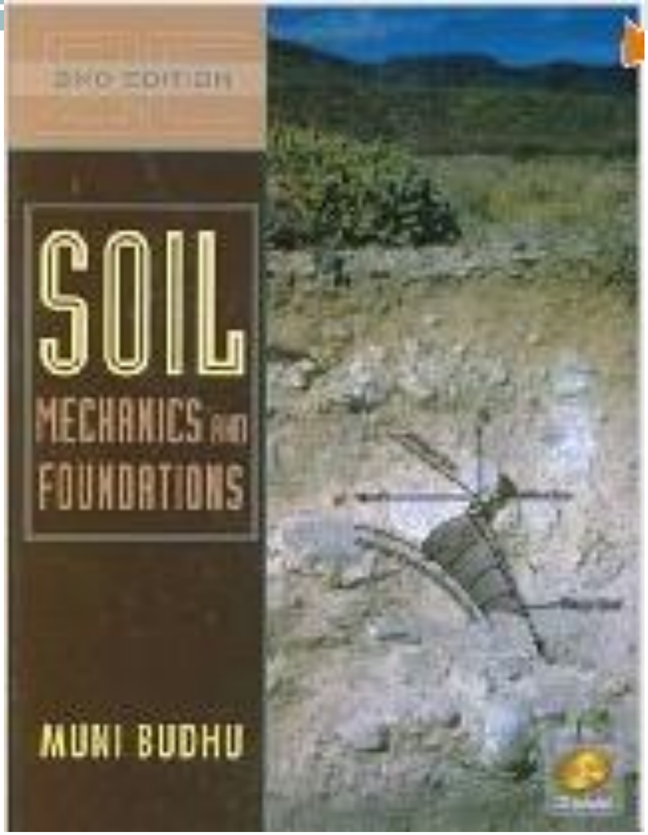
2ND EDITION
SOIL
MECHANICS AND
FOUNDATIONS
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Principles of soil mechanics

WHY SOIL MECHANICS AND FOUNDATIONS?

- The stability and life of any structure, e.g.., buildings, airports, roads, dams, levees, natural slopes, power plants, etc., depend on the stability, strength and deformation of soils.

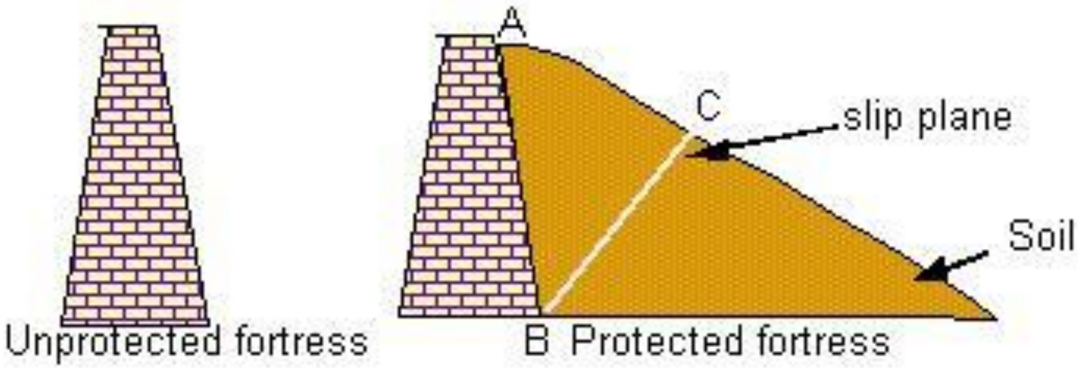
- If the soil fails, structures founded on or within it will fail or be impaired, regardless of how well the structures are designed.

- Soil is the oldest and the most complex engineering material.

- Our ancestors used soils as a construction material to build burial sites, flood protection and shelters.

- Western civilization credits the Romans for recognizing the importance of soils in the stability of structure. Roman engineers, especially Vitruvius, who served during the reign of Emperor Augustus in the first century BC, paid great attention to the soil types (sand, gravel, etc.) and to the design and construction of solid foundations. There were no theoretical bases for design, therefore experience from trial and error was relied upon.

COULOMB'S CONTRIBUTION



Coulomb (1773) is credited as the first person to use mechanics to solve soil problems. He was a member of the French Royal engineers, who were interested in protecting old fortresses that fell easily from cannon fire. To protect the fortresses from artillery attack, sloping masses of soil were placed in front of them. The enemy had to tunnel below the soil mass and the fortress to attack. Of course, the enemy then became an easy target. The mass of soil applies a lateral force to the fortress that could topple it over or cause it to slide away from the soil mass. Coulomb attempted to determine the lateral force so that he could evaluate the stability of the fortress. He postulated that a wedge of soil ABC would fail along a slip plane BC and this wedge would push the wall out or over topple it as it moves down the slip plane.

Movement of the wedge along the slip plane would occur only if the soil resistance along the wedge were overcome. Coulomb assumed that the soil resistance is provided by friction between the particles and the problem now becomes one of a wedge sliding on a rough (frictional) plane, which you may have analyzed in your Physics or Mechanics course. Coulomb has tacitly defined a failure criterion for soils. Today, Coulomb's failure criterion and method of analysis still prevail.

SOIL MECHANICS AND ITS APPLICATIONS TO FOUNDATIONS

- Soil mechanics, also called geotechnique or geotechnics, is the application of engineering mechanics to the solution of problems dealing with soils as a foundation and a construction material. Engineering mechanics is used to understand and interpret the properties, behavior, and performance of soils.

- Soil mechanics is a subset of geotechnical engineering, which involves the application of soil mechanics, geology and hydraulics to the analysis and design of geotechnical systems such as dams, embankments, tunnels, canals, and waterways, foundations for bridges, roads, buildings, etc. Every application of soil mechanics involves uncertainty because of the variability of soils and their compositions. Thus, engineering mechanics can provide only partial solutions to soil problems.

- Experience and approximate calculations are essential for the successful application of soil mechanics to practical problems. Many of the calculations in this textbook/CD are approximations.

INVOLVEMENT

Geotechnical engineers are involved in a variety of problems relating to the analysis and design of foundations, and the stability of earth structures such as retaining walls and slopes.

These marvelous buildings would not exist, regardless of their structural soundness, if the foundations were not adequately designed.

Empire State Building (© Rafael Macia/Photo Researchers)

Sears Tower (© Bill Bachmann/Photo Researchers)

©

Taj Mahal (© Will & Deni McIntyre/Photo Researchers)

INVOLVEMENT

You would be involved in the design of:

Landfill (Courtesy of Todd Mooney)

Hoover Dam
(Courtesy of Bureau of Reclamation, U.S.. Department of the Interior. Photo by E.E. Hertzog)

INVOLVEMENT

You would be involved in the design of transportation systems like these:

Denver airport
(Louis Goldman/Photo Researchers)

Interchange in Los Angeles
(George Gerster/Photo Researchers)

Panama Canal
(Will & Deni McIntyre/Photo Researchers)

FAILURES

You will have to try to avoid failures like these.

Collapse of a sewer system from seepage
(Courtesy of George Tamaro)

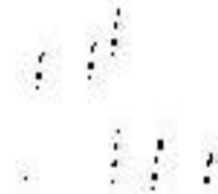
Slope failure
(©Leverett Bradley/Tony stone Images/New York, Inc.)

FORMATION OF SOILS

SOILS are formed from the physical and chemical weathering of rocks

Physical	Chemical
reduction in size without change in composition from parent rock	reduction in size and change in composition from parent rock
agents	agents
exfoliation unloading erosion freezing thawing	hydration carbonation oxidation

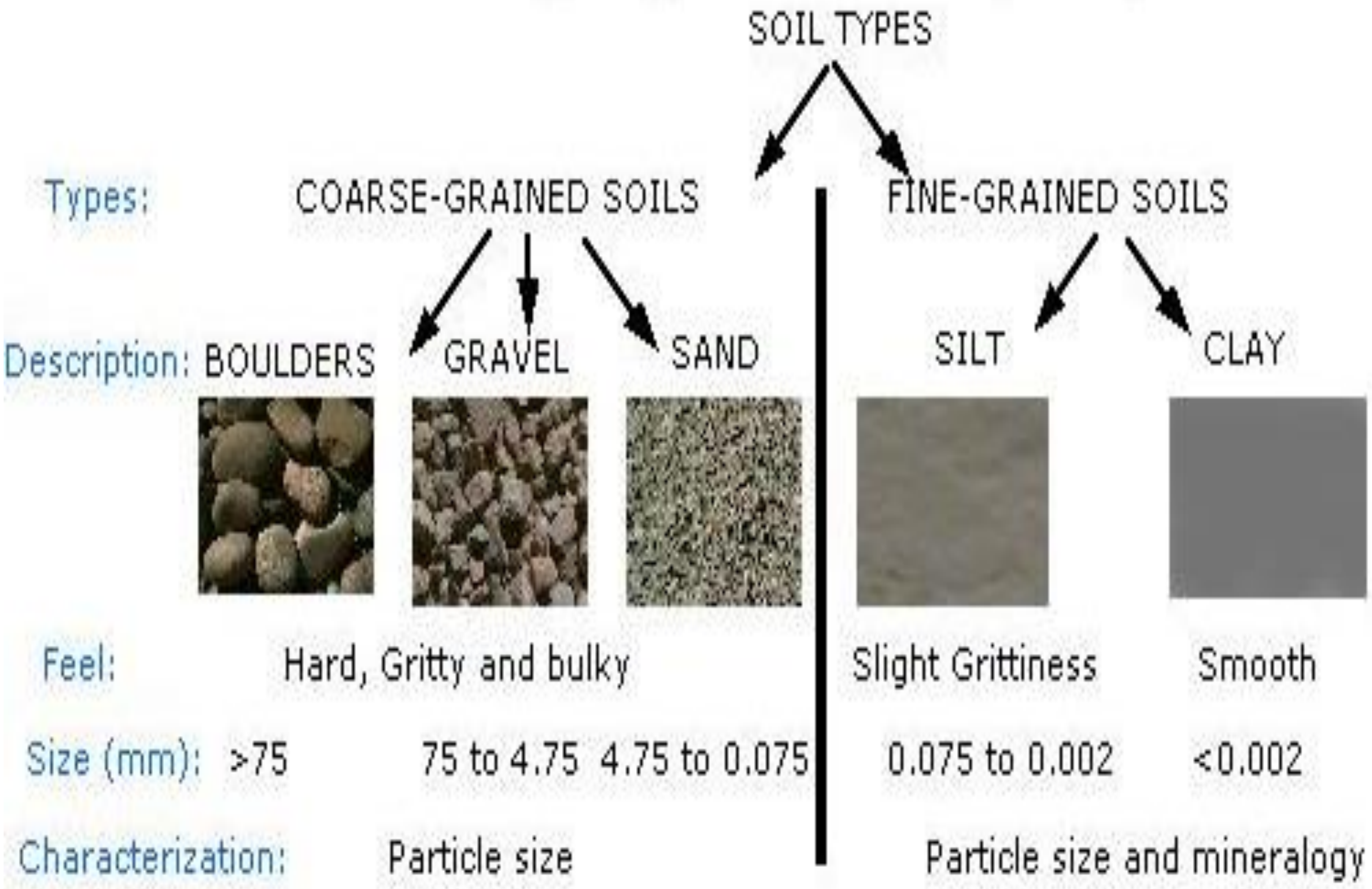
Some agents that cause changes in soils



Rain

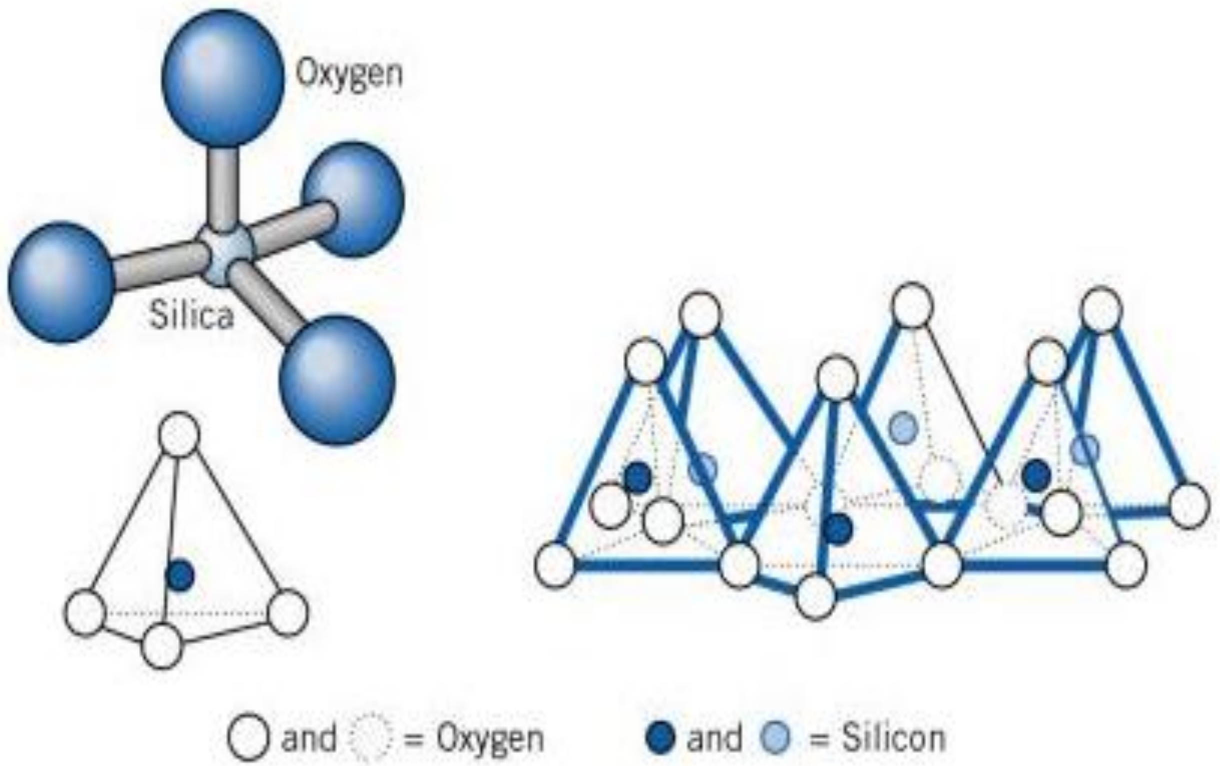
SOIL TYPES

We will deal with the following soil types and call them engineering soils.



CLAY MINERALS

CLAYS are composed of clay minerals. CLAY MINERALS are crystalline materials comprising OXYGEN and SILICA. The structural unit of silicates is a tetrahedron consisting of a silica cation (+charged) surrounded by four oxygen (-charged). Silicates combine to form sheets.

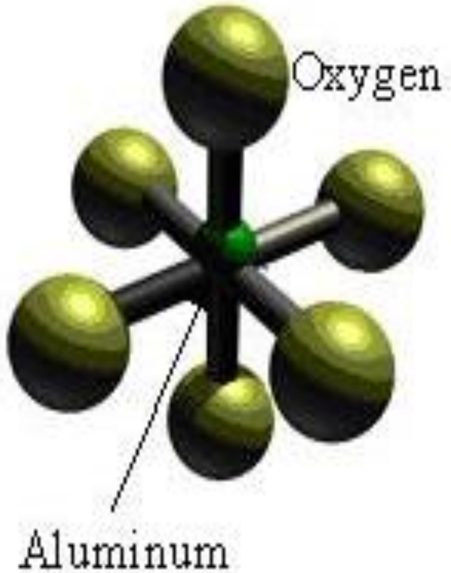


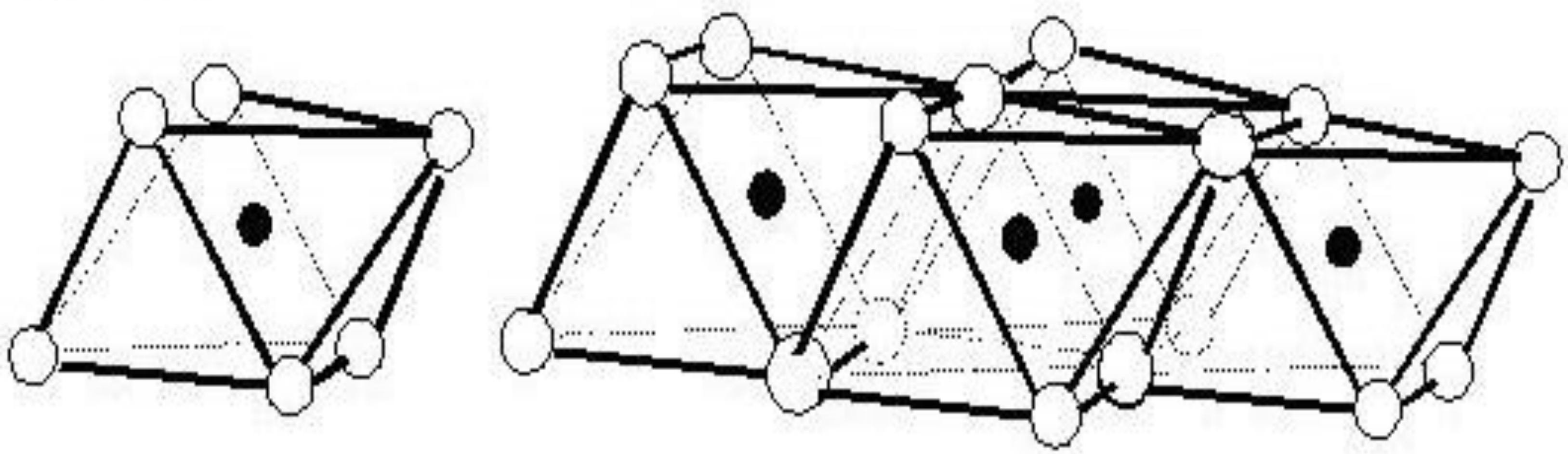
(*a*) Single

(*b*) A tetrahedral

CLAYS MINERALS (cont'd)

Silicate sheets may contain other structural units such as alumina sheets. Alumina sheets are formed by combination of alumina minerals, which consist of an aluminum ion surrounded by six oxygen of hydroxyl atoms in an octahedron.





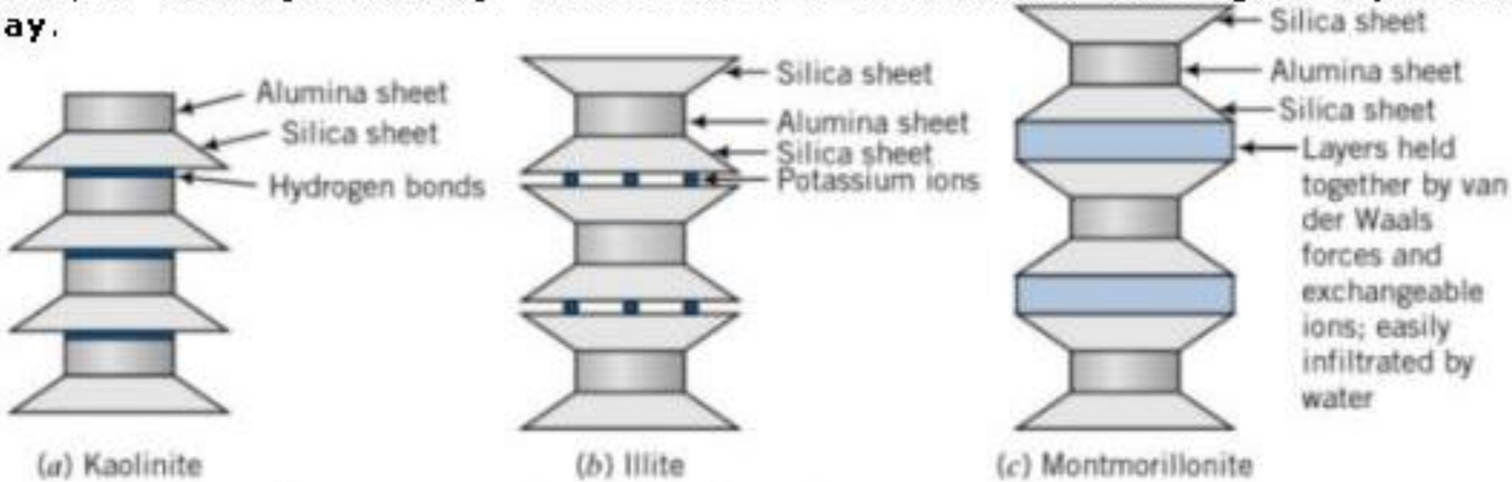
CLAY MINERALS (cont'd)

Minerals are crystalline materials. The The main clay mineral groups are **kaolinite, illite and montmorillonite.**

Kaolinite has a structure that consists of one silica sheet and one alumina sheet bonded together into a layer about 0.72nm thick and stacked repeatedly. The layers are held together by hydrogen bonds. Kaolinite is common in clays in humid tropical regions.

Illite consists of repeated layers of one alumina sheet sandwiched by two silicate sheets. The layers, each of thickness 0.96nm, are held together by potassium ions.

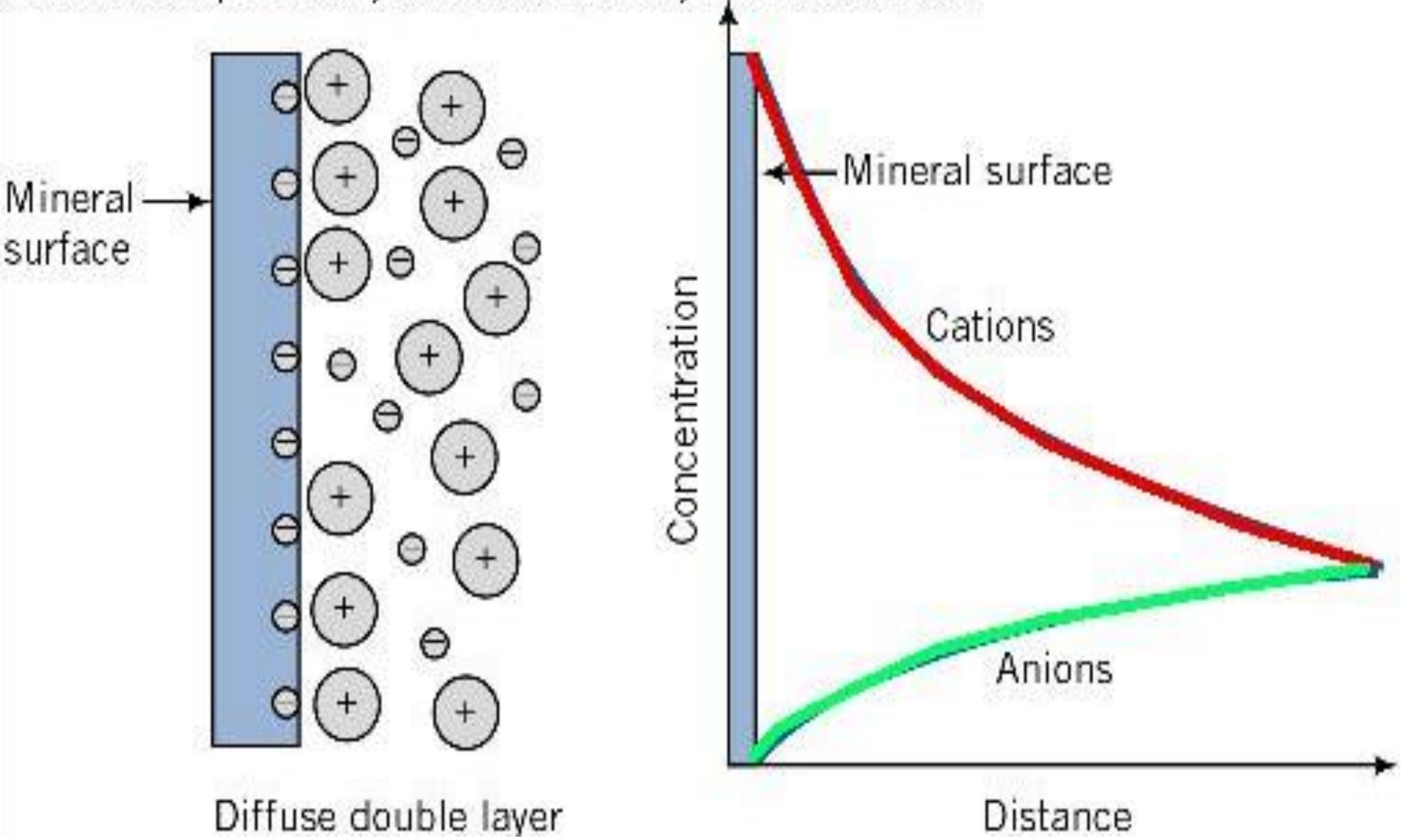
Montmorillonite has a similar structure to illite, but the layers are held together by weak van der Waals forces and exchangeable ions. Water can easily enter the bond and separate the layers resulting in swelling. Montmorillonite is often called a **swelling or expansive clay**.



Observe the structure of kaolinite, illite and montmorillonite

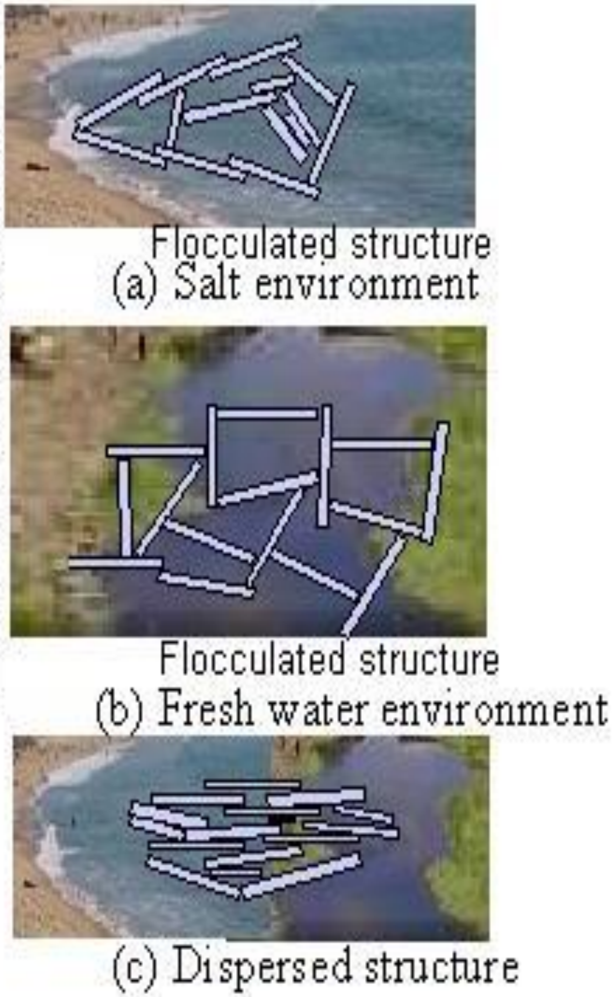
SURFACE FORCES AND ADSORBED WATER

The surface charges on fine-grained soils are negative (anions). These negative surface charges attract cations and the positively charged side of water molecules from surrounding water. Consequently, a thin film or layer of water, called adsorbed water, is bonded to the mineral surfaces. The thin film or layer of water is known as the diffuse double layer. The largest concentration of cations occurs at the mineral surface and decreases exponentially with distance away from the surface.



SOIL FABRIC

During deposition, the mineral particles are arranged into structural frameworks that we call soil fabric. Each particle is in random contact with neighboring particles. The environment under which deposition occurs influences the structural framework that is formed. In particular, the electrochemical environment has the greatest influence on the kind of soil fabric that is formed during deposition. Any loading (tectonic or otherwise) during or after deposition permanently alters the structural arrangement in a way that is unique to that particular loading condition. Consequently, the history of loading and changes in the environment are imprinted in the soil fabric. The soil fabric is the brain; it retains the memory of the birth of the soil and subsequent changes that occur.



Soil fabric formed under salt and fresh water

COMPARISON OF COARSE-GRAINED AND FINE-GRAINED SOILS FOR ENGINEERING USE

Coarse-grained soils
- good load-bearing capacities
- good drainage qualities
- not significantly affected by moisture changes
- engineering properties controlled by grain size and their structural arrangements

Fine-grained soils
- poor load-bearing capacities
- poor drainage qualities; practically impermeable
- change volume and strength with moisture changes
- frost susceptible
- engineering properties controlled by mineralogical factors

ESSENTIAL POINTS

The *essential points* are:

1. Soils are derived from the weathering of rocks and are commonly described by textural terms such as gravels, sands, silts and clays.

2. Particle size is used to distinguish various soil textures.

3. Clays are composed of three main types of minerals - kaolinite, illite and montmorillonite.

4. The clay minerals consist of silica and alumina sheets that are combined to form layers. The bonds between the layers play a very important role in the mechanical behavior of clays. The bond between the layers in montmorillonite is very weak compared with kaolinite and illite. Water can easily enter between the layers in montmorillonite causing swelling.

5. The engineering properties of coarse-grained soils depend mainly on the grain size and the structural arrangement of the particles.

6. The engineering properties of fine-grained soils depend mainly on mineralogical factors.

7. A thin layer of water is bonded to the mineral surfaces of soils and has significant influences on the physical and mechanical characteristics of fine-grained soils.

8. Fine-grained soils have much larger surface areas than coarse-grained soils and are responsible for the major physical and mechanical differences between coarse-grained and fine-grained soils.

WHY A SOIL INVESTIGATION?

A soil investigation is necessary to provide information for design and construction, and for environmental assessment. The purposes of a soil investigation are:

1. To evaluate the general suitability of the site for the proposed project.
2. To enable an adequate and economical design to be made.
3. To disclose and make provision for difficulties that may arise during construction due to ground and other local conditions.

The scope of a soil investigation depends on the type, size and importance of the structure, the client, the engineer's familiarity with the soils at the site and local building codes. Structures that are sensitive to settlement such as machine foundations, high use buildings, etc. usually require a thorough soil investigation compared to a foundation for a house. A client may wish to take a greater risk than normal to save money and set limits on the type and extent of the site investigation. If the geotechnical engineer is familiar with a site, he/she may undertake a very simple soil investigation to confirm his/her experience. Some local building codes have provisions that set out the extent of a site investigation. Regardless of the thoroughness of a site investigation, it is mandatory that a site visit be made to the proposed site.

PHASES OF A SOIL INVESTIGATION PROGRAM

Phase I - Collection of available information such as a site plan, type, size and importance of the structure, loading conditions, previous geotechnical reports, topographic maps, air photographs, geologic maps, newspaper clippings, etc.

Phase II - Preliminary reconnaissance or a site visit to provide a general picture of the topography and geology of the site. It is necessary that you take with you on the site visit all the information gathered in Phase I to compare with the current conditions of the site.

Phase III - Detailed soil exploration. The objectives of a detailed soil exploration are:

1. To determine the geological structure which should include the thickness, sequence and extent of the soil strata.
2. To determine the ground water conditions.
3. To obtain disturbed and undisturbed samples for laboratory tests.
4. To conduct in situ tests.

ESSENTIAL POINTS

The *essential points* are:

1. A site investigation is necessary to determine the nature of the soils at a proposed site for design and construction.
2. A soil investigation needs careful planning and is usually done in phases.

