Ministry of Higher Education and Scientific Research
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Faculty of Science and Technology
Department of Civil Engineering

Course: Soil Mechanics I

(Mécanique des Sols I)

2nd Year – Civil Engineering

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Academic Year 2025–2026

Presentation

This course handout is intended for 2 second-year civil engineering students. It presents the fundamental elements of the Soil Mechanics course. The course contains everything necessary for practical calculations in terms of principles, methods, formulas, and tables. In this context, it represents a handout covering the chapters of the official syllabus. The document is organized into fiver chapters, each presenting the course material, accompanied, where possible, by examples whose solutions are worked out during the lecture.

Teaching objectives: The student will be able to characterize the physical parameters of soils, classify them based on laboratory and in-situ identification tests, and become familiar with seepage in soils.

Recommended prior knowledge: Fundamental subjects from Semesters 1 and 2.

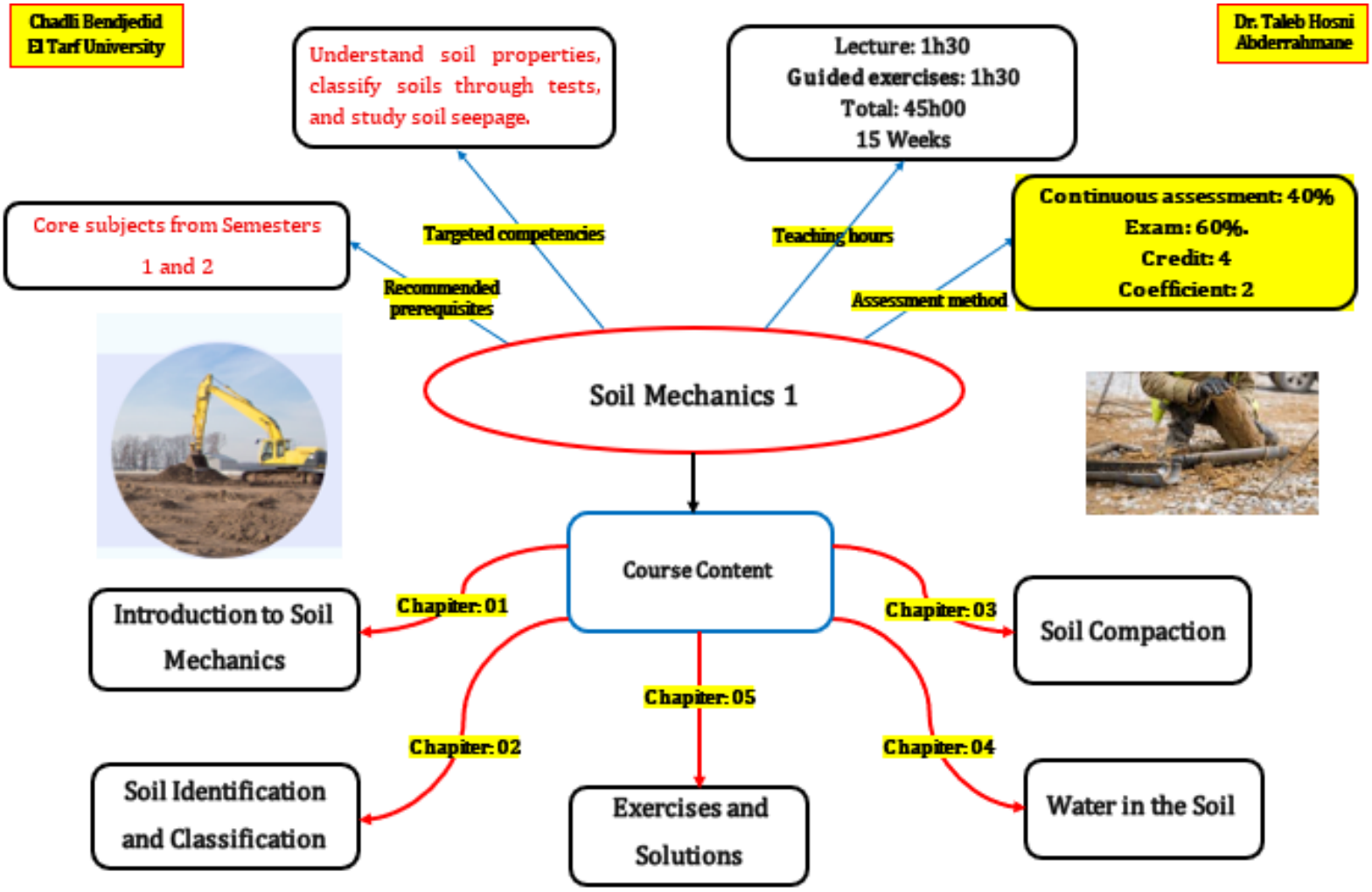
Finally, I would appreciate the reader's comments, corrections, and suggestions on the document's first version.

Dr Taleb Hosni Abderrahmane

El Tarf, March 12, 2026

Mind Map

Teaching Unit: **UEF 2.2.1**



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Chapter I:

Introduction to Soil Mechanics

Chapter I: Introduction to Soil Mechanics

1. Introduction:

Soil mechanics, by exploring the complex behavior of the materials constituting the surface layer of the earth's crust, plays an essential role in the design and construction of sustainable infrastructures. This chapter examines the history and scope of soil mechanics, highlighting the definition, origin, and formation of soils, as well as soil structure (granular soils and fine-grained soils). This course (SM) aims to deepen the understanding of soil behavior and to guide engineers in foundation design, landslide prevention, and other critical aspects of geotechnical engineering.

2. Objeftives of soil mechanics

2.1. History of soil mechanics

a. Geotechnical engineering before the eighteenth (18th) century:

The origin of the initial use of soil as a building material by humans dates back to ancient times. As far as engineering itself is concerned, the understanding of geotechnical engineering as it is known today began in the early **18th** century. For many years, the art of geotechnical engineering has relied mainly on previous experiments, going through a series of experiments with no real scientific basis. From these experiments, various structures were erected, some experiencing collapses while others still remain today.

Starting around 2700 BC (Jesus Christ), several pyramids were built in Egypt, most of which were built as tombs for the country's pharaohs and their wives during the Old and Middle Kingdom periods.

The construction of the pyramids posed formidable challenges regarding the foundations, the stability of the slopes, and the construction of underground chambers.



Figure 1.1: Pyramids, underground chambers

One of the most famous examples of problems related to soil bearing capacity in the construction of structures prior to the eighteenth century is the **Leaning Tower of Pisa in Italy.**

- Construction of the tower began in 1173 AD when the Republic of Pisa was flourishing and continued in several stages for over 200 years.
- The structure weighs approximately 15,700 metric tons and is supported by a circular base with a diameter of 20 m.
- The tower has tilted in the past to the east, north, west, and, finally, south.
- Recent research has shown that a weak clay layer existed at a depth of about 11 m below the ground surface, whose compression caused the tower to tilt. It became more than 5 m out of plumb with the height of 54 m.



Figure 1.2: The Leaning Tower of Pisa, Italy

b. Geotechnical engineering after the eighteenth (18th) century:

Based on the importance and nature of studies in the field of geotechnical engineering, the period from 1700 to 1927 can be divided into four main periods:

Preclassic (1700-1776)

Classical Soil Mechanics – Phase I (1776 to 1856)

Classical Soil Mechanics – Phase II (1856-1910)

Modern soil mechanics (1910 to 1927).

The evolution of soil mechanics can be traced by observing its emergence as an independent science, as well as the development of its main theories (see Table 1.1).

Table 1.1: Summary of the history of soil mechanics

Centenary	Author	Theory or events
18th	Coulomb.	Shear Strength
19th	Collin	Failure in clay slopes
	Darcy	Water flow through sand
	Rankine	Earth pressure on retaining walls
	Gregory	Horizontal drainage, compacted embankment with buttresses to stabilize railway trench slopes
20th	Atterberg	Consistency limits of clay
	Terzaghi	First modern manual of soil mechanics
	Casagrande	Liquid limit tests
	Taylor, D. W	Fundamentals of soil mechanics
	Peck R. B.	History of foundation engineering
	Caquot	Stability of granular or cohesive soil masses

3. Definitions

3.1. Geotechnical engineering

Geotechnics encompasses the study of the physical, mechanical, and hydraulic characteristics of soils and rocks, as well as their applications in the field of civil construction. Although it is based on a theoretical basis shared with other engineering disciplines such as material strength, geotechnics also has an empirical aspect, drawing its conclusions in part from data obtained during laboratory and field tests. In practice, it is closely related to geology, which is devoted to the description of the

materials making up the Earth's crust and the study of their internal and external transformation processes.

Geotechnical engineering is subdivided into three distinct fields: soil mechanics, rock mechanics, and foundation engineering. Our attention will be mainly directed to soil mechanics, while foundation engineering will be studied in the following years in the **M I** and **II** geotechnical master's degree.

In the field of foundation engineering, the use of data from soil mechanics and rock mechanics, as well as geological studies, is crucial to make decisions about the nature and dimensions of structure foundations, thus ensuring their stability.

3.2. Soil mechanics

In soil mechanics, soil is characterized as the stratum of loose materials, of mineral and organic origin, resting on the bedrock. These floors can be used as building materials or as foundations for civil engineering works. The analysis of the physical, mechanical, and hydraulic properties of soils aims to classify them and understand their behavior.

3.3. Rock

In geotechnics, the term "rock" refers to a massive natural aggregate composed of mineral matter. In geology, the term encompasses any constituent element of the earth's crust, which includes rocks in the geotechnical sense but also other elements such as soil, oil, groundwater, etc.

3.4. Soil

Soil is a heterogeneous arrangement of particles or crystals characterized by highly variable properties such as size, shape, and physicochemical characteristics. These elements can be disrupted under relatively light mechanical actions. Soils are typically loose, porous, heterogeneous, and often anisotropic materials. Whether of mineral or organic origin, they generally consist of grains or particles with highly variable shapes and sizes.

Soil can be defined as an uncemented aggregate of mineral grains and decomposed organic matter (solid particles), with liquids and gases occupying the voids between these solids. Soil is widely used as a construction material in various civil engineering projects and serves as the support for structural foundations. For this reason, civil engineers must study soil properties, including origin, grain size, drainage capacity, compressibility, shear strength, and bearing capacity. Soil mechanics is the scientific field dedicated to studying the physical properties of soils and the behavior of soil masses under various types of forces. Soil engineering involves applying the principles of soil mechanics to practical problems. Geotechnical engineering, a sub-discipline of civil engineering,

utilizes natural materials found near the earth's surface. It encompasses the application of soil mechanics and rock mechanics principles to the design of foundations, retaining structures, and earthworks.

4. Fields of application of soil mechanics

Depending on the type of structure and its design method, the foundation may serve as a support for only a few elements of the building. These elements can include isolated footings, continuous footings, rafts, or piles. These types of foundations are typically used in constructions such as buildings, bridges, sheds, and similar structures.

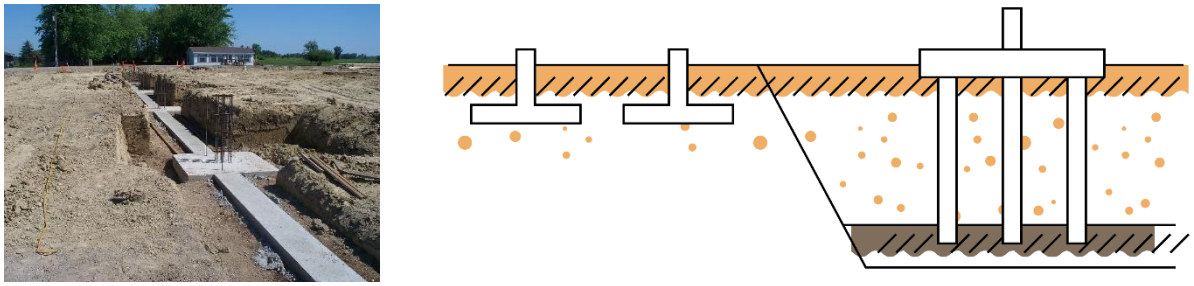


Figure 1.3: The superficial and deep foundations

The role of foundations is to transfer the loads of a structure to the ground. However, since soils vary in their bearing capacity, two categories of foundations are commonly used (Figure 1.3): shallow foundations and deep foundations.

The ground can also serve as a support for an entire structure, such as a road, highway, tunnel, gravity dam, retaining wall, or airfield.



Figure 1.4: Examples of structures supported by the ground: dam, highway, tunnel, road, and retaining wall

Soil mechanics is the science that encompasses all the knowledge and techniques necessary to identify the properties governing the mechanical behavior of soils and to analyze soil-structure interaction. It also addresses issues related to embankments, earth structures, and the stability of slopes and fills (Figure 1.5).

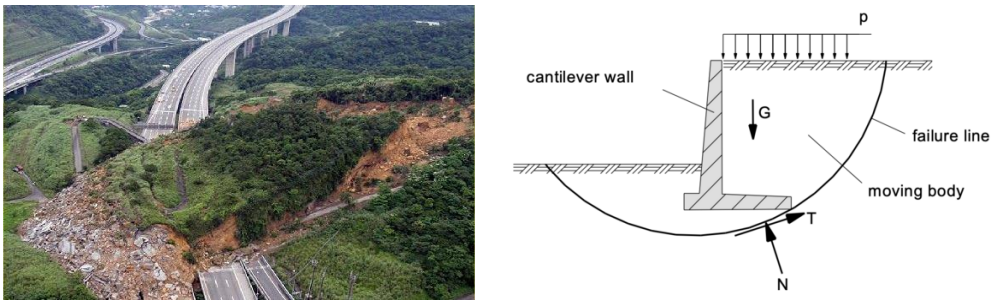


Figure 1.5: Landslide

The supporting soil is considered an integral part of the structure whose role is to transmit the overall loads of the structure to the earth while remaining sufficiently stable and resistant. Therefore, the success of the work depends on the success of the foundation project.



Figure 1.6: Structural instability (Mila, 07/08/2022)

5. Rock cycle and soil origin:

Typically, soils are formed by the process **of weathering** rocks. The **physical characteristics** of a soil are mainly determined by the **minerals** present in **its particles**, influenced by the **original rock**. This section provides an overview of the rock cycle, soil genesis, and particle size distribution within a soil mass.

Depending on how they are formed, rocks can be categorized into three basic types: igneous, sedimentary, and metamorphic. Figures 1.7 and 1.8 show diagrams illustrating the formation cycle of the different types of rocks and the processes related to them. This cycle is commonly referred to as the rock cycle. Brief discussions on each element of the rock cycle will be presented later.

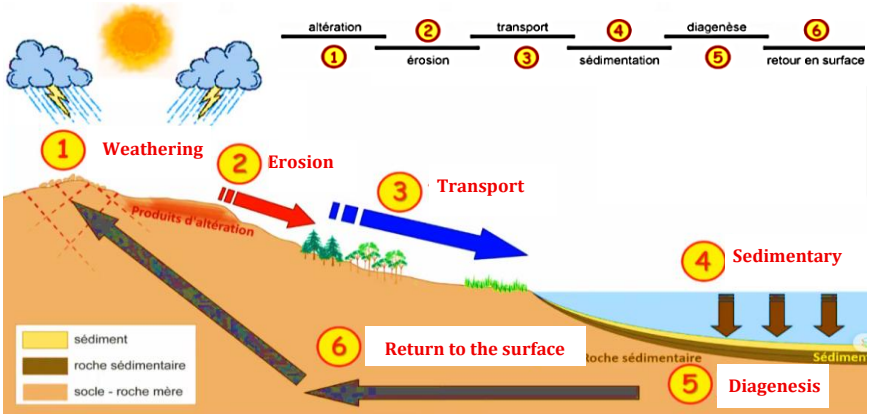


Figure 1.7: The cycle of rocks and soils

5.1. Types of rocks:

A rock: an assemblage of minerals; A mineral is an ordered assembly (crystal) of atoms. There are 3 major groups of rocks that are differentiated by their mode of formation:

5.1.1. Magmatic rocks (or igneous rocks): they result from the crystallization of a magma;

5.1.2. Sedimentary rocks: these are exogenous rocks, that is to say, formed on the surface of the Earth;

5.1.3. Metamorphic rocks: rocks resulting from the transformation, in the solid state, of an original rock due to an increase in pressure and/or temperature. During this transformation, there is usually crystallization of new minerals (newly formed), and the rock acquires a particular texture (e.g., schistosity = flow in sheets).

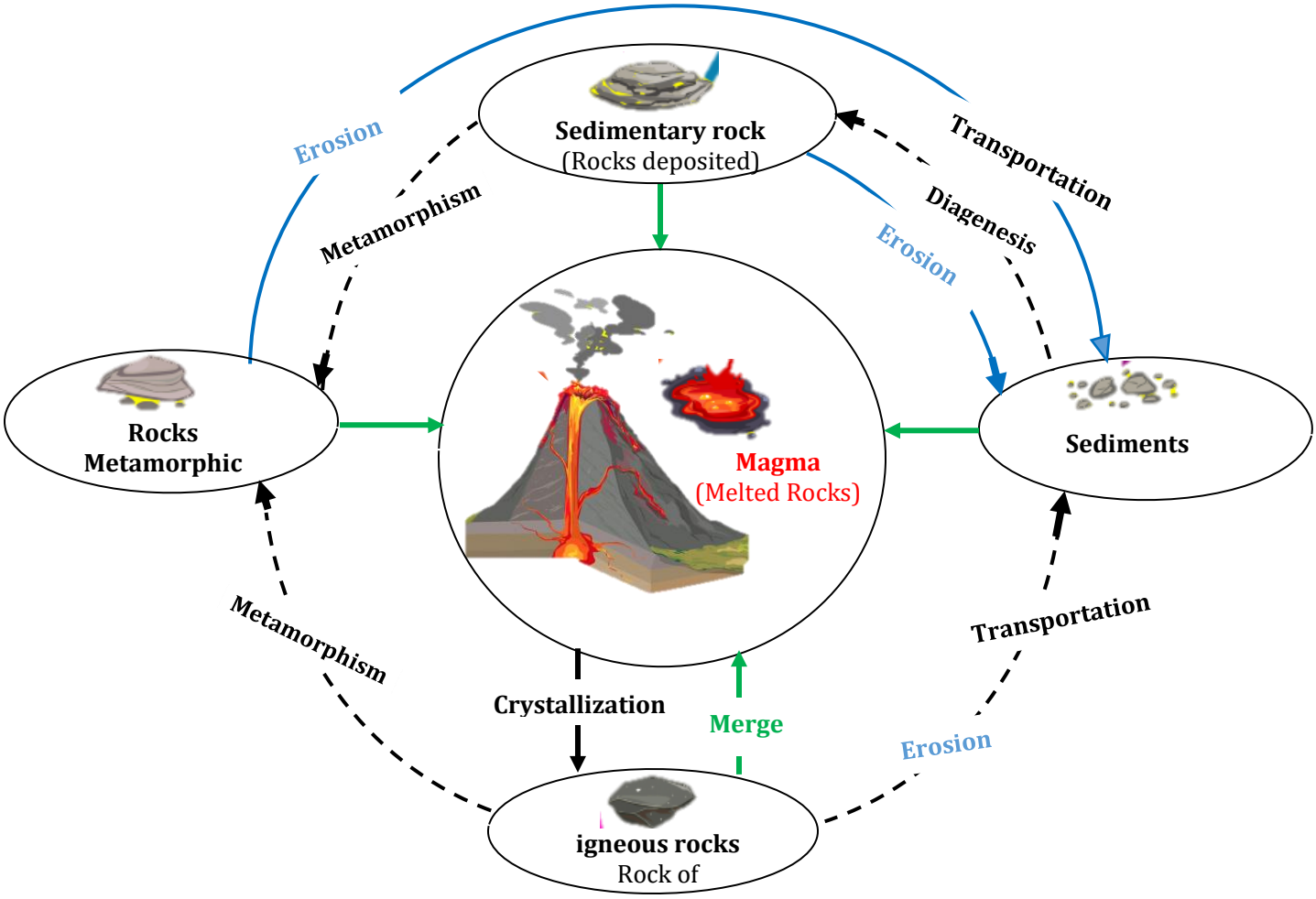


Figure 1.08: The cycle of rocks and soils of magmatic origin

5.2. Weathering:

The processes responsible for transforming intact rock (the 'bedrock') into its decomposition products are called weathering:

a. When the particles remain in place, alteration is at the origin of the formation of soils (called **pedogenesis**).

b. When particles are transported away by agents such as water or wind, the process is referred to as **erosion**.

5.3. Weathering processes:

5.3.1. Physical or mechanical alteration (example: changes in temperature, the effect of impacts, etc.)

5.3.2. Chemical alteration, which is all the stronger as the temperature and humidity are high, especially in a tropical climate (we witness, for example, phenomena of dissolution, hydrolysis)

5.3.3. Biological alteration (example: roots of trees that fracture rocks, burrowing animals).

6. Soil Structure (Granular Soils and Fine-Grained Soils)

The various types of soils are generally classified according to the size of their particles. This rapid approach offers fundamental information on the hydraulic and mechanical properties of soils. In soil mechanics, the most basic classification is divided into two broad categories: coarse-grained soils and fine-grained soils.

Coarse-grained soils include those whose particles are perceptible to the naked eye. Their equivalent diameter is generally greater than 0.08 mm, since below this limit, the human eye has difficulty in discerning shapes, even reaching a few meters in the case of large rocks. Fine-grained soils logically include all soils whose particles are not visible to the naked eye. The equivalent diameter of these particles can vary from about 1 nm (10^{-6} mm) to 0.08 mm.

6.1. Coarse-grained soils:

Large-sized stones, also known as boulders or riprap, have an equivalent diameter greater than **80** mm. They are distinguished by their remarkable permeability. Due to their imposing dimensions and weight, these rock fragments are complex to handle, making them particularly suitable for large-scale engineering projects such as the construction of earth dams or port developments. These works require the use of robust mechanical machinery.

These stones are used as bulk materials to reinforce the stability of structures and prevent erosion caused by water.

Gravel and **sand** consist of rock particles with equivalent diameters ranging from **0.08** mm to **80** mm. These materials generally exhibit excellent permeability. They are easily compacted and remain stable once compacted, enabling them to support large loads without further settlement. Due to their smaller size compared to larger blocks and their favorable properties, gravel and sand are commonly used as foundation materials in numerous engineering projects, including roads, railways, airstrips, and buildings. Additionally, they serve as drainage and filtration materials in applications such as earth dams, building foundations, and filtration systems.

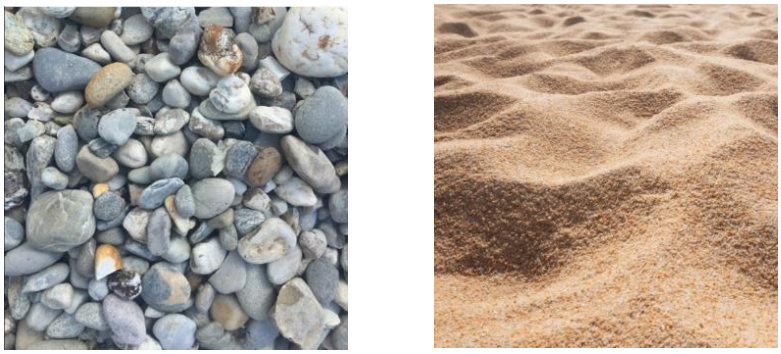


Figure 1.9: Gravies & Sand

6.1.1. Shape:

This is the description of the geometric shape of the grains, as illustrated in Figure 1.10 and Table 1.2. **Cubic or spherical particles:** These dimensions predominate in coarse-grained soils. For a more precise characterization, the following adjectives are used: rounded, subrounded, well-rounded, angular, and subangular.

Table 1.2: Some typical forms of coarse grains

Class	1	2	3	4	5	6
	Very angular	Angular	Subangular	Subrounded	Rounded	Well-rounded
High Sphericity						
Low Sphericity						



Figure 1.10: Some typical forms of coarse grains

6.1.2. Elementary structure of coarse-grained soils

Particles, regardless of their size or shape, blend into the soil to create a diversity of structures. In coarse-grained soils, the particles adopt a fundamental arrangement where each grain is firmly positioned between its neighbors, thus representing the extreme elementary structures (the most compact and loose), the dense structure, the loose structure, and the honeycomb structure (Figure 1.11).

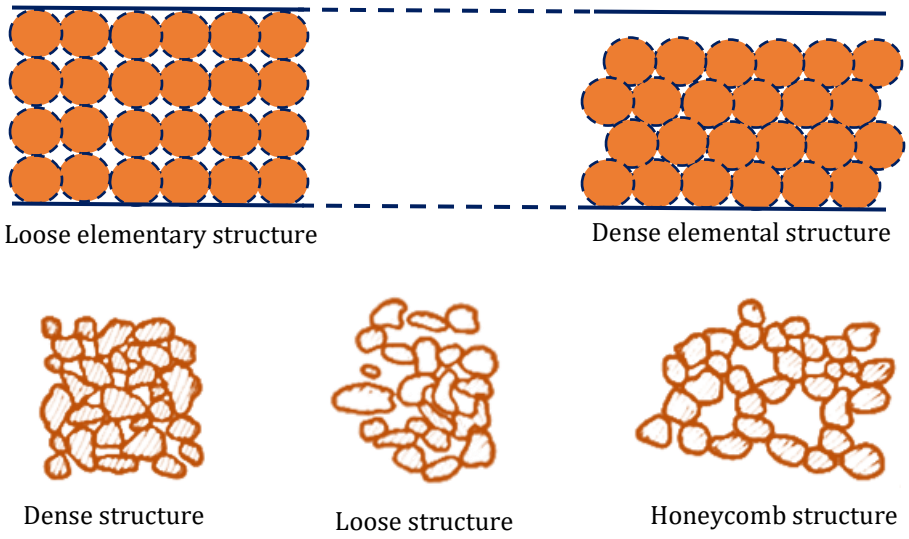


Figure 1.11: Arrangement of coarse-grained soils

6.2. Fine-grained soils

6.2.1. Silt :

Silt consists of fine rock particles with equivalent diameters ranging from 0.002 mm to 0.08 mm, and their shapes can be observed using a magnifying glass or an optical microscope. Silt composed of small, roughly spherical **quartz** particles is sometimes referred to as rock flour. Generally, even at similar degrees of compaction, silt cannot bear loads as high as those supported by gravel or sand.