SOIL EXPLORATION PROGRAM

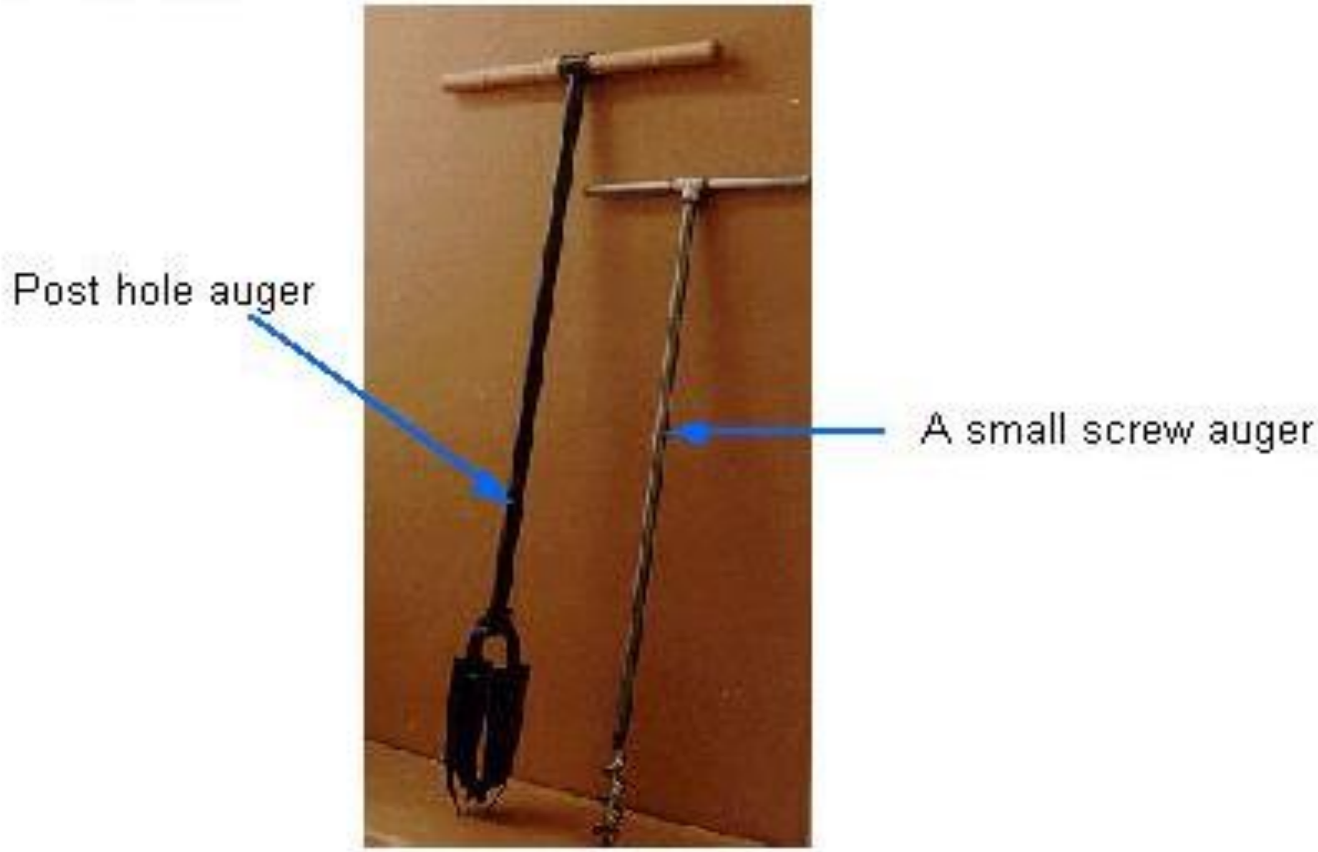
A soils exploration program usually involves test pits and/or soil borings (boreholes). During the site visit (Phase II) you should work out most of the soil exploration program. A detailed soil exploration consists of:

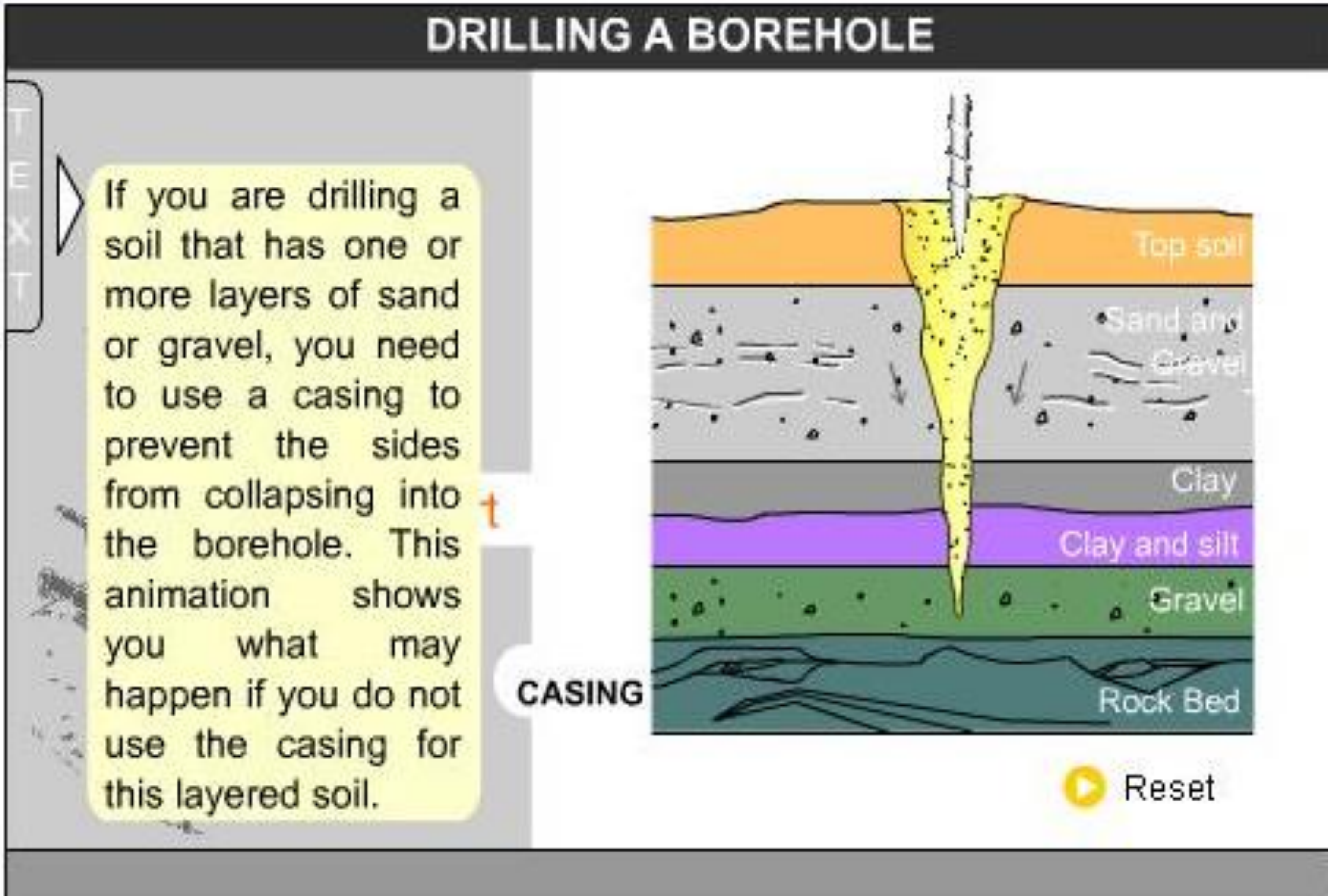
1. Preliminary location of each borehole and/or test pits.
2. Numbering of the boreholes or test pits.
3. Planned depth of each borehole, or test pit.
4. Methods and procedures for advancing the boreholes.
5. Sampling instructions for at least the first borehole. The sampling instructions must include the number of samples and possible locations. Changes in the sampling instructions often occur after the first borehole.
6. Requirements for groundwater observations.

SOIL EXPLORATION METHODS

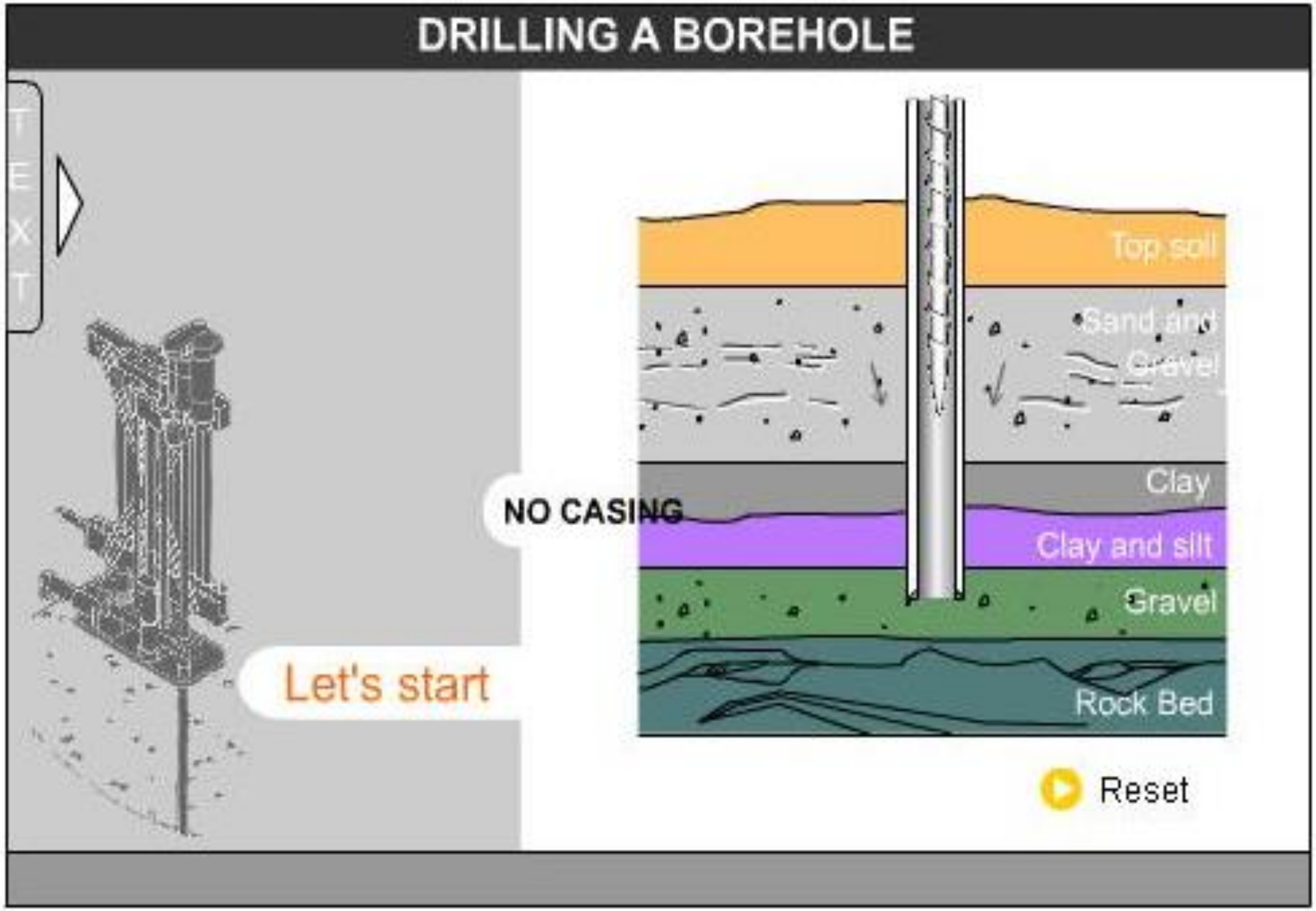
Access to the soil may be obtained by the following methods.

1. Trial pits or test pits.
2. Hand or powered augers.
3. Wash boring.
4. Rotary rigs.





DRILLING A BOREHOLE
TEXT
If you are drilling a soil that has one or more layers of sand or gravel, you need to use a casing to prevent the sides from collapsing into the borehole. This animation shows you what may happen if you do not use the casing for this layered soil.
CASING
Top soil
Sand and Gravel
Clay
Clay and silt
Gravel
Rock Bed
Reset


DRILLING A BOREHOLE
TEXT
NO CASING
Let's start
Top soil
Sand and Gravel
Clay
Clay and silt
Gravel
Rock Bed
Reset

ESSENTIAL POINTS

The *essential points* are:

1. A site investigation is necessary to determine the nature of the soils at a proposed site for design and construction.
2. A soil investigation needs careful planning and is usually done in phases.
3. A number of tools are available for soil exploration. You need to use judgment as to the type appropriate for a given project.

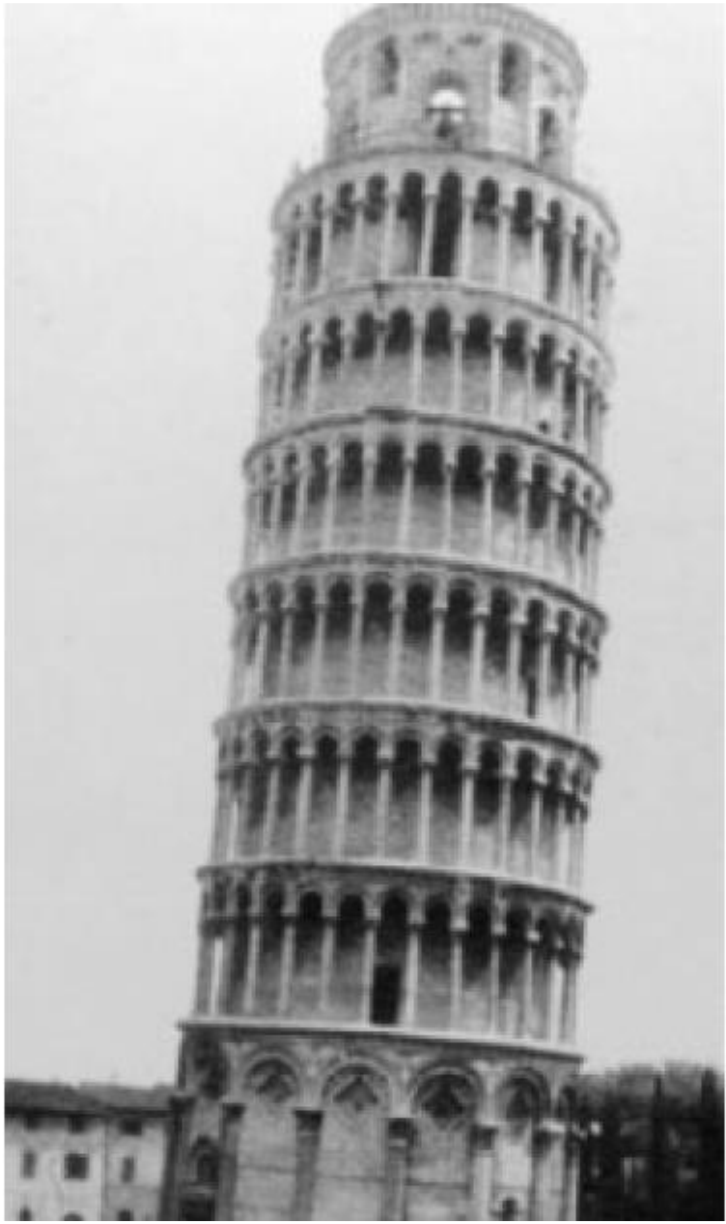
Soil Mechanics

Introduction of soil mechanics - Part 2

Dr Jasim M Abbas Al Shamary

Geotechnical Engineering

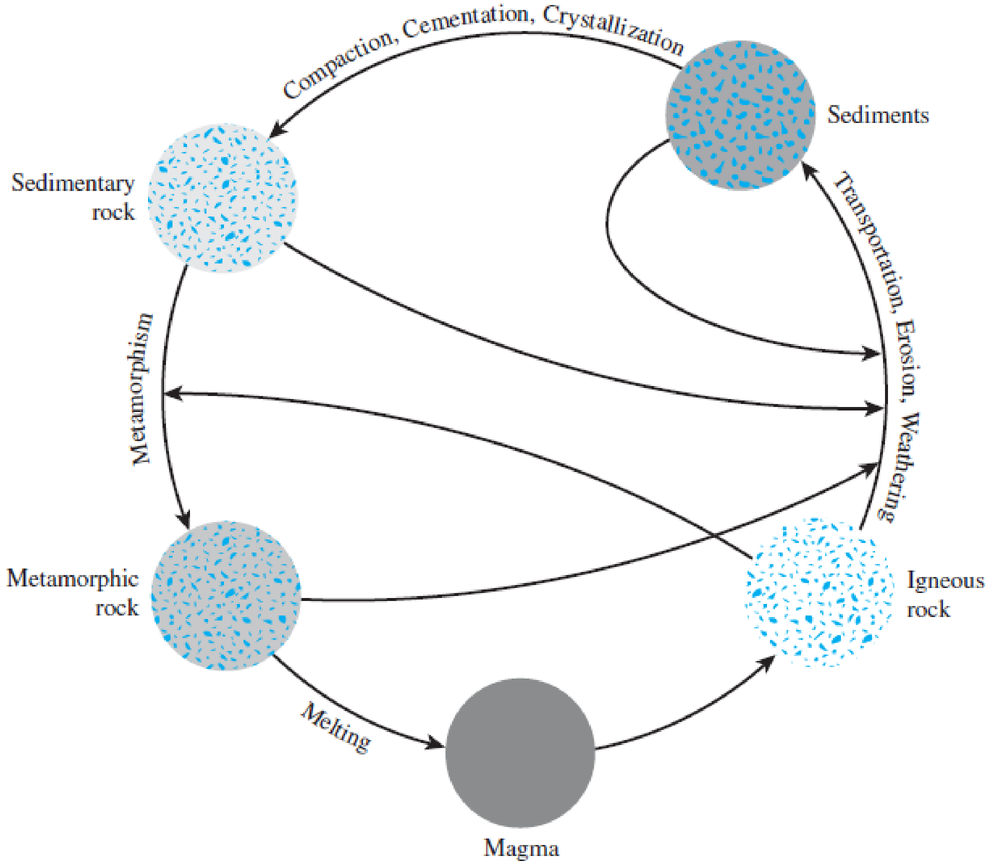
For engineering purposes, ***soil*** is defined as the **uncemented** aggregate of mineral grains and decayed organic matter (solid particles) with liquid and gas in the empty spaces between the solid particles.



Origin of Soil and Grain Size

Rock Cycle and the Origin of Soil

Igneous Rock ***Sedimentary Rock*** ***Metamorphic Rock***



Soil-Particle Size

Table 2.3 Particle-Size Classifications

Name of organization	Grain size (mm) Gravel	Sand	Silt	Clay
Massachusetts Institute of Technology (MIT)	>2	2 to 0.06	0.06 to 0.002	<0.002
U.S. Department of Agriculture (USDA)	>2	2 to 0.05	0.05 to 0.002	<0.002
American Association of State Highway and Transportation Officials (AASHTO)	76.2 to 2	2 to 0.075	0.075 to 0.002	<0.002
Unified Soil Classification System (U.S. Army Corps of Engineers, U.S. Bureau of Reclamation, and American Society for Testing and Materials)	76.2 to 4.75	4.75 to 0.075	Fines (i.e., silts and clays) <0.075	

Note: Sieve openings of 4.75 mm are found on a U.S. No. 4 sieve; 2-mm openings on a U.S. No. 10 sieve; 0.075-mm openings on a U.S. No. 200 sieve. See Table 2.5.

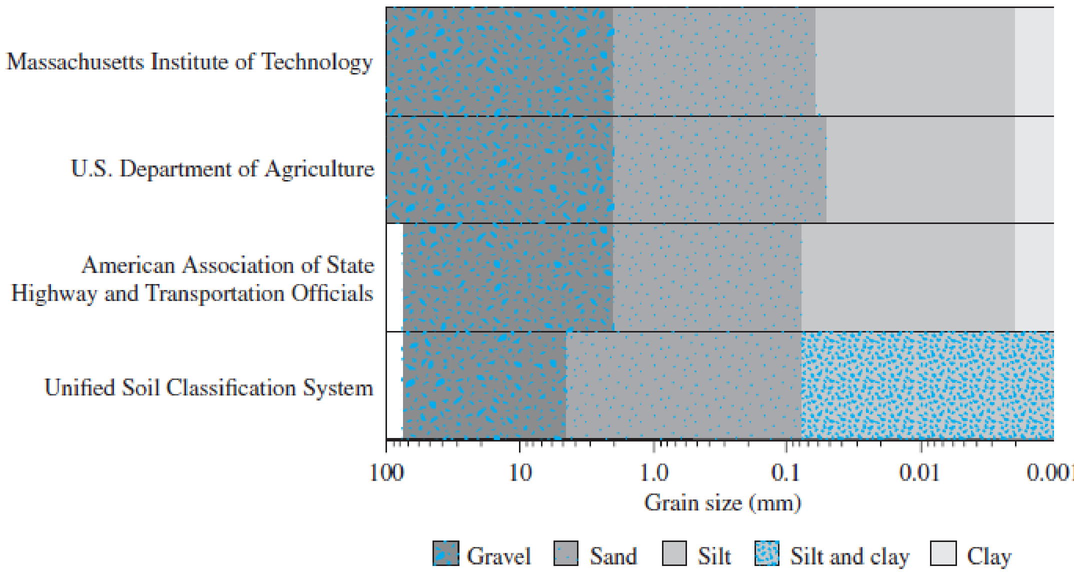


Figure 2.7 Soil-separate-size limits by various systems

Clay Minerals

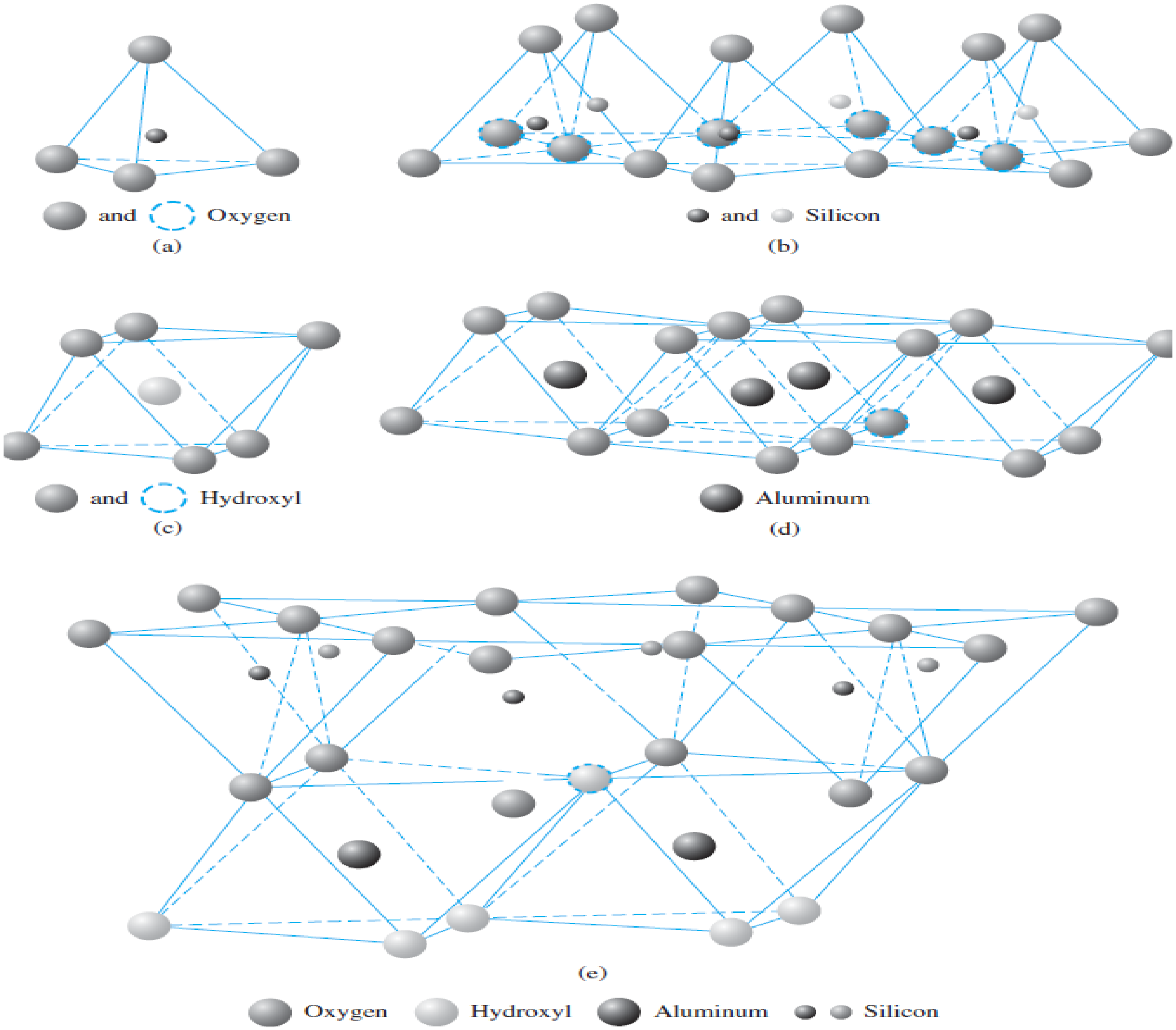


Figure 2.10 (a) Silica tetrahedron; (b) silica sheet; (c) alumina octahedron; (d) octahedral (gibbsite) sheet; (e) elemental silica-gibbsite sheet (*After Grim, 1959. With permission from ASCE.*)

Clay Minerals

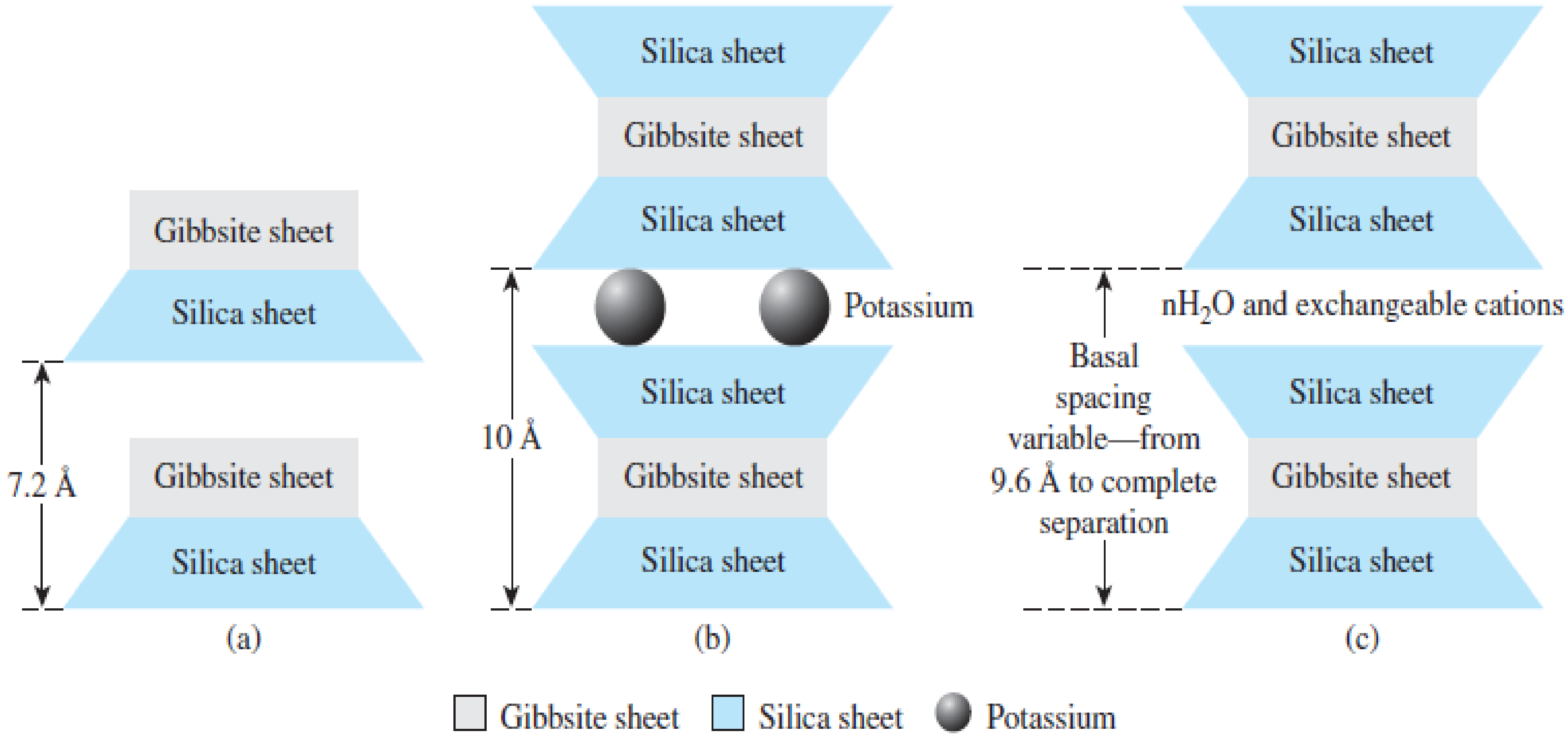


Figure 2.12 Diagram of the structures of (a) kaolinite; (b) illite; (c) montmorillonite

Specific Gravity (G_s)

Specific gravity is defined as the ratio of the unit weight of a given material to the unit weight of water.

Table 2.4 Specific Gravity of Common Minerals

Mineral	Specific gravity, G_s
Quartz	2.65
Kaolinite	2.6
Illite	2.8
Montmorillonite	2.65–2.80
Halloysite	2.0–2.55
Potassium feldspar	2.57
Sodium and calcium feldspar	2.62–2.76
Chlorite	2.6–2.9
Biotite	2.8–3.2
Muscovite	2.76–3.1
Hornblende	3.0–3.47
Limonite	3.6–4.0
Olivine	3.27–3.7

Mechanical Analysis of Soil

Sieve Analysis

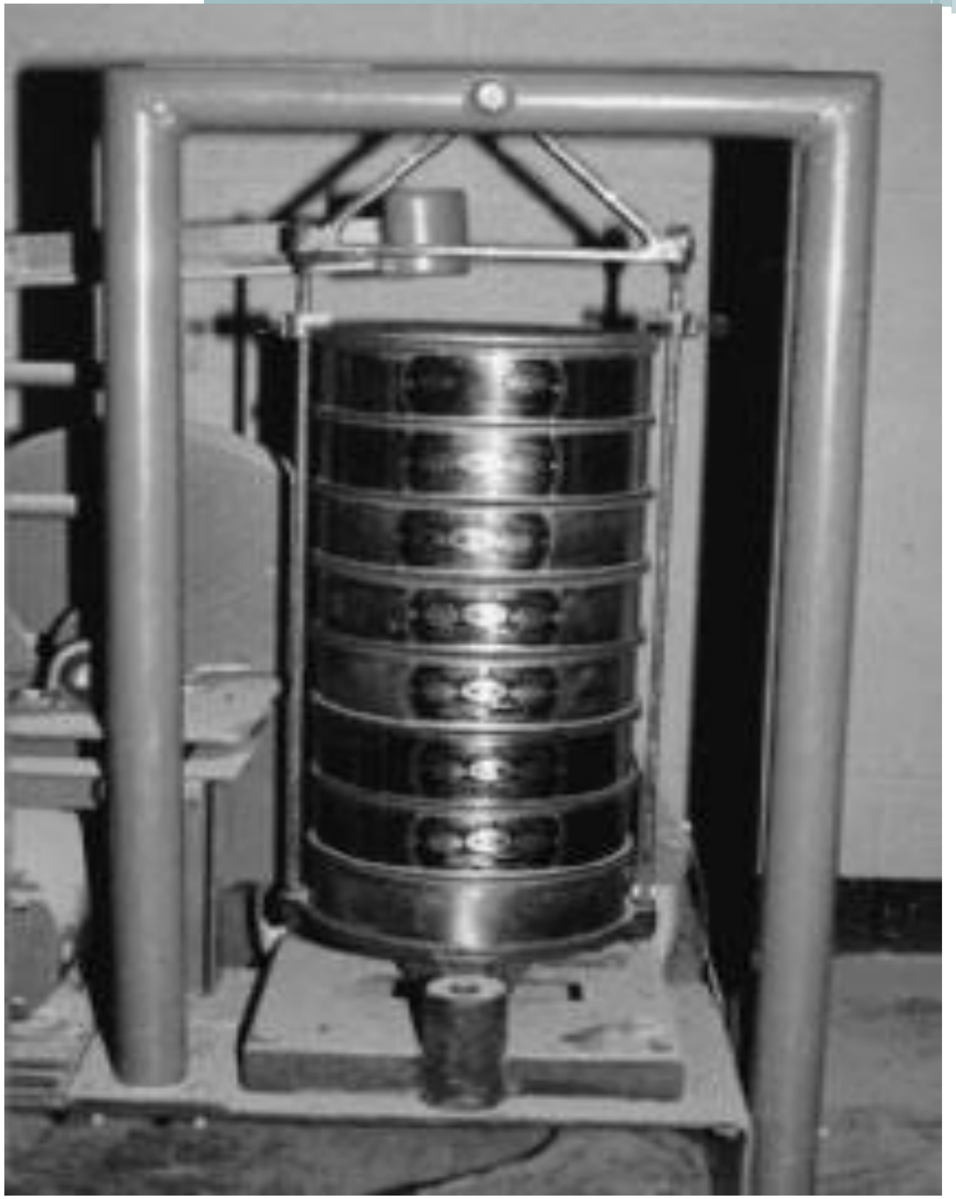
Sieve analysis consists of shaking the soil sample through a set of sieves that have progressively smaller openings

Table 2.5 U.S. Standard Sieve Sizes

Sieve no.	Opening (mm)	Sieve no.	Opening (mm)
4	4.75	35	0.500
5	4.00	40	0.425
6	3.35	50	0.355
7	2.80	60	0.250
8	2.36	70	0.212
10	2.00	80	0.180
12	1.70	100	0.150
14	1.40	120	0.125
16	1.18	140	0.106
18	1.00	170	0.090
20	0.850	200	0.075
25	0.710	270	0.053
30	0.600		

Mechanical Analysis of Soil

Sieve Analysis



Mechanical Analysis of Soil

Hydrometer Analysis

Hydrometer analysis is based on the principle of sedimentation of soil grains in water.

$$v = \frac{\rho_s - \rho_w}{18\eta} D^2$$

where v = velocity

ρ_s = density of soil particles

ρ_w = density of water

η = viscosity of water

D = diameter of soil particles

Particle-Size Distribution Curve

A particle-size distribution curve can be used to determine the following four parameters for a given soil (Figure 2.26):

1. *Effective size* (D_{10}): This parameter is the diameter in the particle-size distribution curve corresponding to 10% finer. The effective size of a granular soil is a good measure to estimate the hydraulic conductivity and drainage through soil.
2. *Uniformity coefficient* (C_u): This parameter is defined as

$$C_u = \frac{D_{60}}{D_{10}}$$

where D_{60} = diameter corresponding to 60% finer.