Its compressibility is also greater than that of these coarser soils, leading to increased settlement. Moreover, its permeability is very low.



Figure 1.12: Siltstone (Siltite rock) containing silt and sand

6.2.2. Clay:

Clay consists of crystalline particles resulting from the chemical decomposition of rock components. According to several authors, their equivalent diameter varies from approximately **1 nm** to **0.002 mm**. Particles with an equivalent diameter of less than 0.0005 mm remain invisible under an optical microscope; their shape and relief require more advanced techniques, such as electron microscopy, to be detected. Due to its almost total impermeability, clay is frequently used as a sealing material in the core of earthen dams or dikes. However, its high compressibility usually involves a much lower carrying capacity than gravel and sand (Figure 1.13).



Figure 1.13: Clay

Unlike particles in other soil types, those in clay are subject to mutual **attraction** that causes them to clump together. This attractive **force**, known as **cohesion**, is responsible for the **consistency** of the material. In the presence of adequate moisture, this consistency gives the clay a **plasticity** that allows it to be shaped, a characteristic that is virtually impossible to achieve with gravel or sand.

6.2.3. Shape:

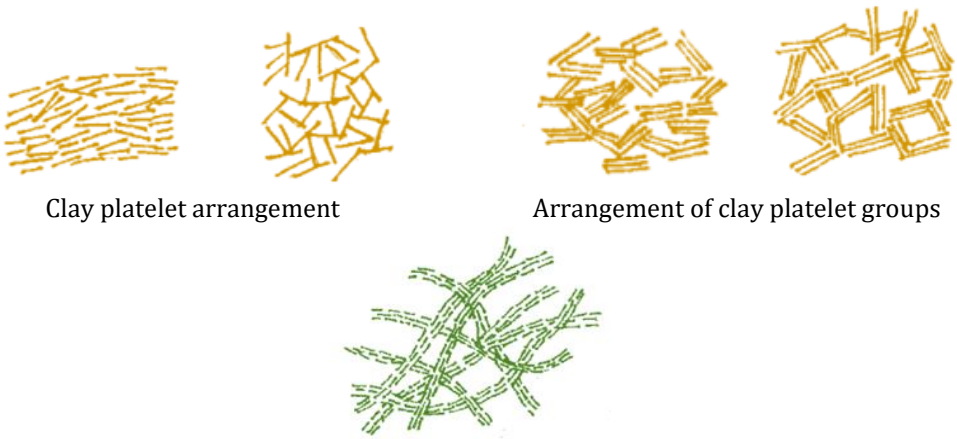
This is the description of the geometric shape of the grains, as illustrated in Figure 1.14.

- **Platelet particles:** Typical for fine-grained soils.
- **Stick or needle particles:** This shape is less responsive in soil.



Honeycomb structure Flocculated structure

Figure 1.14: Arrangement of fine-grained



Clay platelet arrangement Arrangement of clay platelet groups

Entanglement of clay clusters

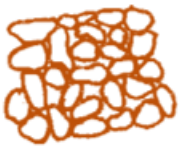
Figure 1.15: Different arrangements of clay platelets
(based on an introduction to geotechnics)



Matrix of clayey particles



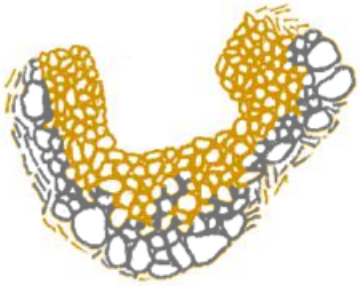
Entanglement of clay clusters with silt inclusions



Silt and sand grains



Silt platelets and sand grains

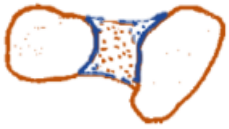


Matrix of granular particles



Partially discernible matrix between particles

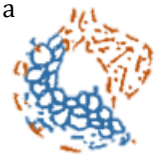
Figure 1.16: Arrangement of solid particles of different sizes



Sand or silt arrangement with a binder



Regular aggregate arrangement with sand or silt grains



Regular aggregate arrangement with a fine particle matrix



Irregular aggregates held by a binder



Irregular aggregates forming a honeycomb

Figure 1.17: Arrangement of solid particles of different sizes

- **Grain size**

When a soil consists of particles of variable sizes, particle size analysis (see the next chapter) makes it possible to study the distribution of particles according to their dimensions. However, an initial qualitative description can be made with the naked eye (See Table. 1.3, and Figure 1.18).

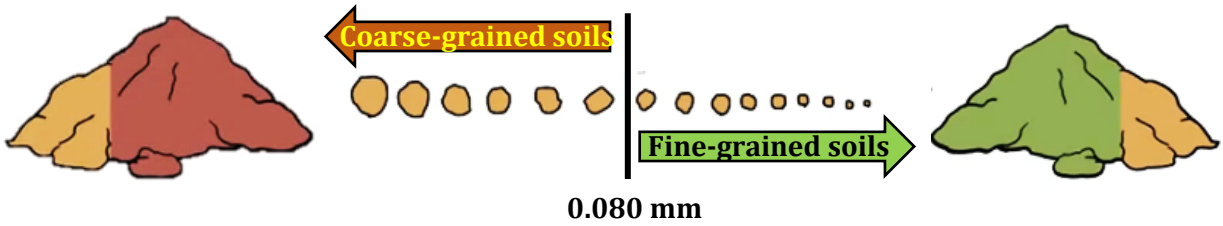


Figure 1.18: Granular and fine-grained particle sizes

Note: Extreme particle sizes may vary slightly depending on the classification system used.

Table 1.3: Textural properties of soils

Property	Gravel	Silt	Clay
Particle size	Coarse particle, visible to the naked eye	Fine particle, invisible to the naked eye	Very fine particle, invisible to the naked eye
Characteristics	Granular, loose (non-cohesive), non-plastic	Granular, loose (non-cohesive), non-plastic	Cohesive, plastic
Effect of water	Of little importance	Important	Very important
Effect of particle size distribution	Important	Of little importance	Of little importance

Chapter II

Soil Identification and Classification

Chapter II: Soil Identification and Classification

1. Introduction

This chapter presents the physical characteristics of soils, grain-size analysis, the consistency of fine-grained soils (Atterberg limits), and soil classification systems.

2. Description of a soil

2.1. Three-phase medium:

A soil is defined by three phases: gaseous, liquid, and solid (Figure 2.1). From the volume and weight proportions, we define geometric and water parameters such as:

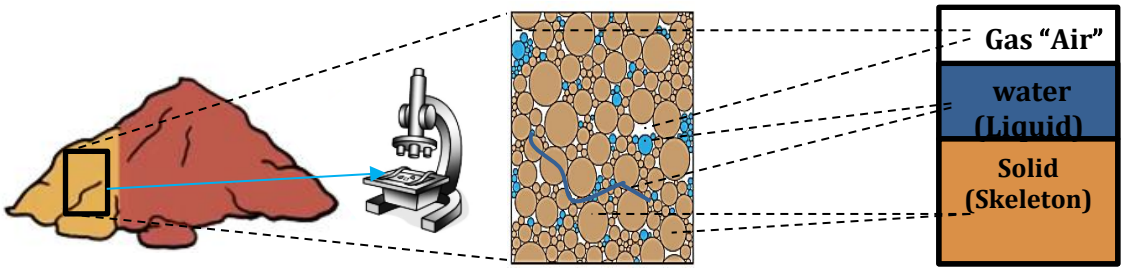


Figure 2.1: Soil element in natural state; three phases of the soil element

Figure 2.1 shows a soil element of volume V and weight W, as it would exist in the natural state. To develop the weight-volume relationships, we need to separate the three phases (i.e., solid, water, and air) as shown in Figure 2.2. Thus, the total volume of a given soil sample can be expressed as follows:

2.2. Weight-Volume Relationships

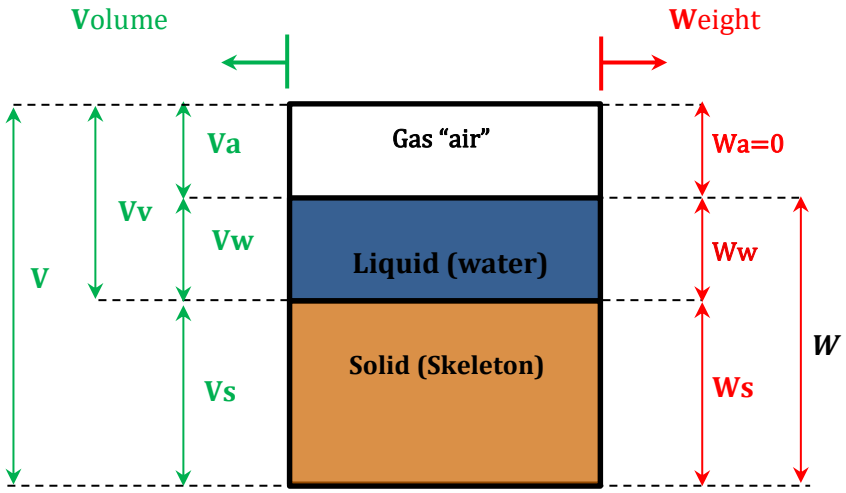


Figure 2.2: Representation of a soil volume

2.3. Conventional ratings:

The weights

W: Total Soil Weight

W_s: Weight of soil solids;

W_w: Weight of water;

W_0: Air weight = 0.

Volumes

V: Total volume

V_s : Volume of soil solid;

V_v: Volume of voids;

V_w: Volume of water in the voids;

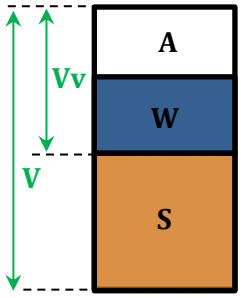
V_a : Volume of air in the voids.

$$W = W_s + W_w;$$

$$V_v = V_w + V_a;$$

$$V = V_s + V_v = V_s + V_w + V_a$$

3. Physical characteristics common to different soils



Porosity, Void Ratio, and Relative Density

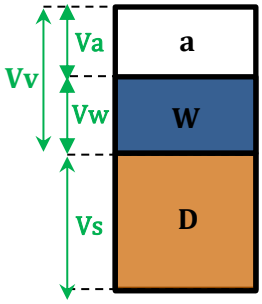
3.1. Porosity: denoted by ***n,*** it is the ratio of the volume of voids (water and air) to the total volume of the soil. It makes it possible to know the importance of voids—that is to say, whether the ground is in a loose or tight state. (n = **0** to **1**).

$$n\,(\%) = \frac{V_v}{V} X\ 100 \qquad (2.01)$$

3.2. Compactness (c): In a volume equal to unity, the solid grains occupy the volume 1-n, called compactness.

$$c = 1 - n \qquad (2.02)$$

3.3. The void ration: Noted as e, it is the ratio of the volume of voids to the volume of solid grains. It is defined by the relationship:



$$e\,(\%) = \frac{V_v}{V_s} X\ 100 \qquad (2.03)$$

This definition results in relationships

$$e = \frac{n}{1-n} \quad et \quad n = \frac{e}{1+e} \qquad (2.04)$$

3.4. Relative density (density index)

It is defined by the expression

$$I_d = \frac{e_{max} - e}{e_{max} - e_{min}} \qquad (2.05)$$

Were

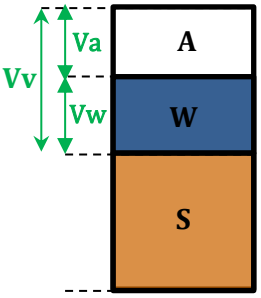
e_{min} is the void ratio corresponding to the most compact (denser) state;

e_{max} is the void ratio corresponding to the loosest state.

e is the void ratio of the soil in place.

The indication of the density index gives an idea of the settlement state of a given soil: $I_d = 0$ for the loosest state ($e = e_{max}$) and $I_d = 1$ for the most compact state ($e = e_{min}$).

3.5. Degree of saturation and water content



a. **Degree of saturation:** Noted, S_r, it is the ratio of the volume occupied by water to the total volume of voids. It is expressed as a percentage (%).

$$S_r(\%) = \frac{V_w}{V_V} X\ 100 \qquad (2.06)$$

The degree of saturation makes it possible to classify the soil, as shown in Figures 2.3 and 2.4 and Table 2.1.

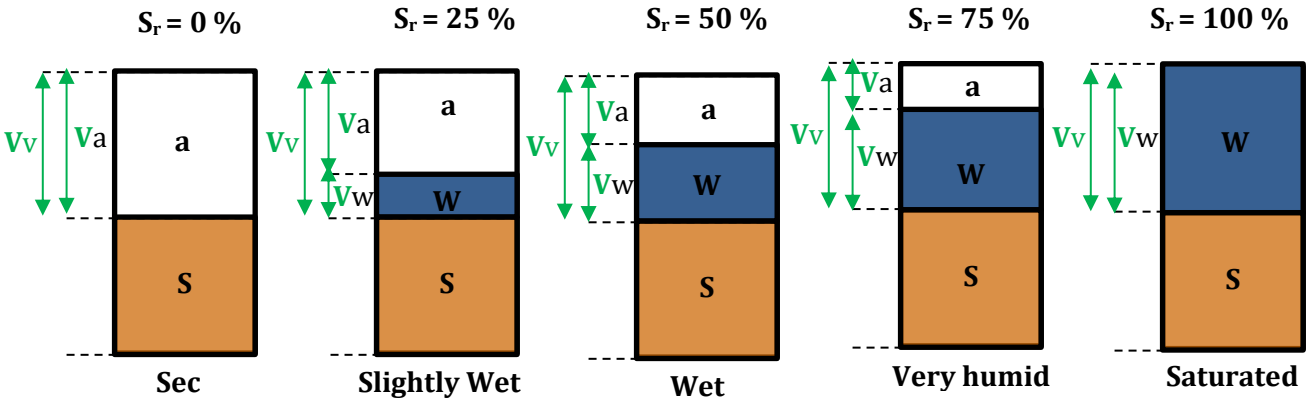


Figure 2.3: Saturation Ratio

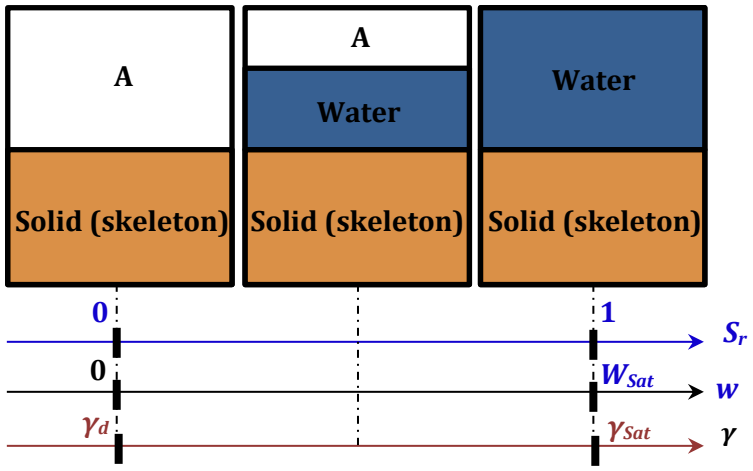


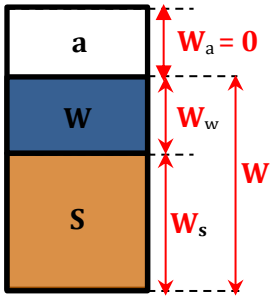
Figure 2.4: Soil description (saturation states)

Table 2.1: Saturation Ratio

Sr	State of soil
(0) %	Sec
(1-25) %	Slightly Wet
(25 – 50) %	Wet
(50 – 75) %	Very humid
(100) %	Saturated

b. Water content: Noted w, it is the ratio of the weight of water to the weight of dry matter. It is expressed as a percentage (%). It is easily measurable in the laboratory. Sand: *w* = 1 to 15%, clay: *w* = 10

$$w\ (\%) = \frac{W_w}{W_s} X\ 100 \qquad (2.07)$$



4. Unit Weights and Densities

Working with soil densities (ρ) can sometimes be convenient. The International Density System unit is (**kg/m³**)

Water content

$$w\ (\%) = \frac{M_w}{M_S} X\ 100 \qquad (2.08)$$

4.1. Apparent density or simply **density is the mass** per unit volume of the soil in question

$$\rho = \frac{M}{V} = \frac{M_S + M_v}{V} = \frac{M_S(1+w)}{V} \qquad (2.09)$$

4.2. The dry density is the mass of the dry matter contained in the unit of volume

$$\rho_d = \frac{M_S}{V} = \frac{\rho}{(1+w)} \qquad (2.10)$$

4.3. The weight in kN/m³ can be obtained from the densities in Kg/m³ as.

$$\gamma\ (\frac{KN}{m^3}) = \frac{g\rho\ (Kg/m^3)}{1000} \qquad (2.11)$$

4.4. The weight of water (γ_w) is equal to **9.81** KN/m³. And ρ_w is equal to **1000** kg/m³

4.5. Volume Weights (Dimensional Parameters):

In most cases, the determining factor is the density. Thus, let us designate by (γ) the weight related to the mass, which leads to the following definitions:

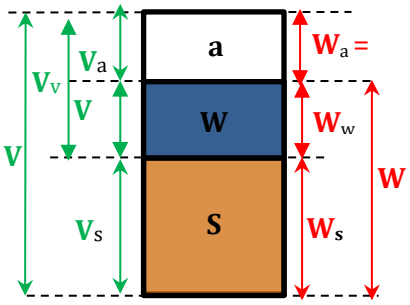


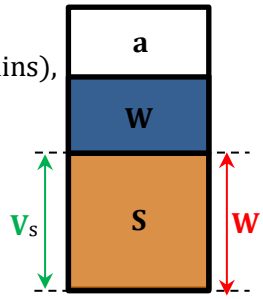
Figure 2.5: Description of a soil (the weight and volume relationship)

$$\gamma = \frac{W}{V} \qquad (2.12)$$

γ_s γ' γ_w γ γ_d γ_{sat}

4.6. The density of the solid particles (of the material constituting the solid grains), rated γ_s

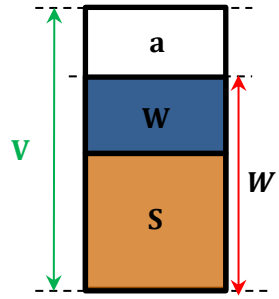
$$\gamma_s = \frac{W_s}{V_s} \qquad (2.13)$$



4.7. The density of the water, noted γ_w

$\gamma_w = 9.81\ kN/m^3$. We often take $\gamma_w =$ 1 t/m³ = 1 g/cm³ = 10 KN/m³ = 1000 Kg/m³. Which leads to 2% error from the outset

$$\gamma_w = \frac{W_w}{V_w} \qquad (2.14)$$



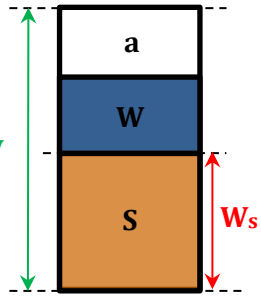
4.8. The density of the soil (or apparent density or weight wet volume), noted γ. This is the ratio of the total weight (particles solids and water) or total soil volume.

$$\gamma = \frac{W}{V} = \gamma_h \quad (2.15)$$

The density of dry soil, denoted γ_d:

If the soil is dry: $\gamma = \gamma_d$.

$$\gamma_d = \frac{W_s}{V} \quad (2.16)$$



For the compaction "**Chapter 03**"

$$\gamma_d = \frac{\gamma_h}{1+w} \quad (2.18)$$

$$\gamma_h = \frac{W}{V} = \frac{M.g}{V} = \rho.g \quad (2.19)$$

4.9. The density of saturated soil, denoted γ_{sat}: when all voids are filled with water.

$$\gamma_{sat} = \frac{W}{V} = \frac{W_s + V_v.\gamma_w}{V} = \quad (2.20)$$

4.10. Submerged unit weight (γ'): This is the apparent weight of solid grains bathing in water. (It is taken into account when the ground is fully immersed. It takes into account the presence of water that fills all the voids and the buoyancy of Archimedes). It is shown that it is given by

$$\gamma' = \gamma_{sat} - \gamma_w \quad (2.21)$$

4.11. Density:

Sometimes we use densities with respect to water:

a. The wet density, noted d_h

$$d_h = \frac{\gamma_h}{\gamma_w} \quad (2.22]$$

b. The dry density, noted d_d

$$d_d = \frac{\gamma_d}{\gamma_w} \quad (2.23)$$

c. The density of solid grains, noted d_s

$$d_s = \frac{\gamma_s}{\gamma_w} \quad (2.24)$$

d. Specific Gravity of Solid Grains, noted G_s

G_S = (2.6 to 2.8).

$$G_s = \frac{\gamma_s}{\gamma_w} = \frac{W_s}{V_s \gamma_w} = \frac{\rho_s}{\rho_w} \quad (2.25)$$

e. The gauged density, noted **from** d'

$$d' = \frac{\gamma'}{\gamma_w} \quad (2.26)$$

5. Relations between weight parameters – volumetric

Relations between weight and volumetric parameters in soil mechanics can be deduced from the following fundamental properties:

1) Void ratio (e)

$$e = \frac{V_v}{V_s}$$

2) Porosity (n)

$$n = \frac{V_v}{V}$$

3) Degree of saturation (S_r)

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

4) Water content (w)

$$w = \frac{W_w}{W_s}$$

5) Unit weight (γ)

$$\gamma = \frac{W}{V}$$

6) Specific Gravity (G_s)

$$G_s = \frac{W_s}{V_s \gamma_w}$$

6. Methods for finding the relationship between properties

There are two ways to find the relationship between the fundamental properties of soil.

6.1. First method:

In this method, the relationship between the properties is inferred through the basic soil properties **(e, n, S_r, w, G_S),** which are simplified (Until the desired relationship is reached).

6.2. Second method (Phases diagram):

In this method, the relationship between the properties is deduced as follows:

- Remember the basic properties (**e, n, S_r, w, G_S**)
- Draw a floor phase diagram.
- Assume one of the following cases (**$V_S = 1$, or $V = 1$, or $W_S = 1$**) depending on the data and complete the rest of the variables
- Find the required relationship from the soil phase diagram.

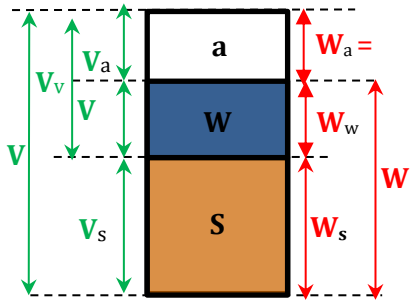
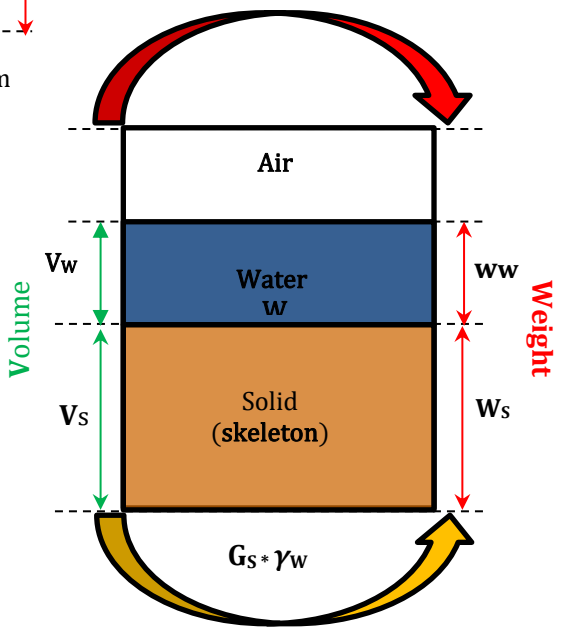


Figure 2.6: Phase diagram

The relationship between volume and weight

$$W_w = \gamma_w \, x \, V_w \quad (2.27)$$

$$W_s = G_s x \, \gamma_w x V_s \quad (2.28)$$



6.3. Relationships between Settings

Not all previously defined parameters are independent. The most important relationships between these different parameters are given as follows:

$$[1]\ n = \frac{Vv}{V} \qquad [5]\ e = \frac{Vv}{Vs} \qquad [9]\ w = \frac{Ww}{Ws}$$

$$[2]\ n = \frac{e}{1+e} \qquad [6]\ e = \frac{n}{1-n} \qquad [10]\ w = e.Sr.\frac{\gamma_w}{\gamma_s}$$

$$[3]\ n = 1 - \frac{\gamma_d}{\gamma_s} \qquad [7]\ e = \frac{\gamma_s}{\gamma_d} - 1 \qquad [11]\ w = \frac{\gamma}{\gamma_d} - 1$$

$$[4]\ n = \frac{\gamma_s - \gamma_{sat}}{\gamma_s - \gamma_w} \qquad [8]\ e = \frac{\gamma_s - \gamma_{sat}}{\gamma_{sat} - \gamma_w} \qquad [12]\ w = Sr\,.\gamma_w\left(\frac{1}{\gamma_d} - \frac{1}{\gamma_s}\right)$$

$$[13]\ Sr = \frac{Vw}{Vv} \qquad [14]\ Sr = \frac{\gamma_s}{\gamma_w}.\frac{w}{e} \qquad [15]\ Sr = \frac{w}{w_{sat}}\ (\gamma_d \text{ constant})$$

$$[16]\ \gamma = (1 + w)(1 - n).\gamma_s \qquad [17]\ \gamma = \frac{1+w}{1+e}.\gamma_s \qquad [18]\ \gamma = (1 + w)\,\gamma_d$$

$$[19]\ \gamma = \gamma_d + n.Sr.\gamma_w \qquad [20]\ \gamma = \frac{\gamma_s + e.Sr.\gamma_w}{1+e} \qquad [21]\ \gamma = (1 - n).\gamma_s + n.Sr.\gamma_w$$

$$[22]\ \gamma_d = (1 - n)\,\gamma_s \qquad [23]\ \gamma_d = \frac{\gamma_s}{1+e} \qquad [24]\ \gamma' = \gamma_{sat} - \gamma_w$$

$$[25]\ \gamma' = (1 - n)(\gamma_s - \gamma_w \qquad [26]\ \gamma' = \frac{\gamma_s - \gamma_w}{1+e} \qquad [27]\ \gamma' = \frac{\gamma_s - \gamma_w}{\gamma_s}.\gamma_d$$

a. Example:

A mass of a soil sample in its natural condition is equal to **2290 g,** with a volume of **1.15×10^{-3} m³.** After being completely dried in an oven, the mass of this sample became **2035 g**. The specific gravity value is **$G_s = 2.68$,** and g = **9.81** m/s².

b. Identify:

1. Determination of sample density
2. Bulk unit weight
3. Water content
4. Void ratio
5. Its porosity
6. Degree of saturation

c. Solution:

1. The density

$$\rho = \frac{M}{V} = \frac{2,290}{1.15 \text{ x } 10^{-3}} = 1991.3\ kg/m^3$$

2. Bulk unit weight

$$\gamma = \frac{W}{V} = \frac{g\rho\ (Kg/m^3)}{1000} = \frac{1991.3\ (9.81)}{1000} = 19.53\ \frac{KN}{m^3}$$

3. Water content

$$W(\%) = \frac{M_w}{M_S}\ X\ 100$$

$$M_w = M - M_s = 2.29 - 2.035 = 0.255\ kg$$

$$W(\%) = \frac{0.255}{2.035}\ X\ 100 = 12.53\ \%$$

4. Void ratio

$$e\ = G_s\ .\ (1 + w)\frac{\rho_w}{\rho} - 1 = 2.68\ .\ (1 + 0.1253)\frac{1000}{1991.3} - 1 = 0.514$$

$$e\ = 51.4\ \%$$

5. The porosity

$$n = \frac{e}{1 + e} = 0.34$$

$$n = 34\ \%$$

6. Degree of saturation

$$S_r = \frac{G_s w}{e} = \frac{2.68x0.1253}{0.514} = 0.6533$$

$$S_r = 65.33\ \%$$

7. Grain Size Analysis, Consistency of Fine Soils (Atterberg Limits)

7.1. Sieve Analysis

This refers to the laboratory determination of the distribution of soil particles according to their size. The procedure includes specific steps, such as sieving for coarse soils (**sieve analysis**) and hydrometer testing for very fine particles (**hydrometer analysis**). In general, to interpret the results, a cumulative percentage passing curve (%) is plotted against the particle diameter (mm), as shown in Figure 2.7. In this context, coefficients such as the coefficient of curvature (**Cc**) and the uniformity

coefficient (**Cu**) are introduced to describe the grain size distribution.