3. *Coefficient of gradation* (C_c): This parameter is defined as

$$C_c = \frac{D_{30}^2}{D_{60} \times D_{10}}$$

Particle-Size Distribution Curve

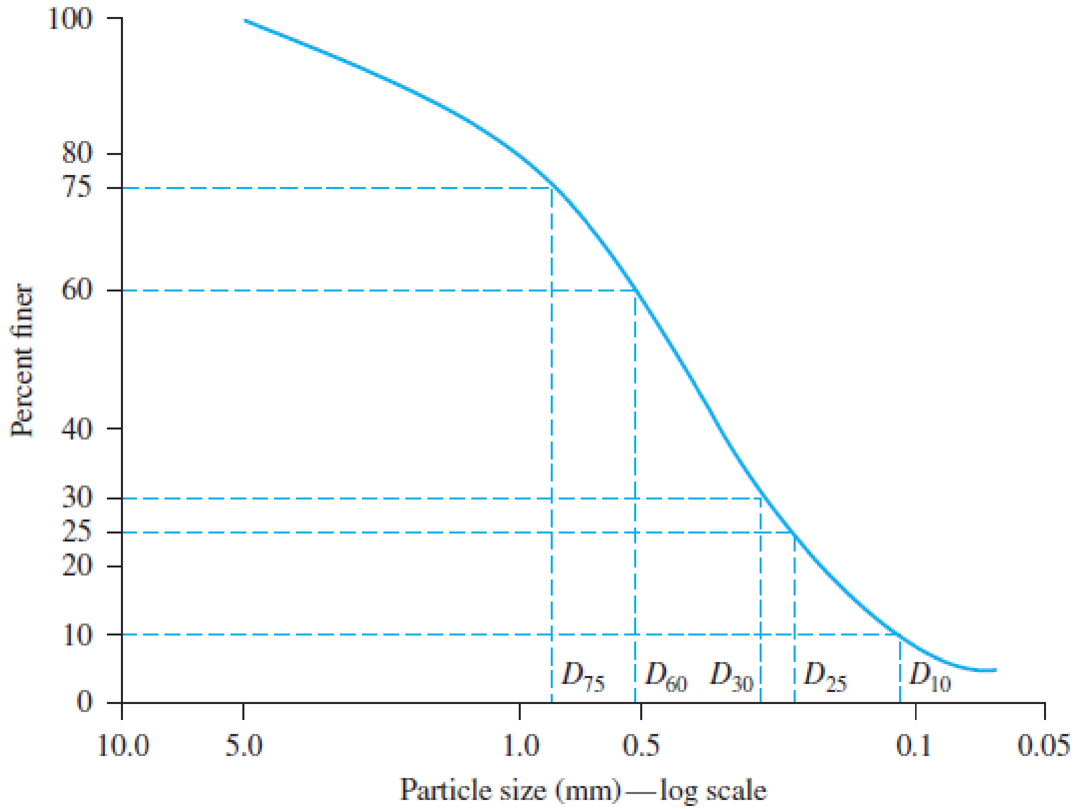


Figure 2.26 Definition of D_{75}, D_{60}, D_{30}, D_{25}, and D_{10}

Particle-Size Distribution Curve

4. *Sorting coefficient* (S_0): This parameter is another measure of uniformity and is generally encountered in geologic works and expressed as

$$S_0 = \sqrt{\frac{D_{75}}{D_{25}}}$$

Size (mm)	Percent finer	
76.2	100	$100 - 100 = 0\%$ gravel
4.75	100	$100 - 62 = 38\%$ sand
0.075	62	$62 - 0 = 62\%$ silt and clay
—	0	

Example 2.1

Following are the results of a sieve analysis. Make the necessary calculations and draw a particle-size distribution curve.

U.S. sieve no.	Mass of soil retained on each sieve (g)
4	0
10	40
20	60
40	89
60	140
80	122
100	210
200	56
Pan	12

Solution

The following table can now be prepared.

U.S. sieve (1)	Opening (mm) (2)	Mass retained on each sieve (g) (3)	Cumulative mass retained above each sieve (g) (4)	Percent finer[a] (5)
4	4.75	0	0	100
10	2.00	40	$0 + 40 = 40$	94.5
20	0.850	60	$40 + 60 = 100$	86.3
40	0.425	89	$100 + 89 = 189$	74.1
60	0.250	140	$189 + 140 = 329$	54.9
80	0.180	122	$329 + 122 = 451$	38.1
100	0.150	210	$451 + 210 = 661$	9.3
200	0.075	56	$661 + 56 = 717$	1.7
Pan	–	12	$717 + 12 = 729 = \Sigma M$	0

[a] $\dfrac{\sum M - \text{col. } 4}{\sum M} \times 100 = \dfrac{729 - \text{col. } 4}{729} \times 100$

The particle-size distribution curve is shown in Figure 2.28.

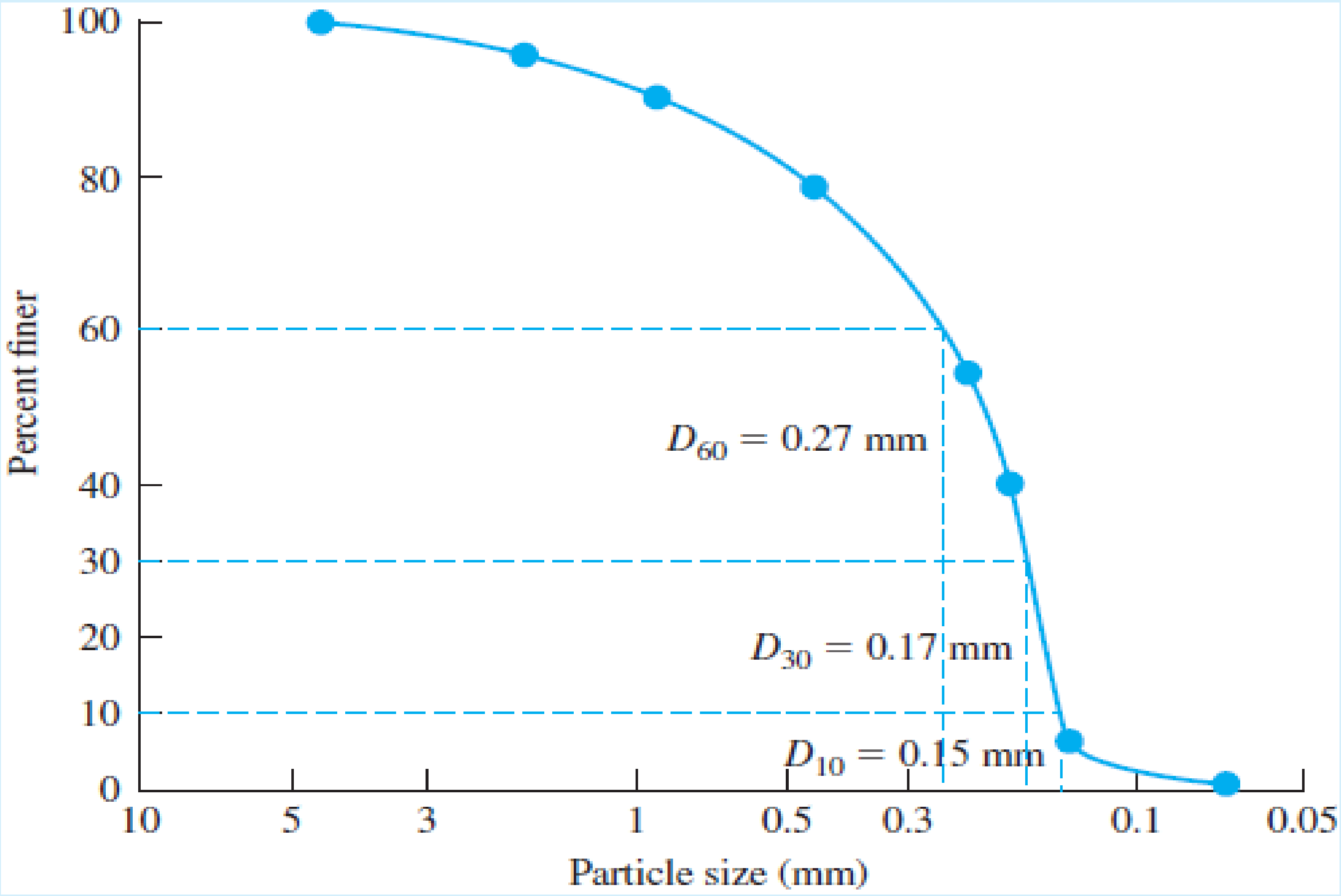


Figure 2.28 Particle-size distribution curve ■

Example 2.2

For the particle-size distribution curve shown in Figure 2.28 determine

a. D_{10}, D_{30}, and D_{60}
b. Uniformity coefficient, C_u
c. Coefficient of gradation, C_z

Solution

Part a

From Figure 2.28,

$$D_{10} = \mathbf{0.15\ mm}$$

$$D_{30} = \mathbf{0.17\ mm}$$

$$D_{60} = \mathbf{0.27\ mm}$$

Part b

$$C_u = \frac{D_{60}}{D_{10}} = \frac{0.27}{0.15} = \mathbf{1.8}$$

$$C_z = \frac{D_{30}^2}{D_{60} \times D_{10}} = \frac{(0.17)^2}{(0.27)(0.15)} = \mathbf{0.71}$$

■

Example 2.3

For the particle-size distribution curve shown in Figure 2.28, determine the percentages of gravel, sand, silt, and clay-size particles present. Use the Unified Soil Classification System.

Solution

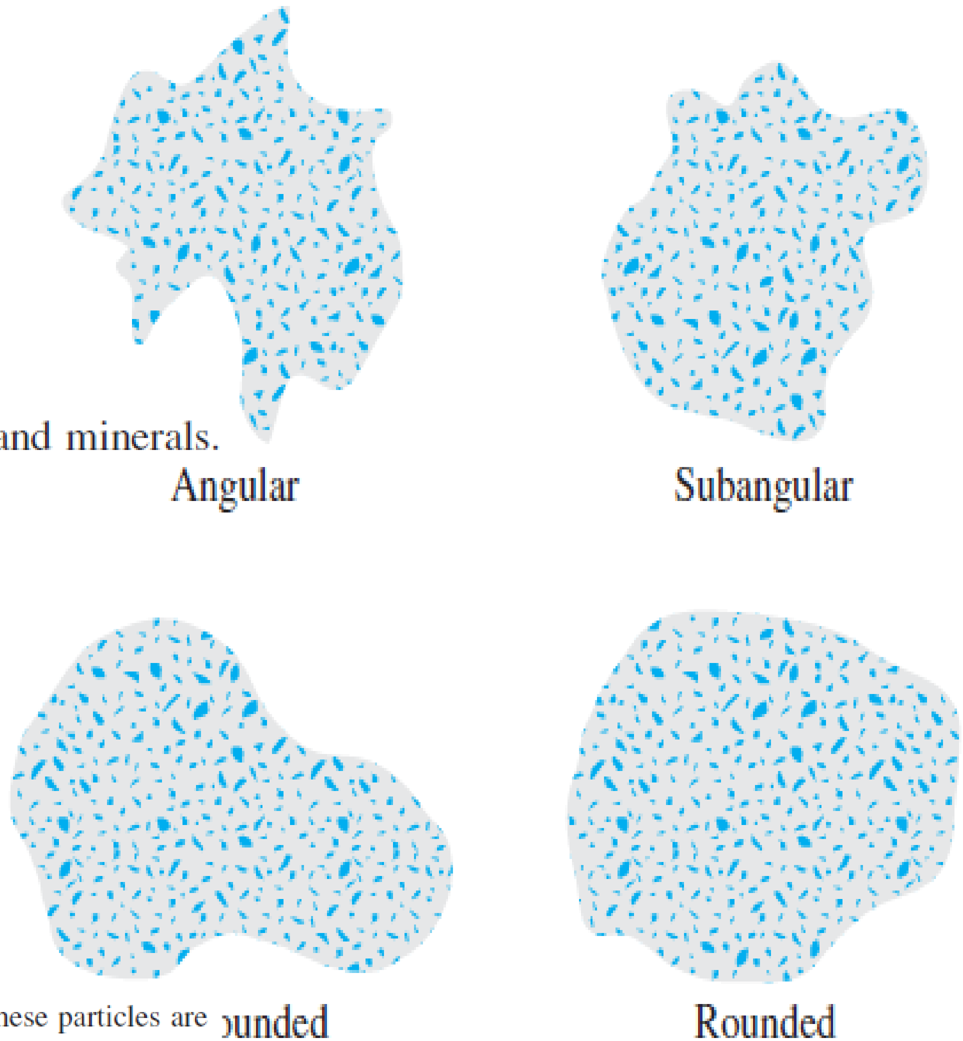
From Figure 2.28, we can prepare the following table.

Size (mm)	Percent finer	
76.2	100	
		100 − 100 = **0% gravel**
4.75	100	
		100 − 1.7 = **98.3% sand**
0.075	1.7	
		1.7 − 0 = **1.7% silt and clay**
–	0	

Particle Shape

1. Bulky
2. Flaky
3. Needle shaped

formed mostly by mechanical weathering of rock and minerals.



9 Shape of bulky particles

Flaky particles have very low sphericity—usually 0.01 or less. These particles are predominantly clay minerals.

Needle-shaped particles are much less common than the other two particle types. Examples of soils containing needle-shaped particles are some coral deposits and attapulgite clays.

Soil Mechanics

Weight – Volume Relationship

Dr Jasim M Abbas Al Shamary

Weight–Volume Relationships

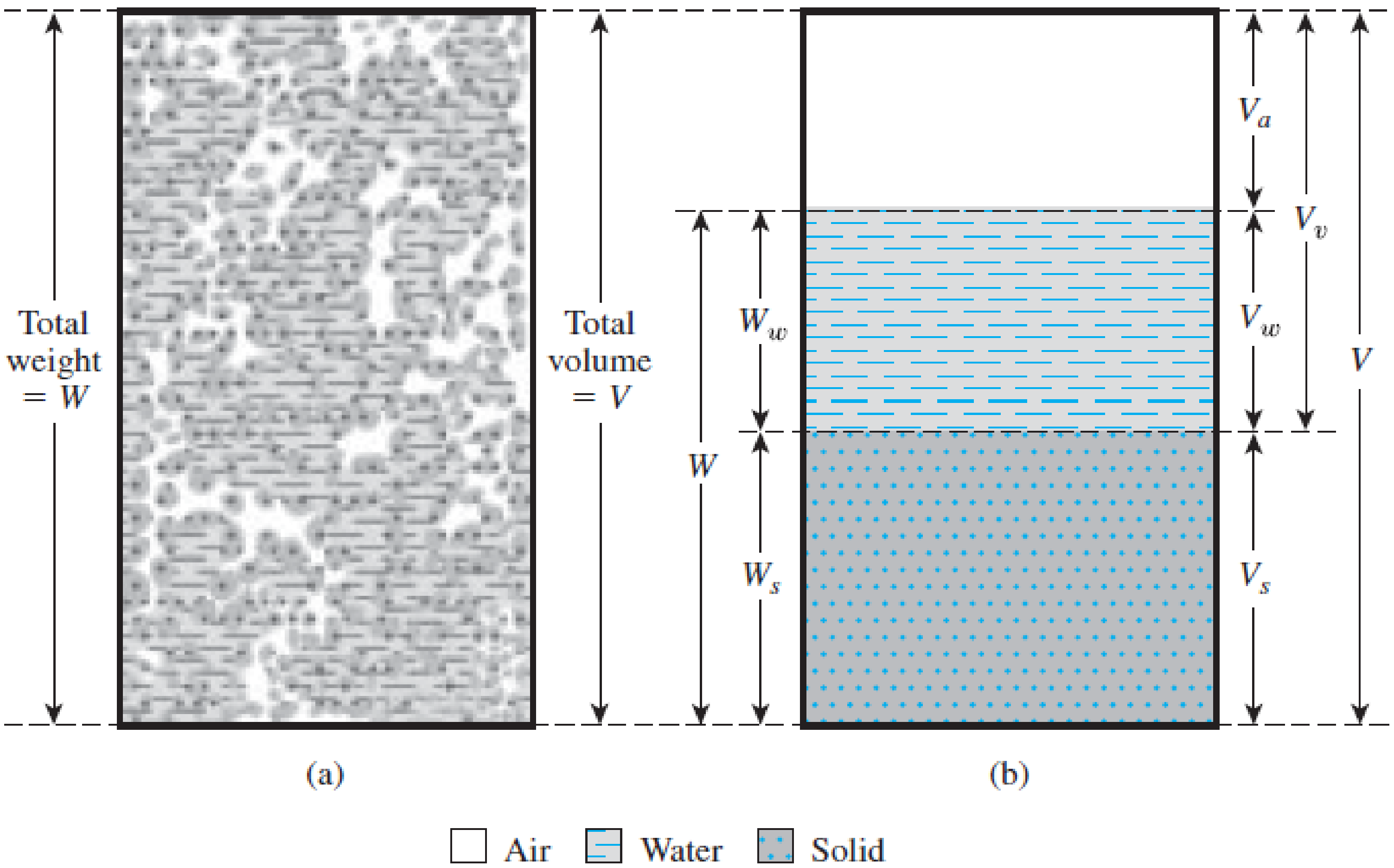


Figure 3.1 (a) Soil element in natural state; (b) three phases of the soil element

$$e = \frac{V_v}{V_s}$$ *Void ratio*

$$S = \frac{V_w}{V_v}$$ *degree of saturation*

$$n = \frac{e}{1 + e}$$ porosity

$$w = \frac{W_w}{W_s}$$ *moisture content*

$$\gamma = \frac{W}{V}$$ *Unit weight*

$$\gamma_d = \frac{W_s}{V}$$

$$\gamma_d = \frac{\gamma}{1 + w}$$ *dry unit weight*

$$\rho = \frac{M}{V}$$

and

$$\rho_d = \frac{M_s}{V}$$

where ρ = density of soil (kg/m^3)
ρ_d = dry density of soil (kg/m^3)
M = total mass of the soil sample (kg)
M_s = mass of soil solids in the sample (kg)

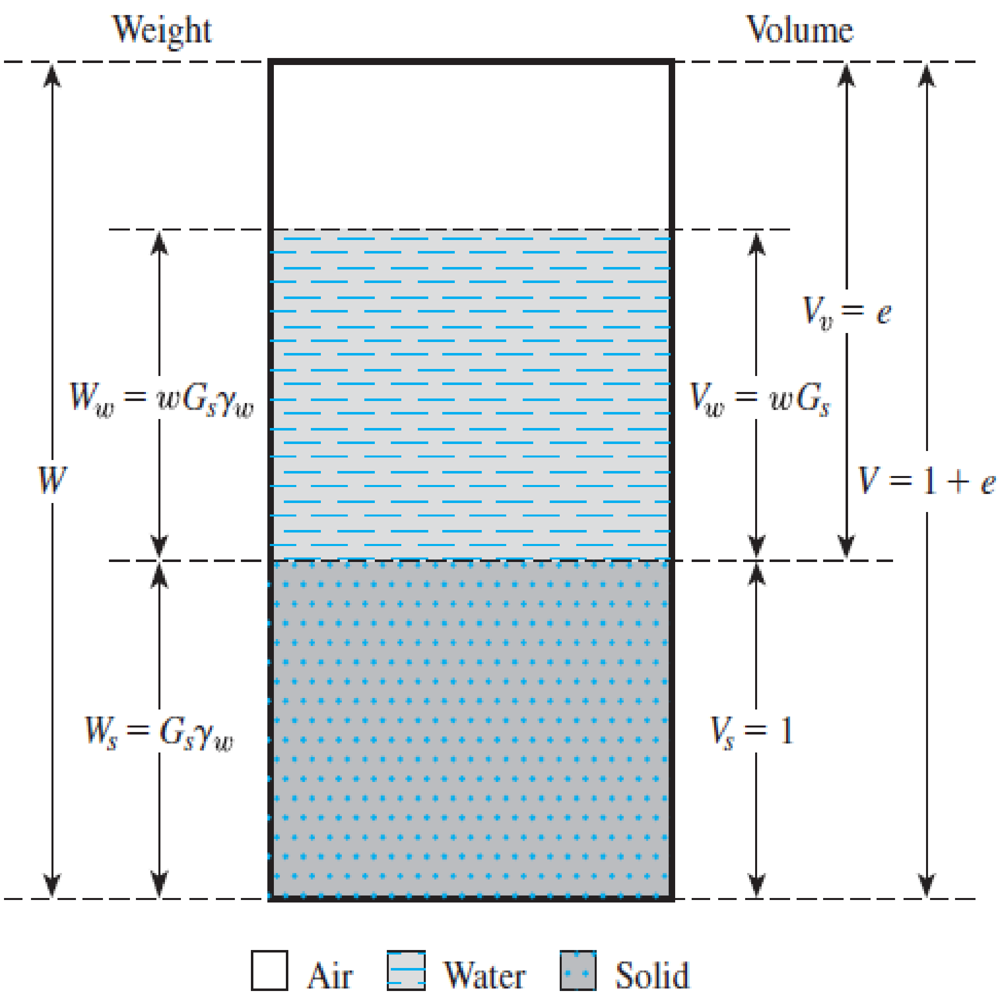


Figure 3.2 Three separate phases of a soil element with volume of soil solids equal to one

$$W_s = G_s\gamma_w$$

$$W_w = wW_s = wG_s\gamma_w$$

where G_s = specific gravity of soil solids
w = moisture content
γ_w = unit weight of water

$$\gamma = \frac{W}{V} = \frac{W_s + W_w}{V} = \frac{G_s\gamma_w + wG_s\gamma_w}{1+e} = \frac{(1+w)G_s\gamma_w}{1+e}$$

$$\gamma_d = \frac{W_s}{V} = \frac{G_s\gamma_w}{1+e}$$

$$e = \frac{G_s\gamma_w}{\gamma_d} - 1$$

$$V_w = \frac{W_w}{\gamma_w} = \frac{wG_s\gamma_w}{\gamma_w} = wG_s$$

$$Se = wG_s$$

$$\gamma_{sat} = \frac{W}{V} = \frac{W_s + W_w}{V} = \frac{G_s\gamma_w + e\gamma_w}{1+e} = \frac{(G_s+e)\gamma_w}{1+e}$$

$$e = wG_s$$

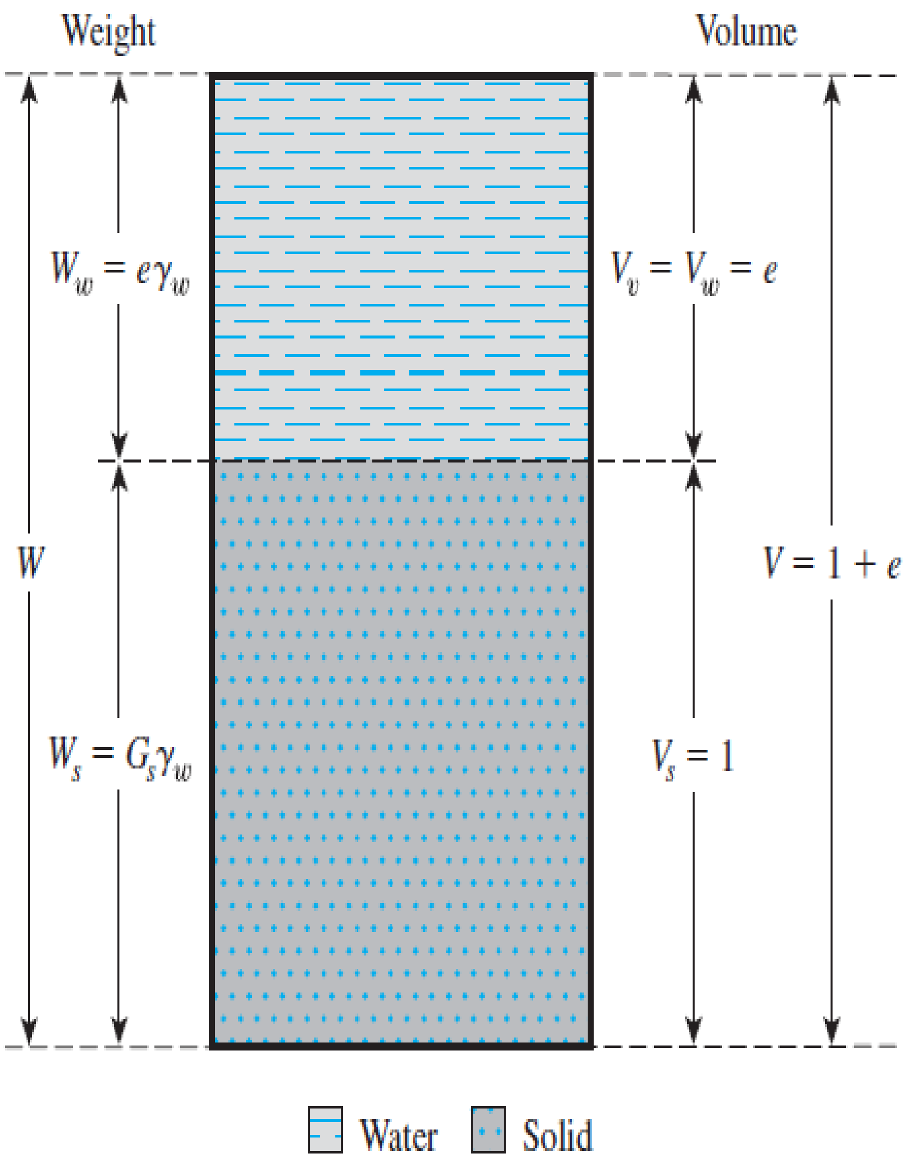


Figure 3.3
Saturated soil element with volume of soil solids equal to one

$$\text{Density} = \rho = \frac{(1 + w)G_s\rho_w}{1 + e}$$

$$\text{Dry density} = \rho_d = \frac{G_s\rho_w}{1 + e}$$

$$\text{Saturated density} = \rho_{\text{sat}} = \frac{(G_s + e)\rho_w}{1 + e}$$

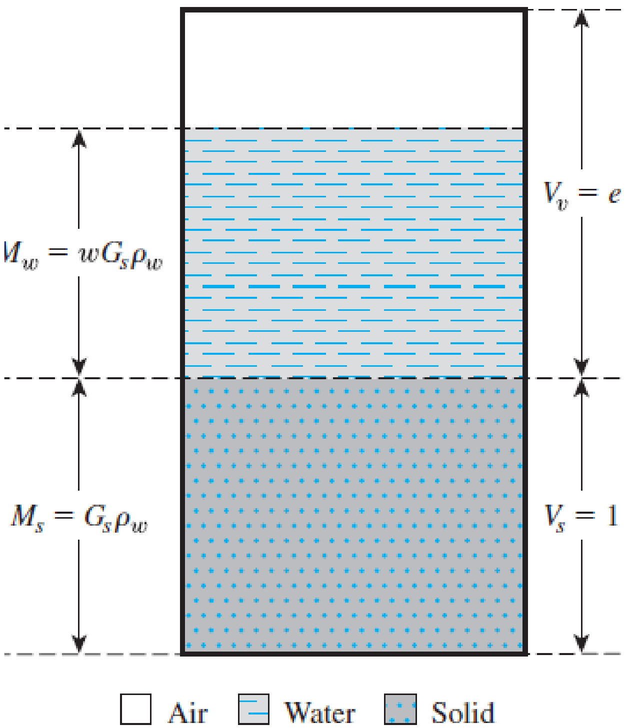


Figure 3.4
Three separate phases of a soil element showing mass–volume relationship

$$w = \frac{W_w}{W_s} = \frac{(\text{mass of water}) \cdot g}{(\text{mass of solid}) \cdot g} = \frac{M_w}{M_s}$$

where M_w = mass of water.

$$M_w = wM_s = wG_s\rho_w$$

$$\rho = \frac{M}{V} = \frac{M_s + M_w}{V_s + V_v} = \frac{G_s\rho_w + wG_s\rho_w}{1 + e} = \frac{(1 + w)G_s\rho_w}{1 + e}$$

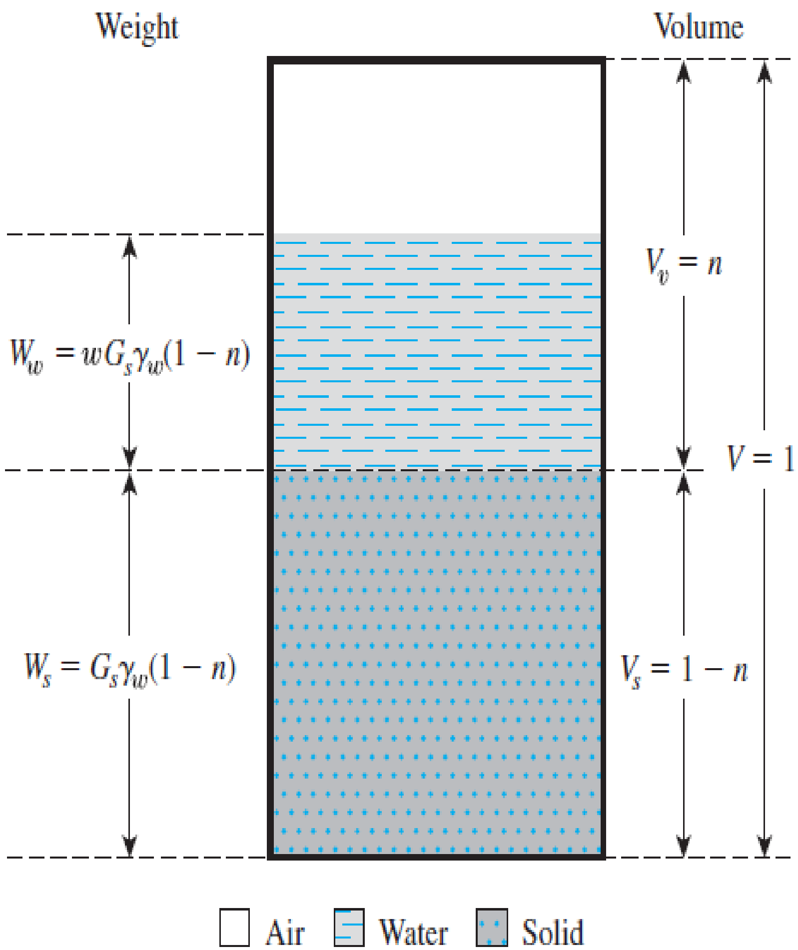


Figure 3.5
Soil element with total volume equal to one

$$W_s = G_s\gamma_w(1 - n)$$

$$W_w = wW_s = wG_s\gamma_w(1 - n)$$

$$\gamma_d = \frac{W_s}{V} = \frac{G_s\gamma_w(1 - n)}{1} = G_s\gamma_w(1 - n)$$

$$\gamma = \frac{W_s + W_w}{V} = G_s\gamma_w(1 - n)(1 + w)$$

$$\gamma_{\text{sat}} = \frac{W_s + W_w}{V} = \frac{(1 - n)G_s\gamma_w + n\gamma_w}{1} = [(1 - n)G_s + n]\gamma_w$$

$$w = \frac{W_w}{W_s} = \frac{n\gamma_w}{(1 - n)\gamma_w G_s} = \frac{n}{(1 - n)G_s}$$