Understanding the grain size of a soil through these tests (sieving and hydrometer analysis) is essential for providing an accurate description of the soil.

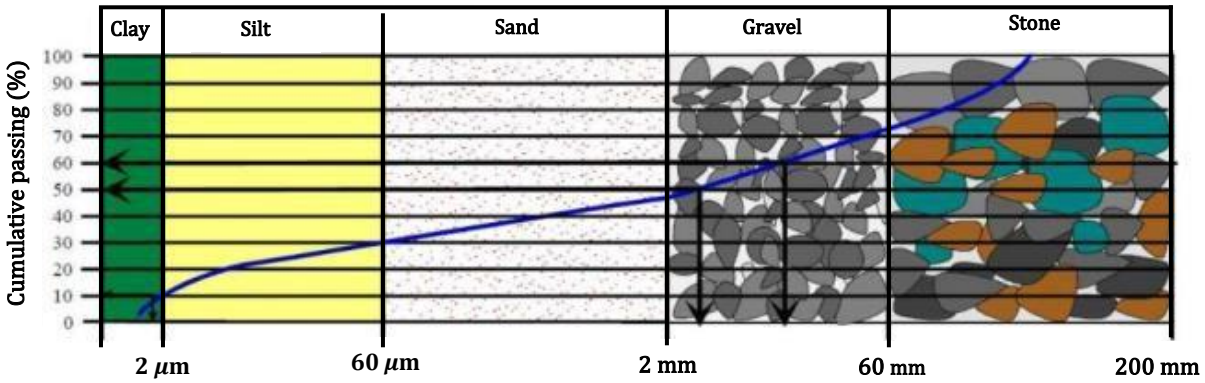


Figure 2.7: Particle-size distribution curve

- **Particle Size Analysis by Sieving (D > 0.08 Mm)**

The experiment involves passing a representative sample of soil through superimposed screens, with openings that decrease from top to bottom. Thus, larger particles are retained by the upper screens, while finer particles pass through the screens below.

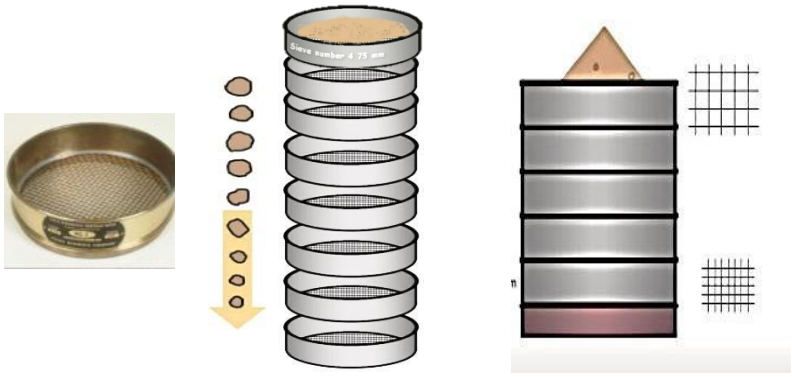


Figure 2.8: Overlay Sieve Series

Horizontal and vertical movements are applied to the sieves, either mechanically or manually, to ensure continuous contact between the soil sample and the sieve surfaces, thereby improving sieving efficiency. Particles retained on a sieve are referred to as **retained fractions**, while the soil passing through a sieve is referred to as the **passing fraction**.

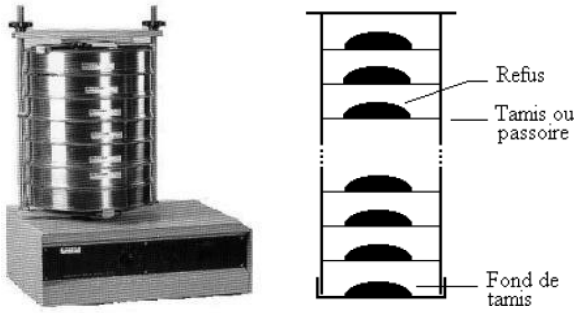


Figure 2.9: Sifter, and Sieve with Rejection

Once the weights retained on each sieve stabilize, the sieving process is considered complete, and the retained fractions are weighed. The mass of each retained fraction is then compared to the total sample mass, allowing the calculation of cumulative percentages of **retained** and **passing** soil.

The particle size distribution of a soil is represented by the sieve opening sizes and the percentage of particles passing through each sieve. Rather than quantifying particles of a specific diameter, the particle size indicates the proportion of particles smaller than the sieve opening, i.e., the fraction of particles that passes through each sieve. These data are then plotted on a **semi-logarithmic** graph to form the particle size curve.

7.2. Particle Size Analysis Using the Hydrometer Method (D < 0.08 Mm)

For particles with an equivalent diameter smaller than 0.08 mm, sieving becomes physically impractical. In this case, particle size analysis by sedimentation is used to estimate the size distribution of silt and clay particles.

This analysis is based on the relationships established in 1850 by the English physicist G.G. Stokes, which link the settling velocity of a sphere in a liquid to its diameter, the relative density of the sphere with respect to the liquid, and the liquid's viscosity.

$$V = \frac{9.8\, D^2 (D_{rs} - D_{rl})}{3\eta} \qquad (2.27)$$

$$\mathbf{D} = \sqrt{\frac{3V\eta}{9.8\,(D_{rs} - D_{rl})}} \qquad (2.28)$$

Were:

D: Sphere diameter (mm)

V: sphere falling speed (cm/min)

$\mathbf{D_{rs}}$: relative density of the sphere

$\mathbf{D_{rl}}$: relative density of the liquid

η: dynamic viscosity of the liquid (Pa.s)

It should be noted that sedimentation analysis provides only equivalent diameters, since Stokes' formula is valid exclusively for spherical particles.

7.3. Analysis of Grain Size Curves

At first glance, the analysis of a particle size curve allows for the identification of the soil components present in the examined sample. By examining the curve for a sample containing gravel, sand, silt, and clay, the respective proportions of each soil type can be determined (see Figure 2.10).

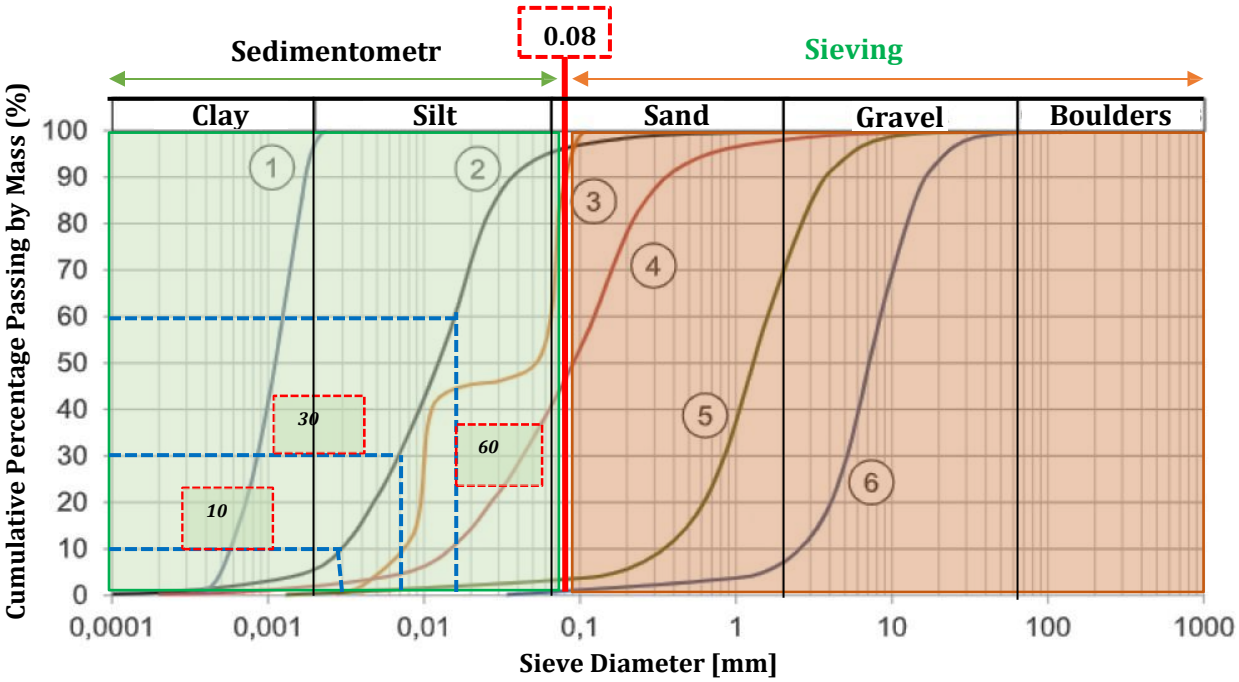


Figure 2.10: Particle Size Curves

Once these proportions are determined, the soil can be classified using the designations listed in Table 2.2.

Table 2.2: Soil Designation According to Proportion of Soil Types

Proportion of soil types	Terminology	Examples
> 35%	Name	Gravel, sand, silt, etc.
20% - 35%	Adjective	Gravelly, sandy, etc.
10% - 20 %	A little	A little silt, a little sand, etc.
<10%	Trace	Traces of clay, silt, etc.

- **Example:** In this case, with a composition of 27% gravel, 38% sand, 29% silt, and 6% clay, the soil would be classified as **gravelly silty sand with traces of clay**.

7.4. Particle size distribution curve

- In this context, coefficients such as the **coefficient of curvature (Cc)** and the **coefficient of uniformity (Cu)** are introduced to describe the particle size distribution.
- A particle size distribution curve can provide information for determining the following parameters for a given soil:

Effective size (**D10**): This is the particle diameter at which 10% of the soil by mass is finer on the particle size distribution curve.

- It is particularly useful for estimating hydraulic conductivity and drainage characteristics in granular soils.

Uniformity Coefficient (Cu): This parameter is calculated as the ratio of the diameter corresponding to 60% finer (D_{60}) to that corresponding to 10% finer (D_{10}).

- It provides an indication of the **uniformity** or **range of particle sizes** within the soil.

Uniformity coefficient Cu.

This coefficient is calculated as follows:

$$C_u = \frac{D_{60}}{D_{10}} \qquad (2.29)$$

Coefficient of curvature Cc

This coefficient is calculated as follows:

$$C_c = \frac{D_{30}^2}{D_{10}\, D_{60}} \qquad (2.30)$$

It is considered that when C_u is greater than **4** for gravel, and greater than **6** for sand, then $1 < C_c < 3$ gives a well-spread particle size.

7.5. Particle Size Distribution Curve

The particle size distribution curve of a soil not only indicates the range of particle sizes present, but also reveals the distribution pattern of these particles. Figure 2. 11 illustrates different types of distributions:

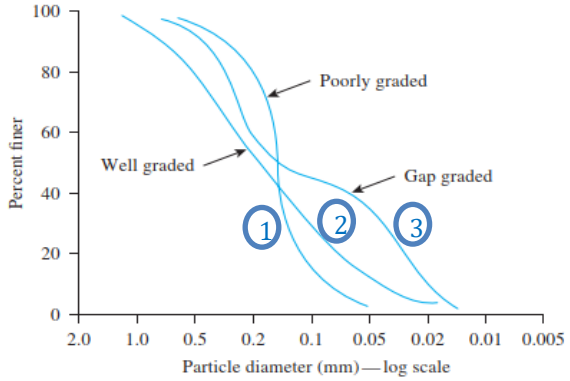


Figure 2.11: Different types of particle size distributions

- **Curve 1:** This represents **poorly graded** soil where the majority of the soil grains are of similar size. It lacks a wide range of particle sizes.
- **Curve 2:** This represents a **well graded** soil where particle sizes are spread over a wide range.

Well-calibrated soils have a **coefficient of uniformity** greater than about **4** for gravel and **6** for sand. The **gradation coefficient** (Curvature) for **well-graded** soils is generally between **1 and 3** for gravel and sand.

- **Curve 3:** This is **gapping graded soil**, i.e. soil with a combination of two or more uniformly graded fractions.

Gap-classified soils have gaps or missing particle sizes in the range of distribution.

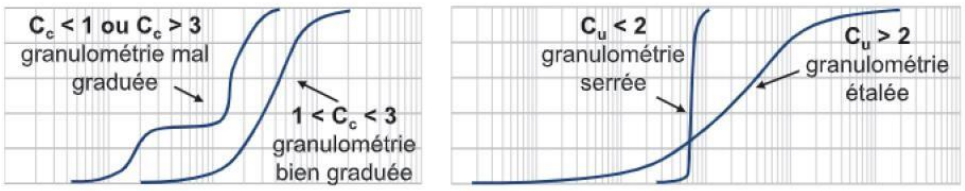


Figure 2.12: Interpretation of coefficients and C_u and C_c

7.6. Determination of Atterberg limits (Consistency of fine soils):

Clays form pastes in which each grain is bonded to neighboring grains through cohesive forces resulting from the presence of adsorbed water layers. The resulting consistency is largely influenced by the soil's water content. Accordingly, clays exhibit four states of consistency: liquid, plastic, semi-solid, and solid (see Figure 2.13).

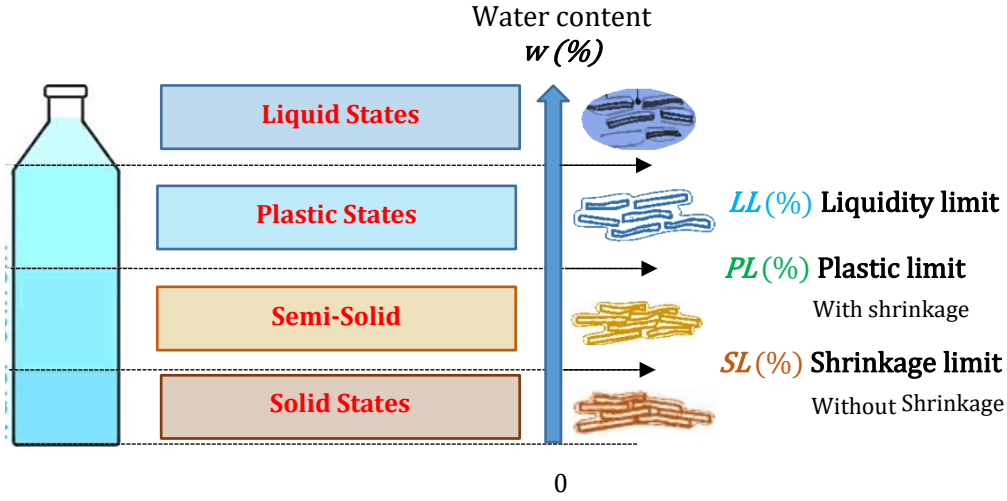


Figure 2.13: Soil consistency states

- **In the liquid state,** the grains are independent of each other, facilitating relative movement between the particles.
- **In the plastic state, the grains come closer together and share their adsorbed water layers.** When there is movement, the grains remain attached to each other without moving away.
- **In a semi-solid state,** the soil has a low water content, characterized by very strong cohesion bonds between its particles. Although the adsorbed water films are still thin, they create a slight separation between the particles. Thus, a drying process would result in soil removal. As a result, ground deformations under the influence of loads are always associated with the formation of cracks.
- **In the solid state,** the inter-granular distances are even smaller. The grains even come into contact at a few points, driving out the adsorbed water. Internal friction then becomes significant. The transition from one state to another is very gradual. Nevertheless, in a practical way, we refer to Atterberg's limits to describe these states.

7.7. Atterberg Limits

The Atterberg limits define the different stages of consistency of a soil according to its water content. These limits, expressed as percentages, include:

- **Liquidity limit LL:** It represents the water content that separates the liquid state from the plastic state of the soil. In some clays, it can reach up to 1000%, although in most cases it remains below 100%.

- **Plasticity limit PL:** It is defined as the water content of a soil that has lost its plasticity, cracking and deforming under low loads. This boundary marks the transition between the **plastic** state and the **semi-solid** state of the soil. It can range from 0% to 100%, but is usually less than 40%.

- **Shrinkage limit SL:** corresponds to the **maximum amount of water** that a soil can contain without undergoing a volume change. This limit marks the transition between **the solid state and the semi-solid state** of the soil, it characterizes the appearance of the **shrinkage** phenomenon.

The determination of liquidity and plasticity limits was initiated by Terzaghi in 1925, who introduced the associated measurement techniques. Casagrande, in 1933, then developed a standard apparatus for measuring the liquidity limit. Subsequently, in 1940, Casagrande used the liquid limit and plasticity index as the basis for soil classification.

7.8. Plasticity and liquidity indices:

- **The plasticity index PI**, expressed as a percentage, is defined as the difference between the liquid limit and the plasticity limit:

$$PI = LL - PL \quad (2.31)$$

It is used to delimit a range in which the soil can be considered to be in a plastic state, as illustrated in Figure 2.14.

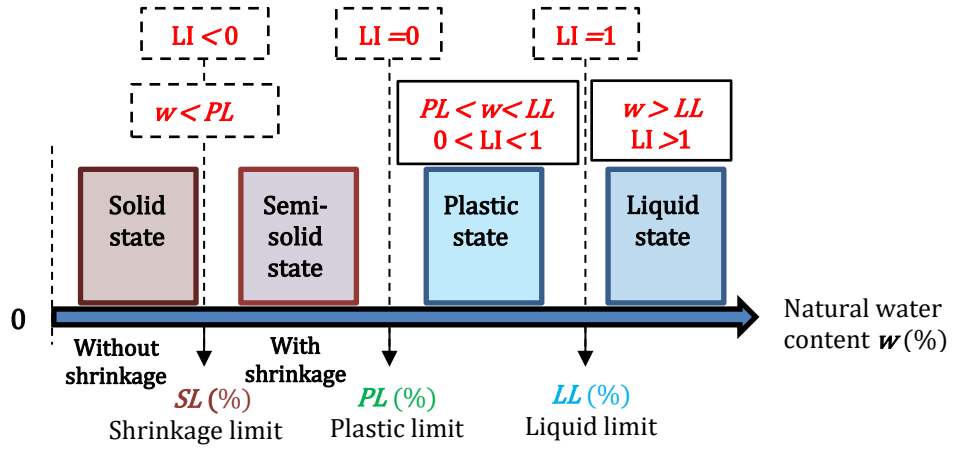


Figure 2.14: Atterberg limits natural water content and liquidity index

- **Consistency index CI:** This is a derived parameter that reflects the water condition of a soil:

$$CI = \frac{(LL - w)}{(LL - PL)} = \frac{(LL - w)}{(PI)} \qquad (2.32)$$

- **Liquidity index LI:** provides a quick indication of the state of a soil, whether it is liquid, plastic, semi-solid or solid. To calculate this index, we compare the natural water content (in situ) (w) of a soil to its plasticity and liquidity limits:

$$LI = \frac{(w - PL)}{(LL - PL)} = \frac{(w - PL)}{(PI)} \qquad (2.33)$$

w: The natural water content of the soil, measured on a sample containing no particles larger than 400 µm.

Table 2.3: Plasticity Index (PI) — Degree of Plasticity

Plasticity Index (PI)	Degree of Plasticity
0 < PI < 5	Non-plastic soils (granular soils)
5 < PI < 30	Low-plasticity soils
30 < PI < 50	Plastic soils
50 < PI	Highly plastic soils

8. Soil classification

Soil categorization is a method for establishing soil classifications in order to anticipate their behaviors. As a general rule, visual identification is sufficient to assign a name to the material, such as blue marl, yellow clay, fine sand, etc. Nevertheless, it is necessary to complete this indication with: Water content, density; A particle size analysis; Determination of Atterberg limits; Density index for powdery soils; and simple compressive strength for coherent soils.

8.1. Textural classification

Agronomists use this triangular classification, which is based on the relative mass proportions of the soil under examination. The classification systems are represented by triangular diagrams whose sides are scaled to show the proportions of sand, silt, and clay measured in a soil sample. The characteristics of sand, silt, and clay are defined with reference to a uniform particle-size distribution. In general, soil texture refers to the surface appearance and feel of the soil, which are influenced by the size of the individual particles present. Figure 2.15 illustrates the textural classification systems developed by the United States Department of Agriculture (USDA). This classification method is based on the grain-size limits defined by the USDA system, as follows:

- **Gravel size: greater than 2mm diameter**
- **Sand size: 2.0 to 0.05 mm diameter**
- **Silt size: 0.05 to 0.002 mm diameter**
- **Clay size: less than 0.002 mm diameter**

Floors must contain less than **30%** of elements with a diameter greater than **2 mm**

In most cases, natural soils are mixtures of particles from several size groups. According to the textural classification system, soils are designated by their main components, such as sandy clay, silty clay, etc.

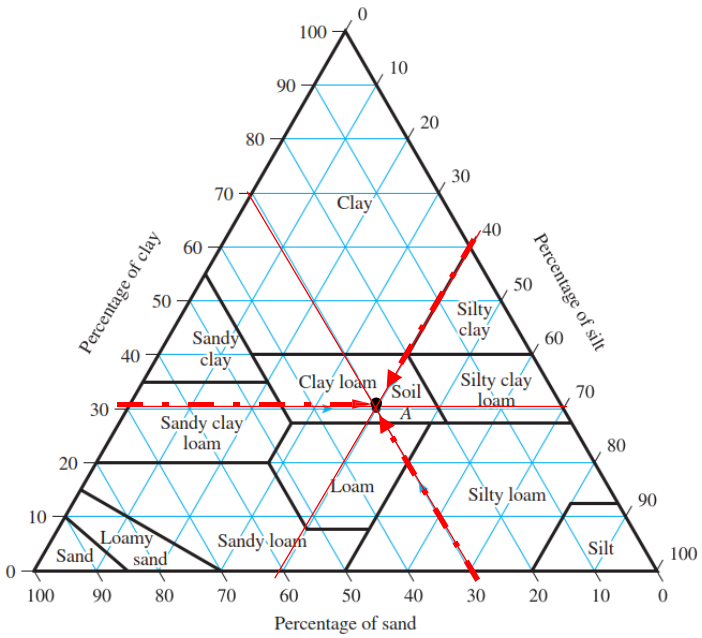


Figure 2.15: Triangular Classification of Agronomists

- **Example:** A soil sample A, which is composed of 30% **sand**, 40% **silt**, and 30% **clay**-sized particles.

By following the arrows on the map, we can determine that this soil falls into the classification of clay loam.

8.2. Classification According to Particle Size

Using the average diameter of soil particles as a reference, it is possible to establish a classification, such as that developed by Atterberg in 1905, which is widely adopted in France, or the MIT classification, which is preferred in Anglo-Saxon countries. However, it should be noted that this type of classification is rather rudimentary and is only suitable for coarse and homogeneous soils.

Table 2.4: Grain size classification

Atterberg Classification		**Anglo-Saxon Classification (MIT)**	
Soil Type	**Size (mm)**	**Soil Type**	**Size (mm)**
Rock fragments	$\Phi > 200$		
Stones	$200 > \Phi > 20$	Stones	$\Phi > 63$
Gravel	$20 > \Phi > 02$	Gravel	$63 > \Phi > 02$
Coarse sand	$02 > \Phi > 0.2$	Sand	$02 > \Phi > 0.06$
Fine sand	$0.2 > \Phi > 0.02$		
Silt	$0.02 > \Phi > 0.002$	Silt	$0.06 > \Phi > 0.002$
Clay	<0.002	Clay	<0.002

8.3. LCPC Classification (Central Bridge and Pavement Laboratory):

The classification is based on grain size for granular soils, and on grain size and Atterberg limits for fine soils. It follows the American classification (U.S.C.S.) but has been adapted to comply with French standards.

Fifteen basic soil classes have been established and defined, each represented by a two-letter symbol. The first letter, in uppercase, identifies the soil family, while the second letter, in lowercase, indicates the main characteristic related to particle size or consistency limits.

Tables 2.5, 2.6, and 2.7 list the soil names, their geotechnical properties, and the symbols used in soil mechanics.