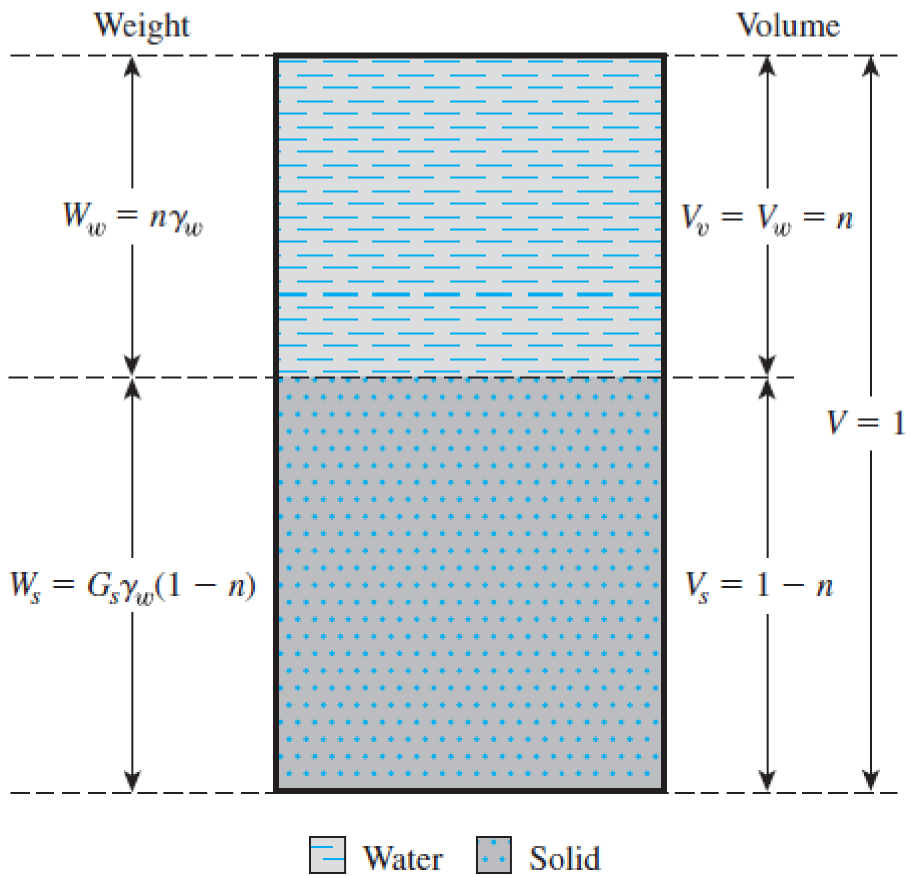


Figure 3.6 Saturated soil element with total volume equal to one

Table 3.1 Various Forms of Relationships for γ, γ_d, and γ_{sat}

Moist unit weight (γ)		Dry unit weight (γ_d)		Saturated unit weight (γ_{sat})	
Given	**Relationship**	**Given**	**Relationship**	**Given**	**Relationship**
w, G_s, e	$\dfrac{(1+w)G_s\gamma_w}{1+e}$	γ, w	$\dfrac{\gamma}{1+w}$	G_s, e	$\dfrac{(G_s+e)\gamma_w}{1+e}$
S, G_s, e	$\dfrac{(G_s+Se)\gamma_w}{1+e}$	G_s, e	$\dfrac{G_s\gamma_w}{1+e}$	G_s, n	$[(1-n)G_s+n]\gamma_w$
w, G_s, S	$\dfrac{(1+w)G_s\gamma_w}{1+\dfrac{wG_s}{S}}$	G_s, n	$G_s\gamma_w(1-n)$	G_s, w_{sat}	$\left(\dfrac{1+w_{sat}}{1+w_{sat}G_s}\right)G_s\gamma_w$
w, G_s, n	$G_s\gamma_w(1-n)(1+w)$	G_s, w, S	$\dfrac{G_s\gamma_w}{1+\left(\dfrac{wG_s}{S}\right)}$	e, w_{sat}	$\left(\dfrac{e}{w_{sat}}\right)\left(\dfrac{1+w_{sat}}{1+e}\right)\gamma_w$
S, G_s, n	$G_s\gamma_w(1-n)+nS\gamma_w$	e, w, S	$\dfrac{eS\gamma_w}{(1+e)w}$	n, w_{sat}	$n\left(\dfrac{1+w_{sat}}{w_{sat}}\right)\gamma_w$
		γ_{sat}, e	$\gamma_{sat}-\dfrac{e\gamma_w}{1+e}$	γ_d, e	$\gamma_d+\left(\dfrac{e}{1+e}\right)\gamma_w$
		γ_{sat}, n	$\gamma_{sat}-n\gamma_w$	γ_d, n	$\gamma_d+n\gamma_w$
		γ_{sat}, G_s	$\dfrac{(\gamma_{sat}-\gamma_w)G_s}{(G_s-1)}$	γ_d, S	$\left(1-\dfrac{1}{G_s}\right)\gamma_d+\gamma_w$
				γ_d, w_{sat}	$\gamma_d(1+w_{sat})$

Table 3.2 Void Ratio, Moisture Content, and Dry Unit Weight for Some Typical Soils in a Natural State

Type of soil	Void ratio, e	Natural moisture content in a saturated state (%)	Dry unit weight, γ_d	
			lb/ft^3	kN/m^3
Loose uniform sand	0.8	30	92	14.5
Dense uniform sand	0.45	16	115	18
Loose angular-grained silty sand	0.65	25	102	16
Dense angular-grained silty sand	0.4	15	121	19
Stiff clay	0.6	21	108	17
Soft clay	0.9–1.4	30–50	73–93	11.5–14.5
Loess	0.9	25	86	13.5
Soft organic clay	2.5–3.2	90–120	38–51	6–8
Glacial till	0.3	10	134	21

Example 3.2

For a moist soil sample, the following are given.

- Total volume: $V = 1.2\ \text{m}^3$
- Total mass: $M = 2350$ kg
- Moisture content: $w = 8.6\%$
- Specific gravity of soil solids: $G_s = 2.71$

Determine the following.

a. Moist density
b. Dry density
c. Void ratio
d. Porosity
e. Degree of saturation
f. Volume of water in the soil sample

Solution

Part a

From Eq. (3.13),

$$\rho = \frac{M}{V} = \frac{2350}{1.2} = \mathbf{1958.3\ kg/m^3}$$

Part b

From Eq. (3.14),

$$\rho_d = \frac{M_s}{V} = \frac{M}{(1 + w)V} = \frac{2350}{\left(1 + \frac{8.6}{100}\right)(1.2)} = \mathbf{1803.3\ kg/m^3}$$

Part c

From Eq. (3.22),

$$\rho_d = \frac{G_s \rho_w}{1 + e}$$

$$e = \frac{G_s \rho_w}{\rho_d} - 1 = \frac{(2.71)(1000)}{1803.3} - 1 = \mathbf{0.503}$$

Part d

From Eq. (3.7),

$$n = \frac{e}{1 + e} = \frac{0.503}{1 + 0.503} = \mathbf{0.335}$$

Part e

From Eq. (3.18),

$$S = \frac{wG_s}{e} = \frac{\left(\frac{8.6}{100}\right)(2.71)}{0.503} = 0.463 = \mathbf{46.3\%}$$

Part f

Volume of water:

$$\frac{M_w}{\rho_w} = \frac{M - M_s}{\rho_w} = \frac{M - \frac{M}{1 + w}}{\rho_w} = \frac{2350 - \left(\frac{2350}{1 + \frac{8.6}{100}}\right)}{1000} = \mathbf{0.186\ m^3}$$

Alternate Solution

Refer to Figure 3.7.

Part a

$$\rho = \frac{M}{V} = \frac{2350}{1.2} = \mathbf{1958.3\ kg/m^3}$$

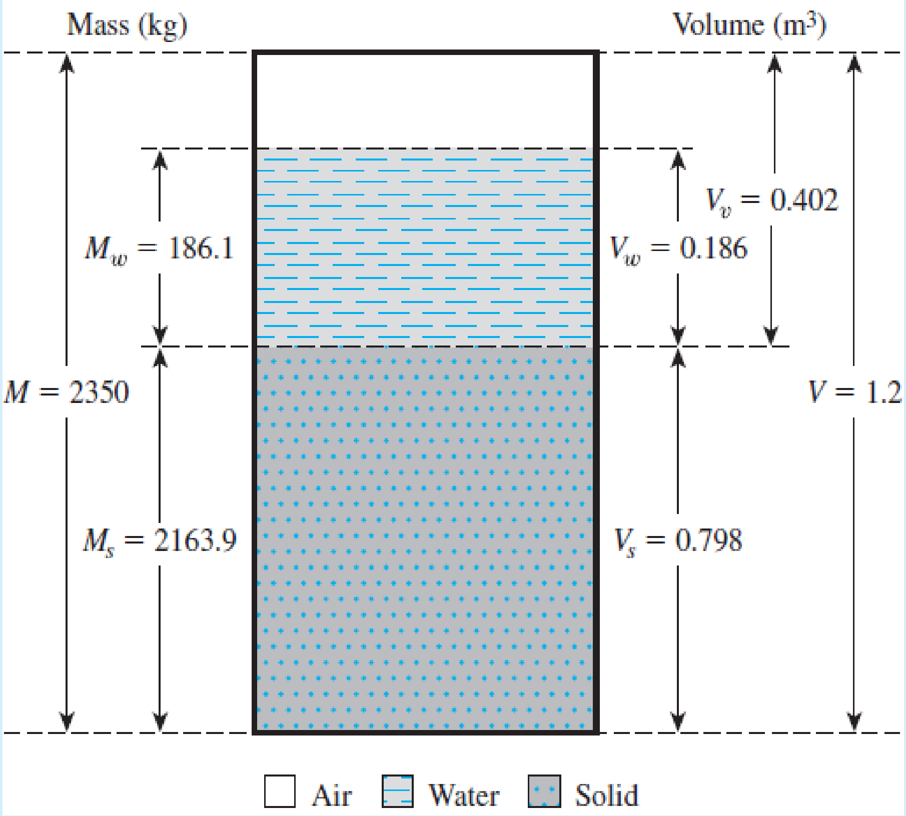


Figure 3.7

Part b

$$M_s = \frac{M}{1 + w} = \frac{2350}{1 + \frac{8.6}{100}} = 2163.9 \text{ kg}$$

$$\rho_d = \frac{M_s}{V} = \frac{M}{(1 + w)V} = \frac{2350}{\left(1 + \frac{8.6}{100}\right)(1.2)} = \mathbf{1803.3\ kg/m^3}$$

Part c

$$\text{The volume of solids: } \frac{M_s}{G_s\rho_w} = \frac{2163.9}{(2.71)(1000)} = 0.798 \text{ m}^3$$

$$\text{The volume of voids: } V_v = V - V_s = 1.2 - 0.798 = 0.402 \text{ m}^3$$

$$\text{Void ratio: } e = \frac{V_v}{V_s} = \frac{0.402}{0.798} = \mathbf{0.503}$$

Part d

$$\text{Porosity: } n = \frac{V_v}{V} = \frac{0.402}{1.2} = \mathbf{0.335}$$

Part e

$$S = \frac{V_w}{V_v}$$

$$\text{Volume of water: } V_w = \frac{M_w}{\rho_w} = \frac{186.1}{1000} = 0.186 \text{ m}^3$$

Hence,

$$S = \frac{0.186}{0.402} = 0.463 = \mathbf{46.3\%}$$

Part f

From Part e,

$$V_w = \mathbf{0.186\ m^3}$$

Relative Density

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}}$$

where D_r = relative density, usually given as a percentage
e = *in situ* void ratio of the soil
e_{max} = void ratio of the soil in the loosest state
e_{min} = void ratio of the soil in the densest state

$$e_{max} = \frac{n_{max}}{1 - n_{max}}$$

$$e_{min} = \frac{n_{min}}{1 - n_{min}}$$

Table 3.3 Qualitative Description of Granular Soil Deposits

Relative density (%)	Description of soil deposit
0–15	Very loose
15–50	Loose
50–70	Medium
70–85	Dense
85–100	Very dense

Example 3.5

For a given sandy soil, $e_{max} = 0.75$ and $e_{min} = 0.4$. Let $G_s = 2.68$. In the field, the soil is compacted to a moist density of 112 lb/f^3 at a moisture content of 12%. Determine the relative density of compaction.

Solution

From Eq. (3.21),

$$\rho = \frac{(1 + w)G_s\gamma_w}{1 + e}$$

or

$$e = \frac{G_s\gamma_w(1 + w)}{\gamma} - 1 = \frac{(2.68)(62.4)(1 + 0.12)}{112} - 1 = 0.67$$

From Eq. (3.30),

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} = \frac{0.75 - 0.67}{0.75 - 0.4} = \mathbf{0.229 = 22.9\%}$$

Comments on e_{max} and e_{min}

Depend on several factors, such as

- Grain size
- Grain shape
- Nature of the grain-size distribution curve
- Fine contents, F_c (that is, fraction smaller than 0.075 mm)

$$e_{max} - e_{min} = 0.23 + \frac{0.06}{D_{50}\ (\text{mm})}$$

Soil Mechanics

Plasticity and Structure of Soil

Dr Jasim M Abbas Al Shamary

Plasticity and Structure of Soil

The moisture content, in percent, at which the transition from solid to semisolid state takes place is defined as the *shrinkage limit*. The moisture content at the point of transition from semisolid to plastic state is the *plastic limit*, and from plastic to liquid state is the *liquid limit*.

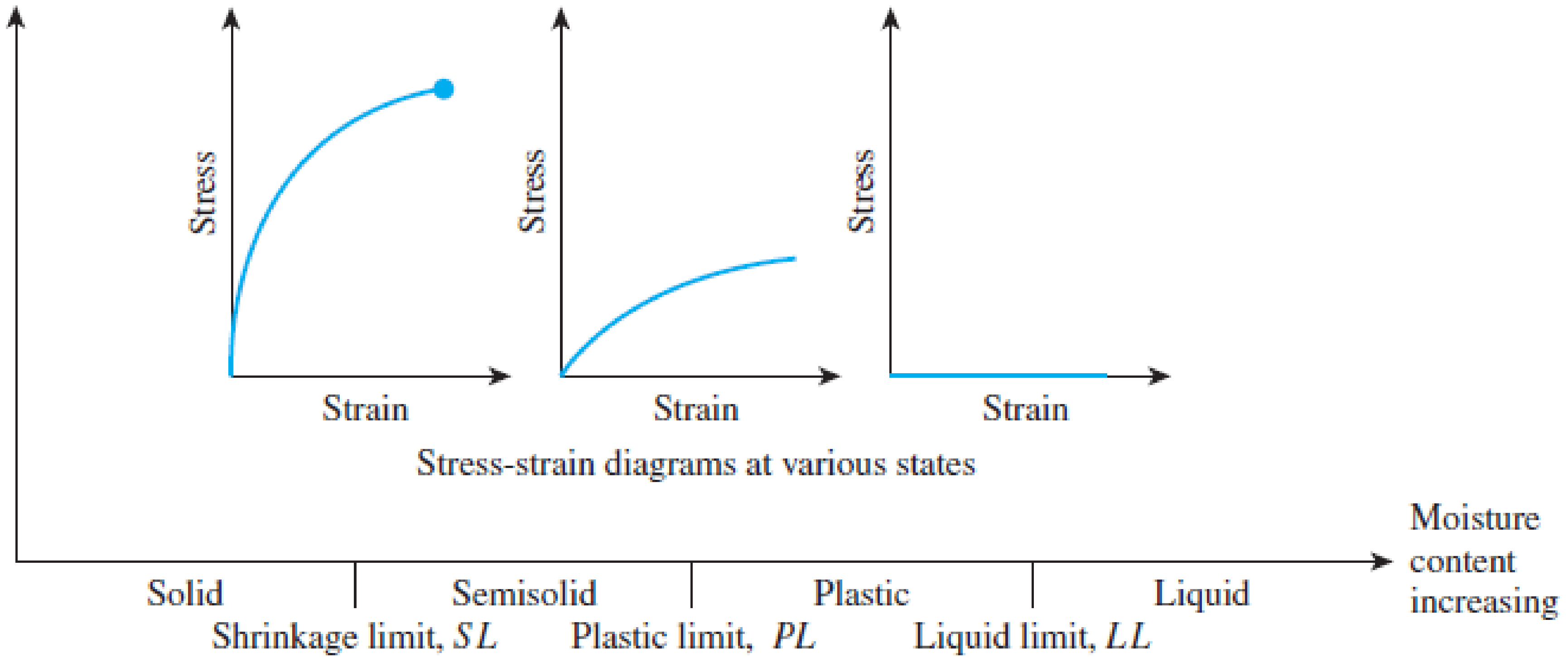
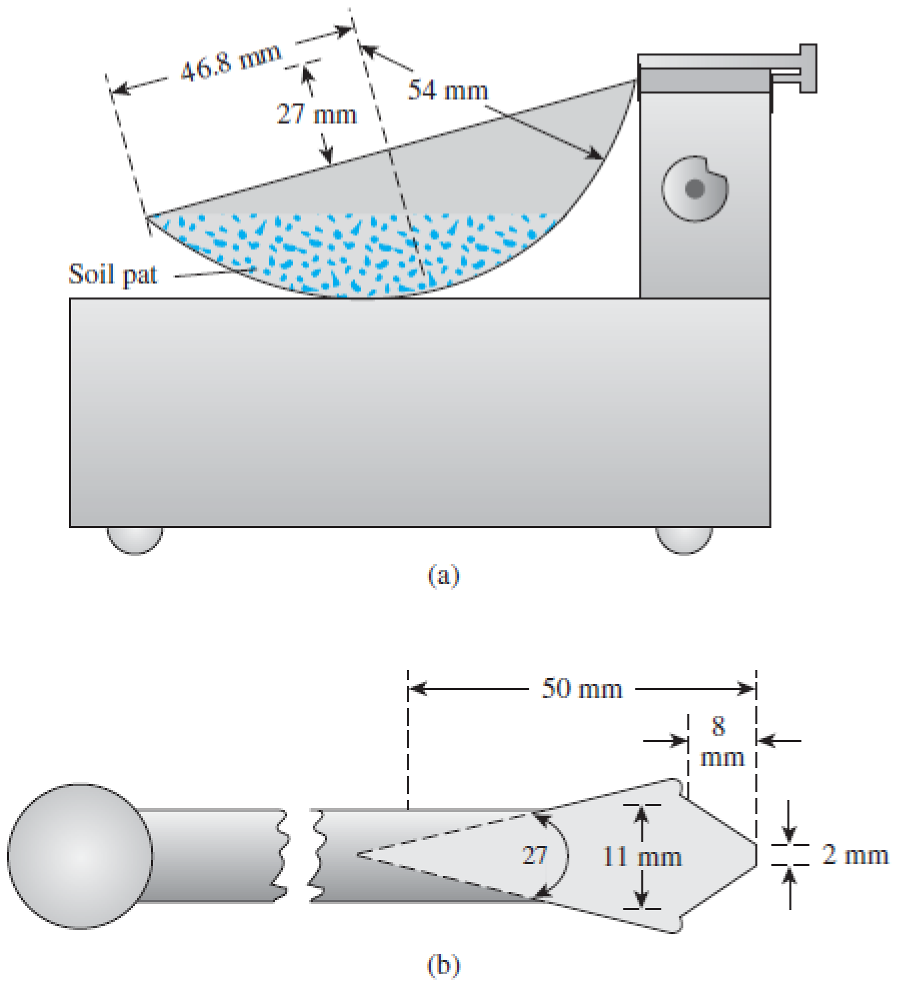


Figure 4.1 Atterberg limits

Liquid Limit (LL)



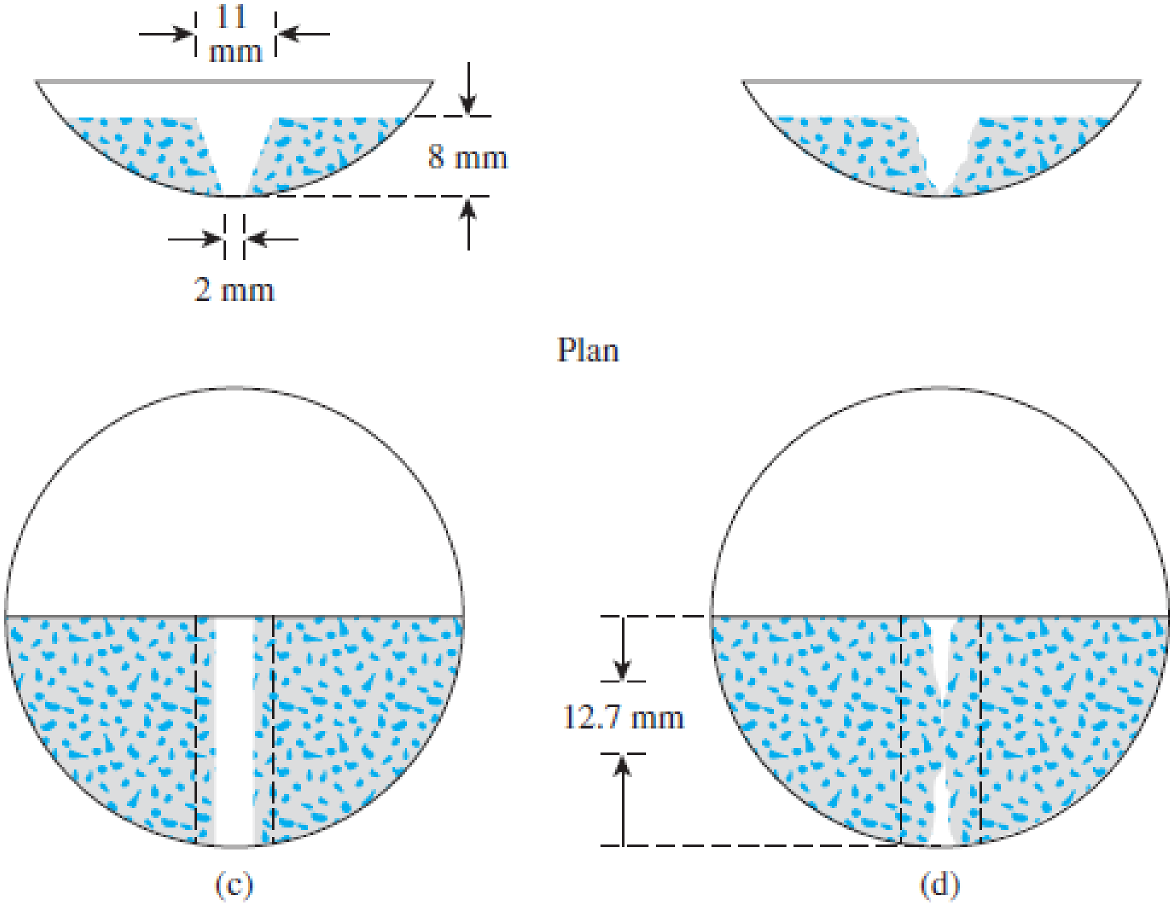


Figure 4.2 Liquid limit test: (a) liquid limit device; (b) grooving tool; (c) soil pat before test; (d) soil pat after test

$$I_F = \frac{w_1 - w_2}{\log\left(\frac{N_2}{N_1}\right)}$$

where I_F = flow index
w_1 = moisture content of soil, in percent, corresponding to N_1 blows
w_2 = moisture content corresponding to N_2 blows

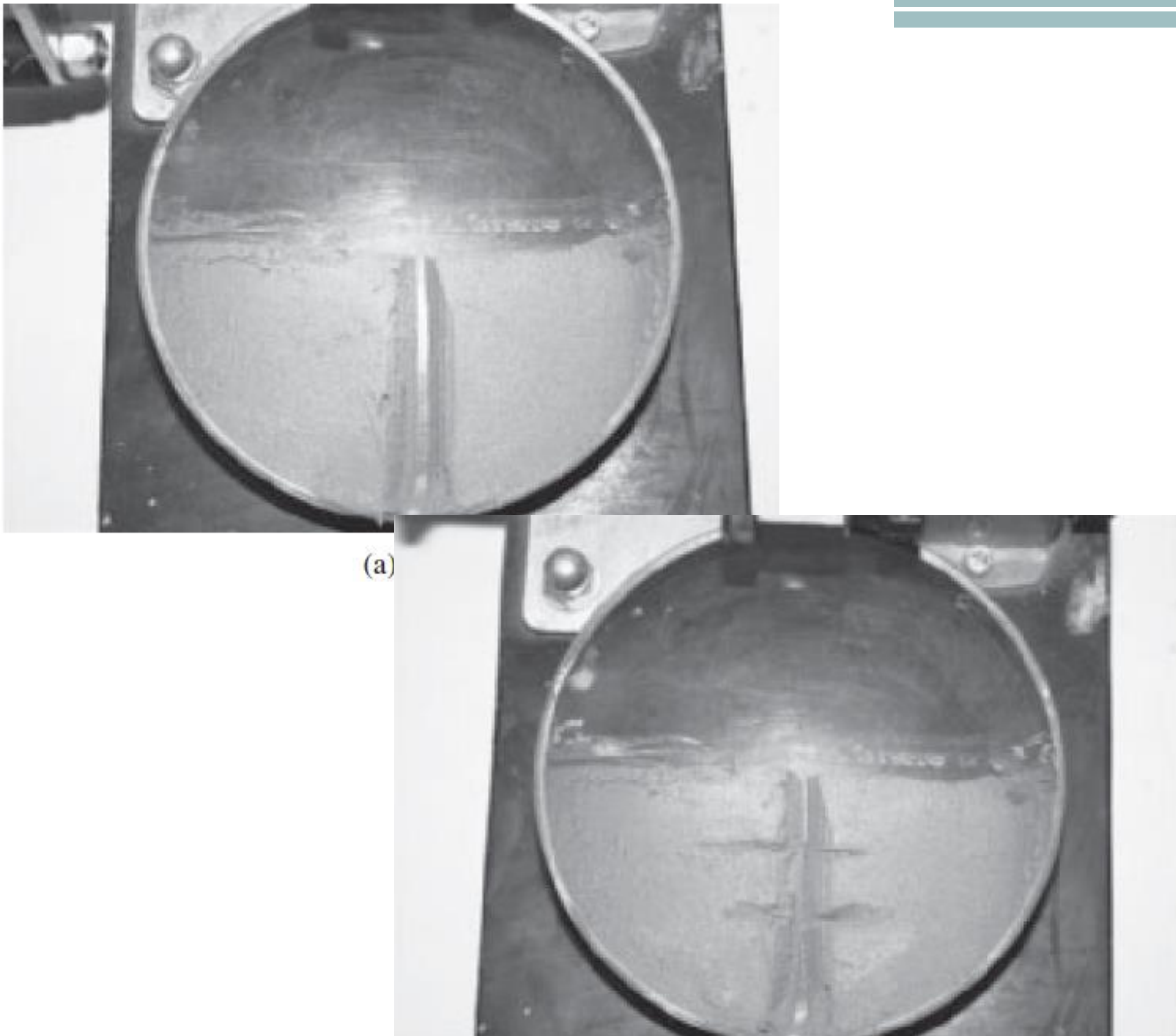
$$w = -I_F \log N + C$$

where C = a constant.

$$LL = w_N\left(\frac{N}{25}\right)^{\tan\beta}$$

where N = number of blows in the liquid limit device for a 12.7 mm (0.5 in.) groove closure
w_N = corresponding moisture content
$\tan\beta$ = 0.121 (but note that $\tan\beta$ is not equal to 0.121 for all soils)

Figure 4.3 Liquid limit test device and grooving tools (*Courtesy of ELE International*)



(a)

Figure 4.4
Photographs showing the soil pat in the liquid limit device: (a) before test; (b) after test [*Note*: The half-inch groove closure in (b) is marked for clarification] (*Courtesy of Braja M. Das, Henderson,*

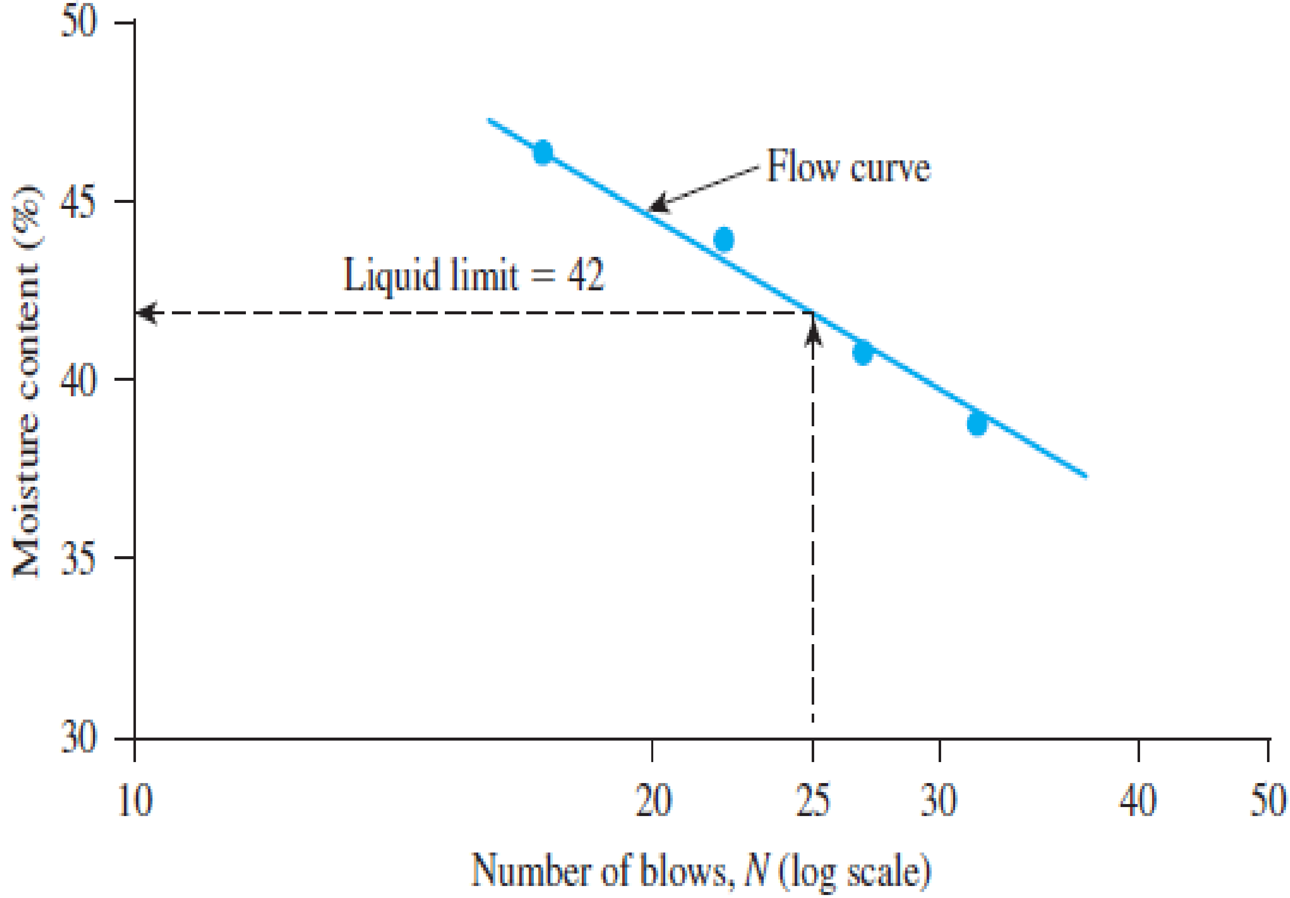
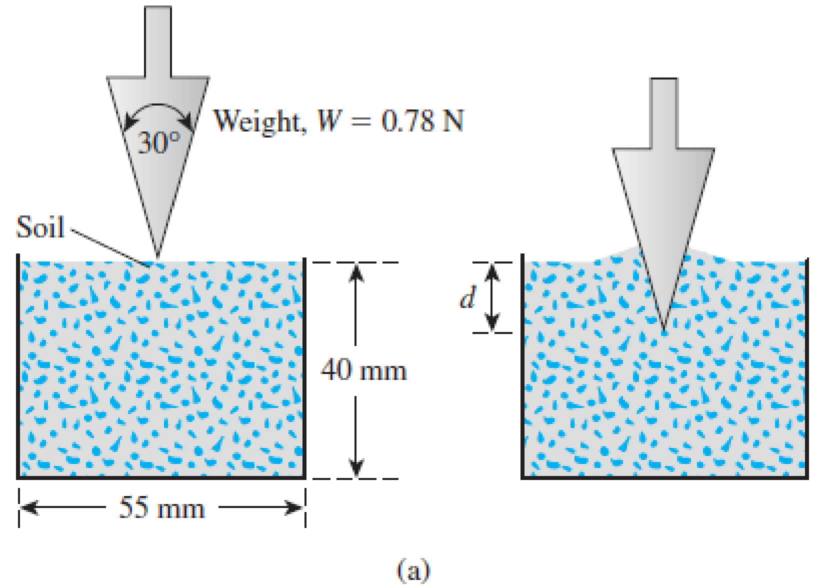


Figure 4.5
Flow curve for liquid limit determination of a clayey silt

$$I_{FC} = \frac{w_2(\%) - w_1(\%)}{\log d_2 - \log d_1}$$

where w_1, w_2 = moisture contents at cone penetrations of d_1 and d_2, respectively.