Table 2.5: Soil designation

In French	In English
Tourbe **(T)**	Peat **(Pt)**
Organique **(O)**	Organic **(O)**
Grave **(G)**	Gravel **(L)**
Sable **(S)**	Sand **(S)**
Limon **(L)**	Silt (**M**)
Argile **(A)**	Clay (**C**)

Table 2.6: Geotechnical Characteristics

In French	In English
Bien Gradué **(b)**	Well Graduated **(W)**
Mal Gradué **(m)**	Poor Graduated **(P)**
Peu Plastique **(p)**	Low Plasticity **(L)**
Très Plastique **(t)**	High Plasticity **(H)**

Table 2.7: Designation of soils and symbols used

Description	**In French**	**IN ENGLISH**
Peat	**T**	**PT**
Well-graded gravel	**GB**	**GW**
Poorly graded gravel	**GM**	**GP**
Silty gravel	**GL**	**GM**
Clayey gravel	**GA**	**GC**
Well-graded sand	**Sb**	**SW**
Poorly graded sand	**Sm**	**SP**
Silt sand	**SL**	**SM**
Clay sand	**SA**	**SC**
Low plasticity silt	**Lp**	**ML**
Low plasticity clay	**AP**	**RT**
Organic soil, low plasticity	**Op**	**OL**
Highly plastic silt	**Lt**	**MH**
Highly plastic clay	**A**	**CH**
Organic soil, high plasticity	**Ot**	**OH**

This classification is based on the results of conventional soil identification tests:

- **Criteria for particle size:**

The proportions (%) of gravel, sand and fine particles (passing through the 2 mm and 0.08 mm sieves);

The configuration (shape) of the particle size curve: by the coefficient of uniformity C_u, and the coefficient of curvature C_c.

8.4. Plasticity characteristics

w_L and I_P, and the line A of equation: IP = 0.73 (w_L – 20) (Cazagrande relation) see Figure 2.16; and the organic matter content.

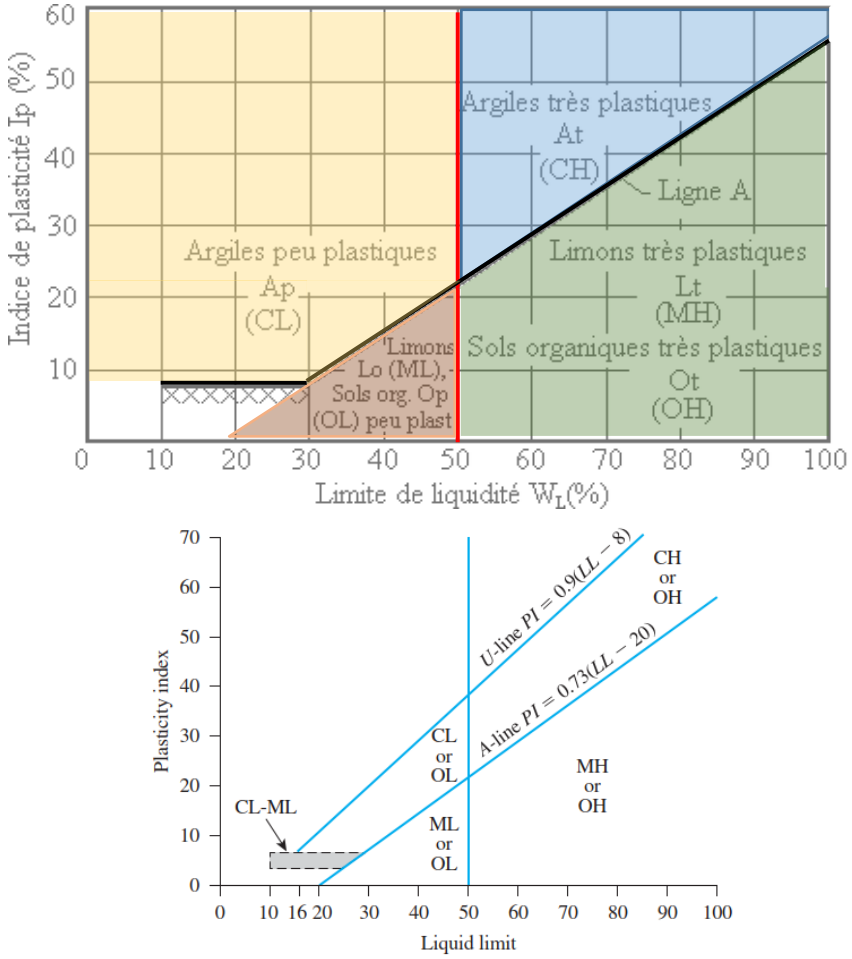


Figure 2.16: Casagrande diagram for the classification of fine soils
(More than 50% of elements < 0.08 mm)

Table 2.8: Classification of granular soils (more than 50% of elements > 0.08 mm)

Definitions			Symbol	Conditions	Names
Gravel	More than 50% of the particles > 0.08 mm have a diameter > 2 mm	Less than 5% of particles < 0.08 mm	GW	$C_u = D_{60}/D_{10} > 4$ and $1 < C_c = (D_{30})^2/D_{10}\,D_{60} < 3$	Well-graded clean Gravel
			GP	One of the GW conditions is not satisfied	Poorly-graded clean Gravel
		More than 12% of particles < 0.08 mm	GM	Atterberg limits below the A-line	Silty Gravel
			GC	Atterberg limits above the A-line	Clayey Gravel
Sand	More than 50% of the particles > 0.08 mm have a diameter < 2 mm.	Less than 5% of particles < 0.08 mm	SW	$C_u = D_{60}/D_{10} > 6$ and $1 < C_c = (D_{30})^2/D_{10}\,D_{60} < 3$	Well-graded clean Sand
			SP	One of the SW conditions is not satisfied	Poorly-graded clean Sand
		More than 12% of particles < 0.08 mm	SM	Atterberg limits below the A-line	Silty Sand
			SC	Atterberg limits above the A-line	Clayey Sand
When the fine particle content (< 0.08 mm) is between 5% and 12%, a dual symbol is used. For example: SW-SM					

8.5. There are other classifications, including:

- **Algerian (Provisional) Classification:** The Algerian classification is codified by the decree of 10 June 1991 issued by the Ministry of Housing. It is based on particle-size distribution and a number of simple laboratory tests.

- **GTR Soil Classification System: The GTR (Guide des Terrassements Routiers)** classification system categorizes soils according to their **particle-size distribution, fines content, plasticity, and water sensitivity**.

- **HRB** (Highway Research Board) Classification;

- **AASHTO (American Association of State Highway and Transportation Officials) classification**;

- **Unified Soil Classification System, Casagrande Classification (1948); and Organic Soils Classification.**

Chapter III

Soil Compaction

Chapter III: Soil Compaction

1. Introduction

It is likely that the in-situ soil is highly compressible, highly permeable, and of low consistency. If the use of another construction site is not possible, the only viable solution is soil stabilization, that is, the improvement of the properties of the soil in question. This improvement in soil properties can be achieved through various methods:

- **Chemical process:** This process involves mixing or injecting chemicals into the soil, such as Portland cement, lime, asphalt, calcium or sodium chloride, as well as pulp and paper residues;
- **Thermal treatment:** This method consists of heating the soil to modify its properties.
- **Electrical process:** This involves the application of an electric current to the soil in order to induce changes in its characteristics.
- **Mechanical process:** This process mainly focuses on soil compaction and densification.
- **Other methods:** These include vertical drains to reduce pore water pressures, as well as preloading and temporary loading to limit settlement.

2. Definitions

Compaction is defined as the set of mechanical operations involving the application of mechanical energy, aimed at increasing the density of a soil.

$$\gamma_d = \frac{W_s}{V} \qquad (2.16)$$

So, if γ_d the total volume V

Through this process, the soil structure is densified, thereby reducing deformations and compressibility, while increasing soil density and improving its bearing capacity. Structures commonly subjected to compaction include road embankments, earth dams, and airfields.

The purpose of compaction is to rearrange the soil grains by bringing them closer together (see Figure 3.1). To achieve this, the material must contain an appropriate amount of water. The

presence of water on the surface of the grains facilitates their movement by acting as a lubricant and ensuring cohesion. As the grains are brought closer together, the contact surfaces increase, thereby improving intergranular bonding (Figure 3.2).

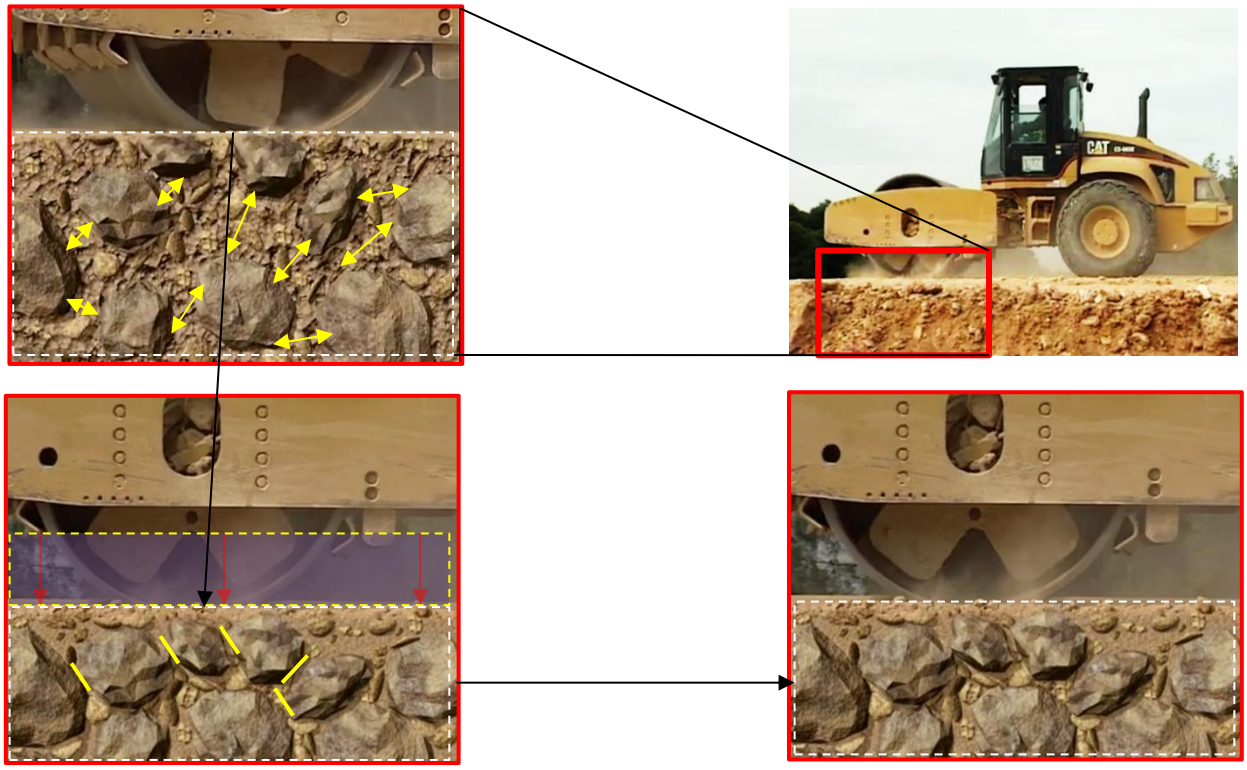


Figure 3.1: Compaction of a soil layer

Figures 3.2, 3.3, and 3.4 illustrate the behavior of soil grains under the effect of compaction with different water contents. With an optimum water content, the grains can move more easily relative to one another, as shown in Figure 3.2.

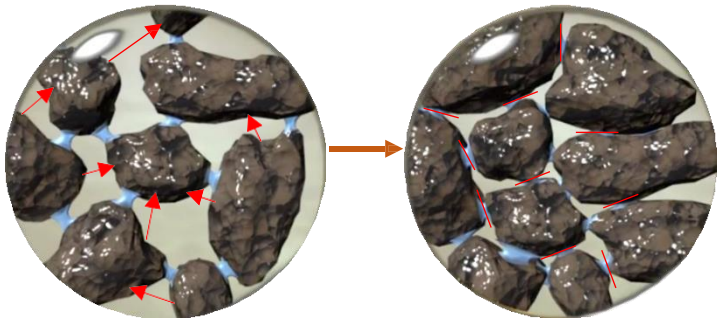


Figure 3.2: Compaction with good dosage

When the water content is insufficient, the grains cannot slide relative to one another and do not fill the voids. As a result, the contact surface is reduced and intergranular bonding is weaker.

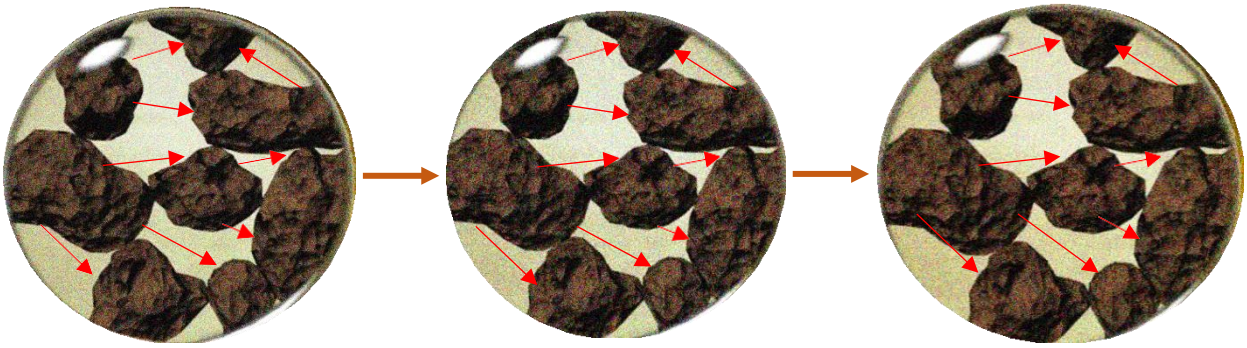


Figure 3.3: Compaction with insufficient water between grains

When the water content is excessive, intergranular bonding is no longer ensured. The grains slide past one another and become separated.

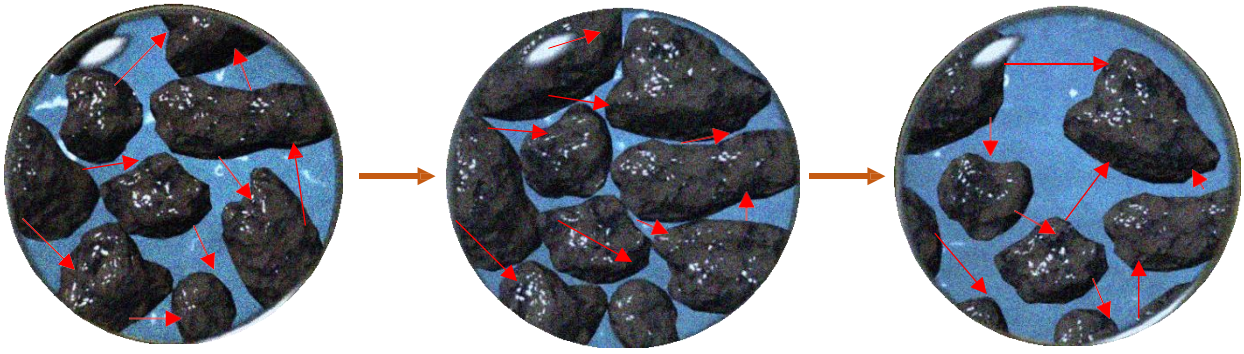


Figure 3.4: Compaction with excessive water between grains

Mechanical soil densification can result in:

- Modification of particle arrangement (structure);
- Change in water content;
- Reduction or elimination of settlement hazards;
- Increased soil strength and slope stability;
- Improved load-bearing capacity;
- Limitation of volume variations caused by frost action, swelling, and shrinkage.

3. Compaction theory (Proctor theory)

Proctor showed that the compaction process depends on four variables: the dry density of the soil, ρ_d; the moisture (water) content, ω; the compaction energy; and the nature of the soil (such as grain size distribution, mineralogy, etc.).

Equation (2.09) in Chapter 02 defines the bulk density, also referred to as apparent density, as the mass per unit volume of the soil considered.

$$\rho = \frac{M}{V} = \frac{M_s + M_v}{V} = \frac{M_s(1+w)}{V} \quad (2.09)$$

The **dry density** is the mass of the solid (dry) particles contained in a unit volume of soil.

$$\rho_d = \frac{M_s}{V} = \frac{\rho}{1+w} \quad (2.10)$$

The **weights** in kN/m^3 can be obtained from the densities in kg/m^3 using the following relation:

$$\gamma\left(\frac{KN}{m^3}\right) = \frac{g\rho\ (Kg/m^3)}{1000} \quad (2.11)$$

Dry soil density (γ_d):

$$\gamma_d = \frac{W_s}{V} \quad (2.16)$$

$$\gamma_d = \frac{\gamma_h}{1+w} et\ \gamma_h = \frac{W}{V} = \frac{M.g}{V} = \rho.g \quad (2.\ 15, \text{et } 16)$$

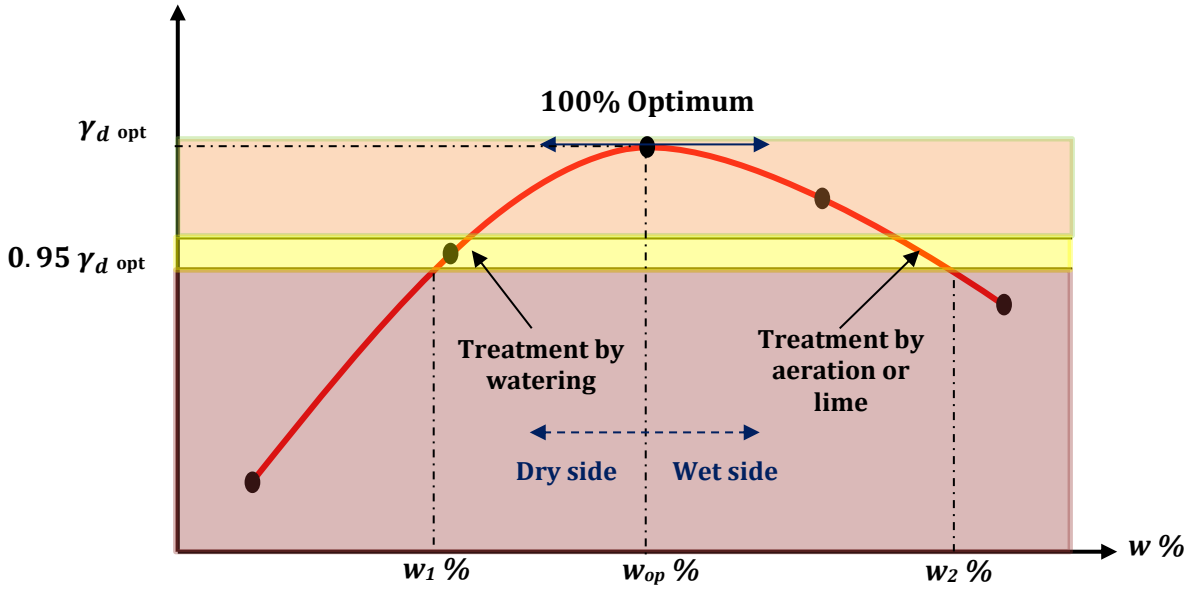


Figure 3.5: Compaction Curve

4. Factors Affecting the Compaction Process are Numerous and Include:

4.1. Water content

When the water content (moisture content) is high, a large portion of the compaction energy is absorbed by the water without contributing to soil densification. Conversely, when the water content is low, it acts as an important lubricant, facilitating particle rearrangement and increasing the dry density according to the water content (Figure 3.5).

4.2. Soil Type

The compaction curves show variations depending on the nature of the soil (Figure 3.6). They show great flatness in the case of sands, indicating that their compaction is little influenced by the water content. Materials in this category are therefore considered to be high quality embankments.

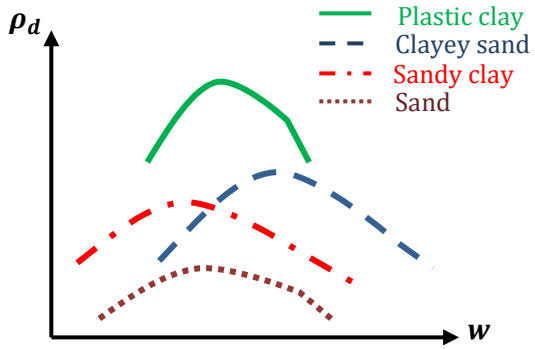


Figure 3.6: Influence of soil type

4.3. Compaction energy

Increasing the compaction energy results in a higher optimum dry density and a lower optimum water content (Figure 3.7).

Saturation Lines: When all the air present in the soil is expelled during the compaction process, the soil reaches a state of full saturation (S_r = 100% and A = 0). At this point, the dry density of the soil attains its maximum possible value for a given water content. This maximum value, known as the "saturation value," can be determined using the following expression:

$$\rho_d = \frac{G_s * \rho_w}{1 + G_s * w} = \frac{\rho_w}{\frac{1}{G_s} + w} \quad (3.01)$$

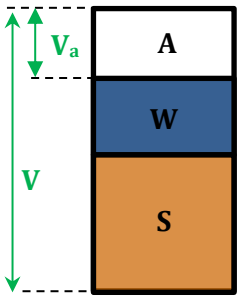
$$\gamma_d = \frac{G_s * \gamma_w}{(1 + G_s * w)} = \frac{\gamma_w}{\frac{1}{G_s} + w} \quad (3.02)$$

In practice, it is impossible to achieve full saturation. Regardless of the compaction method or effort applied, a certain amount of air always remains in the soil. Therefore, the dry density can be calculated based on the air content **A**:

$$\rho_d = \frac{(1 - A)G_s * \rho_w}{1 + G_s * w} = \frac{\rho_w}{\frac{1}{G_s} + w} \quad (3.03)$$

$$\gamma_d = \frac{(1 - A) * G_s * \gamma_w}{(1 + G_s * w)} = \frac{\gamma_w}{\frac{1}{G_s} + w} \quad (3.04)$$

$$A = \frac{V_a}{V} \quad (3.05)$$



For an air content of 5% (A=0.05)

$$\rho_d = \frac{(1 - 0.05)G_s * \rho_w}{1 + G_s * w} \quad (3.06)$$

$$\rho_d = \frac{0.95 * G_s * \rho_w}{1 + G_s * w} \quad (3.07)$$

For an air content of 10% (A=0.1)

$$\rho_d = \frac{0.90 * G_s * \rho_w}{1 + G_s * w} \quad (3.08)$$

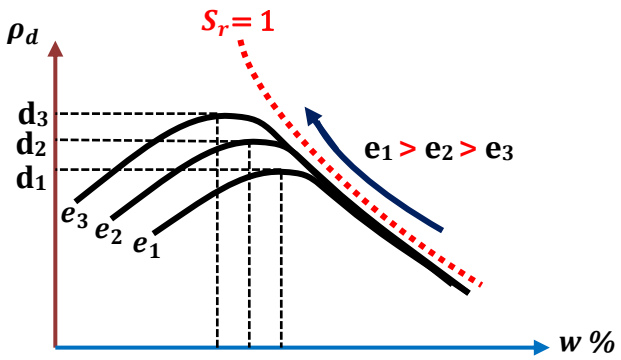


Figure 3.7: Influence of compaction energy

5. Effects of the Compaction Process on Various Soil Properties:

Compaction increases the stability of soils and their ability to withstand loads; however, it simultaneously alters several of their characteristics. The most affected properties include soil structure, permeability, swelling and shrinkage, compressibility, and shear strength.

5.1. Soil Structure:

Soil structure is influenced by compaction, resulting in a reduction of the void ratio. In granular soils, this process leads to densification of the individual grain structure. In cohesive soils, compaction causes a reorganization of clay particles, which tend to align parallel and adopt a more dispersed structure.

5.2. Permeability:

Soil permeability is affected by compaction, as the reduction of voids decreases the soil's ability to transmit water. Increasing the compaction energy further reduces the void ratio, leading to a corresponding decrease in permeability.

5.3. Swelling and Shrinkage:

Soil swelling and shrinkage are reduced due to the decreased void ratio and lower permeability. This reduction limits both the soil's capacity to retain water and its susceptibility to additional water intake. Consequently, volume changes, including those caused by swelling and shrinkage, are

mitigated. It is important to note, however, that compacted clay with a water content above the optimum is more prone to shrinkage, whereas compacted clay with a water content below the optimum is more prone to swelling.

5.4. Compressibility:

The compressibility of a soil is directly related to the volume of voids within it. As the void volume increases, soil compressibility also increases, potentially leading to significant settlements. Therefore, by reducing the voids through compaction, the compressibility of the soil is decreased, thereby mitigating the risk of excessive settlement.

5.5. Shear Strength:

In granular soils, compaction brings the particles closer together, increasing interparticle friction and interlocking, which enhances shear strength.

For **cohesive soils**, shear strength is primarily governed by cohesive forces, which increase as the distance between particles decreases. Thus, compaction, by reducing this distance, improves shear strength.

In general, the shear strength of clay soils is higher when the water content is below the optimum during compaction. Compaction not only increases shear strength but also enhances the internal friction angle.

Since the internal friction angle is directly related to the soil's bearing capacity, its increase through compaction contributes to enhancing the soil's load-bearing capacity.

6. Laboratory Compaction Tests (Proctor and CBR – California Bearing Ratio)

6.1. Objective:

The Proctor test aims to determine the optimum water content (W_{opt})for a specific backfill soil under defined compaction conditions. This optimum water content corresponds to the level at which maximum compaction or the highest bearing capacity of the soil can be achieved. The purpose of the test is to establish the compaction curve.

- **Procedure:**

The test consists of compacting a soil sample in a standardized mold using a standard tamper, following a defined procedure. After compaction, the water content and dry density of the sample are measured. This operation is repeated on several samples with different water contents.

As a result, multiple points on a (γ_d; w) curve are obtained. This curve exhibits a maximum, where the abscissa corresponds to the optimum water content and the ordinate corresponds to the optimum dry density ($\gamma_{d\,opt}$).

6.2. Types of tests

It may be necessary to choose either the Standard Proctor test or the Modified Proctor test. The former is recommended for embankment works, while the latter is intended for pavement layers.

6.2.1. Standard Proctor Test:

Compaction is only moderately applied. This method is commonly used in earth embankment studies, particularly for dams and dikes. The procedure consists of compacting three layers using the *Standard Proctor rammer.* The compaction energy is defined as follows:

- 25 blows per layer in the C.B.R. mould,
- 25 blows per layer in the Standard Proctor mould.

6.2.2. Modified Proctor Test:

Compaction is significantly more intensive and, in principle, corresponds to the maximum level of compaction achievable in the field using modern heavy sheepfoot rollers or pneumatic-tired rollers. In general, the Modified Proctor test is used to evaluate compaction characteristics such as the optimum moisture content and the maximum dry density of materials intended for use in road foundations or pavement structures, as well as airfield pavements.

In this case, compaction is carried out in five successive layers using the *Modified Proctor rammer*. The compaction energy is defined as follows:

- 55 blows per layer in the C.B.R. mould,
- 25 blows per layer in the Proctor mould.

6.3. Moulds and Rammers:

Two types of moulds with different dimensions are used for these tests:

- **Proctor mould:** used when the material is sufficiently fine (no particles larger than 5 mm; $\Phi \leq$ 5 mm)
- **CBR (California Bearing Ratio) mould:** used for materials with particle sizes greater than 5 mm and less than 20 mm.

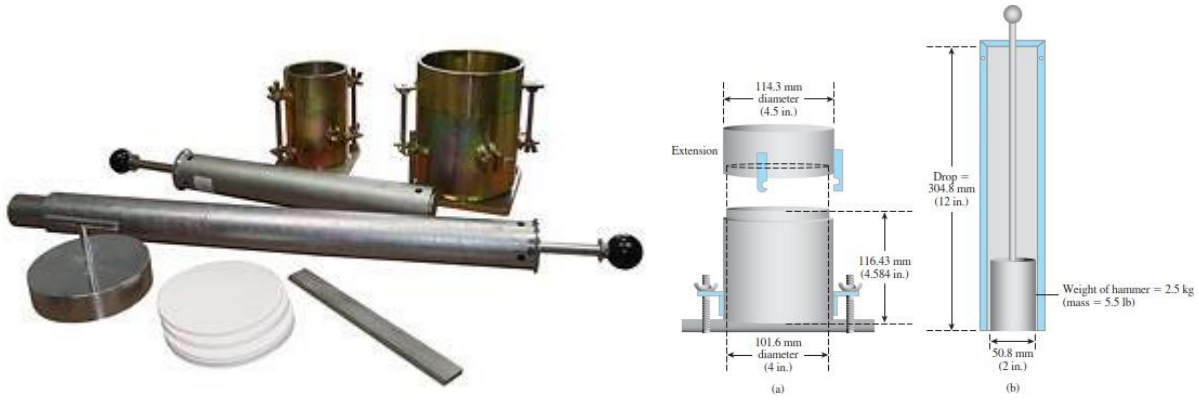


Figure 3.8: Proctor Test Equipment

The following table, specifies the required parameters for each test.

Table 3.1: Characteristics of laboratory compaction tests

Characteristics	**Proctor Mould**		**C.B.R. Mould**	
Mould volume Mould diameter Mould height	948.55 cm^3 10.16 cm 11.70 cm		2316.66 cm^3 15.24 cm 12.70 cm	
Test	**Standard**	**Modified**	**Standard**	**Modified**
Rammer weight Rammer diameter Drop height	2.49 kg 5.08 cm 30.5 cm	4.54 kg 5.08 cm 45.7 cm	2.49 kg 5.08 cm 30.5 cm	4.54 kg 5.08 cm 45.7 cm
Number of layers	3	5	3	5
Number of blows per layer	25	25	55	55
Compaction energy	0.59 KJ/dm^3	2.68 KJ/dm^3	0.53 KJ/dm^3	2.41 KJ/dm^3

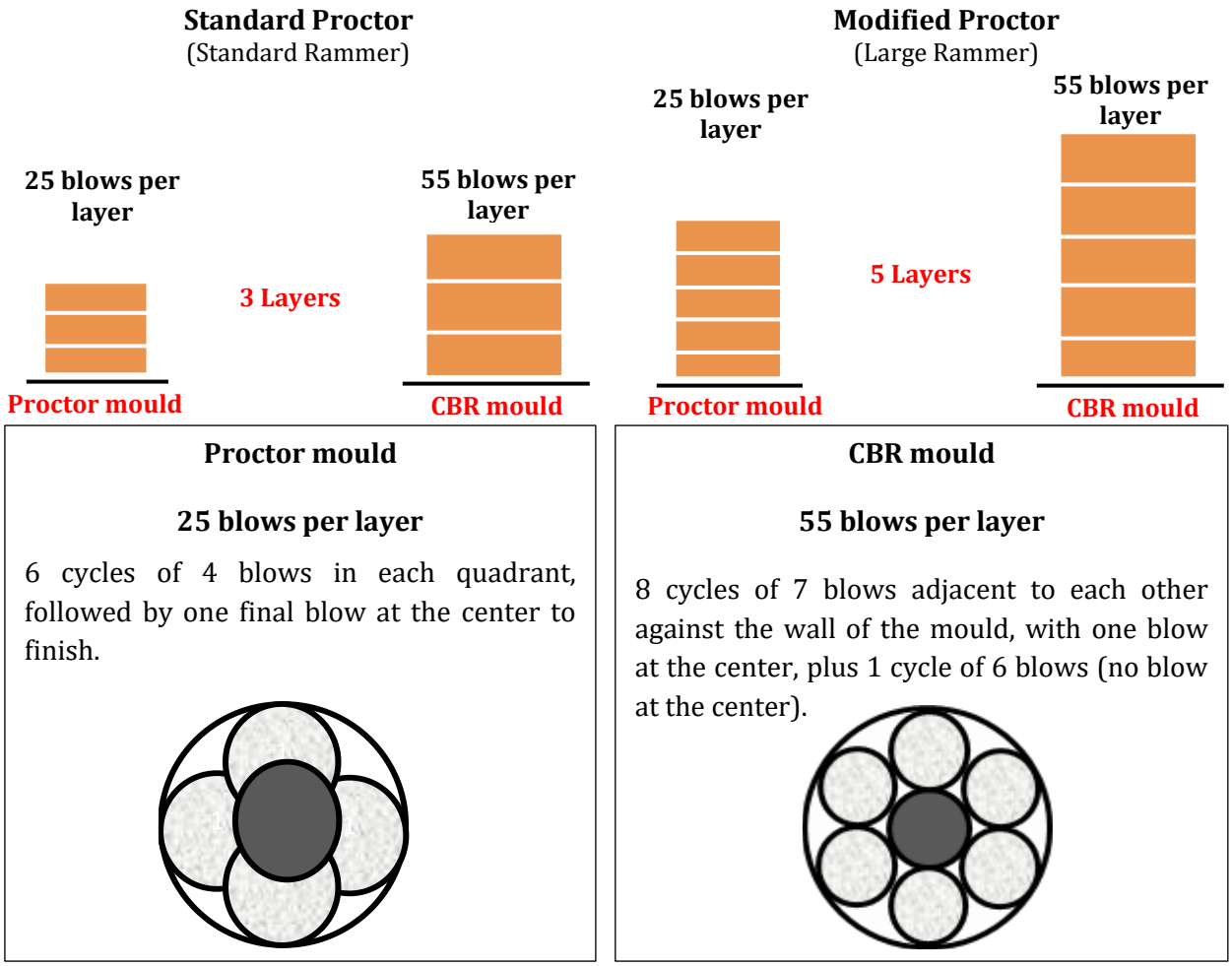


Figure 3.9: Normal and Modified Proctor Execution

7. Special in-situ compaction equipment and processes

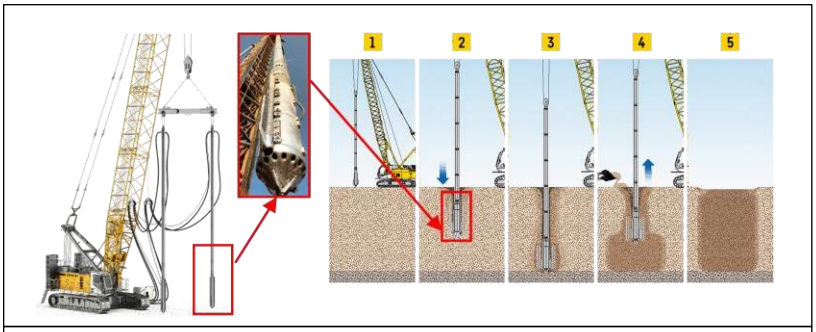
Various compaction equipment used in the field, such as smooth-wheeled rollers, sheepsfoot rollers, wheeled rollers and vibrating rollers.

Vibratory rollers are particularly effective for densifying granular soils.

Vibroflot devices, a technique known as *vibroflotation*, are used to compact granular soil deposits to significant depths.



Vibratory Rollers



Vibroflotation

Figure 3.10: Vibrating Rollers and Vibroflotation

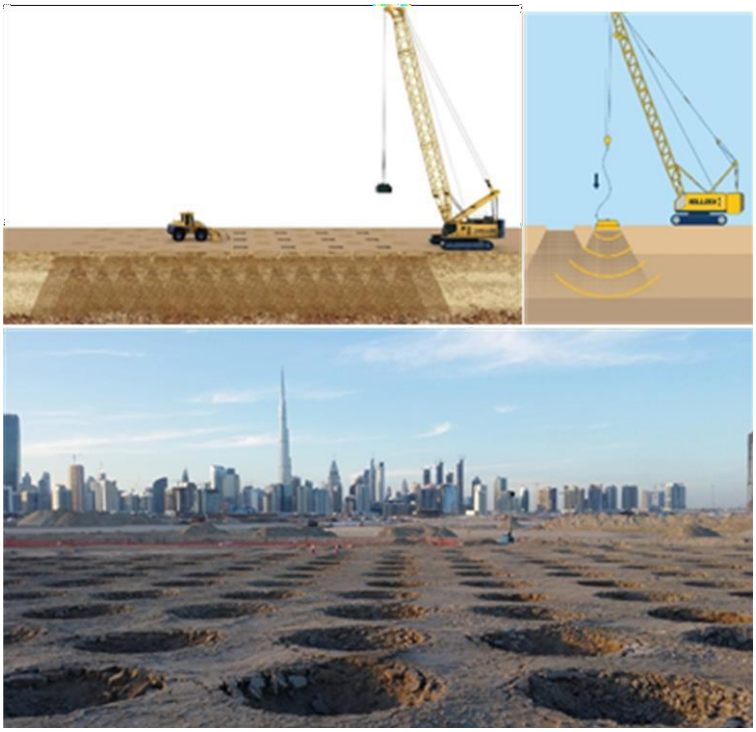
For powdery and granular soils, effective compaction is achieved by vibration using manual vibrating plates, self-propelled vibratory rollers, pneumatic-tired rollers, and large free-falling masses.



Vibrating plate



Vibratory roller



Dynamic compaction

Figure 3.11: Self-Propelled Vibratory Rollers, Manual Vibrating Plate, and Dynamic Compaction Equipment

It is applicable to all soil types. Its principle is based on the transmission of high-energy shocks to the soil surface, achieved by dropping a mass of 10 to 30 tons, and in exceptional cases up to 140 tons, from a height of 15 to 30 meters. The depth of influence is determined according to the

formula proposed by Leonardo and colleagues (1980):

$$\boldsymbol{D = \frac{1}{2}\sqrt{W.h}} \quad [\boldsymbol{m}] \qquad (3.09)$$

Here, W represents the falling mass in tons, and h denotes the drop height in meters.

- **Smooth rollers:** used for compacting cohesive, non-clayey soils.
- **Pneumatic rollers:** used for compacting non-cohesive soils.
- **Sheep-foot rollers:** used for compacting cohesive soils, and are particularly essential for clayey soils.



Smooth Wheel Rollers



Pneumatic rollers



Sheepsfoot Rolls

Figure 3.12: Smooth Wheel Rollers, Pneumatic rollers, Sheepsfoot Rolls

Compressed-air drum rollers: used for compacting thin layers.

- Explosive tampers (Road roller): for cohesive or non-cohesive soils over small areas.
- Mounted on a traveling crane: suitable for all soil types but mainly useful for small surfaces.



Vibratory Rammer



Road roller

Figure 3.13: Vibratory Rammer and Road roller

8. Requirements and Control of Compaction

In addition to water content and compaction energy, several other factors significantly influence the quality of compaction and the final desired outcome. These factors include the thickness of the layer to be compacted, the pressure exerted by the compactor, and the surface to which this pressure is applied (see Figure 3.14)

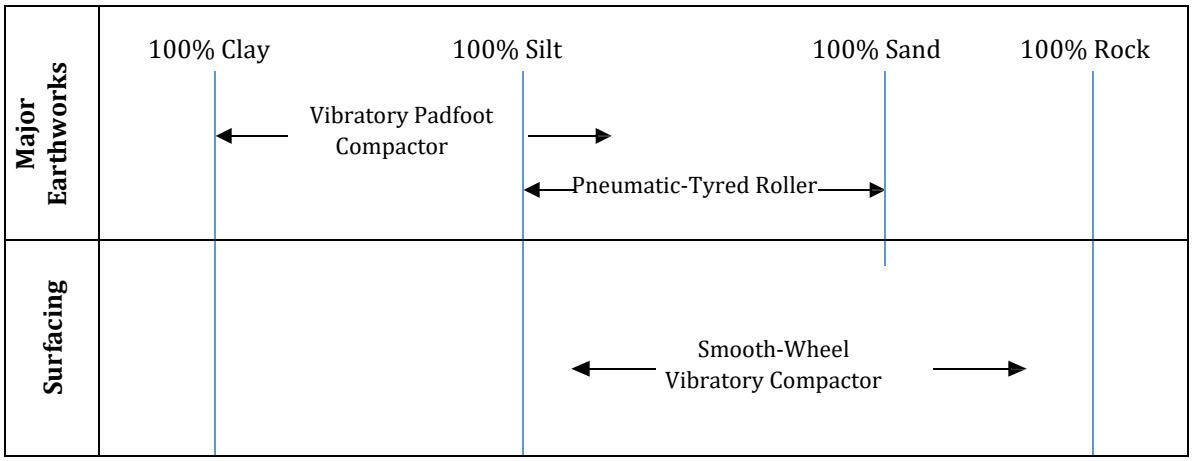


Figure 3.14: Typical Applications of Main Compaction Machinery (Road Works)

Note: The compaction operation on site must be performed in thin layers—20 to 30 cm for roadworks and 10 to 15 cm for building works—regardless of the machinery used.

8.1. Concept of a Test Strip

The concept of a test strip is aimed at addressing a large part of the challenges associated with earthwork projects, which often depend on the weather conditions at the time of construction, particularly when placing a soil, such as in an embankment. Before the actual start of the earthworks, the test strip allows for the definition of compaction parameters related to the equipment used, the soil characteristics at the time of work (water content, equipment speed, number of passes, etc.). The objective is to achieve the specified compaction by adequately adjusting these parameters.

8.2. Effect of Machine Speed

The speed of the compaction machine has a significant influence on the compaction process. For a given machine and predetermined quality criteria, there is an optimal speed that depends on the layer thickness and the material properties, allowing maximum compaction to be achieved. The more stringent the quality requirements, the lower the optimal travel speed. In general, it is recommended to limit the speed of most compactors to 8 km/h. For vibrating compactors, the optimum speed is approximately 5 km/h, ensuring that the vibrations effectively act throughout the entire thickness of the layer to be compacted.

8.3. Degree of Compaction (Compaction Control)

Controlling the in-situ density of a platform or pavement layer is essential. The degree of compactness D_c (%), is the primary criterion for accepting or rejecting the compaction. It is calculated by comparing the dry density of the soil on site ($\gamma_{d\ chant}$) with the optimum dry density obtained in the laboratory ($\gamma_{d\ opt}$).

$$D_C = \frac{\gamma_{d\ chantie}}{\gamma_{d\ opt}} \quad (3.10)$$

This degree approaches 100% when the value of ($\gamma_{d\ chant}$) approaches that of ($\gamma_{d\ opt}$). In general, the specifications impose a $\mathbf{D_c \geq 95\%}$. The higher $\mathbf{D_c}$, the greater the soil compactness and the more effective the compaction.

8.4. Influence of the number of passes

For a given machine and predetermined quality parameters, there is an optimal number of passes, which depends on the machine speed, the layer thickness, and the material properties, aimed at achieving maximum compaction. As the quality requirements become more stringent, the optimal number of passes increases.

As a general rule, between 3 and 8 passes are required to compact a 30 cm thick soil layer. However, this number can increase up to 12 depending on the soil type, water content, and the mass of the compactor. If, even under optimal moisture conditions, the desired compaction is not achieved after 12 passes, it can be concluded that the compaction operations were unsuccessful, indicating that the compactor used may not be suitable for the task.

In practice, the water content is adjusted to the optimal value determined from the Proctor test by calibrating the water supply. For each operating speed of the machine, the number of passes required to meet the specified compaction criteria is determined. This allows the creation of a curve plotting machine speed against the number of passes, from which the optimum can be identified, thereby providing the parameters associated with the machine.

9. Compaction Control on the Site

Compaction control tests carried out on site are used to determine the dry density and water content of a compacted soil layer. By comparing these results with those obtained from the Proctor test or the vibrating table test, the degree of compaction or the relative density index can be

established. This procedure allows the quality of the compaction work to be evaluated and either accepted or rejected, in accordance with the guidelines provided in the construction specifications.

Five tests are commonly used to verify the quality of compaction on site:

Compaction control tests carried out on site are used to determine the dry density and water content of a soil layer that has been compacted. By comparing these results with those of the Proctor test, the degree of compaction or the relative density index can be established. This procedure allows the quality of the compaction work to be assessed and either accepted or rejected in accordance with the guidelines provided in the construction specifications. To ensure proper compaction on site, five tests are generally performed.

9.1. Nuclear densimeter test

The Nuclear densimeter is a relatively accurate device with which you can quickly determine the density of dry soil on the site as well as its water content. It is also used to check the density of concrete and bituminous coatings.



Figure 3.15: Nuclear densimeter equipment

9.2. The sand cone test:

The sand cone test involves determining the in-situ dry density of soil by filling a hole, from which a soil sample has been removed, with calibrated sand of known density using a device called a sand cone. The sand cone consists of a removable bottle for storing the sand, a funnel (or cone), and a valve located between the two.

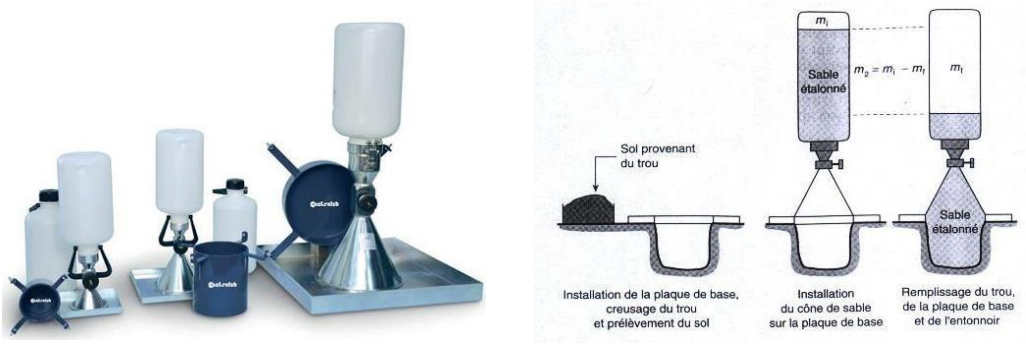


Figure 3.16: Sand cone device

9.3. Plate bearing test

The plate loading test is a conventional geotechnical test that allows a relatively simple experiment to be conducted on the surface of a soil or on a layer forming part of a road structure, the results of which are related to the mechanical properties of the materials tested.

The test results are primarily used to control the implementation of materials (compaction control). In some cases, plate loading tests are also performed as part of the design of a new pavement or the reinforcement of an existing one. When the plate modulus is not measured directly, it can be estimated using the formula provided in pavement design guidelines: E=5×CBR.

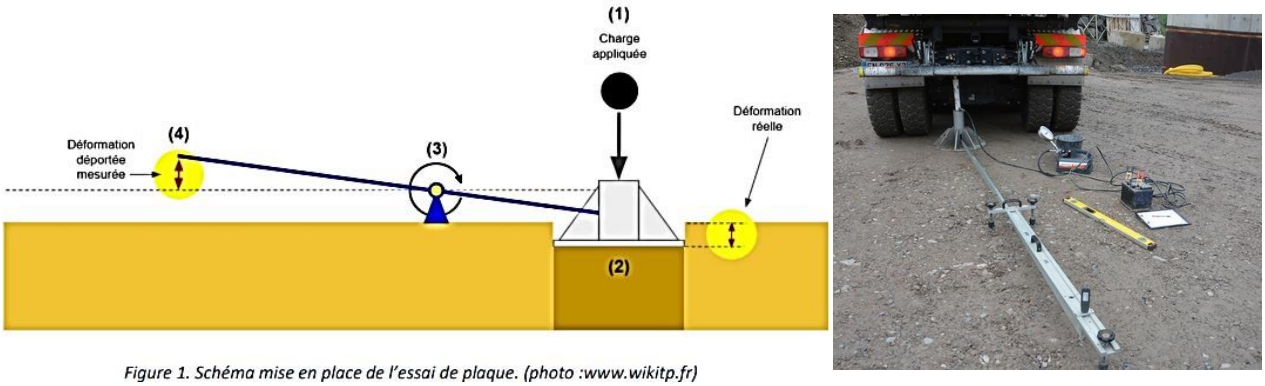


Figure 3.17: Plate bearing test

9.4. Membrane Densitometer

The membrane densitometer is a traditional method used to measure the volume of a cavity created in the soil layer to be tested. The soil is carefully extracted from this cavity to determine both the total wet weight and the total dry weight (Figure 3.18). Knowing the wet and dry mass of the extracted sample allows for the straightforward calculation of the wet and dry bulk densities of the in-situ soil.

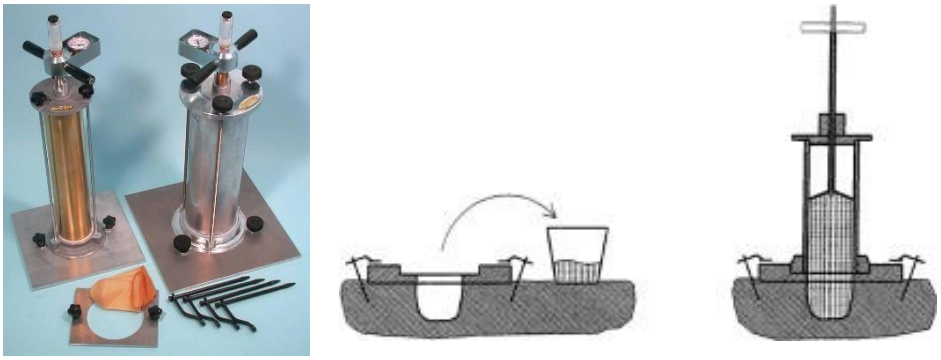


Figure 3.18: Membrane Densitometer Equipment

9.5. Other Compaction Control Tests

- The Washington-type apparatus test; and
- The flexible membrane test.

Chapter IV:

Water in the Soil

Chapter IV: Water in the soil

1. Introduction

In the field of geotechnical engineering, water is generally considered a disruptive element, or even an enemy. The presence of water in soil and the resulting soil–water interactions complicate construction problems.

As a first approach, the issue can be summarized by two fundamental questions:

- **Where is the water in the ground?** This leads to the identification of saturated and unsaturated zones, as well as the determination of piezometric levels.
- **How does water move through the ground?** This depends on two main factors: first, a soil property (permeability), and second, an external factor (the hydraulic gradient).

In practice, knowledge of soil hydraulic parameters is required in several engineering problems, such as:

- The calculation of seepage through and beneath dams;
- Stability problems related to pore water pressure or seepage forces in structures such as embankments, excavations, and dams.



Figure 4.1: Collapse caused by the presence of gypsum (Le Luc, Var, France). *French Geological and Mining Research Bureau (BRGM).*

Collapses due to the dissolution of soluble materials occur in sandy or gypsum soils and may be accelerated by the presence of aggressive water.



Figure 4.2: Natural hazards related to the shrink–swell behavior of clays

Once the presence of water at a site has been identified, it becomes necessary to address the problems it causes. Almost all of these problems can be reduced either by eliminating the water (dewatering of excavations) or by reducing its hydraulic head (drainage) (CAMBFORT, 1980).

Drainage and the temporary or permanent lowering of the groundwater table are often essential for the construction of structures and for ensuring their stability.

2. Groundwater types

Groundwater originates mainly from precipitation in the form of rain, snow, and hail. Less than 20% of this precipitation infiltrates the soil and percolates downward under the effect of gravity until it reaches impermeable soil layers or bedrock. This process generally leads to the formation of two main zones:

- The unsaturated zone, located in the upper part;
- The saturated zone, located in the lower part.

In the unsaturated zone, the soil moisture content may range from a dry state to a condition close to full saturation. Under certain conditions, this zone may be absent; in such cases, the soil consists solely of a saturated zone.

2.1. Types of Water in Soil

Water can exist in several states within soil. These states are mainly distinguished by the intensity of the forces binding the water to soil particles. Conventionally, four types of water are identified:

2.1.1. Constitution water, which is part of the chemical or mineralogical structure of certain minerals, mainly clays;

2.1.2. Adsorbed water, which is bound to the surface of soil particles and is closely associated with them;

2.1.3. Capillary water, which is retained in the finest soil pores above the groundwater table. The capillary fringe may be fully or partially saturated;

2.1.4. Free water, which can move through the soil pores under the action of gravity. The volume occupied by this water defines the concept of effective porosity, which is smaller than the total (geometric) porosity.

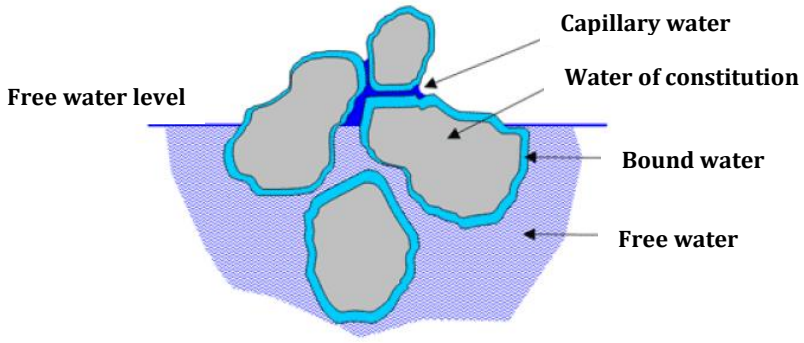


Figure 4.3: The different states of water in soils, except the case of ice.