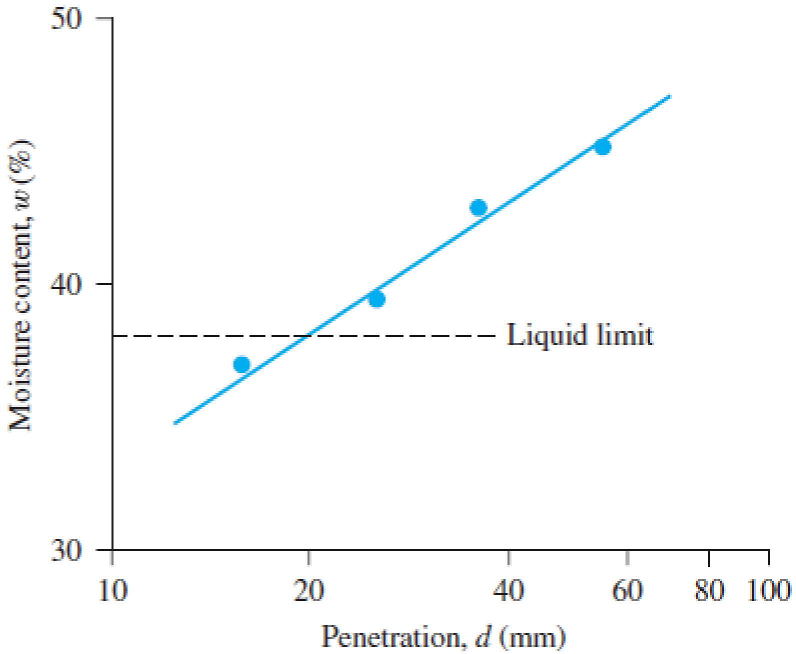


Figure 4.6
(a) Fall cone test (b) plot of moisture content vs. cone penetration for

Plastic Limit (PL)

As in the case of liquid limit determination, the fall cone method can be used to obtain the plastic limit. This can be achieved by using a cone of similar geometry but with a mass of 2.35 N (240 gf). Three to four tests at varying moisture contents of soil are conducted, and the corresponding cone penetrations (d) are determined. The moisture content corresponding to a cone penetration of $d = 20$ mm is the plastic limit. Figure 4.8 shows the liquid and plastic limit determination of Cambridge Gault clay reported by Wroth and Wood (1978).

Figure 4.7 Rolling of soil mass on ground glass plate to determine plastic limit (*Courtesy of Braja M. Das, Henderson, Nevada*)

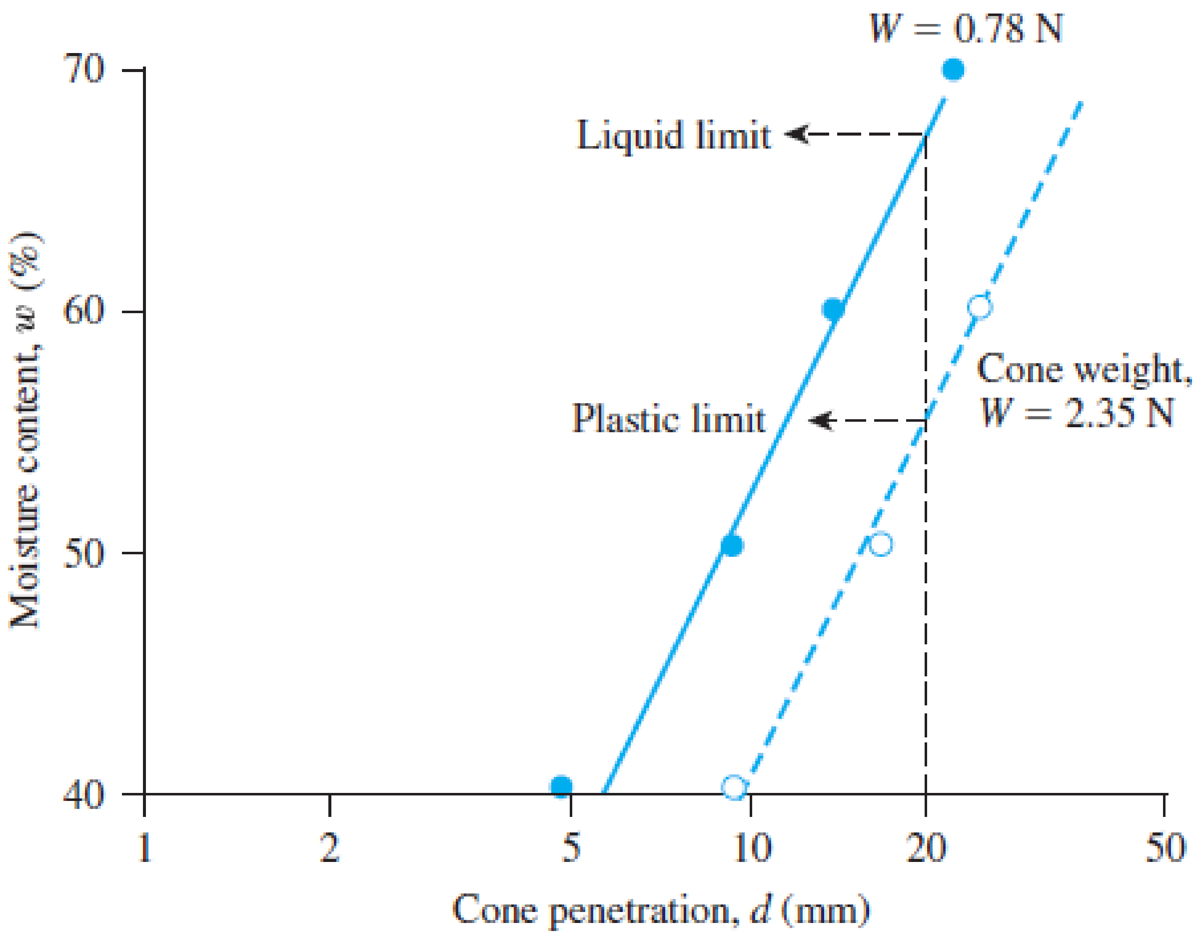


Figure 4.8
Liquid and plastic limits for Cambridge Gault clay determined by fall cone test

$PI = LL - PL$

plasticity index (PI)

Table 4.1 Typical Values of Liquid Limit, Plastic Limit, and Activity of Some Clay Minerals

Mineral	Liquid limit, *LL*	Plastic limit, *PL*	Activity, *A*
Kaolinite	35–100	20–40	0.3–0.5
Illite	60–120	35–60	0.5–1.2
Montmorillonite	100–900	50–100	1.5–7.0
Halloysite (hydrated)	50–70	40–60	0.1–0.2
Halloysite (dehydrated)	40–55	30–45	0.4–0.6
Attapulgite	150–250	100–125	0.4–1.3
Allophane	200–250	120–150	0.4–1.3

PI	Description
0	Nonplastic
1–5	Slightly plastic
5–10	Low plasticity
10–20	Medium plasticity
20–40	High plasticity
>40	Very high plasticity

$$PI\ (\%) = 4.12 I_F (\%)$$

$$PI\ (\%) = 0.74 I_{FC} (\%)$$

$$PL = 0.04(LL) + 0.26(CF) + 10$$

$$PI = 0.96(LL) - 0.26(CF) - 10$$

where CF = clay fraction ($<2\ \mu$m) in %. The experimental results of Polidori (2007) show that the preceding relationships hold good for CF approximately equal to or greater than 30%.

Shrinkage Limit (SL)

$$SL = w_i(\%) - \Delta w(\%) \qquad (4$$

where w_i = initial moisture content when the soil is placed in the shrinkage limit dish
Δw = change in moisture content (that is, between the initial moisture content and the moisture content at the shrinkage limit)

$$w_i(\%) = \frac{M_1 - M_2}{M_2} \times 100$$

where M_1 = mass of the wet soil pat in the dish at the beginning of the test (g)
M_2 = mass of the dry soil pat (g) (see Figure 4.10)

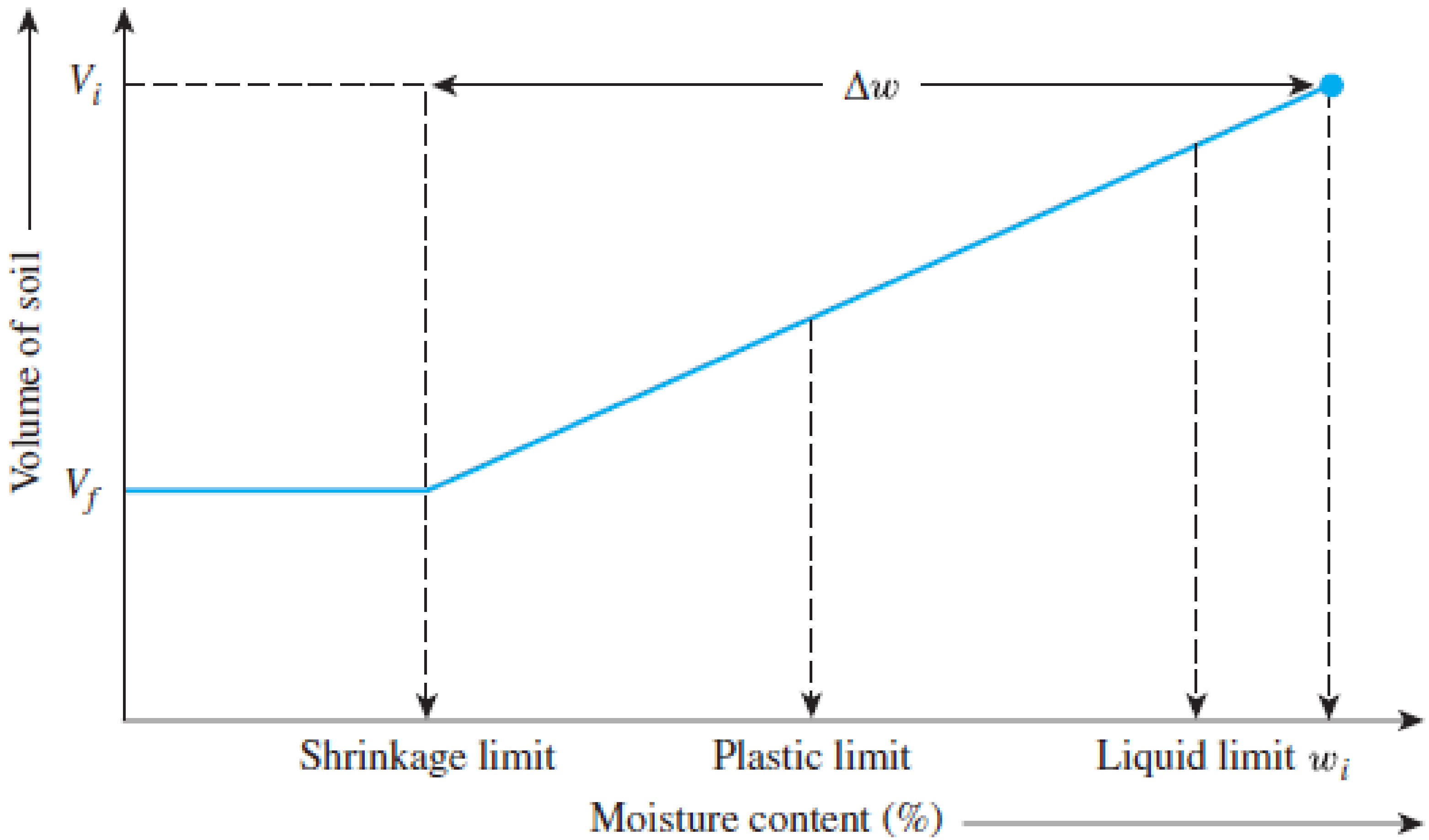


Figure 4.9 Definition of shrinkage limit

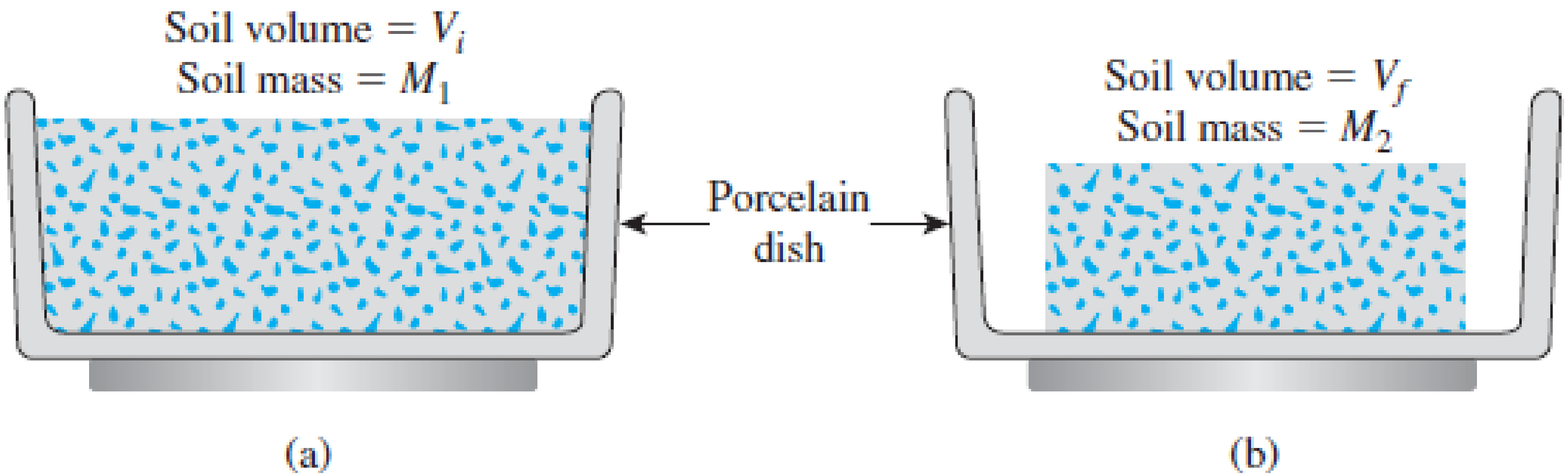


Figure 4.10 Shrinkage limit test: (a) soil pat before drying; (b) soil pat after drying

Also,

$$\Delta w\,(\%) = \frac{(V_i - V_f)\rho_w}{M_2} \times 100$$

where V_i = initial volume of the wet soil pat (that is, inside volume of the dish, cm^3)

V_f = volume of the oven-dried soil pat (cm^3)

ρ_w = density of water (g/cm^3)

$$SL = \left(\frac{M_1 - M_2}{M_2}\right)(100) - \left(\frac{V_i - V_f}{M_2}\right)(\rho_w)(100)$$

$$SR = \frac{\left(\frac{\Delta V}{V_f}\right)}{\left(\frac{\Delta M}{M_2}\right)} = \frac{\left(\frac{\Delta V}{V_f}\right)}{\left(\frac{\Delta V \rho_w}{M_2}\right)} = \frac{M_2}{V_f \rho_w}$$

where ΔV = change in volume
ΔM = corresponding change in the mass of moisture

$$G_s = \frac{1}{\frac{1}{SR} - \left(\frac{SL}{100}\right)}$$

where G_s = specific gravity of soil solids.

Example 4.1

Following are the results of a shrinkage limit test:

- Initial volume of soil in a saturated state = 24.6 cm^3
- Final volume of soil in a dry state = 15.9 cm^3
- Initial mass in a saturated state = 44.0 g
- Final mass in a dry state = 30.1 g

Determine the shrinkage limit of the soil.

Solution

From Eq. (4.13),

$$SL = \left(\frac{M_1 - M_2}{M_2}\right)(100) - \left(\frac{V_i - V_f}{M_2}\right)(\rho_w)(100)$$

$$M_1 = 44.0\text{g} \qquad V_i = 24.6\text{ cm}^3 \qquad \rho_w = 1\text{ g/cm}^3$$

$$M_2 = 30.1\text{g} \qquad V_f = 15.9\text{ cm}^3$$

$$SL = \left(\frac{44.0 - 30.1}{30.1}\right)(100) - \left(\frac{24.6 - 15.9}{30.1}\right)(1)(100)$$

$$= 46.18 - 28.9 = \mathbf{17.28\%}$$

■

Mineral	Shrinkage limit
Montmorillonite	8.5–15
Illite	15–17
Kaolinite	25–29

Liquidity Index and Consistency Index

$$LI = \frac{w - PL}{LL - PL}$$

where w = *in situ* moisture content of soil.

$LI > 1$ sensitive clay

$LI < 0$ overconsolidated

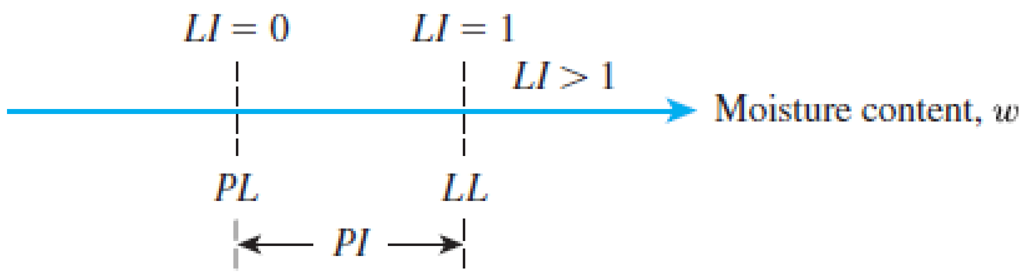


Figure 4.11 Liquidity index

$$CI = \frac{LL - w}{LL - PI}$$ *consistency index (CI)*

where w = *in situ* moisture content. If w is equal to the liquid limit, the consistency index is zero. Again, if $w = PI$, then $CI = 1$.

Plasticity Chart

- The important feature of this chart is the empirical *A*-line that is given by the equation $PI = 0.73(LL - 20)$.
- An *A*-line separates the inorganic clays from the inorganic silts.
- Inorganic clay values lie above the *A*-line, and values for inorganic silts lie below the *A*-line.
- Organic silts plot in the same region (below the *A*-line and with *LL* ranging from 30 to 50) as the inorganic silts of medium compressibility.
- Organic clays plot in the same region as inorganic silts of high compressibility (below the *A*-line and *LL* greater than 50).

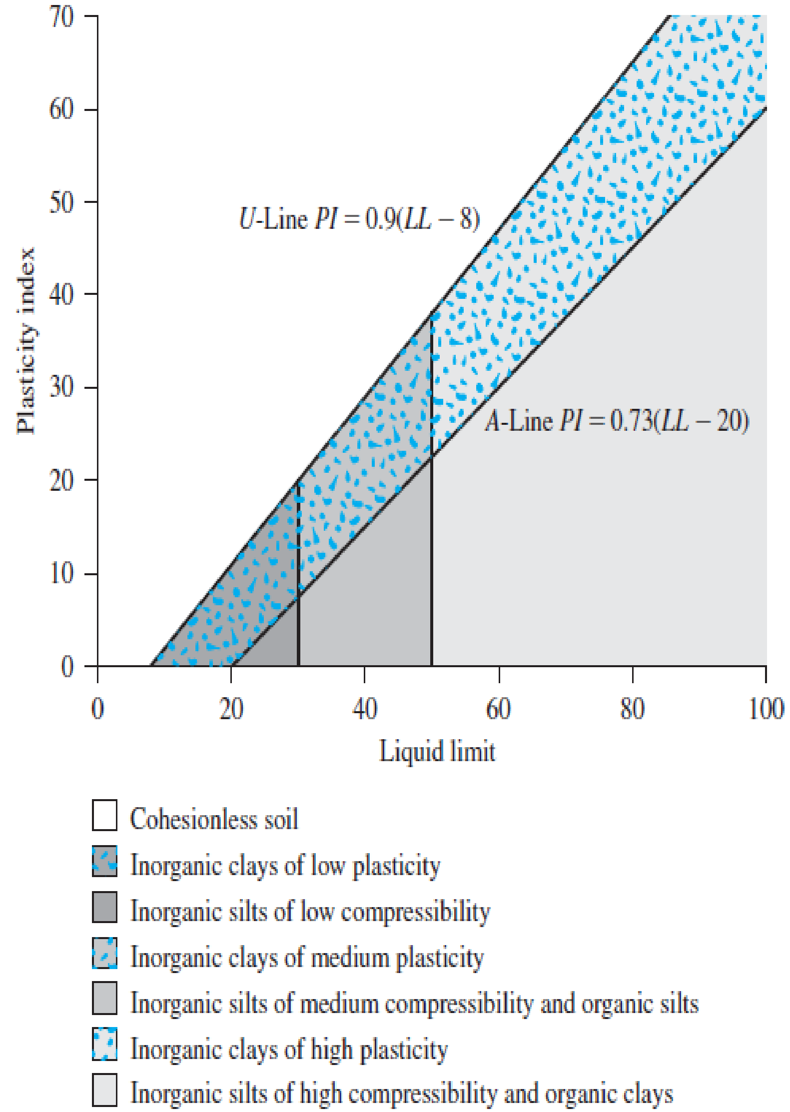


Figure 4.16 Pl chart

Soil Structure

Soil structure is defined as the geometric arrangement of soil particles with respect to one another. Among the many factors that affect the structure are the shape, size, and mineralogical composition of soil particles, and the nature and composition of soil water. In general, soils can be placed into two groups: cohesionless and cohesive. The structures found in soils in each group are described next.

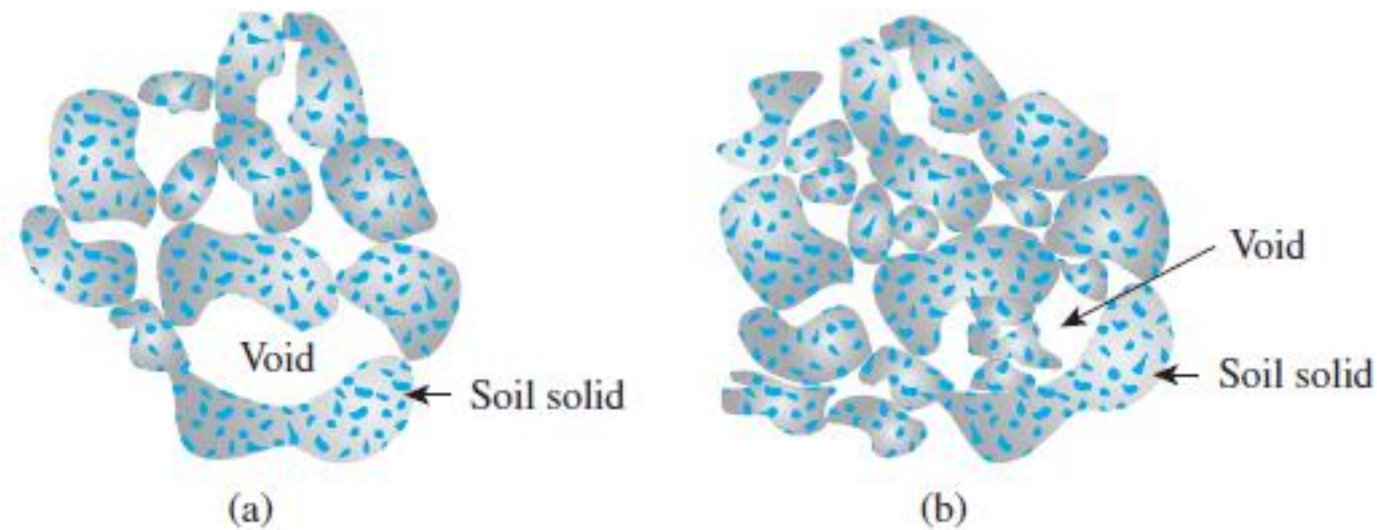


Figure 4.18 Single-grained structure: (a) loose; (b) dense

Structures in Cohesionless Soil

The structures generally encountered in cohesionless soils can be divided into two major categories: *single grained* and *honeycombed*.

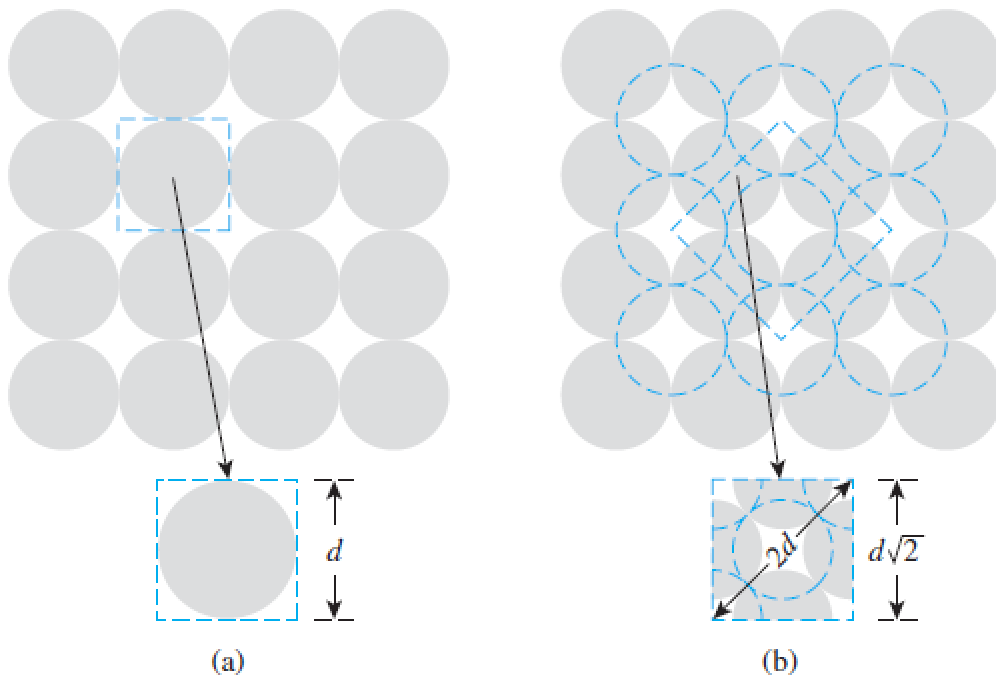


Figure 4.19 Mode of packing of equal spheres (plan views): (a) very loose packing ($e = 0.91$); (b) very dense packing ($e = 0.35$)

Structures in Cohesive Soils

The negative charge on the surface of the clay particles and the diffuse double layer surrounding each particle. When two clay particles in suspension come close to each other, the tendency for interpenetration of the diffuse double layers results in repulsion between the particles.

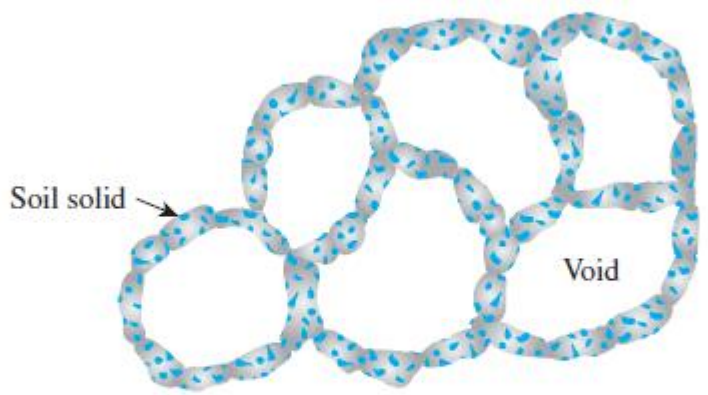


Figure 4.20 Honeycombed structure

Soil Mechanics

Classification of Soil

Dr Jasim M Abbas Al Shamary

Textural Classification

In a general sense, *texture* of soil refers to its surface appearance. Soil texture is influenced by the size of the individual particles present in it.

- *Sand size:* 2.0 to 0.05 mm in diameter
- *Silt size:* 0.05 to 0.002 mm in diameter
- *Clay size:* smaller than 0.002 mm in diameter

USDA system

$$\text{Modified \% sand} = \frac{\%\text{sand}}{100 - \%\text{gravel}} \times 100$$

$$\text{Modified \% silt} = \frac{\%\text{silt}}{100 - \%\text{gravel}} \times 100$$

$$\text{Modified \% clay} = \frac{\%\text{clay}}{100 - \%\text{gravel}} \times 100$$

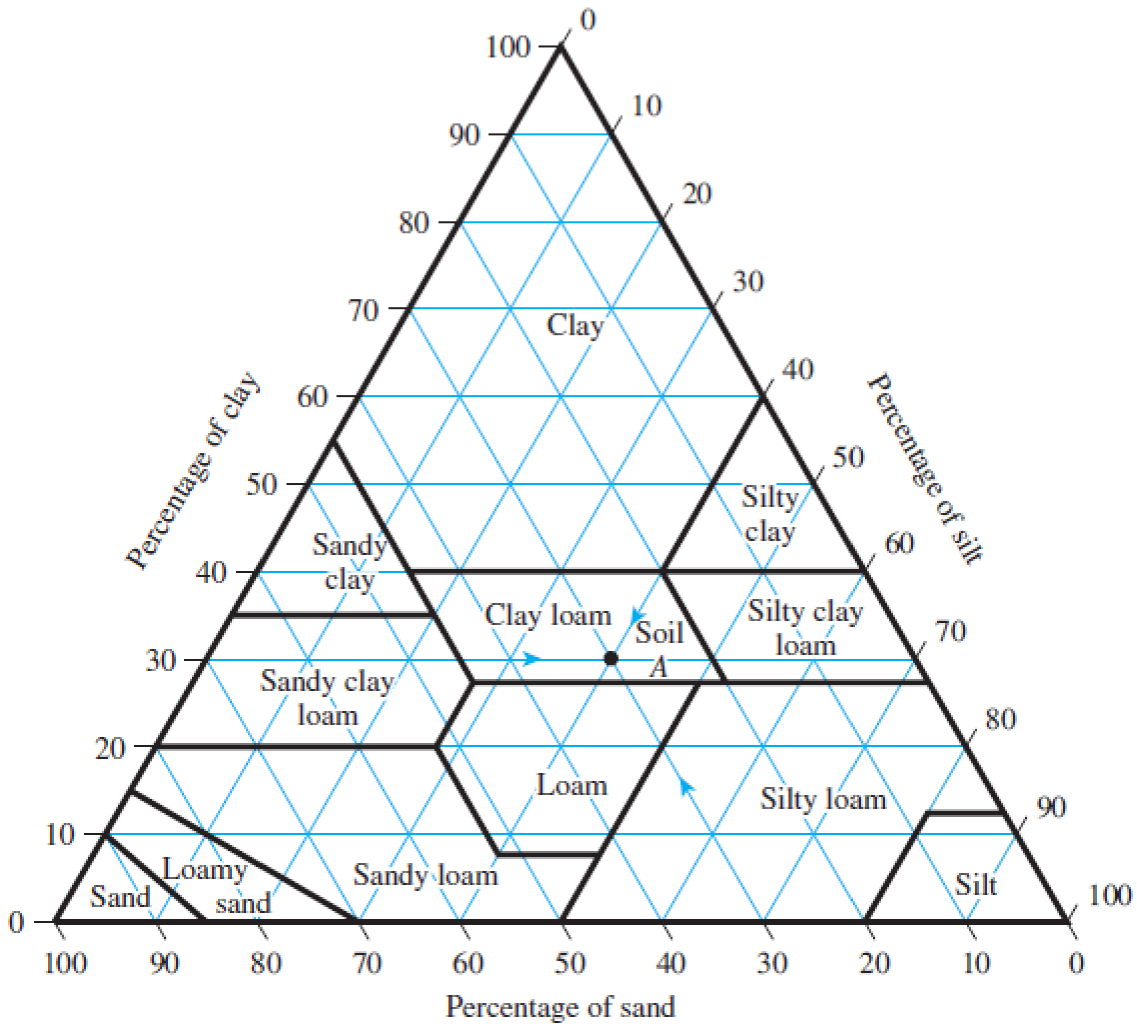


Figure 5.1 U.S. Department of Agriculture textural classification

Example 5.1

Classify the following soils according to the USDA textural classification system.