3. Assumptions for the Study of Water Flow in Soils

- The soil is fully saturated;
- Water and soil grains are incompressible;
- The flow occurs only in the liquid phase.

3.1. Continuity Condition

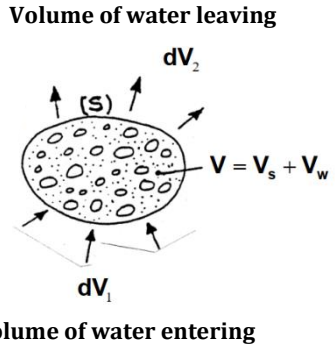


Figure 4.4: saturated soil crossed

Consider a volume of saturated soil crossed by a flow.

During a time interval *dt*, a volume dV_1 enters the soil element, while a volume dV_2 leaves it.

If the soil grains remain fixed and taking into account the second assumption (incompressibility of water and grains), the continuity condition requires that the volume of water entering the soil element equals the volume leaving it.

In soil hydraulics → steady state (the flow speed does not change).

4. Properties of Free Water

4.1. Discharge (or Flow or Seepage) Velocity

Water in soil is said to be in a free state when the voids are completely filled with water. Under these conditions, the granular medium can be the seat of flow.

The discharge, denoted by q, represents the quantity (volume) of water passing through a given cross-section per unit time.

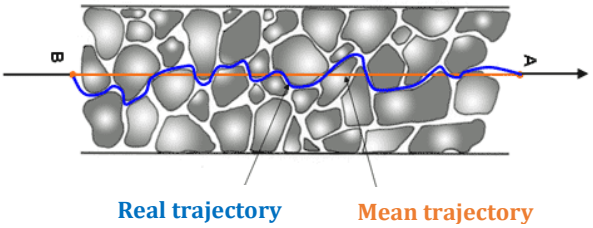


Figure 4.5: Actual and Average Trajectories

Three different flow velocities are then defined:

v: **Discharge velocity** (also called fictitious or apparent velocity),

Average velocity

$$v = \frac{q}{s} \tag{4.01}$$

v': Real average velocity (taking porosity into account)

$$v' = \frac{q}{S_v} \tag{4.02}$$

v_r: **Actual local velocity** of each water particle, following the true flow path (accounting for tortuosity).

Let *n* denote the porosity of the medium.

$$n = \frac{V_v}{V} \tag{4.03}$$

$$n\,V = V_v \tag{4.04}$$

For a cylinder of cross-sectional area S and height H, we have:

$$\mathbf{V_v = S_v . H = n . S . H}$$

where:

- $\boldsymbol{S}$ is the total cross-sectional area,
- $\boldsymbol{S_v}$ is the area occupied by voids,
- $\boldsymbol{n}$ is the porosity of the medium.

Since $\mathbf{S_v = n . S}$ the volume of voids can also be expressed as $\mathbf{V_v = S_v . H}$.

$$v' = \frac{q}{n \, . \, s} \qquad (4.05)$$

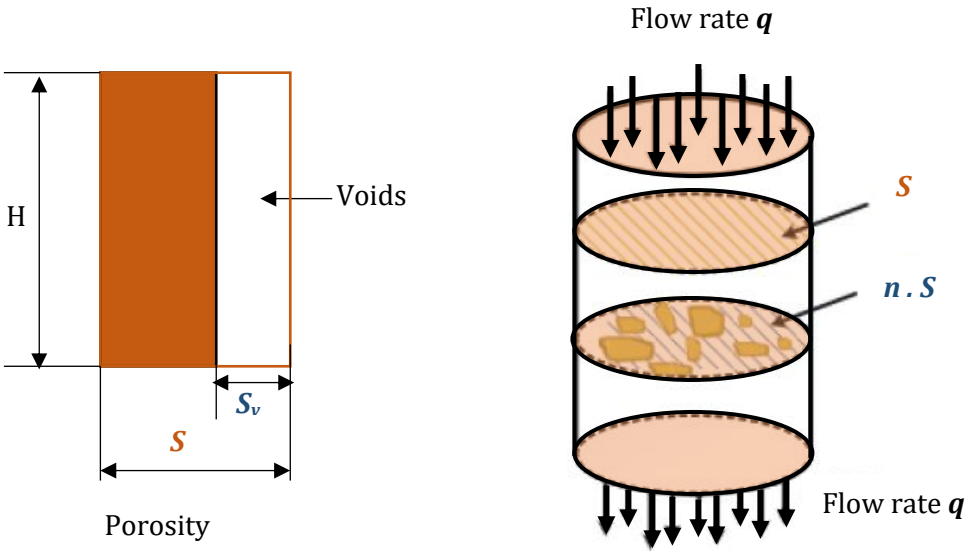


Figure 4.6: Apparent and actual passage surface

4.2. Hydraulic load

The hydraulic load represents the energy per unit mass of a flowing particle. The hydraulic load, $\mathbf{h_M}$, at a point **M** is evaluated using the **Bernoulli equation**.

$$h_M = \frac{u_M}{\gamma_w} + Z_M + \frac{v_M^2}{2.g} \qquad (4.06)$$

Potential Energy *Kinetic energy*

Energy of a fluid particle per unit mass (expressed in meters of water)

Let

V_M: velocity of water at point M

U_M: pore pressure at point M, here $\frac{u_w}{\gamma_w}$ is measured directly by the height of water in a piezometer (see Figure 4.8),

Z_M: elevation of point M with respect to a reference plane,

L: distance between points M and N,

U_N : pore pressure at point N,

Z_N: elevation of point N,

g = acceleration due to gravity.

Note: In soils, flow velocities (v) are very low, and therefore the kinetic energy term $\frac{v_M^2}{2.g}$ is **negligible**.

The hydraulic load can then be expressed as:

$$h_M = \frac{u_M}{\gamma_w} + Z_M \qquad (4.07)$$

4.3. Piezometer and piezometric line

"Open" piezometers are simple vertical tubes in which the water level is measured. This measurement can be performed using a weighted tape or, alternatively, an electrical contact sensor at the end of a wire.

More sophisticated systems, of course, use a pressure sensor at the end of the tube.



Figure 4.7: Piezometer apparatus

The pore pressure $\boldsymbol{u}$ is measured using the height of water in a piezometer, a tube extending up to point **M**. In the absence of flow between **M** and the high point of the water in tube **A**, the hydraulic head is the same at these two points: $\mathbf{h_M = h_A}$.

With

$$h_M = \frac{u_M}{\gamma_w} + Z_M \qquad (4.08)$$

And: $\mathbf{h_A = Z_A}$

$$u_M = \gamma_w (Z_A - Z_M) \qquad (4.09)$$

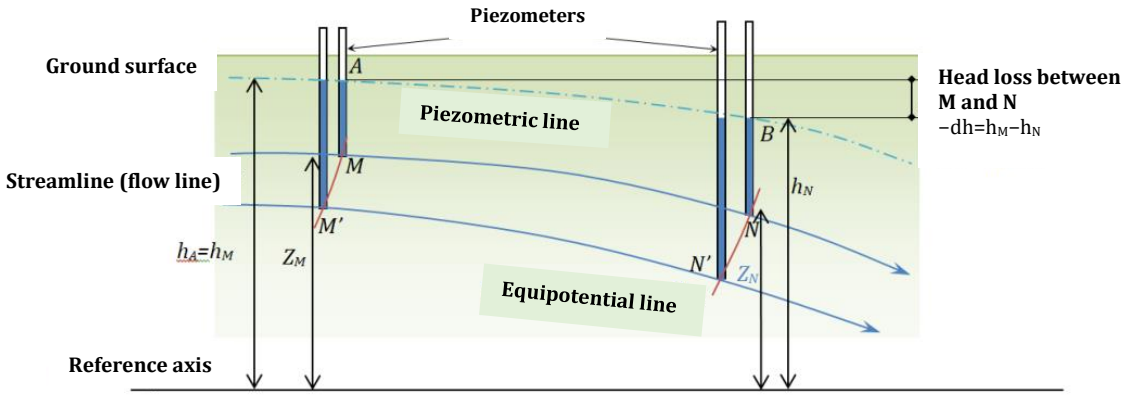


Figure 4.8: Illustration of the measurement by piezometer

4.4. Hydraulic Gradient

The hydraulic gradient $\boldsymbol{i}$ is defined as the head loss per unit length of flow.

$$i = \frac{\Delta h}{\Delta L} \qquad (4.10)$$

- Dimensionless
- In the direction of flow

In Figure 4.8, the distance traveled by water from point $\boldsymbol{M}$ to point $\boldsymbol{N}$ is denoted $\mathbf{L_{MN}}$; the corresponding hydraulic gradient is then given by:

$$i = \frac{\Delta h_{MN}}{L_{MN}} = \frac{h_M - h_N}{L_{MN}} \qquad (4.11)$$

- **Example of Gradient Calculation**

Calculate the hydraulic gradient in the soil (between points B and D).

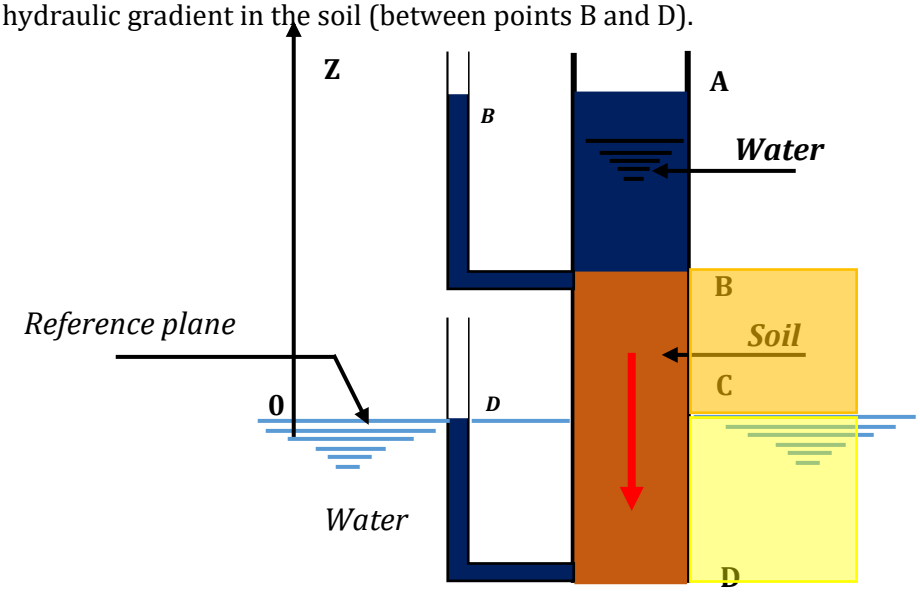


Figure 4.9: Downward vertical flow

$$i = \frac{perte\ de\ charge}{longueur\ traversée}$$

$$h_M = Z_M + \frac{u_M}{\gamma_w} \qquad (4.12)$$

Reference plane

Head at point B:

$h_B = Z_B + (U_B/\gamma_w)$

$h_B = AB + BC = AC$

Head at point D

$h_D = Z_D + (U_D/\gamma_w)$

$h_D = -CD + CD = 0$

Head loss: $\Delta h = h_B - h_D = AC - 0 = \mathbf{AC}$

$$U_B = \gamma_w\, AB$$

$$h_B = \frac{\gamma_w\, AB}{\gamma_w} + Z_B$$

$$h_B = \frac{\gamma_w\, AB}{\gamma_w} + BC = AC$$

$$U_D = \gamma_w\, CD$$

$$h_D = \frac{\gamma_w\, CD}{\gamma_w} - Z_D$$

$$h_B = CD - CD = 0$$

Hydraulic Gradient

$$i = \frac{\Delta h}{\Delta L} = \frac{AC}{BD}$$

4.5. DARCY's Law

DARCY's law is the fundamental law of soil hydraulics:

Darcy's original law, published in 1856, expresses the proportionality between flow velocity and hydraulic gradient. It is an experimental law:

$$v = K \cdot i \quad (4.13)$$

Where:

- v: Flow velocity (m/s)
- k: Soil permeability coefficient (m/s); it quantifies the soil's ability to transmit water. A highly permeable soil has a high coefficient, while a poorly permeable soil has a low coefficient.
- i: Hydraulic gradient (dimensionless)

The equation for the flow through a soil section S can then be written as a function of i and k:

$$q = K \cdot i \cdot S \quad (4.14)$$

Other representation of Darcy's law

$$q = v \cdot S = K \cdot i \cdot S = K \cdot \frac{\Delta h}{\Delta L} \cdot S \quad (4.15)$$

Orders of magnitude of k are given in the table below (for guidance: for $\mathbf{k = 10^{-8}\ m/s}$, $\mathbf{v = 30\ cm/year}$ for a unit hydraulic gradient!).

Table 4.1: Soil Permeability Classification by Hydraulic Conductivity

Nature	Order of magnitude of K in cm/s	Permeability
Medium to coarse gravel	10^{-1} à 10	Very high
Small gravel, sand	10^{-1} à 10^{-3}	Fairly high
Very fine sand, silty sand, loess	10^{-3} à 10^{-5}	Low
Compact silt, silty clay	10^{-5} à 10^{-7}	Very low
Pure clay	10^{-7} à 10^{-10}	Practically impermeable

For well-graded sands, the value of k can be estimated using Hazen's empirical formula:

$$K = 10^4 (d_{10})^2$$

Where k is in m/s and d_{10} (the effective grain size) is in meters.

5. Permeability measurements

5.1. Laboratory tests

5.1.1. Constant-Head Permeameter

This test is used for soils with **high permeability** (k > 10^{-5} m/s).

1. A soil sample is placed in a permeameter as shown in **Figure 4.10.** A constant pressure drop Δh causes water to flow from top to bottom through the soil.
2. The water flow rate q is determined by collecting a volume of water V over a given time t ($q = V/t$).
3. Knowing the length of the sample L cross-sectional area S, the permeability coefficient k can be calculated using Darcy's law:

$$v = k \cdot i$$
$$v \cdot S = k \cdot i \cdot S$$
$$q = k \cdot i \cdot S$$
$$k = \frac{q}{i \cdot S}$$

Now $q = V/t$ and $i = \Delta h/L$, so finally k is deduced from the relationship:

So k is deduced from the relation

$$k = \frac{V \cdot L}{\Delta h \cdot S \cdot t} \qquad (4.16)$$

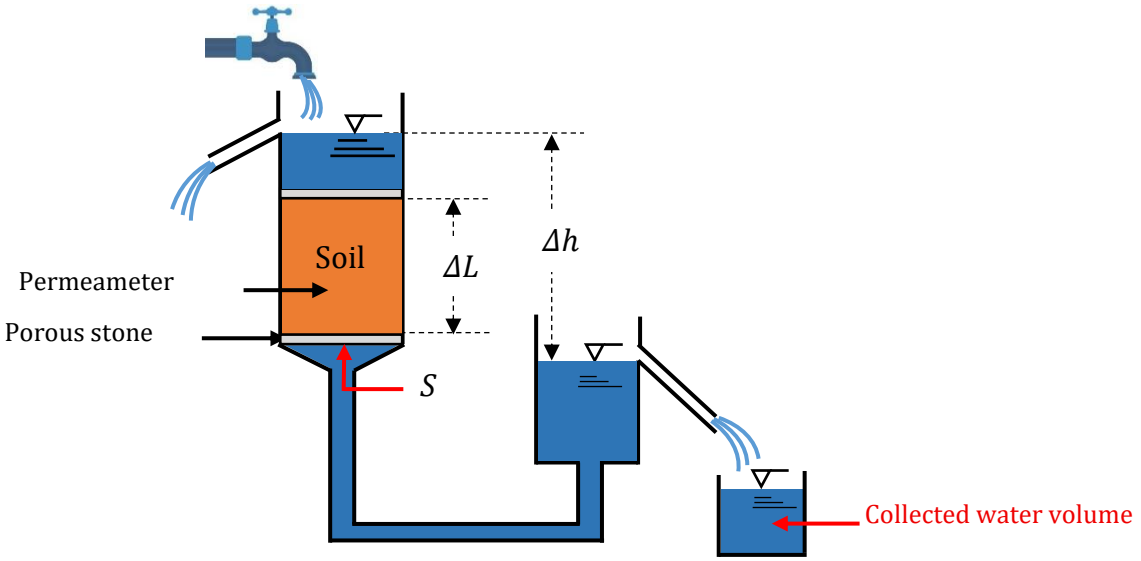


Figure 4.10: Constant-head permeameter

5.1.2. Falling Head Test

This test is used for **soils with low permeability** ($k < 10^{-5}$ m/s), where a constant-head test would take too long due to the low flow rate.

During the test, water flows through the soil sample, causing a pressure head that decreases over time. The water comes from a narrow-diameter tube (Figure 4.11). As the test proceeds, the water level in the tube drops. The time t is then measured for the water to descend from an initial level h_1 to a final level h_2.

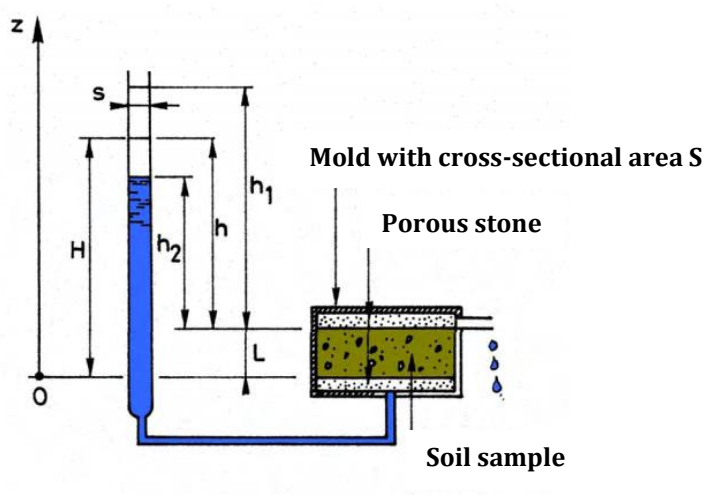


Figure 4.11: Falling Head Test

Using the notations from Figure 4.11, with the reference plane at the inlet of the sample, at an intermediate time we have:

At A:

$$\boldsymbol{h_A = \frac{u_A}{\gamma_w} + Z_A = H + 0}$$

At B:

$$\boldsymbol{h_B = \frac{u_B}{\gamma_w} + Z_B = 0 + L}$$

Head loss:

$$\boldsymbol{h_A - h_B = H - L = h}$$

hydraulic gradient:

$$\boldsymbol{i = \frac{h}{L}}$$

Flow rate:

$$q = v \,.\, S = k \,.\, \frac{h}{L} \,.\, S \tag{4.17}$$

In addition, during a time interval ***dt***, the volume of water ***dV*** passing through the sample is equal to the loss of the volume of water in the tube:

$$dV = q \,.\, dt = -s \,.\, dh$$

During a time interval ***dt***, the volume of water ***dV*** passing through the sample is equal to the loss of water volume in the tube:

$$dV = q \,.\, dt = -s \,.\, dh$$

dV=volume decrease in the tube

Permeability is then expressed as:

$$k.\frac{h}{L}.S.dt = -s \,.\, dh$$

$$k \int_0^t dt = -\frac{s.L}{S} \int_{h1}^{h2} \frac{dh}{h}$$

$$k = \frac{s.L}{S.t} \ln \frac{h_1}{h_2} \tag{4.18}$$

5.2. In-Situ Measurements

A drawback of laboratory measurements is that they are performed on small samples, which may not necessarily provide a good representation of the average permeability of a soil, especially when the soil is heterogeneous.

There are several methods to determine the permeability coefficient in the field. For example, one method requires a steady-state flow: the Dupuit-Lefranc test.

5.2.1. Unconfined Aquifer Case

It consists of **drilling a well** into the soil down to a level below the water table and **pumping at a constant flow rate q** until a **steady state** is reached, in the case of a **phreatic (unconfined) aquifer** (Figure 4.12).

It can then be shown that:

$$k = Q \frac{\ln(R/r)}{\pi(H^2 - h^2)}$$

$$q = \pi \,.\, k \frac{(H^2 - h^2)}{\ln(R/r)} \tag{4.19}$$

$$k = q\frac{\ln(R/r)}{\pi(H^2 - h^2)} \quad (4.20)$$

Where:

- **r** is the radius of the well,
- **R** is the radius of influence, i.e., the distance beyond which the water table is not affected by pumping,
- **H** is the initial water table height, and
- **h** is the water height in the well when a steady pumping regime is reached.

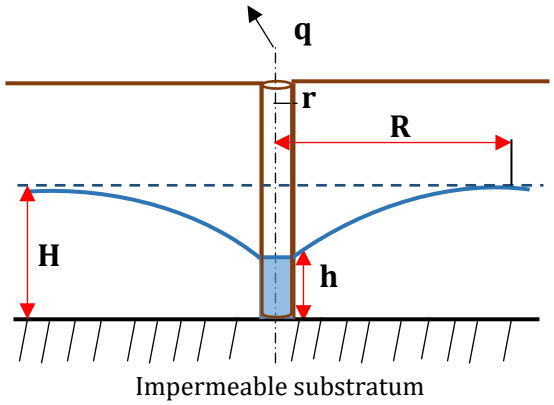


Figure 4.12: Dupuit Pumping Test – Unconfined Aquifer Case

5.2.2. In the case of a confined aquifer (Figure 4.13)

The water table surface is not the same as the piezometric surface.

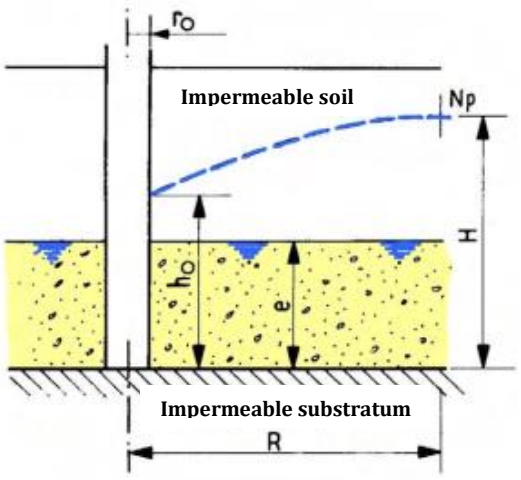


Figure 4.13: Dupuit Pumping Test – Confined Aquifer Case

$$q = 2\pi \,.\, e \,.\, k\frac{(H - h)}{\ln(R/r)} \quad (4.21)$$

$$k = \frac{q}{2\pi \,.\, e(H-h)} \ln(R/r) \qquad (4.22)$$

6. Average Horizontal and Vertical Permeability of Layered Soils

Most soils are layered (a succession of altered materials of different origins). It appears that permeability is much higher along the bedding planes than in the direction perpendicular to the layers.

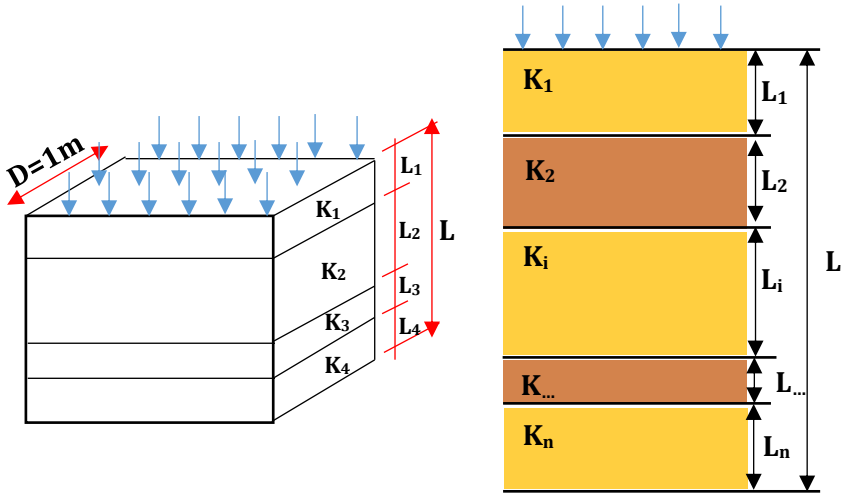


Figure 4.14: Average Vertical Permeability of Layered Soils

K_i: *permeability of layer i*

L_i: *thickness of layer i*

L: *total thickness*

h: *total head loss, such that:*

$$h = \sum_{i=1}^{n} h_i$$

v*: Flow velocity*

$$v = k.\, i$$

i: *hydraulic gradient*

$$i = h/H$$

6.1. Average Permeability Coefficient Perpendicular to Bedding Planes (Average Vertical Permeability)

Let us write the equation for the conservation of interstitial water mass:

$$v = \frac{h}{H} . k_v = k_1 i_1 = k_2 i_2 = k_n i_n \rightarrow i_i = \frac{H\, k_v}{L\, k_i}$$

With:

$$H = L_1 i_1 + L_2 i_2 + \cdots + L_n i_n = \sum_{i=1}^{n} L_i \frac{H\, k_v}{L\, k_i} = \frac{H}{L} k_v \sum_{i=1}^{n} \frac{L_i}{k_i}$$

where:

$$k_v = \frac{L}{\frac{L_1}{k_1} + \frac{L_2}{k_2} + \cdots + \frac{L_n}{k_n}} = k_v = \frac{L}{\sum_{i=1}^{n} \frac{L_i}{k_i}} \leftrightarrow \frac{L}{k_v} = \sum_{i=1}^{n} \frac{L_i}{k_i} \qquad (4.23)$$

6.2. Average Permeability Coefficient Parallel to the Bedding Plane (Average Horizontal Permeability)

The total flow is the sum of the flows in each layer for a unit thickness slice and for a hydraulic gradient "***i***".

Where:

- **Q:** total flow rate
- $\mathbf{q_i}$: flow rate through each elementary layer *i*

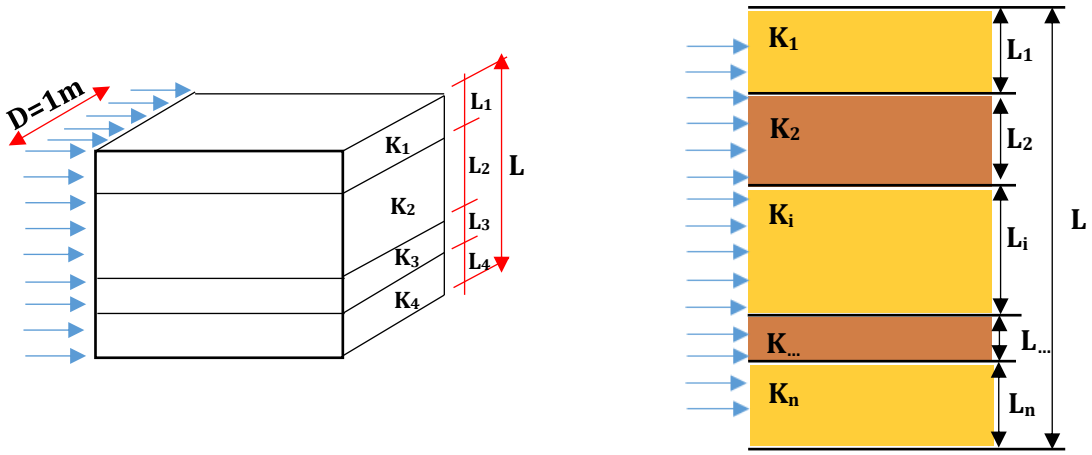


Figure 4.15. Average horizontal permeability of bedded soils

$$Q = k_h . i . L = \sum_{i=1}^{n} q_i = \sum_{i=1}^{n} v_i L_i = \sum_{i=1}^{n} k_i . i . L_i = i . \sum_{i=1}^{n} k_i . L_i$$

where:

$$\boldsymbol{k_h = \frac{\sum_{i=1}^{n} k_i . L_i}{L} \leftrightarrow k_h . L = \sum_{i=1}^{n} k_i . L_i} \qquad (4.24)$$

Note that:

- $K_h \geq k_v$
- For a natural soil, $k_h \cong 3 . k_v$
- For a natural soil, $k_h \cong 10 . k_v$

6.3. Equivalent Permeability Coefficient

We are considering the case of both vertical and horizontal flow. The following quantity is then defined:

$$\boldsymbol{k_{equivalent} = \sqrt{k_h . k_v}} \qquad (4.25)$$

Note: The flow direction in sedimentary soils is important. Indeed, since the successive layers are horizontal, water flows more easily in the horizontal direction than in the vertical direction.

7. Flow Network

In the examples mentioned above, water flowed through the soil in only **one direction**, either horizontally or vertically. This type of flow, called **one-dimensional flow**, is the simplest. The **Darcy equation** can be applied directly to calculate the flow of water through the soil element.

In most civil engineering works, where the **circulation of water in the ground** plays an important role, the flow is **three-dimensional**, meaning that water can move in all three spatial directions simultaneously. Due to the complexity of studying such flow, it is often simplified by considering it as a **two-dimensional flow**, in which water moves along both horizontal and vertical paths (Figure 4.16).

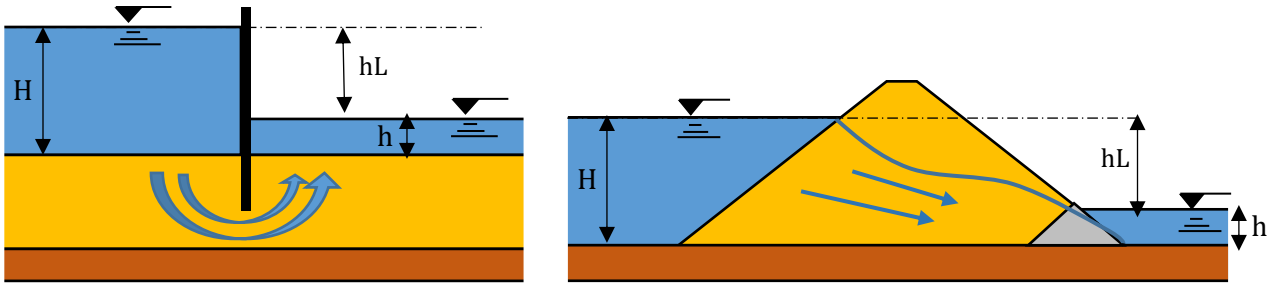


Figure 4.16: Two-Dimensional Soil Water Flow Problems

7.1. Analytical description

Darcy's law remains valid for two-dimensional flows. However, since the flow paths can take various shapes, the water threads have variable directions and lengths. As a result, the mathematical expressions that describe the flow become more complex. In the case of steady-state flow, the continuity equation can be written as:

$$\frac{\partial v_x}{\partial x} + \frac{\partial v_y}{\partial y} = 0 \qquad (4.26)$$

This equation reflects the fact that, since water is incompressible, the volume of water entering a soil element—as illustrated in Figure 4.17 must equal the volume of water leaving it.

However, Darcy's law gives (assuming that the permeability *k* is the same in both the horizontal and vertical directions):

$$v_x = k\frac{\partial h}{\partial x} \; et \; v_y = k\frac{\partial h}{\partial y} \qquad (4.27)$$

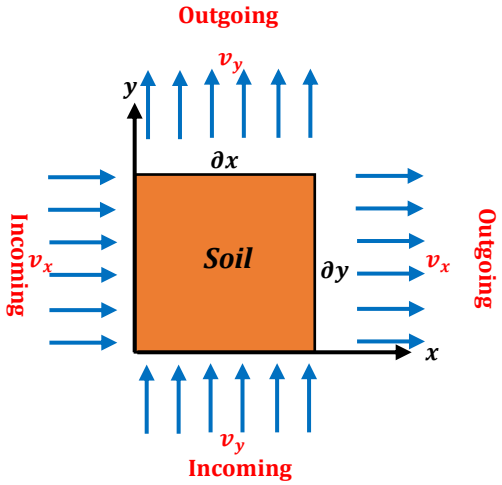


Figure 4.17: Flow of water through an elemental surface of a soil

The continuity equation can then be written as:

$$k\frac{\partial^2 h}{\partial x^2} + k\frac{\partial^2 h}{\partial y^2} = 0 \qquad (4.28)$$

Simplifying by k, we then obtain:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0 \qquad (4.29)$$

This last equation is a **Laplace equation**, which provides information on the hydraulic head h at all points in our two-dimensional problem. However, to obtain a solution, the equation must be solved analytically in simple cases, or using numerical methods in more complex situations.

Once the equation is solved, the hydraulic head is known, and the water velocity can be determined from **Darcy's law**:

$$v_x = k \cdot \partial h/\partial x \text{ and } v_y = k \cdot \partial h/\partial y$$

7.2. Graphical Representation

The solution of the Laplace equation can be represented graphically by a network of lines called a flow net.

7.2.1. Simple Case: One-Dimensional Flow

Figure 4.18 shows the flow net for a horizontal, one-dimensional flow. The flow net is composed of flow lines and equipotential lines.

- **Flow lines** indicate the average paths followed by the water. In reality, a flow net contains an infinite number of flow lines. In our example, for simplicity, only three have been drawn. These lines divide the soil into **flow channels**, the number of which is denoted Nt (here $Nt=2$). The flow lines are drawn such that the water discharge in each flow channel is equal.
- **Equipotential lines** connect all points having the same hydraulic head. They are perpendicular to the direction of water flow, thus perpendicular to the flow lines, and are as numerous as the flow lines. Here, only five equipotential lines are drawn, so that the head losses between consecutive equipotential lines are equal (denoted here as $\Delta h'$).

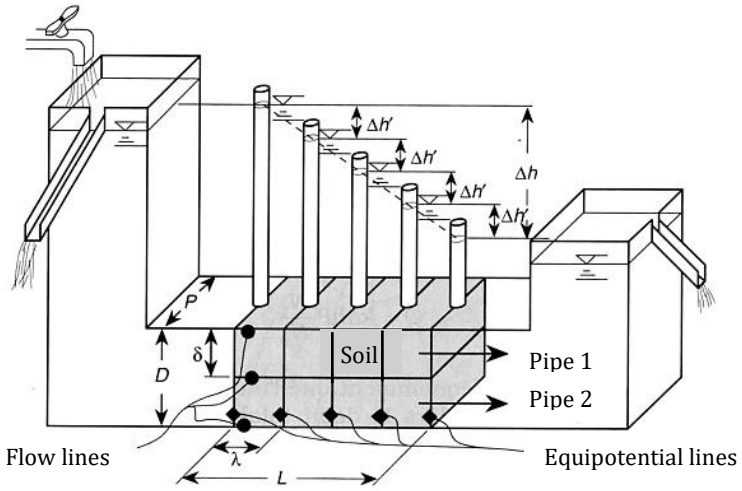


Figure 4.18: One-Dimensional Flow Net

Knowing the total head loss $\boldsymbol{\Delta h}$ and the number of spaces $\boldsymbol{N_p}$ between the equipotential lines, we have:

$$\Delta \boldsymbol{h'} = \frac{\Delta \boldsymbol{h}}{\boldsymbol{N_p}}$$

Flow lines and equipotential lines form curvilinear squares (in our simple example, the squares are perfect). Each square experiences the same head loss and is traversed by the same water flow.

Darcy's equation for a single cell (a square) of the flow net can be written as:

$$\boldsymbol{v} = \boldsymbol{k}\frac{\Delta \boldsymbol{h'}}{\boldsymbol{\lambda}}$$

where $\frac{\Delta \boldsymbol{h'}}{\boldsymbol{\lambda}}$ represents the hydraulic gradient for the cell in question.

Given that, for a cell, water flows through an area of $\boldsymbol{\delta.P}$, the flow rate $\boldsymbol{q_t}$ passing through the cell—and thus through an entire flow channel—is expressed as:

$$\boldsymbol{q_t} = \boldsymbol{k}\frac{\Delta \boldsymbol{h'}}{\boldsymbol{\lambda}}\boldsymbol{\delta}.\boldsymbol{P}$$

Since the cells are (curvilinear) squares, λ=δ; in addition, $\boldsymbol{\Delta h' = \Delta h / N_p}$. Therefore:

$$\boldsymbol{q_t} = \boldsymbol{k}\frac{\Delta \boldsymbol{h}}{\boldsymbol{N_p}}\boldsymbol{P}$$

To calculate the total flow rate $\boldsymbol{Q}$, multiply the flow rate in a single flow channel by the number of flow channels:

$$\boldsymbol{Q} = \boldsymbol{k}\Delta \boldsymbol{h}\frac{\boldsymbol{N_t}}{\boldsymbol{N_p}}\boldsymbol{P}$$

or a unit depth *P*, the total flow rate per unit depth is:

$$Q = k\Delta h \frac{N_t}{N_p} \quad (4.30)$$

This expression for the flow rate is also valid for two-dimensional flow, as presented below.

7.2.2. Case of Two-Dimensional Flows

Figure 4.19 shows a cross-sectional view of a sheet pile embedded in the bottom of a lake. The downstream side is kept dry by pumping, so that water flows through the soil deposit beneath the sheet pile.

- The drawn two-dimensional flow network consists of **6 streamlines**, forming **5 flow tubes** identified by Arabic numerals.
- There are **eleven equipotential lines**, creating **ten equal head losses**, represented by Roman numerals. The first equipotential line, where there is no head loss yet, coincides with line AB, while the last one, having the lowest hydraulic head, corresponds to line CD.
- Streamlines and equipotential lines intersect at **right angles**, forming **curvilinear squares** whose average length **λ** is equal to the average height δ. It can be verified that the average height equals the average length when a **circle** is inscribed in the curvilinear square.
- For a given curvilinear square, the shorter the distance λ between two equipotential lines, the higher the **hydraulic gradient** $i=\Delta h'/\lambda$ and the greater the water velocity $v=k\,i$. Thus, water following **path 2** flows faster than water following **path 1**.

  In fact, the smaller the square, the higher the water velocity inside it. Water reaches its maximum velocity along the sheet pile, because this is where it travels the shortest distance for the same head loss (highest hydraulic gradient).

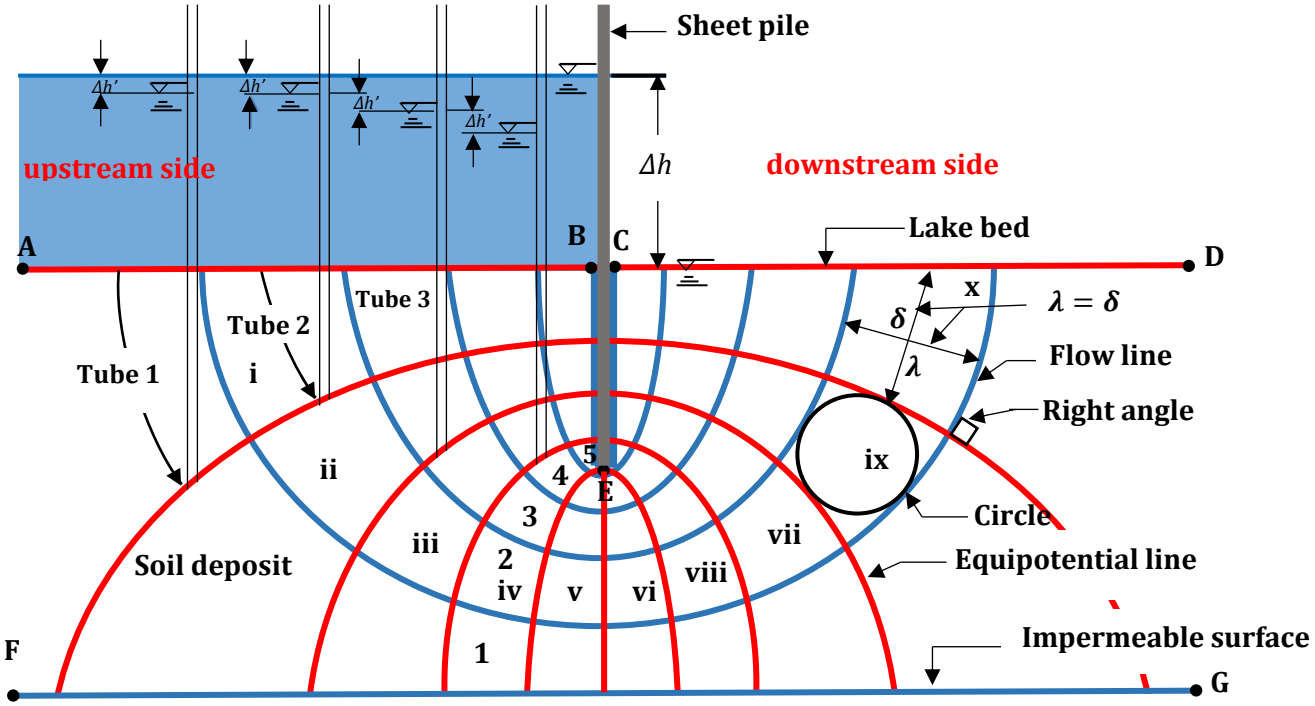


Figure 4.19: Two-dimensional flow network under a sheet pile

7.3. Conditions for drawing a two-dimensional flow network:

- Draw at a reasonable scale.
- Equipotential lines must intersect streamlines at right angles.

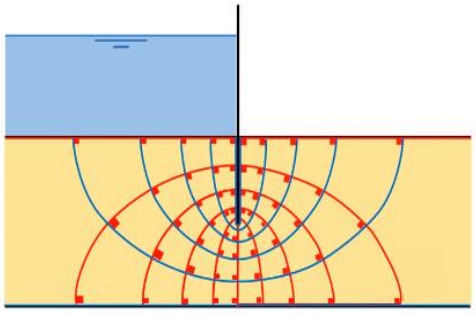


Figure 4.20: Two-dimensional flow network under a sheet pile

- The formed flow elements should be approximately square.

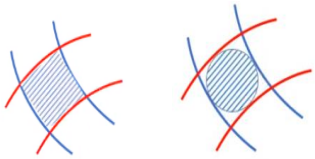


Figure 4.21: Two-dimensional flow network under a sheet pile

8. Flow forces

8.1. Forces on a Soil Element – Boiling Phenomenon

In a groundwater table in hydrostatic equilibrium (without flow), the action of water on the soil skeleton is limited to the Archimedes force acting upward on the soil grains. However, when flow occurs, a head loss appears, reflecting an energy dissipation due to the viscous friction of the fluid on the soil grains. This gives rise to a force on the soil grains created by the water, directed in the direction of flow.

Consider the case of a vertical upward (homogeneous) flow, as shown in Figure 4.20. Point ***M*** is located at a depth ***D***, and point ***A*** is vertically above ***M***.

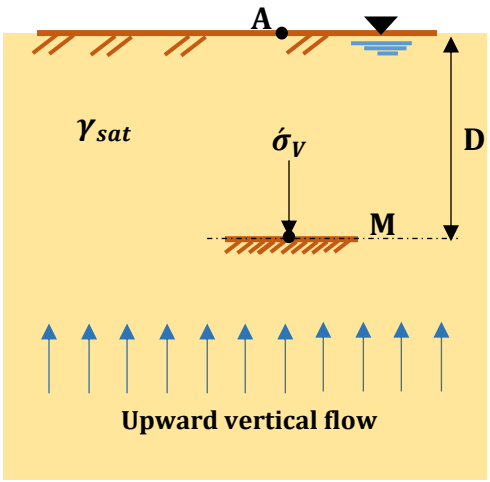


Figure 4.22: Upward vertical flow within a mass of saturated soil

The vertical effective stress at point *M* is expressed as:

$$\acute{\sigma}_V = \sigma_V - u_M$$

$$\acute{\sigma}_V = \gamma_{sat} . D - u_M$$

Furthermore, we have:

- The hydraulic head at *M*:

$$h_M = \frac{u_M}{\gamma_w} + Z_M$$

- The hydraulic head at *A*:

$$h_A = Z_A$$

- The head loss between M and A:

$$\Delta h_{MA} = h_M - h_A = \frac{u_M}{\gamma_w} + Z_M - Z_A = \frac{u_M}{\gamma_w} - D$$

From this, the hydraulic gradient between M and A is:

$$i = \frac{\Delta h_{MA}}{D} = \frac{\frac{u_M}{\gamma_w} - D}{D} = \frac{u_M}{\gamma_w \,.\, D} - 1$$

And the pore water pressure at M as a function of the hydraulic gradient:

$$u_M = (i + 1)\gamma_w \,.\, D = (i \,.\, \gamma_w + \gamma_w) \,.\, D$$

Finally, the vertical effective stress at M is given by:

$$\acute{\sigma}_V = \gamma_{sat} \,.\, D - (i \,.\, \gamma_w + \gamma_w) \,.\, D$$
$$\acute{\sigma}_V = (\gamma_{sat} \,.\, -i \,.\, \gamma_w - \gamma_w) \,.\, D$$
$$\acute{\sigma}_V = (\acute{\gamma} \,.\, -i \,.\, \gamma_w) \,.\, D$$

Thus, upward flow generates a force on the granular skeleton that opposes gravity. This results in a reduction of the effective stress equal to $i \,.\, \gamma_w . D$.

For a sufficiently high value of i, the effective stress becomes zero and can even become negative (directed upward). In this state, the soil grains are suspended in water and cannot support any load. This condition is referred to as **boiling**.

The critical hydraulic gradient, i_c, is defined as the hydraulic gradient at which the effective stress becomes zero (the onset of boiling).

$$\acute{\sigma}_V = (\acute{\gamma} \,.\, - i_c \,.\, \gamma_w) . D = 0 \rightarrow i_c = \frac{\acute{\gamma}}{\gamma_w} \qquad (4.31)$$

The phenomenon of **boiling** can lead to serious accidents if structures are founded on soil where it occurs, or if the soil itself forms part of the structure, such as in dikes, earthen dams, or excavation bottoms.

In all soil hydraulics problems, it is crucial to ensure that the actual upward hydraulic gradients remain well below the critical gradient, i_c.

In the case of sands, the critical gradient is generally very close to 1.

8.2. Piping

The phenomenon of piping occurs in the case of upward vertical flow. In the general case of flow, whether vertical or not, through a permeable medium, water can locally reach high velocities capable of entraining fine soil particles. As a result, the soil becomes locally more permeable, the water velocity increases, and the phenomenon intensifies. Larger soil elements may be transported as erosion progresses regressively (from downstream to upstream) along a flow path (Figure 4.21). A conduit forms through which water rushes, completely disrupting the soil structure (Figure 4.22). This is known as the **"fox phenomenon."**

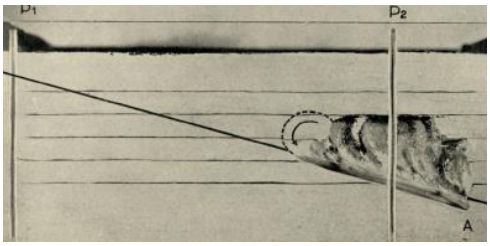


Figure 4.23: Progression of a fox phenomenon in a soil mass



Figure 4.24: Disruption in an earthen dam caused by the development of a fox phenomenon along a conduit

Chapter V:

Exercises

and Solutions for Selected Exercise

Chapter V: Exercises and Solutions

1. Soil Identification

Exercise 1.1

A soil sample has a volume of 0.1 m^3 and weighs 1.8 kN. The water content determined in the laboratory is 12.6%. The solid grains have a specific gravity of 2.71, and g = 9.81 m/s^2.

Required:

1. The moist unit weight γ
2. The void ratio e
3. The dry unit weight γd
4. The porosity n
5. The degree of saturation Sr

Solution 1.1:

1. The moist unit weight

$$\gamma = \frac{W}{V} = \frac{1.8}{0.1} = \mathbf{18}\ \frac{\boldsymbol{KN}}{\boldsymbol{m^3}}$$

2. The void ratio

$$e = G_s \,.\, (1 + w)\frac{\gamma_w}{\gamma} - 1 = 2.71\,.\,(1 + 0.126)\frac{9.81}{18} - 1 = \mathbf{0.663}$$

$$e\ (\%) = \mathbf{66.3}\%$$

3. The dry unit weight

$$\gamma_d = \frac{G_s}{1 + e}\gamma_w = \frac{2.71}{1 + 0.663}9.81 = \mathbf{15.986}\frac{\boldsymbol{KN}}{\boldsymbol{m^3}}$$

4. The porosity

$$n = \frac{e}{1 + e} = \frac{0.663}{1 + 0.663} = \mathbf{0.3987}$$

$$n\ (\%) = \mathbf{39.87}\%$$

5. The degree of saturation

$$S_r = \frac{G_s\, w}{e} = \frac{2.71\ x\ 0.126}{0.663} = 0.5150$$
$$S_r\ (\%)\ =\ 51.50\%$$

Exercise 1.2

Prove the following relationships:

$$e = \frac{n}{1-n}$$
$$\gamma_d = \frac{\gamma}{1+w}$$
$$S_r = \frac{w\,\gamma_s}{\gamma_w\, e}$$

Solution 1.2:

Proof of the following relationships:

$$e = \frac{V_v}{V_s} = \frac{V_v V}{V_s V} = \frac{n\,V}{V_s} = \frac{n\,V}{V - V_v} = \frac{n}{1-n}$$
$$\gamma_d = \frac{W_s}{V} = \frac{W\,W_s}{W\,V} = \gamma\frac{W_s}{W} = \gamma\frac{W_s}{W_s + W_w} = \frac{\gamma}{1+w}$$
$$S_r = \frac{V_w}{V_v} = \frac{V_w\,W_w}{V_v\,W_w} = \frac{W_w}{V_v\,\gamma_w} = \frac{W_w\,W_s}{V_v\,\gamma_w\,W_s} = \frac{w\,W_s}{V_v\gamma_w} = \frac{w\,W_s\,V_s}{\gamma_w\,V_s\,V_v} = \frac{w\,\gamma_s}{\gamma_w\,e}$$

Exercise 1.3

Find the relationship between the void ratio (e) and the porosity (n), using two methods.

Solution 1.3:

- Method No. 01

$$e = \frac{V_v}{V_s} = \frac{V_v}{V - V_v} = \frac{\left(\frac{V_v}{V}\right)}{1 - \left(\frac{V_v}{V}\right)} = \frac{n}{1-n}$$
$$n = \frac{e}{1+e}$$

- Method No. 02

Given

$$e = \frac{V_v}{V_s}$$

We propose

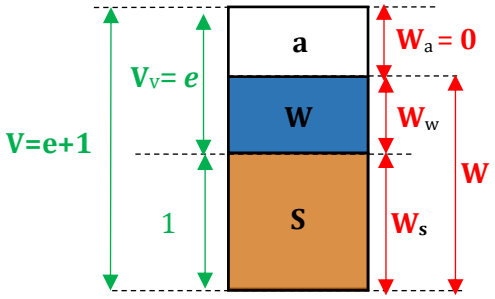
$\mathbf{V_S = 1}$

$$e = \frac{V_v}{1}$$

$$V = e + 1$$

$$n = \frac{V_v}{V} = \frac{e}{1+e}$$

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



Exercise 1.4

Find the relationship between the following parameters (e, S_r, W, G_s), using two methods.