Particle-size distribution (%)	Soil			
	A	*B*	*C*	*D*
Gravel	10	21	0	12
Sand	20	12	18	22
Silt	41	35	24	26
Clay	29	32	58	40

Solution

Step 1. Calculate the modified percentages of sand, gravel, and silt as follows:

$$\text{Modified \% sand} = \frac{\%\text{sand}}{100 - \%\text{gravel}} \times 100$$

$$\text{Modified \% silt} = \frac{\%\text{silt}}{100 - \%\text{gravel}} \times 100$$

$$\text{Modified \% clay} = \frac{\%\text{clay}}{100 - \%\text{gravel}} \times 100$$

Thus, the following table results:

Particle-size distribution (%)	Soil			
	A	*B*	*C*	*D*
Sand	22.2	15.2	18	25
Silt	45.6	44.3	24	29.5
Clay	32.2	40.5	58	45.5

Step 2. With the modified composition calculated, refer to Figure 5.1 to determine the zone into which each soil falls. The results are as follows:

Classification of soil			
A	***B***	***C***	***D***
Gravelly clay loam	Gravelly silty clay	Clay	Gravelly clay

Note: The word *gravelly* was added to the classification of soils *A*, *B*, and *D* because of the large percentage of gravel present in each.

Classification by Engineering Behavior

AASHTO Classification System

Proposed by the Committee on Classification of Materials for Subgrades and Granular Type Roads of the Highway Research Board in 1945 (ASTM designation D-3282; AASHTO method M145).

This classification system is based on the following criteria:

1. *Grain size*
   a. *Gravel:* fraction passing the 75-mm (3-in.) sieve and retained on the No. 10 (2-mm) U.S. sieve
   b. *Sand:* fraction passing the No. 10 (2-mm) U.S. sieve and retained on the No. 200 (0.075-mm) U.S. sieve
   c. *Silt and clay:* fraction passing the No. 200 U.S. sieve
2. *Plasticity:* The term *silty* is applied when the fine fractions of the soil have a plasticity index of 10 or less. The term *clayey* is applied when the fine fractions have a plasticity index of 11 or more.
3. If cobbles and *boulders* (size larger than 75 mm) are encountered, they are excluded from the portion of the soil sample from which classification is made. However, the percentage of such material is recorded.

Table 5.1 Classification of Highway Subgrade Materials

General classification	Granular materials (35% or less of total sample passing No. 200)						
	A-1			A-2			
Group classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7
Sieve analysis (percentage passing)							
No. 10	50 max.						
No. 40	30 max.	50 max.	51 min.				
No. 200	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.
Characteristics of fraction passing No. 40							
Liquid limit				40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		NP	10 max.	10 max.	11 min.	11 min.
Usual types of significant constituent materials	Stone fragments, gravel, and sand		Fine sand	Silty or clayey gravel and sand			
General subgrade rating	Excellent to good						

General classification	Silt-clay materials (more than 35% of total sample passing No. 200)			
Group classification	A-4	A-5	A-6	A-7 A-7-5[a] A-7-6[b]
Sieve analysis (percentage passing)				
No. 10				
No. 40				
No. 200	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 40				
Liquid limit	40 max.	41 min.	40 max.	41 min.
Plasticity index	10 max.	10 max.	11 min.	11 min.
Usual types of significant constituent materials	Silty soils		Clayey soils	
General subgrade rating	Fair to poor			

[a] For A-7-5, $PI \leq LL - 30$

[b] For A-7-6, $PI > LL - 30$

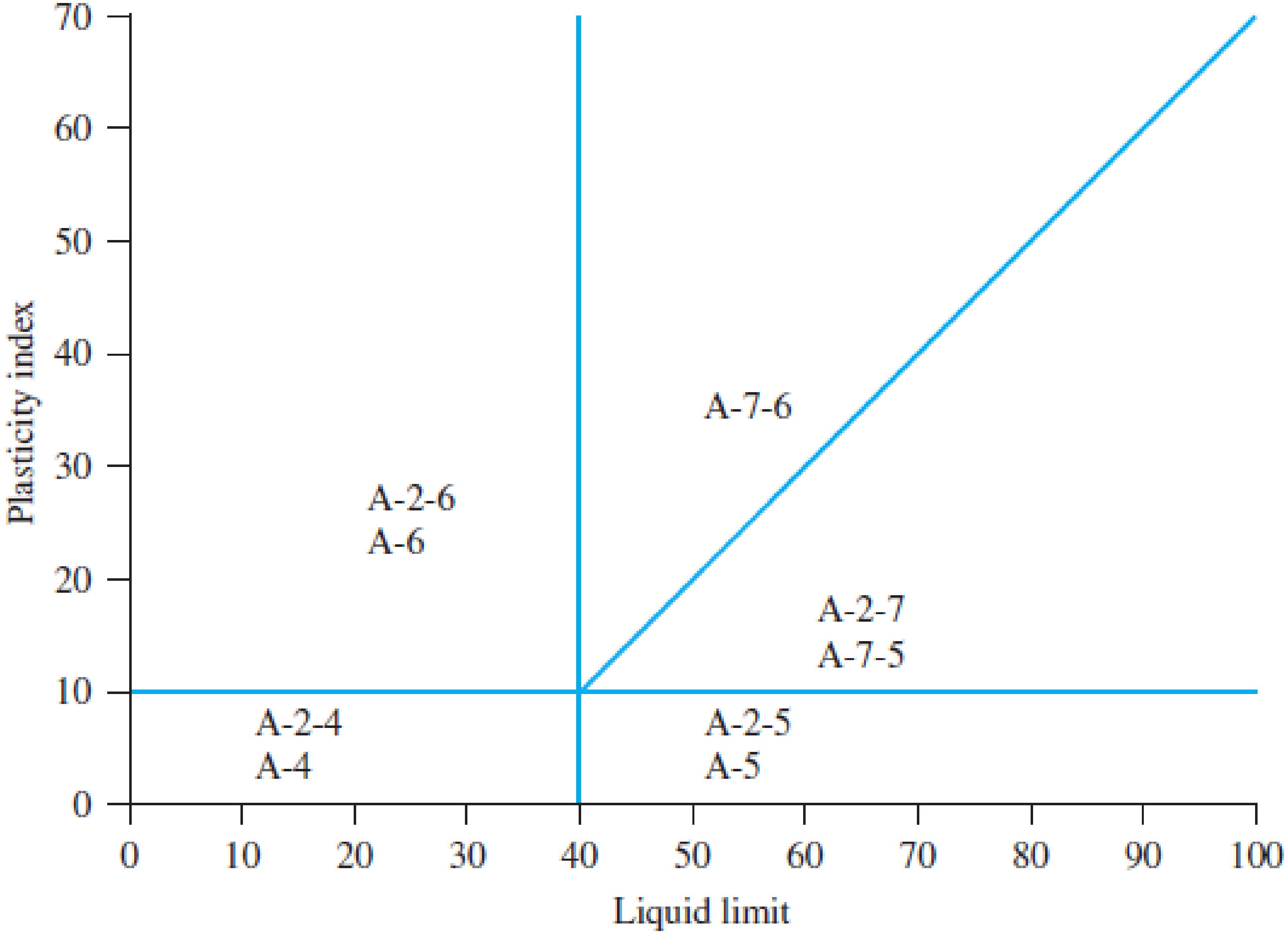


Figure 5.2 Range of liquid limit and plasticity index for soils in groups A-2, A-4, A-5, A-6, and A-7

$$GI = (F_{200} - 35)[0.2 + 0.005(LL - 40)] + 0.01(F_{200} - 15)(PI - 10)$$

group index

where F_{200} = percentage passing through the No. 200 sieve
LL = liquid limit

1. If Eq. (GI) yields a negative value for *GI*, it is taken as 0.
2. The group index calculated from Eq. (GI) is rounded off to the nearest whole number (for example, $GI = 3.4$ is rounded off to 3; $GI = 3.5$ is rounded off to 4).
3. There is no upper limit for the group index.
4. The group index of soils belonging to groups A-1-a, A-1-b, A-2-4, A-2-5, and A-3 is always 0.
5. When calculating the group index for soils that belong to groups A-2-6 and A-2-7, use the partial group index for *PI*, or

$$GI = 0.01(F_{200} - 15)(PI - 10)$$

Example 5.2

The results of the particle-size analysis of a soil are as follows:

- Percent passing the No. 10 sieve = 100
- Percent passing the No. 40 sieve = 80
- Percent passing the No. 200 sieve = 58

The liquid limit and plasticity index of the minus No. 40 fraction of the soil are 30 and 10, respectively. Classify the soil by the AASHTO system.

Solution

Using Table 5.1, since 58% of the soil is passing through the No. 200 sieve, it falls under silt-clay classifications—that is, it falls under group A-4, A-5, A-6, or A-7. Proceeding from left to right, it falls under group A-4.

From Eq. (5.1)

$$\begin{aligned} GI &= (F_{200} - 35)[0.2 + 0.005(LL - 40)] + 0.01(F_{200} - 15)(PI - 10) \\ &= (58 - 35)[0.2 + 0.005(30 - 40)] + (0.01)(58 - 15)(10 - 10) \\ &= 3.45 \simeq 3 \end{aligned}$$

So, the soil will be classified as A-4(3). ■

Classification by Engineering Behavior

Unified Soil Classification System

This system classifies soils into two broad categories:

1. Coarse-grained soils that are gravelly and sandy in nature with less than 50% passing through the No. 200 sieve. The group symbols start with a prefix of G or S. G stands for gravel or gravelly soil, and S for sand or sandy soil.
2. Fine-grained soils are with 50% or more passing through the No. 200 sieve. The group symbols start with prefixes of M, which stands for inorganic silt, C for inorganic clay, or O for organic silts and clays. The symbol Pt is used for peat, muck, and other highly organic soils.

Other symbols used for the classification are:

- W—well graded
- P—poorly graded
- L—low plasticity (liquid limit less than 50)
- H—high plasticity (liquid limit more than 50)

Table 5.2 Unified Soil Classification System (Based on Material Passing 76.2-mm Sieve)

Criteria for assigning group symbols				Group symbol
Coarse-grained soils More than 50% of retained on No. 200 sieve	**Gravels** More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[a]	$C_u \geq 4$ and $1 \leq C_c \leq 3$[c]	GW
			$C_u < 4$ and/or $1 > C_c > 3$[c]	GP
		Gravels with Fines More than 12% fines[a,d]	$PI < 4$ or plots below "A" line (Figure 5.3)	GM
			$PI > 7$ and plots on or above "A" line (Figure 5.3)	GC
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[b]	$C_u \geq 6$ and $1 \leq C_c \leq 3$[c]	SW
			$C_u < 6$ and/or $1 > C_c > 3$[c]	SP
		Sands with Fines More than 12% fines[b,d]	$PI < 4$ or plots below "A" line (Figure 5.3)	SM
			$PI > 7$ and plots on or above "A" line (Figure 5.3)	SC
Fine-grained soils 50% or more passes No. 200 sieve	**Silts and clays** Liquid limit less than 50	Inorganic	$PI > 7$ and plots on or above "A" line (Figure 5.3)[e]	CL
			$PI < 4$ or plots below "A" line (Figure 5.3)[e]	ML
		Organic	$\frac{\text{Liquid limit — oven dried}}{\text{Liquid limit — not dried}} < 0.75$; see Figure 5.3; OL zone	OL
	Silts and clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line (Figure 5.3)	CH
			PI plots below "A" line (Figure 5.3)	MH
		Organic	$\frac{\text{Liquid limit — oven dried}}{\text{Liquid limit — not dried}} < 0.75$; see Figure 5.3; OH zone	OH
Highly Organic Soils	Primarily organic matter, dark in color, and organic odor			Pt

[a] Gravels with 5 to 12% fine require dual symbols: GW-GM, GW-GC, GP-GM, GP-GC.

[b] Sands with 5 to 12% fines require dual symbols: SW-SM, SW-SC, SP-SM, SP-SC.

[c] $C_u = \dfrac{D_{60}}{D_{10}}$; $C_c = \dfrac{(D_{30})^2}{D_{60} \times D_{10}}$

[d] If $4 \leq PI \leq 7$ and plots in the hatched area in Figure 5.3, use dual symbol GC-GM or SC-SM.

[e] If $4 \leq PI \leq 7$ and plots in the hatched area in Figure 5.3, use dual symbol CL-ML.

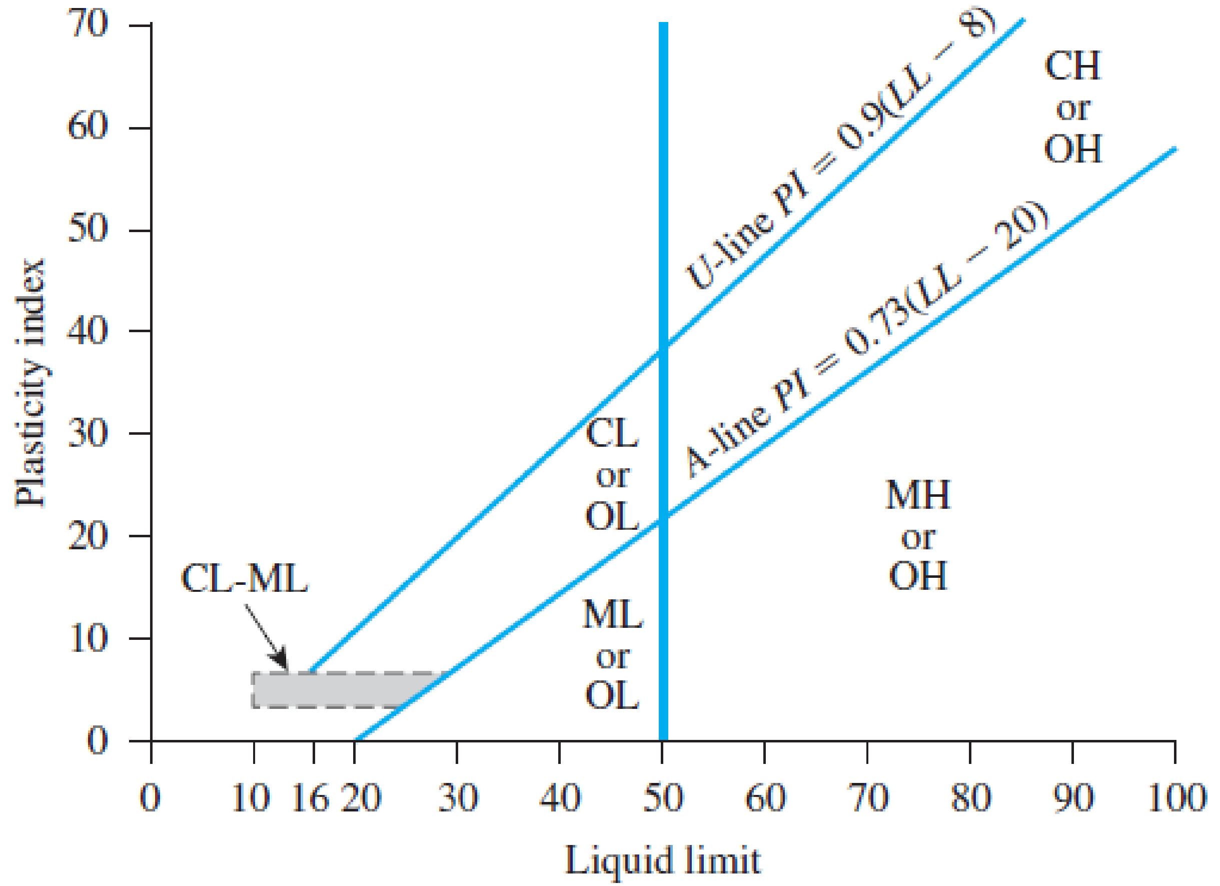


Figure 5.3 Plasticity chart

For proper classification according to this system, some or all of the following information must be known:

1. Percent of gravel—that is, the fraction passing the 76.2-mm sieve and retained on the No. 4 sieve (4.75-mm opening)
2. Percent of sand—that is, the fraction passing the No. 4 sieve (4.75-mm opening) and retained on the No. 200 sieve (0.075-mm opening)
3. Percent of silt and clay—that is, the fraction finer than the No. 200 sieve (0.075-mm opening)
4. Uniformity coefficient (C_u) and the coefficient of gradation (C_c)
5. Liquid limit and plasticity index of the portion of soil passing the No. 40 sieve

The group symbols for coarse-grained gravelly soils are GW, GP, GM, GC, GC-GM, GW-GM, GW-GC, GP-GM, and GP-GC. Similarly, the group symbols for fine-grained soils are CL, ML, OL, CH, MH, OH, CL-ML, and Pt.

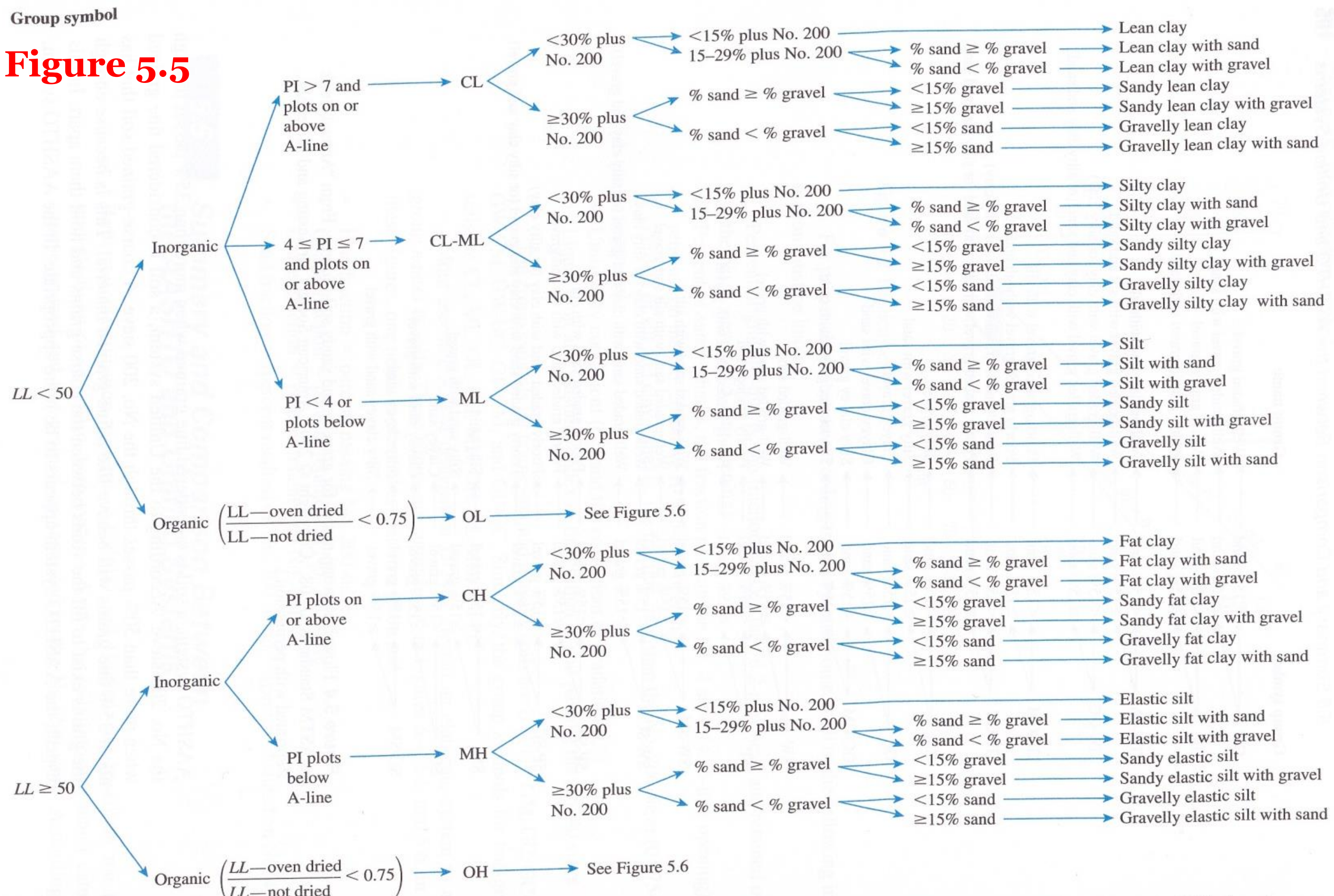
one needs to remember that, in a given soil,

- Fine fraction = percent passing No. 200 sieve
- Coarse fraction = percent retained on No. 200 sieve
- Gravel fraction = percent retained on No. 4 sieve
- Sand fraction = (percent retained on No. 200 sieve) − (percent retained on No. 4 sieve)

Group symbol		Group name
GW	<15% sand	Well-graded gravel
	≥15% sand	Well-graded gravel with sand
GP	<15% sand	Poorly graded gravel
	≥15% sand	Poorly graded gravel with sand
GW-GM	<15% sand	Well-graded sand with silt
	≥15% sand	Well-graded gravel with silt and sand
GW-GC	<15% sand	Well-graded gravel with clay (or silty clay)
	≥15% sand	Well-graded gravel with clay and sand (or silty clay and sand)
GP-GM	<15% sand	Poorly graded gravel with silt
	≥15% sand	Poorly graded gravel with silt and sand
GP-GC	<15% sand	Poorly graded gravel with clay (or silty clay)
	≥15% sand	Poorly graded gravel with clay and sand (or silty clay and sand)
GM	<15% sand	Silty gravel
	≥15% sand	Silty gravel with sand
GC	<15% sand	Clayey gravel
	≥15% sand	Clayey gravel with sand
GC-GM	<15% sand	Silty clayey gravel
	≥15% sand	Silty clayey gravel with sand
SW	<15% gravel	Well-graded sand
	≥15% gravel	Well-graded sand with gravel
SP	<15% gravel	Poorly graded sand
	≥15% gravel	Poorly graded sand with gravel
SW-SM	<15% gravel	Well-graded sand with silt
	≥15% gravel	Well-graded sand with silt and gravel
SW-SC	<15% gravel	Well-graded sand with clay (or silty clay)
	≥15% gravel	Well-graded sand with clay and gravel (or silty clay and gravel)
SP-SM	<15% gravel	Poorly graded sand with silt
	≥15% gravel	Poorly graded sand with silt and gravel
SP-SC	<15% gravel	Poorly graded sand with clay (or silty clay)
	≥15% gravel	Poorly graded sand with clay and gravel (or silty clay and gravel)
SM	<15% gravel	Silty sand
	≥15% gravel	Silty sand with gravel
SC	<15% gravel	Clayey sand
	≥15% gravel	Clayey sand with gravel
SC-SM	<15% gravel	Silty clayey sand
	≥15% gravel	Silty clayey sand with gravel

Figure 5.4

Figure 5.5



Group symbol
LL < 50
Inorganic
PI > 7 and plots on or above A-line
CL
<30% plus No. 200
<15% plus No. 200
Lean clay
15–29% plus No. 200
% sand ≥ % gravel
Lean clay with sand
% sand < % gravel
Lean clay with gravel
≥30% plus No. 200
% sand ≥ % gravel
<15% gravel
Sandy lean clay
≥15% gravel
Sandy lean clay with gravel
% sand < % gravel
<15% sand
Gravelly lean clay
≥15% sand
Gravelly lean clay with sand
4 ≤ PI ≤ 7 and plots on or above A-line
CL-ML
<30% plus No. 200
<15% plus No. 200
Silty clay
15–29% plus No. 200
% sand ≥ % gravel
Silty clay with sand
% sand < % gravel
Silty clay with gravel
≥30% plus No. 200
% sand ≥ % gravel
<15% gravel
Sandy silty clay
≥15% gravel
Sandy silty clay with gravel
% sand < % gravel
<15% sand
Gravelly silty clay
≥15% sand
Gravelly silty clay with sand
PI < 4 or plots below A-line
ML
<30% plus No. 200
<15% plus No. 200
Silt
15–29% plus No. 200
% sand ≥ % gravel
Silt with sand
% sand < % gravel
Silt with gravel
≥30% plus No. 200
% sand ≥ % gravel
<15% gravel
Sandy silt
≥15% gravel
Sandy silt with gravel
% sand < % gravel
<15% sand
Gravelly silt
≥15% sand
Gravelly silt with sand
Organic
(LL—oven dried / LL—not dried < 0.75)
OL
See Figure 5.6
LL ≥ 50
Inorganic
PI plots on or above A-line
CH
<30% plus No. 200
<15% plus No. 200
Fat clay
15–29% plus No. 200
% sand ≥ % gravel
Fat clay with sand
% sand < % gravel
Fat clay with gravel
≥30% plus No. 200
% sand ≥ % gravel
<15% gravel
Sandy fat clay
≥15% gravel
Sandy fat clay with gravel
% sand < % gravel
<15% sand
Gravelly fat clay
≥15% sand
Gravelly fat clay with sand
PI plots below A-line
MH
<30% plus No. 200
<15% plus No. 200
Elastic silt
15–29% plus No. 200
% sand ≥ % gravel
Elastic silt with sand
% sand < % gravel
Elastic silt with gravel
≥30% plus No. 200
% sand ≥ % gravel
<15% gravel
Sandy elastic silt
≥15% gravel
Sandy elastic silt with gravel
% sand < % gravel
<15% sand
Gravelly elastic silt
≥15% sand
Gravelly elastic silt with sand
Organic
(LL—oven dried / LL—not dried < 0.75)
OH
See Figure 5.6

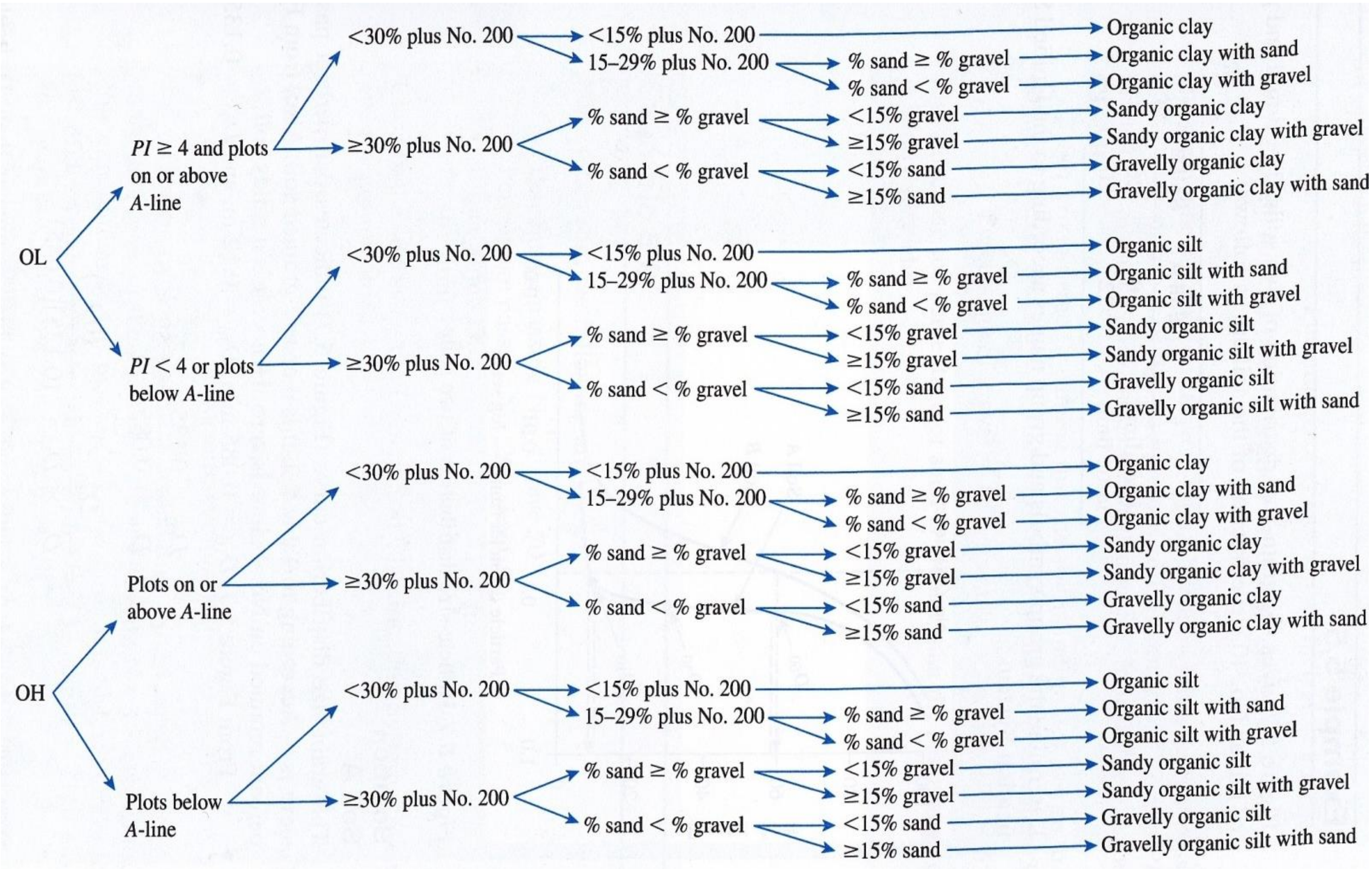

OL
PI ≥ 4 and plots on or above A-line
<30% plus No. 200
<15% plus No. 200
Organic clay
15–29% plus No. 200
% sand ≥ % gravel
Organic clay with sand
% sand < % gravel
Organic clay with gravel
≥30% plus No. 200
% sand ≥ % gravel
<15% gravel
Sandy organic clay
≥15% gravel
Sandy organic clay with gravel
% sand < % gravel
<15% sand
Gravelly organic clay
≥15% sand
Gravelly organic clay with sand
PI < 4 or plots below A-line
<30% plus No. 200
<15% plus No. 200
Organic silt
15–29% plus No. 200
% sand ≥ % gravel
Organic silt with sand
% sand < % gravel
Organic silt with gravel
≥30% plus No. 200
% sand ≥ % gravel
<15% gravel
Sandy organic silt
≥15% gravel
Sandy organic silt with gravel
% sand < % gravel
<15% sand
Gravelly organic silt
≥15% sand
Gravelly organic silt with sand
OH
Plots on or above A-line
<30% plus No. 200
<15% plus No. 200
Organic clay
15–29% plus No. 200
% sand ≥ % gravel
Organic clay with sand
% sand < % gravel
Organic clay with gravel
≥30% plus No. 200
% sand ≥ % gravel
<15% gravel
Sandy organic clay
≥15% gravel
Sandy organic clay with gravel
% sand < % gravel
<15% sand
Gravelly organic clay
≥15% sand
Gravelly organic clay with sand
Plots below A-line
<30% plus No. 200
<15% plus No. 200
Organic silt
15–29% plus No. 200
% sand ≥ % gravel
Organic silt with sand
% sand < % gravel
Organic silt with gravel
≥30% plus No. 200
% sand ≥ % gravel
<15% gravel
Sandy organic silt
≥15% gravel
Sandy organic silt with gravel
% sand < % gravel
<15% sand
Gravelly organic silt
≥15% sand
Gravelly organic silt with sand

Figure 5.6

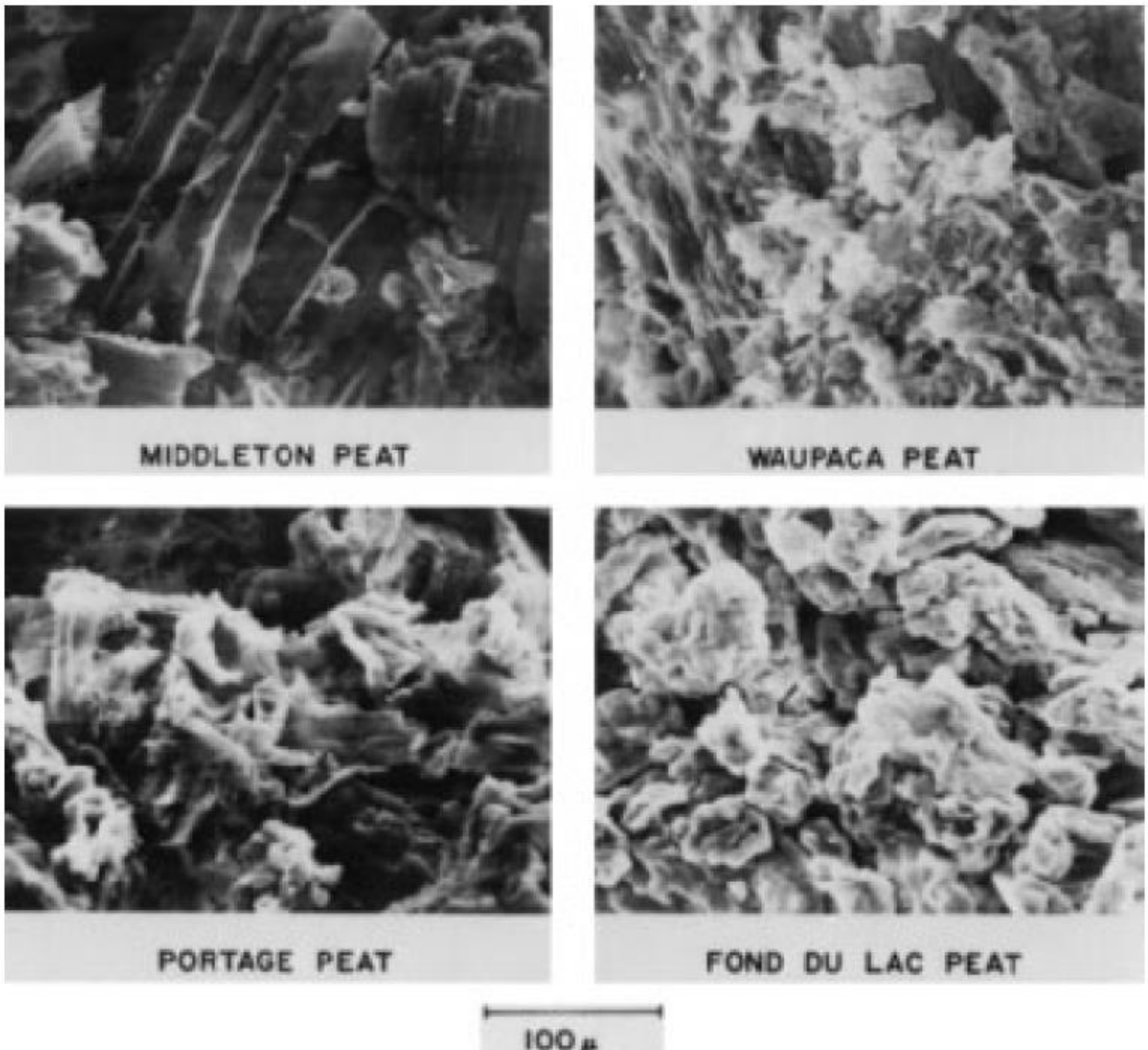

MIDDLETON PEAT
WAUPACA PEAT
PORTAGE PEAT
FOND DU LAC PEAT
100 µ

Table 5.3 Properties of the Peats Shown in Figure 5.8

Source of peat	Moisture content (%)	Unit weight kN/m^3	Unit weight lb/ft^3	Specific gravity, G_s	Ash content (%)
Middleton	510	9.1	57.9	1.41	12.0
Waupaca County	460	9.6	61.1	1.68	15.0
Portage	600	9.6	61.1	1.72	19.5
Fond du Lac County	240	10.2	64.9	1.94	39.8

Table 5.4 Comparison of the AASHTO System with the Unified System*

Soil group in AASHTO system	Comparable soil groups in Unified 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

* After Liu (1967)

Source: From A Review of Engineering Soil Classification Systems. In Highway Research Record 156, Highway Research Board, National Research Council, Washington, D.C., 1967, Table 5, p. 16. Reproduced with permission of the Transportation Research Board.

Table 5.5 Comparison of the Unified System with the AASHTO System*

Soil group in Unified system	Comparable soil groups in AASHTO system		
	Most probable	Possible	Possible but improbable
GW	A-1-a	—	A-2-4, A-2-5, A-2-6, A-2-7
GP	A-1-a	A-1-b	A-3, A-2-4, A-2-5, A-2-6, A-2-7
GM	A-1-b, A-2-4, A-2-5, A-2-7	A-2-6	A-4, A-5, A-6, A-7-5, A-7-6, A-1-a
GC	A-2-6, A-2-7	A-2-4	A-4, A-6, A-7-6, A-7-5
SW	A-1-b	A-1-a	A-3, A-2-4, A-2-5, A-2-6, A-2-7
SP	A-3, A-1-b	A-1-a	A-2-4, A-2-5, A-2-6, A-2-7
SM	A-1-b, A-2-4, A-2-5, A-2-7	A-2-6, A-4	A-5, A-6, A-7-5, A-7-6, A-1-a
SC	A-2-6, A-2-7	A-2-4, A-6, A-4, A-7-6	A-7-5
ML	A-4, A-5	A-6, A-7-5, A-7-6	—
CL	A-6, A-7-6	A-4	—
OL	A-4, A-5	A-6, A-7-5, A-7-6	—
MH	A-7-5, A-5	—	A-7-6
CH	A-7-6	A-7-5	—
OH	A-7-5, A-5	—	A-7-6
Pt	—	—	—

*After Liu (1967)

Source: From A Review of Engineering Soil Classification Systems. In Highway Research Record 156, Highway Research Board, National Research Council, Washington, D.C., 1967, Table 6, p. 17. Reproduced with permission of the Transportation Research Board.

Example 5.5

Figure 5.7 gives the grain-size distribution of two soils. The liquid and plastic limits of minus No. 40 sieve fraction of the soil are as follows:

	Soil A	Soil B
Liquid limit	30	26
Plastic limit	22	20

Determine the group symbols and group names according to the Unified Soil Classification System.

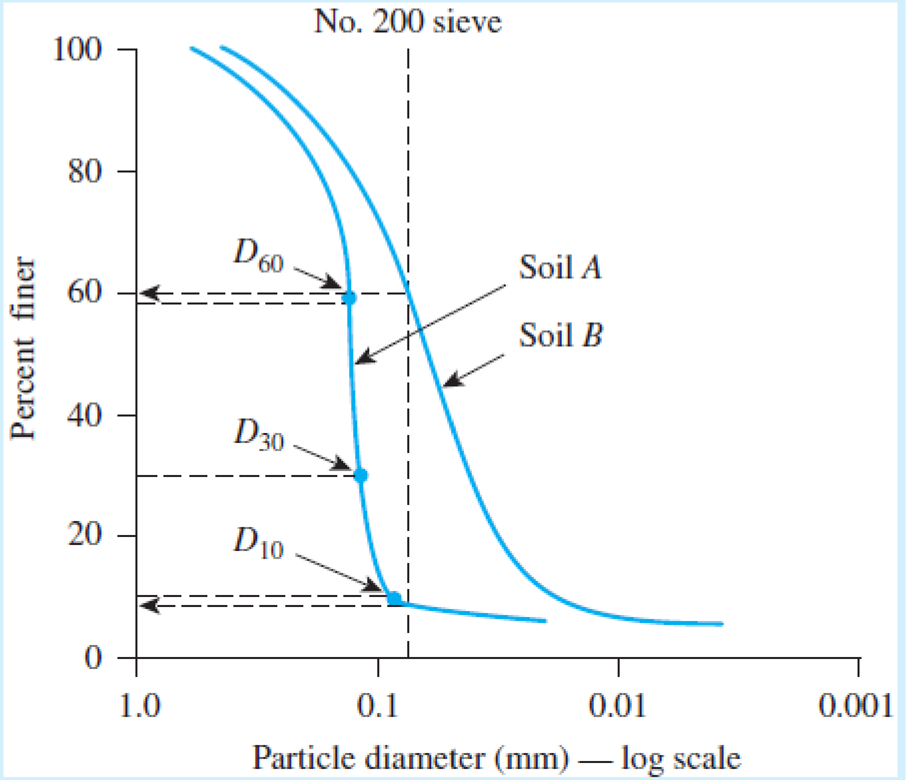


Figure 5.7 Particle-size distribution of two soils

Solution

Soil *A*

The grain-size distribution curve (Figure 5.7) indicates that percent passing No. 200 sieve is 8. According to Table 5.2, it is a coarse-grained soil. Also, from Figure 5.7, the percent retained on No. 4 sieve is zero. Hence, it is a sandy soil.

From Figure 5.7, $D_{10} = 0.085$ mm, $D_{30} = 0.12$ m, and $D_{60} = 0.135$ mm. Thus,

$$C_u = \frac{D_{60}}{D_{10}} = \frac{0.135}{0.085} = 1.59 < 6$$

$$C_c = \frac{D_{30}^2}{D_{60} \times D_{10}} = \frac{(0.12)^2}{(0.135)(0.085)} = 1.25 > 1$$

With $LL = 30$ and $PI = 30 - 22 = 8$ (which is greater than 7), it plots above the *A*-line in Figure 5.3. Hence, the group symbol is **SP-SC**.

In order to determine the group name, we refer to Figure 5.4.

$$\text{Percentage of gravel} = 0 \text{ (which is} < 15\%\text{)}$$

So, the group name is **poorly graded sand with clay**.

Soil *B*

The grain-size distribution curve in Figure 5.7 shows that percent passing No. 200 sieve is 61 (>50%); hence, it is a fine-grained soil. Given: $LL = 26$ and $PI = 26 - 20 = 6$. In Figure 5.3, the *PI* plots in the hatched area. So, from Table 5.2, the group symbol is **CL-ML**.

For group name (assuming that the soil is inorganic), we go to Figure 5.5 and obtain Plus No. 200 sieve $= 100 - 61 = 39$ (which is greater than 30).

$$\text{Percentage of gravel} = 0; \text{ percentage of sand} = 100 - 61 = 39$$

Thus, because the percentage of sand is greater than the percentage of gravel, the soil is **sandy silty clay**. ■

Soil Mechanics

Soil Compaction

Dr Jasim M Abbas Al Shamary

General

In the construction of highway embankments, earth dams, and many other engineering structures, loose soils must be compacted to increase their unit weights. Compaction' increases the strength characteristics of soils, which increase the bearing capacity of foundations constructed over them. Compaction also decreases the amount of undesirable settlement of structures and increases the stability of slopes of embankments.

Principles

Is the densification of soil by removal of air, which requires mechanical energy.

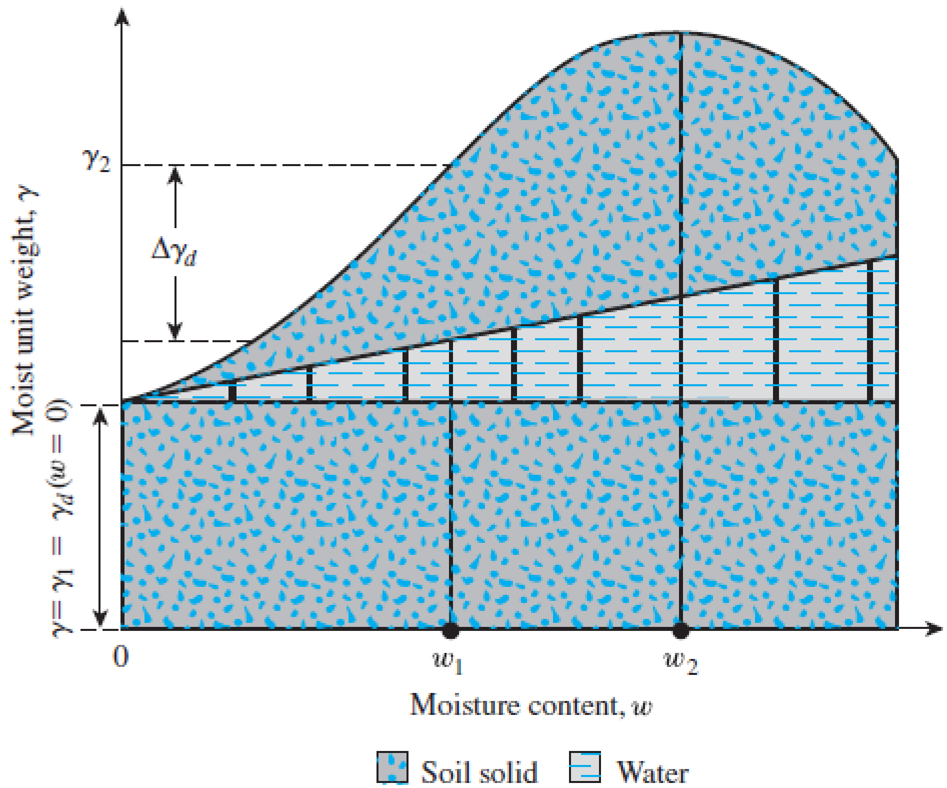


Figure 6.1 Principles of compaction

Standard Proctor Test

For each test, the moist unit weight of compaction, γ, can be calculated as

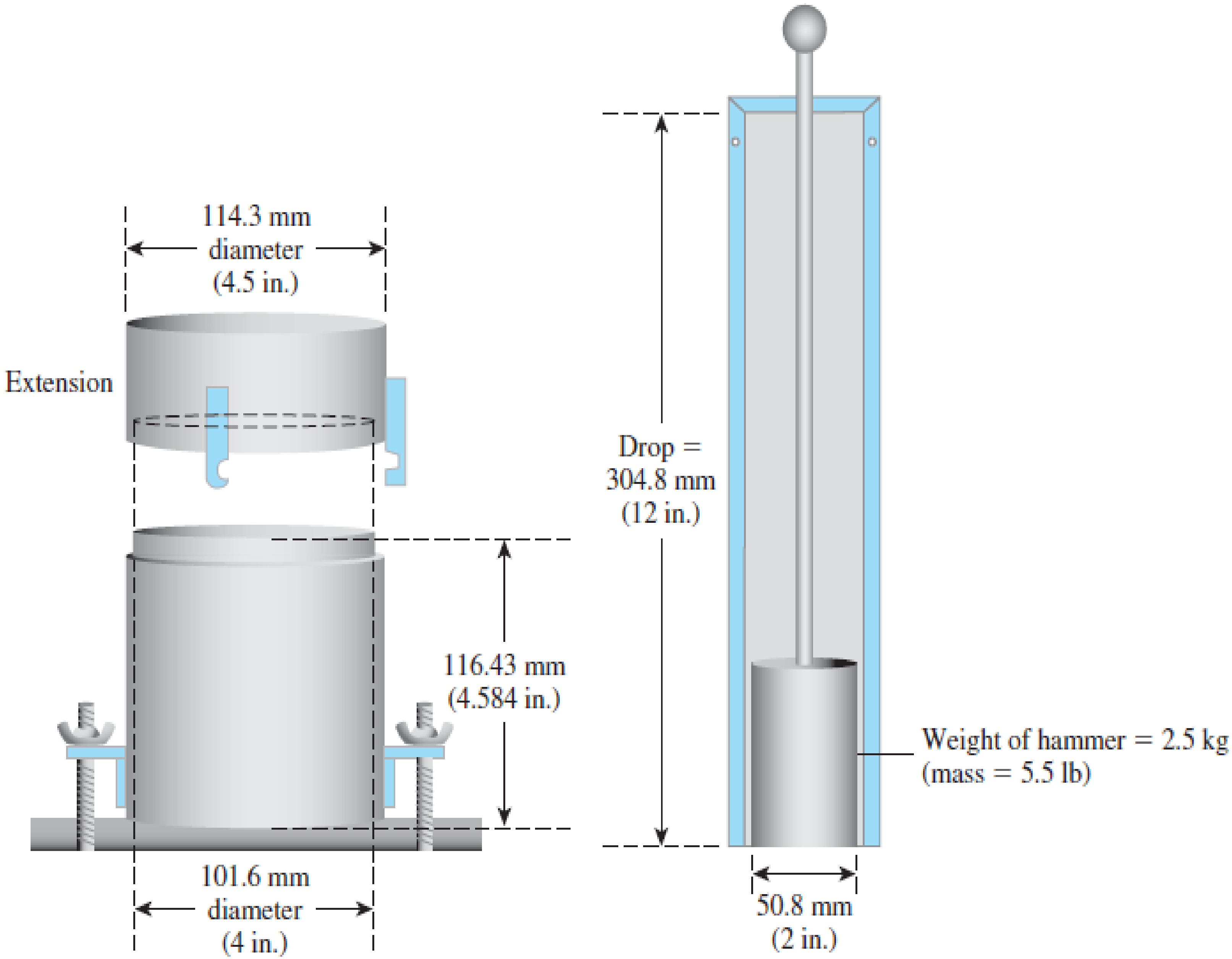
$$\gamma = \frac{W}{V_{(m)}}$$

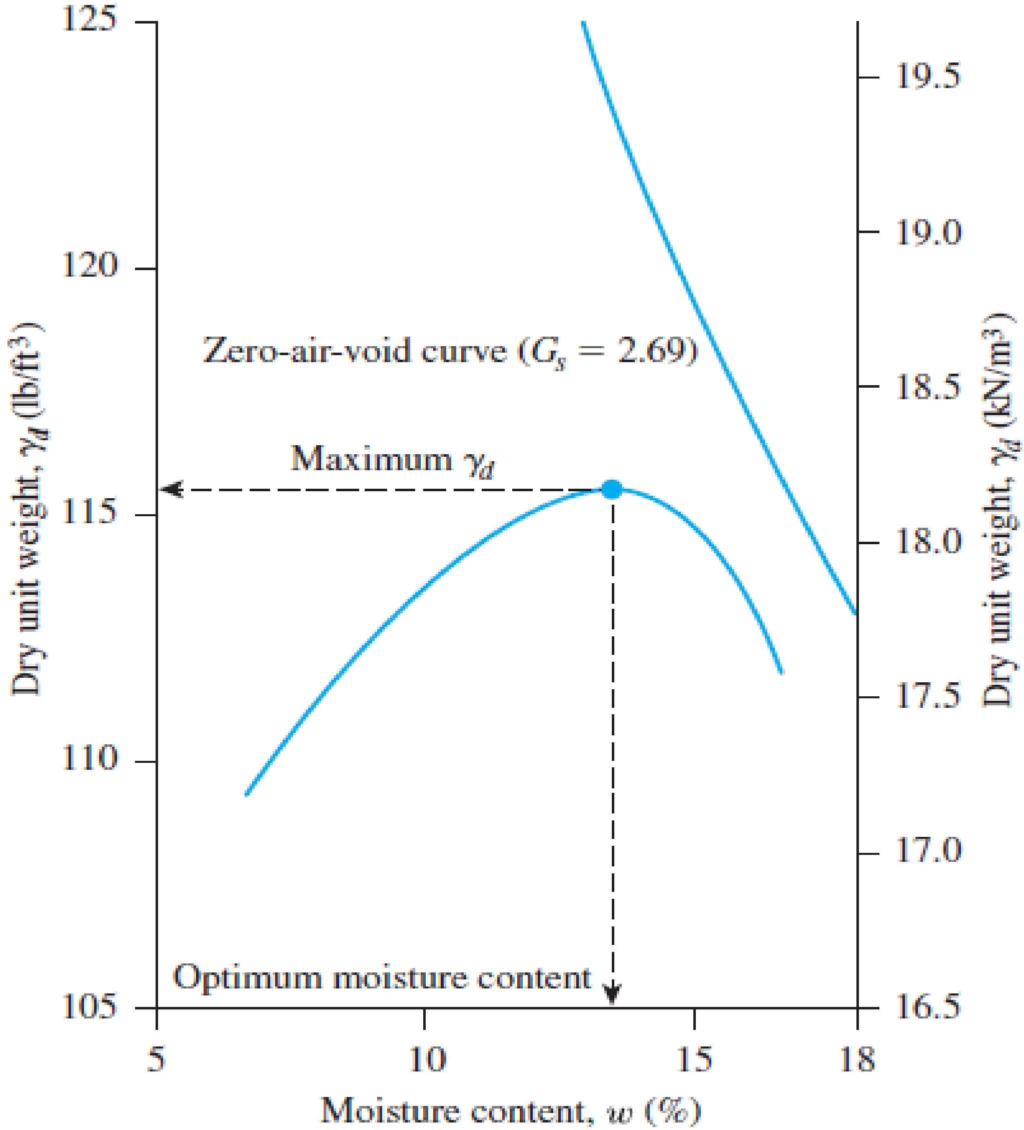
where W = weight of the compacted soil in the mold

$V_{(m)}$ = volume of the mold [944 cm^3 ($\frac{1}{30}$ ft^3)]

$$\gamma_d = \frac{\gamma}{1 + \frac{w\,(\%)}{100}}$$

where w (%) = percentage of moisture content.


114.3 mm
diameter
(4.5 in.)
Extension
Drop =
304.8 mm
(12 in.)
116.43 mm
(4.584 in.)
Weight of hammer = 2.5 kg
(mass = 5.5 lb)
101.6 mm
diameter
(4 in.)
50.8 mm
(2 in.)



125
120
115
110
105
19.5
19.0
18.5
18.0
17.5
17.0
16.5
5
10
15
18
Zero-air-void curve (G_s = 2.69)
Maximum γ_d
Optimum moisture content
Dry unit weight, γ_d (lb/ft^3)
Dry unit weight, γ_d (kN/m^3)
Moisture content, w (%)

$$\gamma_{zav} = \frac{G_s\gamma_w}{1 + wG_s} = \frac{\gamma_w}{w + \dfrac{1}{G_s}}$$

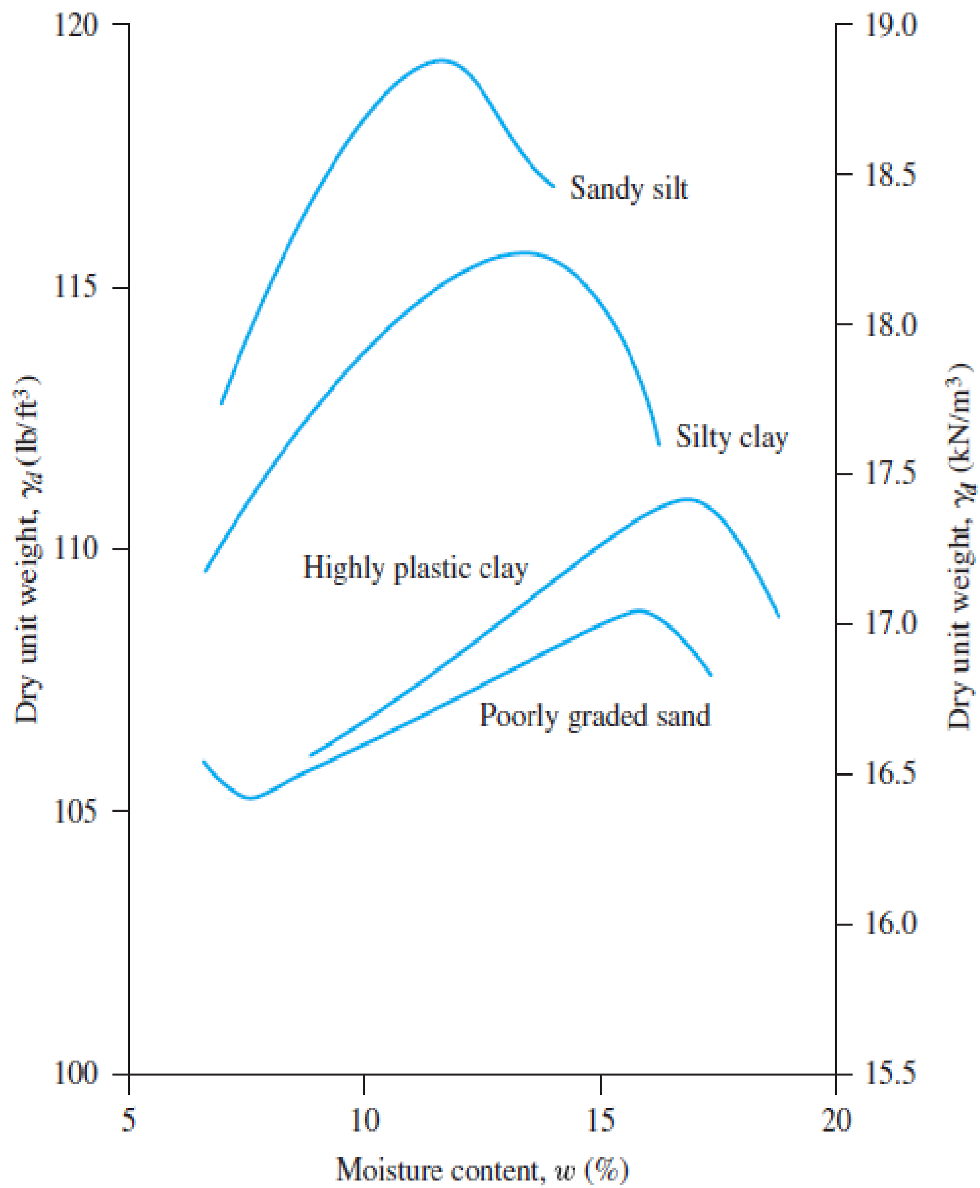
where γ_{zav} = zero-air-void unit weight.

To obtain the variation of γ_{zav} with moisture content, use the following procedure:

1. Determine the specific gravity of soil solids.
2. Know the unit weight of water (γ_w).
3. Assume several values of w, such as 5%, 10%, 15%, and so on.
4. Use Eq. (6.4) to calculate γ_{zav} for various values of w.

Factors Affecting Compaction

<u>Soil Type</u>: Grain-size distribution, shape of the soil grains, specific gravity of soil solids, and amount and type of clay minerals



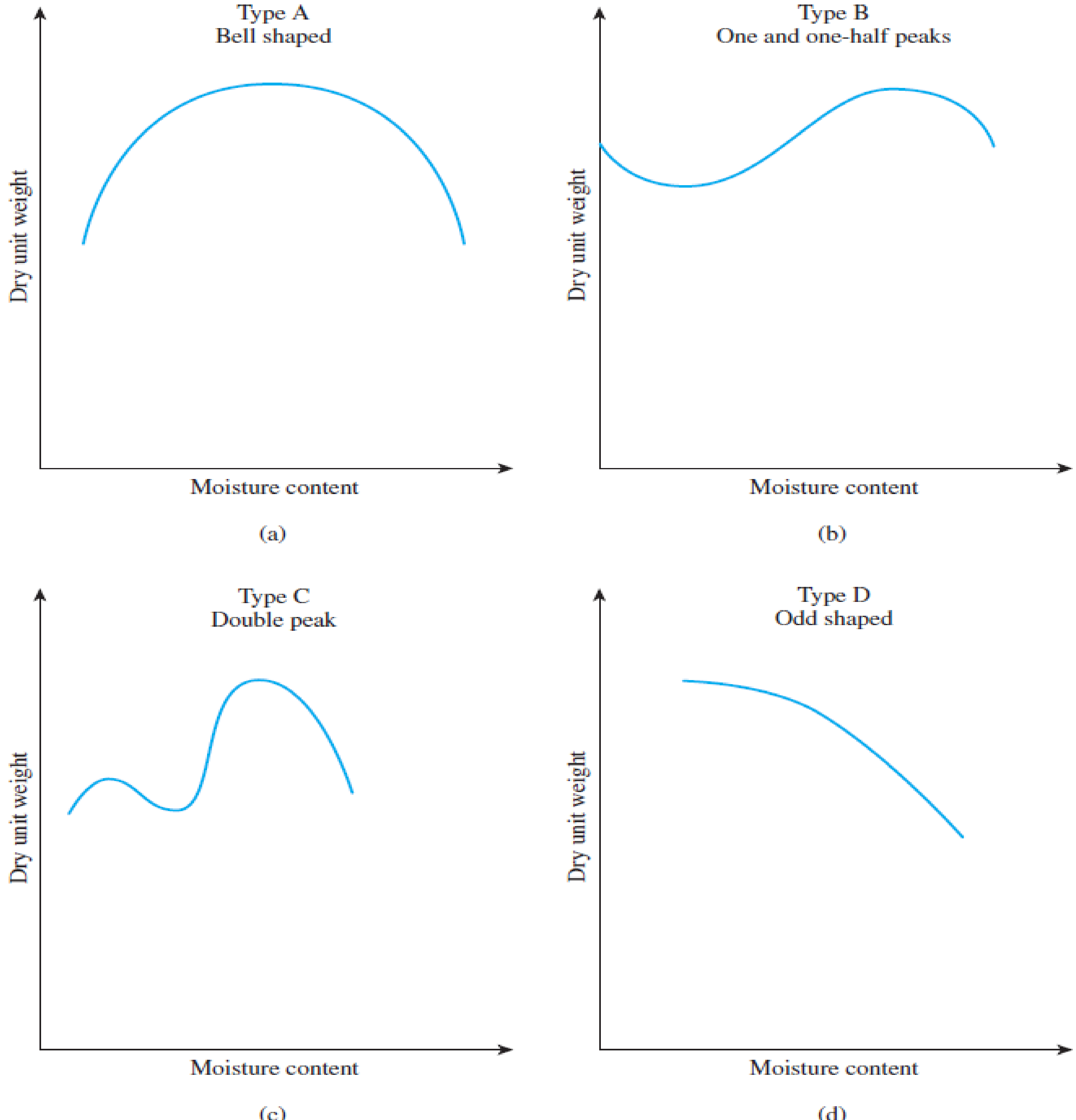
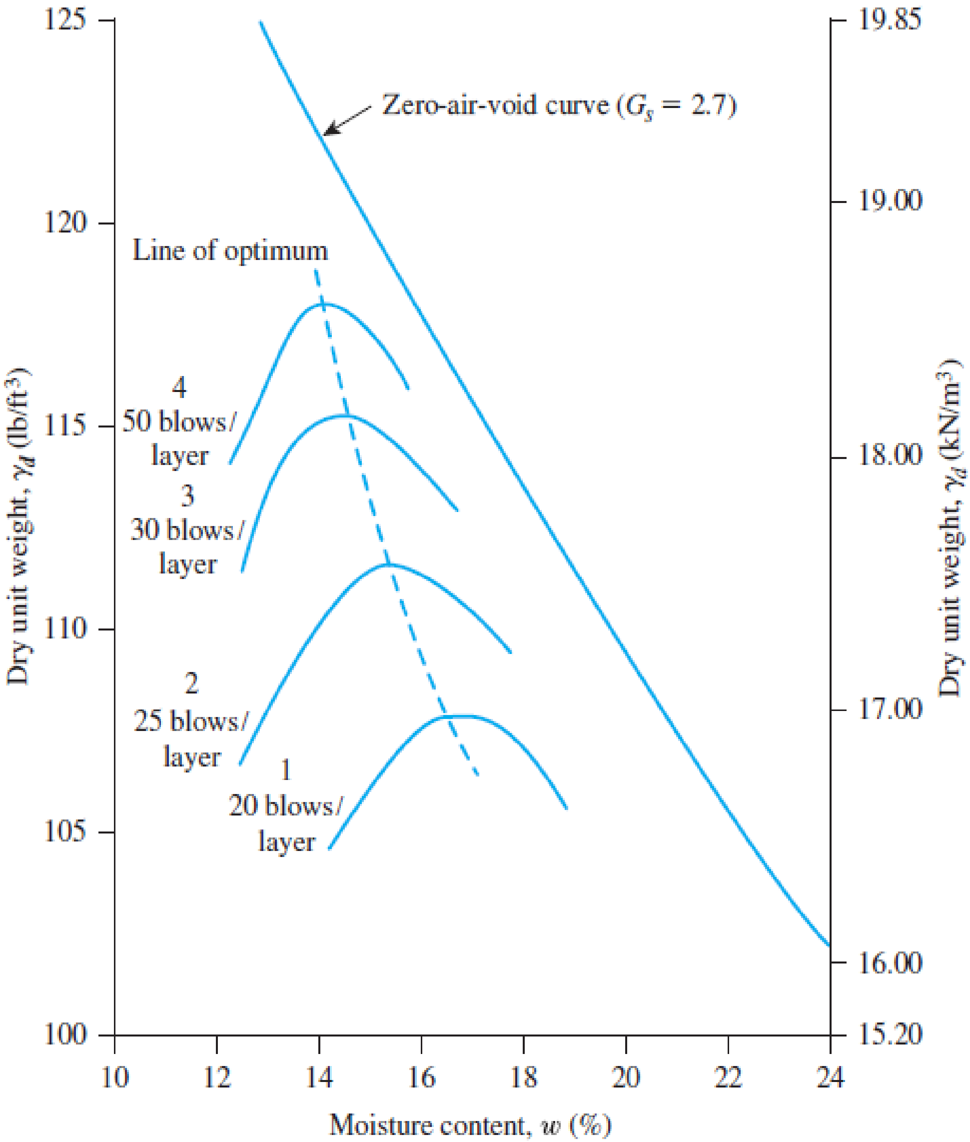


Figure 6.5
Various types of compaction curves encountered in soils

<u>Compaction Effort</u>: The compaction energy per unit volume used for the standard Proctor test described in previous section can be given as:

$$E = \frac{\begin{pmatrix}\text{Number}\\ \text{of blows}\\ \text{per layer}\end{pmatrix} \times \begin{pmatrix}\text{Number}\\ \text{of}\\ \text{layers}\end{pmatrix} \times \begin{pmatrix}\text{Weight}\\ \text{of}\\ \text{hammer}\end{pmatrix} \times \begin{pmatrix}\text{Height of}\\ \text{drop of}\\ \text{hammer}\end{pmatrix}}{\text{Volume of mold}}$$



Sandy clay: Liquid limit = 31 Plastic limit = 26

Modified Proctor Test

$$\rho_{d(\max)}\,(\text{kg/m}^3) = [4{,}804{,}574G_s - 195.55(LL)^2 + 156{,}971(\text{R\#4})^{0.5} - 9{,}527{,}830]^{0.5}$$

$$\ln(w_{\text{opt}}) = 1.195 \times 10^{-4}(LL)^2 - 1.964G_s - 6.617 \times 10^{-5}(\text{R\#4}) + 7.651$$

where $\rho_{d(\max)}$ = maximum dry density (kg/m^3)
w_{opt} = optimum moisture content (%)
G_s = specific gravity of soil solids
LL = liquid limit, in percent
R#4 = percent retained on No. 4 sieve

$$w_{\text{opt}}(\%) = [1.95 - 0.38(\log CE)](PL)$$

$$\gamma_{d(\max)}\,(\text{kN/m}^3) = 22.68e^{-0.0183w_{\text{opt}}(\%)}$$

where PL = plastic limit (%)
CE = compaction energy (kN-m/m^3)

For modified Proctor test, CE = 2700 kN/m^3. Hence,

$$w_{\text{opt}}(\%) \approx 0.65(PL)$$

and

$$\gamma_{d(\max)}\,(\text{kN/m}^3) = 22.68e^{-0.012(PL)}$$

Table 6.1 Summary of Standard and Modified Proctor Compaction Test Specifications (*ASTM D-698 and D-1557*)

	Description	Method A	Method B	Method C
Physical data for the tests	Material	Passing No. 4 sieve	Passing 9.5 mm ($\frac{3}{8}$ in.) sieve	Passing 19 mm ($\frac{3}{4}$ in.) sieve
	Use	Used if 20% or less by weight of material is retained on No. 4 (4.75 mm) sieve	Used if more than 20% by weight of material is retained on No. 4 (4.75 mm) sieve and 20% or less by weight of material is retained on 9.5 mm ($\frac{3}{8}$ in.) sieve	Used if more than 20% by weight of material is retained on 9.5 mm ($\frac{3}{8}$ in.) sieve and less than 30% by weight of material is retained on 19 mm ($\frac{3}{4}$ in.) sieve
	Mold volume	944 cm^3 ($\frac{1}{30}$ ft^3)	944 cm^3 ($\frac{1}{30}$ ft^3)	2124 cm^3 ($\frac{1}{13.33}$ ft^3)
	Mold diameter	101.6 mm (4 in.)	101.6 mm (4 in.)	152.4 mm (6 in.)
	Mold height	116.4 mm (4.584 in.)	116.4 mm (4.584 in.)	116.4 mm (4.584 in.)
Standard Proctor test	Weight of hammer	24.4 N (5.5 lb)	24.4 N (5.5 lb)	24.4 N (5.5 lb)
	Height of drop	305 mm (12 in.)	305 mm (12 in.)	305 mm (12 in.)
	Number of soil layers	3	3	3
	Number of blows/layer	25	25	56
Modified Proctor test	Weight of hammer	44.5 N (10 lb)	44.5 N (10 lb)	44.5 N (10 lb)
	Height of drop	457 mm (18 in.)	457 mm (18 in.)	457 mm (18 in.)
	Number of soil layers	5	5	5
	Number of blows/layer	25	25	56

Example 6.1

The laboratory test results of a standard Proctor test are given in the following table.

Volume of mold (ft^3)	Weight of moist soil in mold (lb)	Moisture content, w (%)
$\frac{1}{30}$	3.78	10
$\frac{1}{30}$	4.01	12
$\frac{1}{30}$	4.14	14
$\frac{1}{30}$	4.12	16
$\frac{1}{30}$	4.01	18
$\frac{1}{30}$	3.90	20

a. Determine the maximum dry unit weight of compaction and the optimum moisture content.

b. Calculate and plot γ_d versus the moisture content for degree of saturation, $S =$ 80, 90, and 100% (i.e., γ_{zav}). Given: $G_s = 2.7$.

Solution

Part a

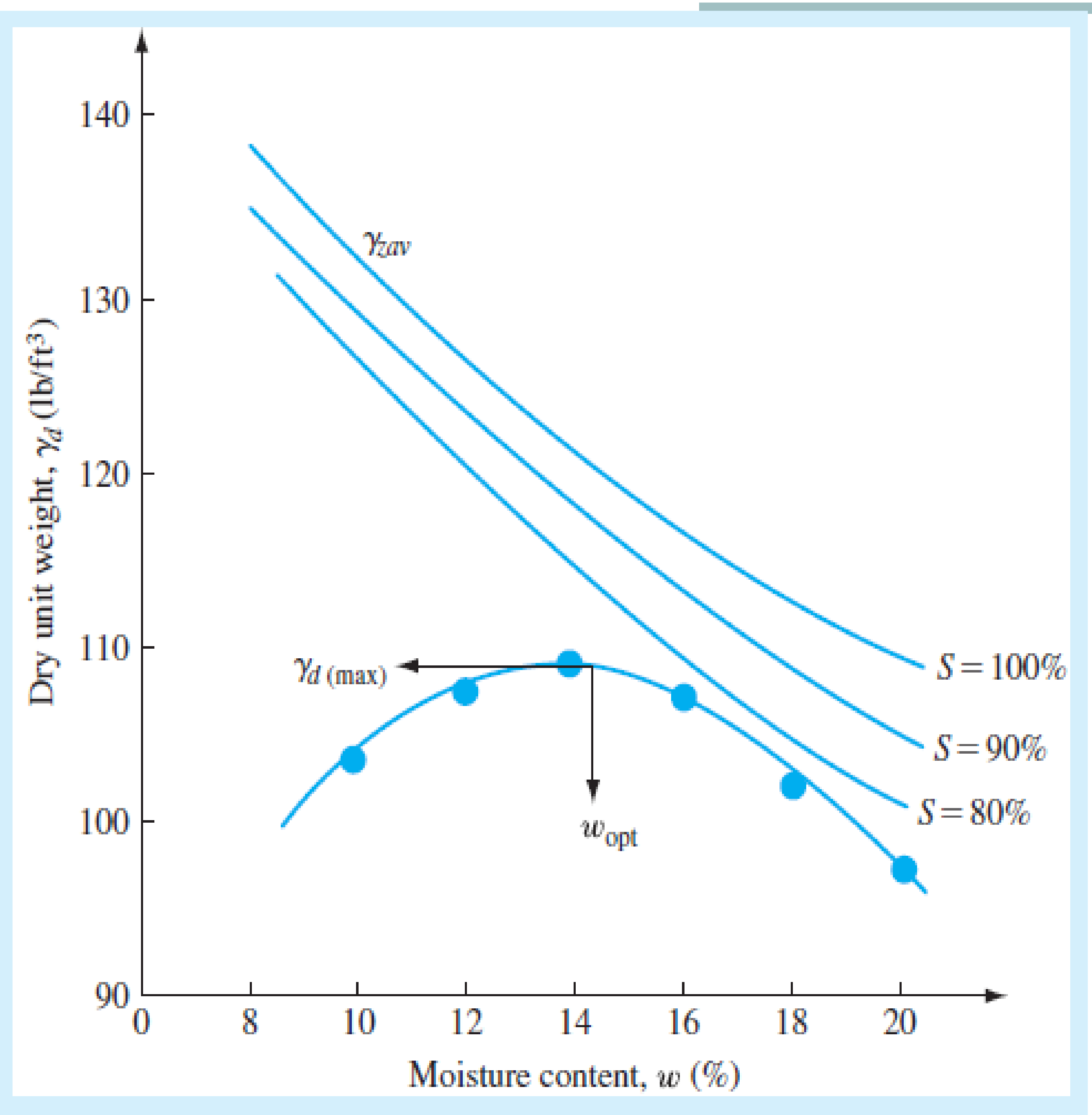
The following table can be prepared.

Volume of mold V (ft^3)	Weight of soil, W (lb)	Moist unit weight, γ (lb/ft^3)[a]	Moisture content, w (%)	Dry unit weight, γ_d (lb/ft^3)[b]
$\frac{1}{30}$	3.78	113.4	10	103.1
$\frac{1}{30}$	4.01	120.3	12	107.4
$\frac{1}{30}$	4.14	124.2	14	108.9
$\frac{1}{30}$	4.12	123.6	16	106.6
$\frac{1}{30}$	4.01	120.3	18	101.9
$\frac{1}{30}$	3.90	117.0	20	97.5

[a] $\gamma = \dfrac{W}{V}$

[b] $\gamma_d = \dfrac{\gamma}{1 + \dfrac{w\%}{100}}$

The plot of γ_d versus w is shown at the bottom of Figure 6.8. From the plot, we see that the maximum dry unit weight, $\gamma_{d(\max)} = 109\ lb/ft^3$ and the optimum moisture content is **14.4%**. ■



140
130
120
110
100
90
0
8
10
12
14
16
18
20
Dry unit weight, γ_d (lb/ft^3)
Moisture content, w (%)
γ_{zav}
$\gamma_{d\,(max)}$
w_{opt}
$S = 100\%$
$S = 90\%$
$S = 80\%$

Part b

From Eq. (6.3),

$$\gamma_d = \frac{G_s \gamma_w}{1 + \dfrac{G_s w}{S}}$$

The following table can be prepared.

		γ_d (lb/ft^3)		
G_s	w (%)	S = 80%	S = 90%	S = 100%
2.7	8	132.7	135.9	138.6
2.7	10	126.0	129.6	132.7
2.7	12	119.9	123.9	127.3
2.7	14	114.4	118.6	122.3
2.7	16	109.4	113.8	117.7
2.7	18	104.8	109.4	113.4
2.7	20	100.6	105.3	109.4

The plot of γ_d versus w for the various degrees of saturation is also shown in Figure 6.8.

Field Compaction

1. Smooth-wheel rollers (or smooth-drum rollers)
2. Pneumatic rubber-tired rollers
3. Sheepsfoot rollers
4. Vibratory rollers

Figure 6.15 Smooth-wheel roller (*Ingram Compaction LLC*)

Figure 6.16 Pneumatic rubber-tired roller (*Ingram Compaction LLC*)

Figure 6.17 Sheepsfoot roller (*SuperStock/Alamy*)

Specifications for Field Compaction

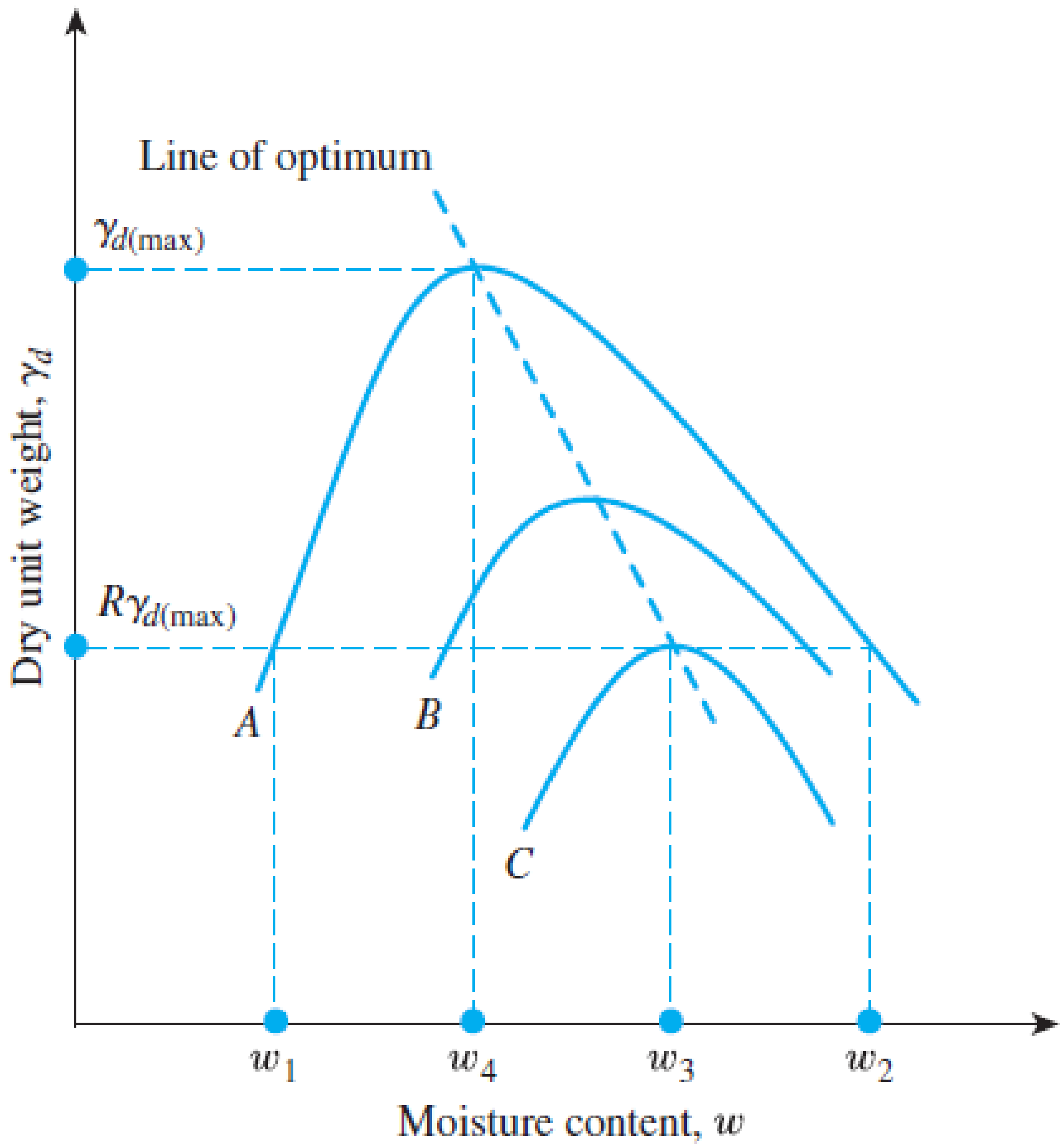
$$R(\%) = \frac{\gamma_{d(\text{field})}}{\gamma_{d(\text{max—lab})}} \times 100$$ where R = relative compaction

90 to 95% of the maximum dry unit weight determined in the lab

$$D_r = \left[\frac{\gamma_{d(\text{field})} - \gamma_{d(\min)}}{\gamma_{d(\max)} - \gamma_{d(\min)}}\right]\left[\frac{\gamma_{d(\max)}}{\gamma_{d(\text{field})}}\right]$$ relative density

$$R = \frac{R_0}{1 - D_r(1 - R_0)}$$

$$R_0 = \frac{\gamma_{d(\min)}}{\gamma_{d(\max)}}$$



Line of optimum
$\gamma_{d(\max)}$
$R\gamma_{d(\max)}$
Dry unit weight, γ_d
A
B
C
w_1
w_4
w_3
w_2
Moisture content, w

Table 6.2 Requirements to Achieve R = 95 to 100% (based on standard Proctor maximum dry unit-weight)*

		Requirements for compaction of 95 to 100% standard Proctor maximum dry unit weight					
Equipment type	**Applicability**	**Compacted lift thickness**	**Passes or coverages**	**Dimensions and weight of equipment**			**Possible variations in equipment**
				Soil type	*Foot contact area*	*Foot contact pressures*	
Sheepsfoot rollers	For fine-grained soils or dirty coarse-grained soils with more than 20% passing the No. 200 sieve. Not suitable for clean, coarse-grained soils. Particularly appropriate for compaction of impervious zone for earth dam or linings where bonding of lifts is important.	150 mm (6 in.)	4 to 6 passes for fine-grained soil	Fine-grained soil, $PI > 30$	30 to 80 cm^2 (5 to 12 in^2)	1700 to 3400 kN/m^2 (250 to 500 psi)	For earth dam, highway, and airfield work, a drum of 1.5 m (60-in.) diameter, loaded to 45 to 90 kN per linear meter (1.5 to 3 tons per linear ft) of drum is generally utilized. For smaller projects, a 1 m (40-in.) diameter drum, loaded to 22 to 50 kN per linear meter (0.75 to 1.75 tons per linear ft) of drum, is used. Foot contact pressure should be regulated to avoid shearing the soil on the third or fourth pass.
				Fine-grained soil, $PI < 30$	45 to 90 cm^2 (7 to 14 in^2)	1400 to 2800 kN/m^2 (200 to 400 psi)	
			6 to 8 passes for coarse-grained soil	Coarse-grained soil	65 to 90 cm^2 (10 to 14 in^2)	1000 to 1800 kN/m^2 (150 to 250 psi)	
				Efficient compaction of soils on the wet side of the optimum requires less contact pressure than that required by the same soils at lower moisture contents.			
Rubber-tired rollers	For clean, coarse-grained soils with 4 to 8% passing the No. 200 sieve.	250 mm (10 in)	3 to 5 coverages	Tire inflation pressures of 400 to 550 kN/m^2 (60 to 80 psi) for clean granular material or base course and subgrade compaction. Wheel load, 80 to 110 kN (18,000 to 25,000 lb.)			A wide variety of rubber-tired compaction equipment is available. For cohesive soils, light-wheel loads, such as provided by wobble-wheel equipment, may be substituted for heavy-wheel loads if lift thickness is decreased. For cohesionless soils, large-size tires are desirable to avoid shear and rutting.
	For fine-grained soils or well-graded, dirty, coarse-grained soils with more than 8% passing the No. 200 sieve.	150 to 200 mm (6 to 8 in)	4 to 6 coverages	Tire inflation pressures in excess of 450 kN/m^2 (65 psi) for fine-grained soils of high plasticity. For uniform clean sands or silty fine sands, use larger-size tires with pressures of 280 to 350 kN/m^2 (40 to 50 psi.)			

Smooth-wheel rollers	Appropriate for subgrade or base course compaction of well-graded sand-gravel mixtures.	200 to 300 mm (8 to 12 in.)	4 coverages	Tandem-type rollers for base-course or sub-grade compaction, 90 to 135 kN (10 to 15 ton) weight, 53 to 88 kN per linear meter (300 to 500 lb per linear in.) of width of rear roller.	Three-wheel rollers are obtainable in a wide range of sizes. Two-wheel tandem rollers are available in the 9 to 180 kN (1- to 20-ton) weight range. Three-axle tandem rollers are generally used in the 90 to 180 kN (10- to 20-ton) weight range. Very heavy rollers are used for proof rolling of subgrade or base course.
	May be used for fine-grained soils other than in earth dams. Not suitable for clean, well-graded sands or silty, uniform sands.	150 to 200 mm (6 to 8 in.)	6 coverages	Three-wheel roller for compaction of fine-grained soil; weights from 45 to 55 kN (5 to 6 tons) for materials of low plasticity to 90 kN (10 tons) for materials of high plasticity.	
Vibrating baseplate compactors	For coarse-grained soils with less than about 12% passing the No. 200 sieve. Best suited for materials with 4 to 8% passing the No. 200 sieve, placed thoroughly wet.	200 to 250 mm (8 to 10 in.)	3 coverages	Single pads or plates should weigh no less than 0.9 kN (200 lb). May be used in tandem where working space is available. For clean, coarse-grained soil, vibration frequency should be no less than 1600 cycles per minute.	Vibrating pads or plates are available, hand-propelled or self-propelled, single or in gangs, with width of coverage from 0.45 to 4.5 m ($1\frac{1}{2}$ to 15 ft). Various types of vibrating-drum equipment should be considered for compaction in large areas.
Crawler tractor	Best suited for coarse-grained soils with less than 4 to 8% passing the No. 200 sieve, placed thoroughly wet.	250 to 300 mm (10 to 12 in.)	3 to 4 coverages	No smaller than D8 tractor with blade, 153 kN (34,500 lb) weight, for high compaction.	Tractor weights up to 265 kN (60,000 lb.)
Power tamper or rammer	For difficult access, trench backfill. Suitable for all inorganic soils.	100 to 150 mm (4 to 6 in.) for silt or clay; 150 mm (6 in.) for coarse-grained soils	2 coverages	130 N (30 lb) minimum weight. Considerable range is tolerable, depending on materials and conditions.	Weights up to 1.1 kN (250 lb); foot diameter, 100 to 250 mm (4 to 10 in.)

** After U.S. Navy (1971). Published by U.S. Government Printing Office*

Determination of Field Unit Weight of Compaction

1. Sand cone method
2. Rubber balloon method
3. Nuclear method

Sand Cone Method

$$W_3 = \frac{W_2}{1 + \frac{w(\%)}{100}}$$

where w = moisture content.

$$W_5 = W_1 - W_4 \quad ($$

where W_5 = weight of sand to fill the hole and cone.

The volume of the excavated hole can then be determined as

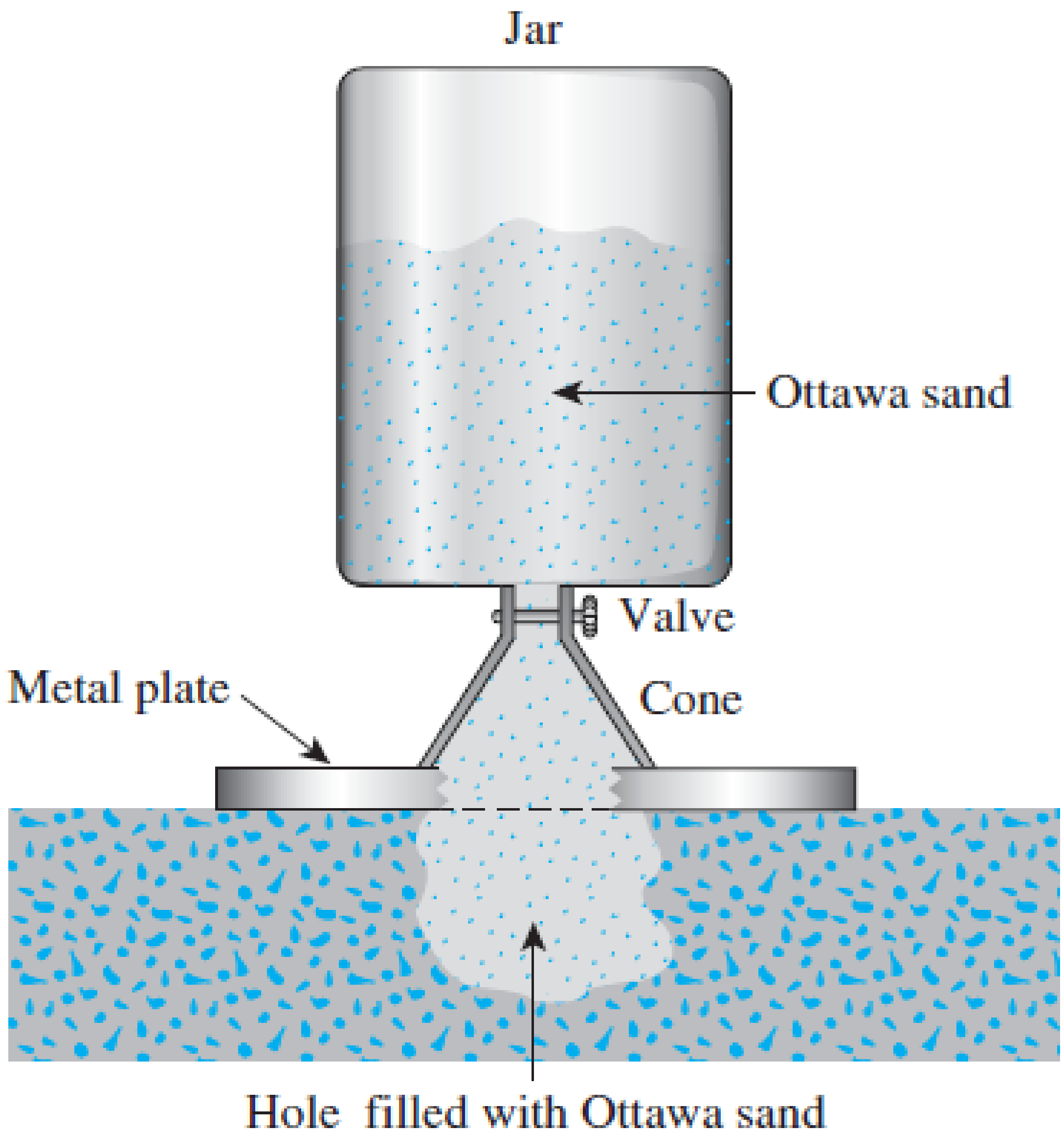
$$V = \frac{W_5 - W_c}{\gamma_{d(\text{sand})}} \quad ($$

where W_c = weight of sand to fill the cone only

$\gamma_{d(\text{sand})}$ = dry unit weight of Ottawa sand used

The values of W_c and $\gamma_{d(\text{sand})}$ are determined from the calibration done in the laboratory dry unit weight of compaction made in the field then can be determined as follows:

$$\gamma_d = \frac{\text{Dry weight of the soil excavated from the hole}}{\text{Volume of the hole}} = \frac{W_3}{V} \quad ($$


Jar
Ottawa sand
Valve
Metal plate
Cone
Hole filled with Ottawa sand

Rubber Balloon Method

The procedure for the rubber balloon method
is similar to that for the sand cone method; a test
hole is made and the moist weight of soil removed
from the hole and its moisture content are determined.

Nuclear Method

It uses a radioactive isotope source. The isotope gives off Gamma rays that radiate back to the meter’s detector. Dense soil absorbs more radiation than loose soil.

Example 6.3

Laboratory compaction test results for a clayey silt are given in the following table.

Moisture content (%)	Dry unit weight (kN/m^3)
6	14.80
8	17.45
9	18.52
11	18.9
12	18.5
14	16.9

Following are the results of a field unit-weight determination test performed on the same soil by means of the sand cone method:

- Calibrated dry density of Ottawa sand = 1570 kg/m^3
- Calibrated mass of Ottawa sand to fill the cone = 0.545 kg
- Mass of jar + cone + sand (before use) = 7.59 kg
- Mass of jar + cone + sand (after use) = 4.78 kg
- Mass of moist soil from hole = 3.007 kg
- Moisture content of moist soil = 10.2%

Determine:

a. Dry unit weight of compaction in the field
b. Relative compaction in the field

Solution

Part a

In the field,

$$\text{Mass of sand used to fill the hole and cone} = 7.59\text{ kg} - 4.78\text{ kg} = 2.81\text{ kg}$$

$$\text{Mass of sand used to fill the hole} = 2.81\text{ kg} - 0.545\text{ kg} = 2.265\text{ kg}$$

$$\text{Volume of the hole } (V) = \frac{2.265\text{ kg}}{\text{Dry density of Ottawa sand}}$$

$$= \frac{2.265\text{ kg}}{1570\text{ kg/m}^3} = 0.0014426\text{ m}^3$$

$$\text{Moist density of compacted soil} = \frac{\text{Mass of moist soil}}{\text{Volume of hole}}$$

$$= \frac{3.007}{0.0014426} = 2.084.4\text{ kg/m}^3$$

$$\text{Moist unit weight of compacted soil} = \frac{(2084.4)(9.81)}{1000} = 20.45\text{ kN/m}^3$$

Hence,

$$\gamma_d = \frac{\gamma}{1 + \dfrac{w(\%)}{100}} = \frac{20.45}{1 + \dfrac{10.2}{100}} = \mathbf{18.56\ kN/m^3}$$

Part b

The results of the laboratory compaction test are plotted in Figure 6.26. From the plot, we see that $\gamma_{d(\max)} = 19\ \text{kN/m}^3$. Thus, from Eq. (6.10).

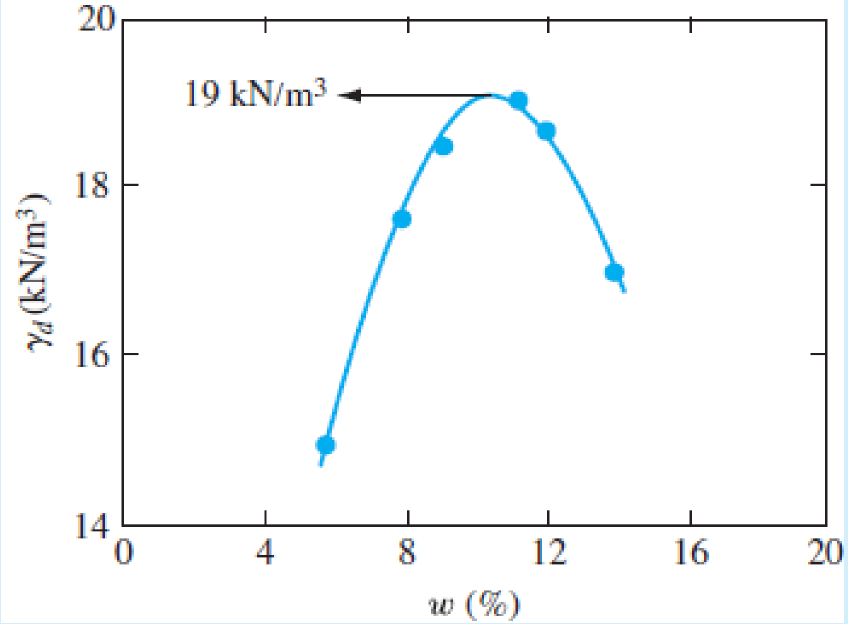


Figure 6.26 Plot of laboratory-compaction test results

$$R = \frac{\gamma_{d(\text{field})}}{\gamma_{d(\text{field})}} = \frac{18.56}{19.0} = \mathbf{97.7\%}$$

END