Solution 1.4:

- Method No. 01

$$W = \frac{W_w}{W_s} = \frac{\gamma_w V_w}{\gamma_s V_s} = \frac{\gamma_w V_w}{\gamma_w G_s V_s} = \frac{V_w}{G_s V_s} = \frac{S_r V_v}{G_s V_s} = \frac{S_r\, e}{G_s}$$

$$W G_s = S_r\, e$$

- Method No. 02

Given

$$(e, S_r, G_s)$$

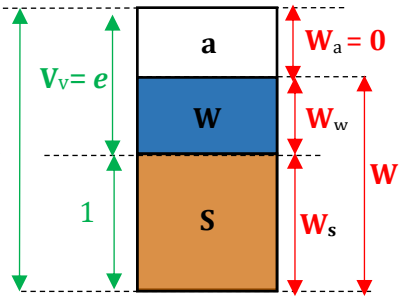
And find W

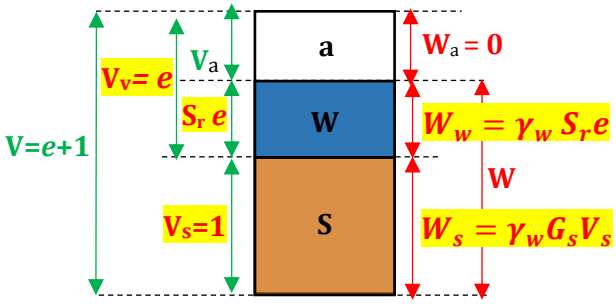
$$e = \frac{V_v}{V_s} = \frac{V_v}{1} = V_v$$

V=e+1

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

$$V_w = V_v\, S_r = e\, S_r$$





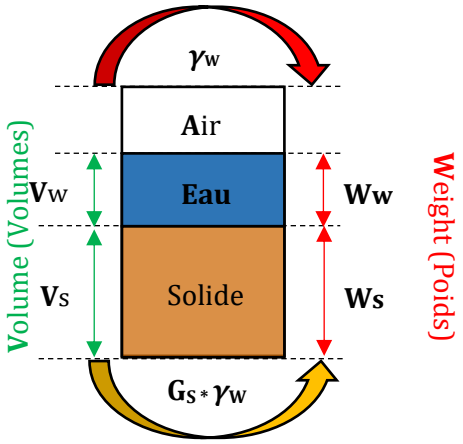
We propose

V_S = 1

$$W = \frac{W_w}{W_s} = \frac{\gamma_w S_r e}{\gamma_w G_s 1}$$

$$W = \frac{e\, S_r}{G_s 1}$$

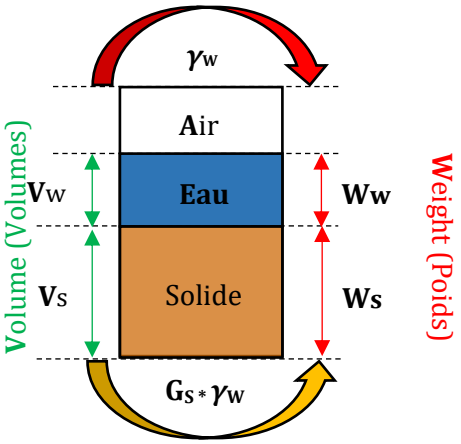
$$W G_s = e\, S_r$$



Justify the relationships using the phase diagram method in 4 steps:

Steps:

1- If the relationship contains (**γ, G_S**), eliminate these parameters.
2- Start with any of the remaining parameters and assume the denominator = 1.
3- Substitute into the prism (phase diagram).
4- The prism results are substituted into the first parameter (the remaining parameters after eliminating γ and GS).



Phase Diagram

Exercise 1.5

Justify the following relationship

$$\gamma = \left(\frac{G_s + S_r\, e}{1 + e}\right)\gamma_w$$

Or find the relationship between:

$$\gamma, G_s, S_r, e, \gamma_w$$

Solution 1.5:

Justify the relationships using the phase diagram method in 4 steps.

Steps:

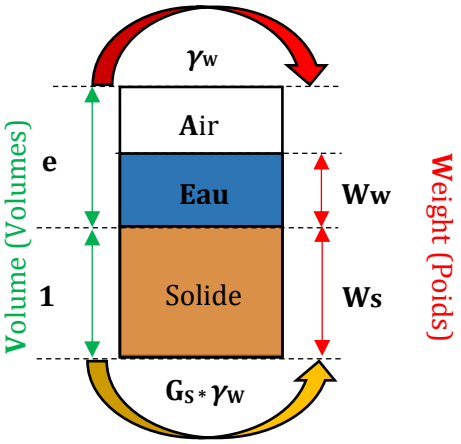
1. If the relationship contains (γ, G_S), eliminate these parameters.

$$\cancel{\gamma}, \cancel{G_s}, S_r, e, \cancel{\gamma_w}$$

2. Start with any of the remaining parameters (S_r, e), and assume the denominator = 1.

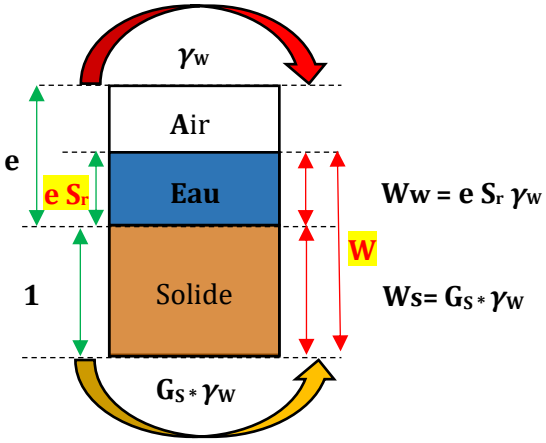
$$e = \frac{V_v}{V_s} = \frac{V_v}{1} = V_v$$

3. Substitute into the prism (phase diagram).



$$S_r = \frac{V_w}{V_v} = \frac{V_w}{e}$$

$$e\,S_r = V_w$$



4. The prism results are substituted into the first parameter (the remaining parameters after eliminating γ,γ_w et G_S).

The parameters (γ_w et G_S), are constants and are used accordingly (γ).

$$\gamma = \frac{W}{V} = \frac{e\,S_r\,\gamma_w + G_s\,\gamma_w}{e+1} = \frac{(e\,S_r + G_s)\,\gamma_w}{e+1}$$

$$\gamma = \frac{(e\,S_r + G_s)\,\gamma_w}{e+1}$$

2. Soil Classification

Exercise 2.1

A soil sample has a particle size distribution of 10% sand, 20% silt, and 70% clay.

- Determine the soil type according to the textural (triangular) classification.

Solution 2.1:

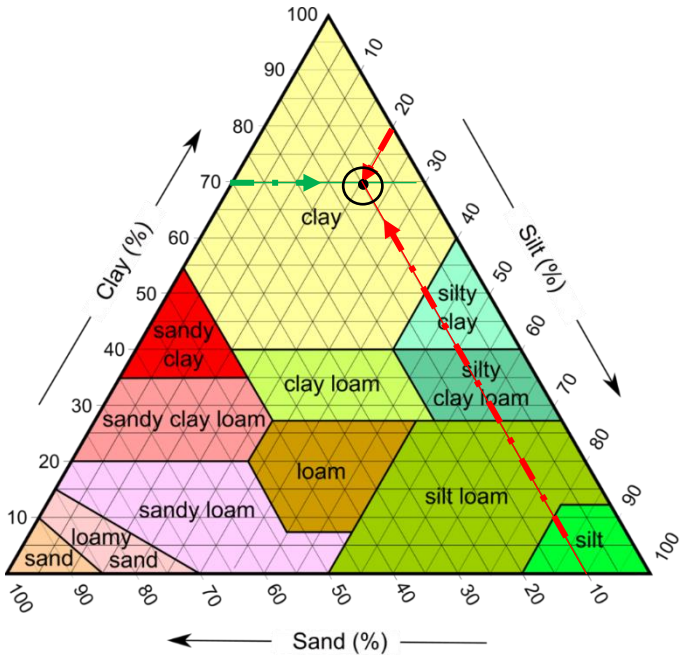


Figure 5.1: Triangular Classification

Based on the grain size percentages and their position on the chart, the soil textural classification is clay.

Exercise 2.2

A soil sample has a grain-size distribution of 10% sand, 20% gravel, 30% silt, and 40% clay-sized particles.

- Determine the soil type according to the textural (triangular) classification.

Solution 2.2:

Calculate the modified percentage:

- Sand (%) = (10 × 100) / (100 − 20) = 12.5%
- Silt (%) = (30 × 100) / (100 − 20) = 37.5%
- Clay (%) = (40 × 100) / (100 − 20) = 50%

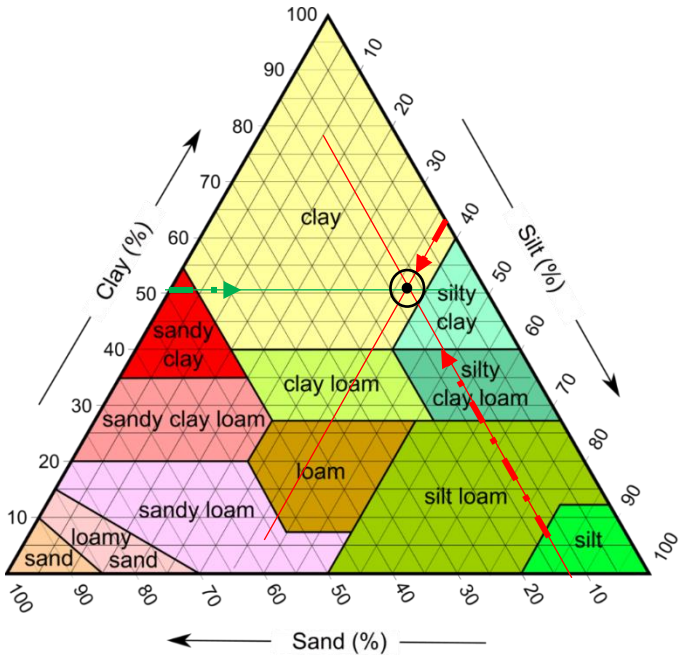


Figure 5.2: Triangular Classification

- Based on the modified percentages, the textural classification is **clay**.
- However, due to the significant percentage of gravel, it can be referred to as **gravelly clay**.

Exercise 2.3

Four soil samples (A, B, C, and D) with different grain-size distributions (Table 2.1) are given. Classify the soils according to the textural classification system.

Table 5.1: Normal Soil Samples

Particle Size (%)	Natural Soils (%)			
	A	B	C	D
Gravel	12	18	0	12
Sand	25	31	15	22
Silt	32	30	30	26
Clay	31	21	55	40

Solution 2.3:

Calculate the modified percentage:

- Modified **Sand (%)**

$$= \frac{\text{Sand \%}}{(100 - \text{gravier})} \textbf{ x } 100$$

- Modified **Silt (%)**

$$= \frac{\text{Silt \%}}{(100 - \text{gravier})} \textbf{ x } 100$$

- Modified **Clay (%)**

$$= \frac{\text{Clay \%}}{(100 - \text{Gravel})} \textbf{ x } 100$$

Table 5.2: Modified Soils

Particle Size (%)	Modified Soils (%)			
	A	**B**	**C**	**D**
Sand	28.4	37.8	15	**25**
Silt	36.4	36.6	30	**29.5**
Clay	35.2	25.6	55	**45.5**

A Gravelly Clay loam

B Gravelly Silt

C Clay

D Gravelly Clay

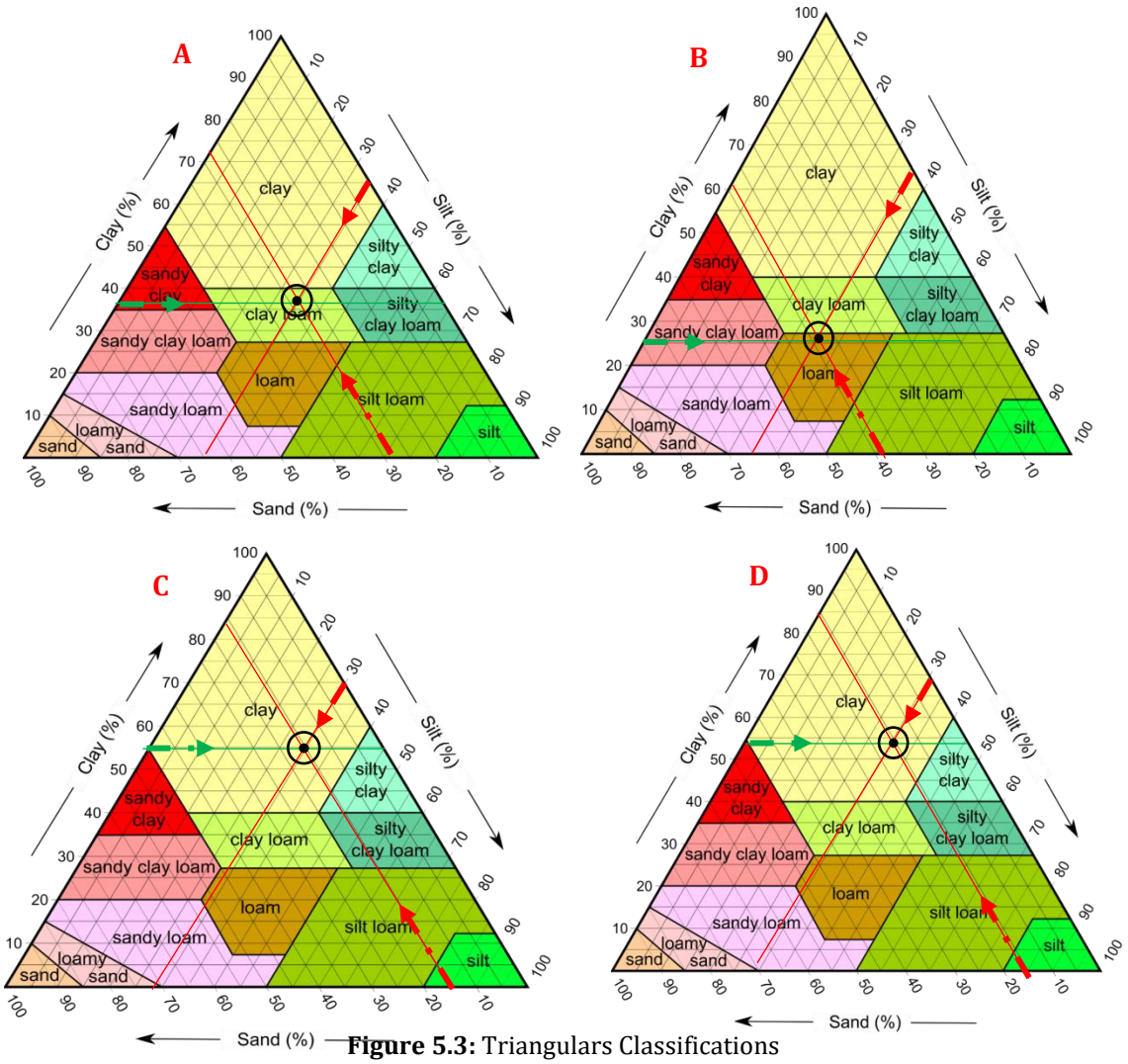


Figure 5.3: Triangulars Classifications

Exercise 2.4

Two types of soils, A and B, with their liquid and plastic limits (Table 2.2), are given. Classify the soils according to the Casagrande chart (Figure 5.3) for fine soils under the following condition: more than 50% of particles are < 0.08 mm.

Table 5.3: Results of Atterberg Limits

Soil	A	B
Liquid Limit (%)	40	70
Plastic Limit (%)	22	48

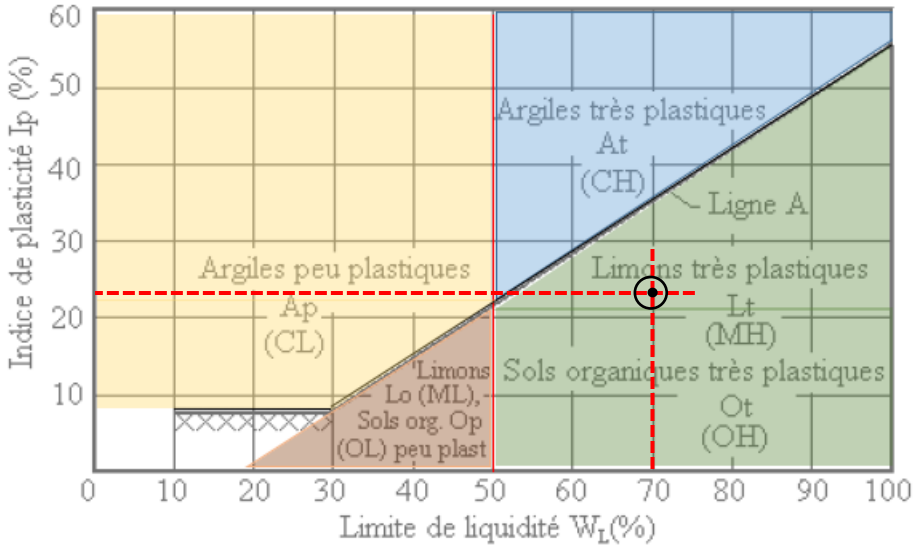


Figure 5.4: Casagrande Chart for Fine Soil Classification (More than 50% of particles < 0.08 mm)

Solution 2.4:

Calculation of the Plasticity Index (PI):

The plasticity index is calculated as:

$$P_I = L_L - P_L$$

- **Soil A**

P_I= 40 - 22 = **18**

P_I = 18 et **W_L** = 40 Therefore, the soil is low-plasticity clay (CL). (Argile peu plastiques AP).

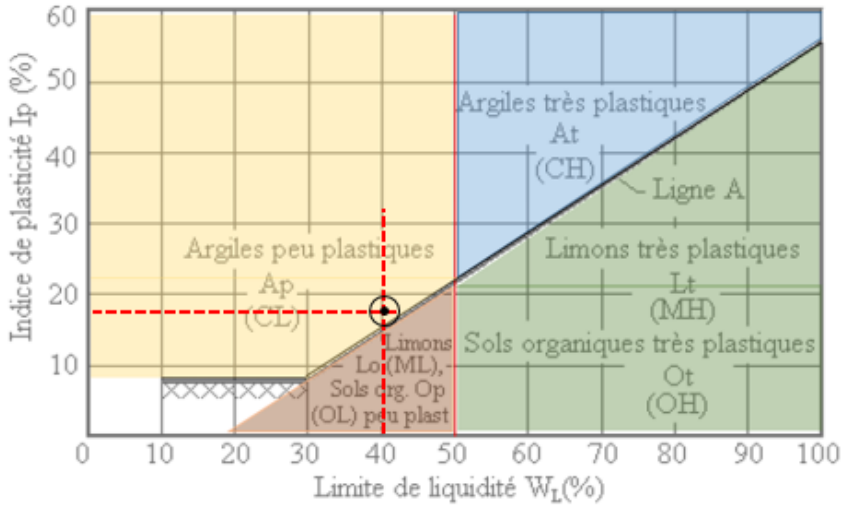


Figure 5.5: Casagrande Chart for Fine Soil Classification

- **Soil B**

I_P = 70 - 48 = **22**

Ip = 22 et **W_L** = 70 Therefore, the soil is highly plastic silt (MH), limons très plastiques Lt.

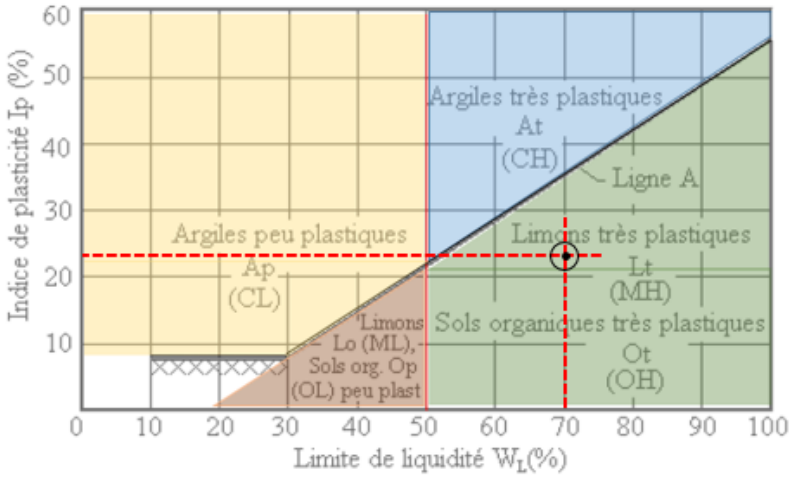


Figure 5.6: Casagrande Chart for Fine Soil Classification

Exercise 2.5

From a grain size curve of a soil: $D_{10} = 0{,}085$ mm, $D_{30} = 0{,}12$ mm and $D_{60} = 0{,}135$ mm.

- Determine the uniformity coefficient, C_u
- Determine the coefficient of curvature, C_c

Solution 2.5

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

$$C_c = \frac{(0.12)^2}{(0.135)\ \mathrm{x}\ (0.085)} = 1.25$$

Exercise 2.6

Sieving of a 1000 g sand sample gives the retained mass on each of the following sieves:

Table 5.4: Sieve opening, with Mass retained on each sieve

Sieve opening (mm)	12.5	5	2	1	0.5	0.2	0.1
Mass retained on each sieve (g)	0	62	248	313	231	127	11

- Plot the grain size curve.
- Determine the soil uniformity and curvature coefficients.
- Give the soil classification according to the LCPC system.

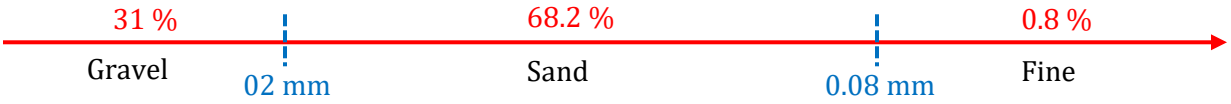
Solution 2.6

The results of the grain size analysis are represented in the form of a curve showing the percentage by weight of soil particles passing through a sieve with opening D (mm). The grain size distribution curve is plotted on a semi-logarithmic scale, with log 10D on the horizontal axis (abscissa) and a linear scale for the percentage passing on the vertical axis.

Table 5.5: Sieve Retained, Cumulative retained, and Passing

Sieve (mm)	Retained (g)	Cumulative retained (g)	Cumulative retained (%)	Passing (%)
12.5	0	0	0	100
5	62	62	6.2	93.8
2	248	310	31	69
1	313	623	62.3	37.7
0.5	231	854	85.4	14.6
0.2	127	981	98.1	1.9
0.1	11	992	99.2	0.8
Pan	0	0	0	0

Cumulative retained = (62*100) /1000 = 6.2



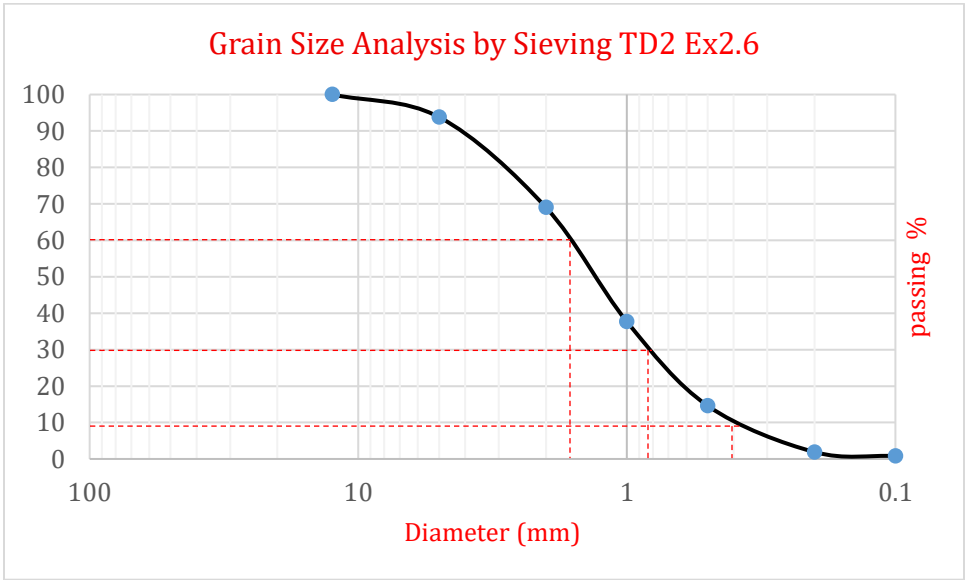


Figure 5.7: Curve of Grain Size Analysis by Sieving

- Determine D60, D30, and D10

D60= 1.71246, **D30=**0.833333**, and D10=**0.391339.

- Determine the coefficients Cc et Cu

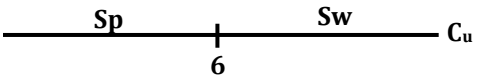
C_u = 4.375904; **C_c =**1.03625

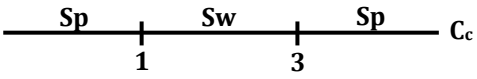
Soil classification

99.2 % of the particles have D > 0.1 mm, therefore, more than 50% have a diameter> 0.08 mm.

- More than 50% (69% passing) have D < 2 mm
- Also, 0.08 % (D<0.08) < 5%

The soil is sand





- **C_c =**1.03625
- **C_u**=4.37 < 6 The condition for sand is not satisfied

Therefore, the soil is a clean poorly graded sand (SP)

Table 5.6: Classification of granular soils

Definitions			Symbol	Conditions	Names
Gravel	**More** than **50%** of the particles **> 0.08 mm** have a diameter **> 2 mm**	**Less** than **5%** of particles **< 0.08 mm**	**GW**	$C_u = D_{60}/D_{10} > 4$ and $1 < C_c = (D_{30})^2/D_{10}D_{60} < 3$	**Well-graded clean Gravel**
			GP	**One of the GW conditions is not satisfied**	**Poorly-graded clean Gravel**
		More than **12%** of particles **< 0.08 mm**	**GM**	**Atterberg limits below the A-line**	**Silty Gravel**
			GC	**Atterberg limits above the A-line**	**Clayey Gravel**
Sand	**More** than **50%** of the particles **> 0.08 mm** have a diameter **< 2 mm.**	**Less** than **5%** of particles **< 0.08 mm**	**SW**	$C_u = D_{60}/D_{10} > 6$ and $1 < C_c = (D_{30})^2/D_{10}D_{60} < 3$	**Well-graded clean Sand**
			SP	**One of the SW conditions is not satisfied**	**Poorly-graded clean Sand**
		More than **12%** of particles **< 0.08 mm**	**SM**	**Atterberg limits below the A-line**	**Silty Sand**
			SC	**Atterberg limits above the A-line**	**Clayey Sand**
When the fine particle content (< 0.08 mm) is between 5% and 12%, a dual symbol is used. For example: SW-SM					

Exercise 2.7

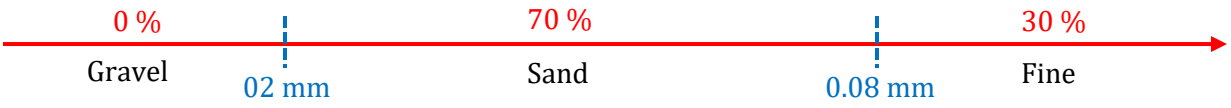
The following Table 2.4 presents the characteristics of soil samples A and B. Classify the following soils according to the LCPC classification.

Table 5.7: Characteristics of Soil A and B

Soil	Percent passing 0.08 mm	Percent passing 2 mm	Cu	Cc	LL	PL
A	30 (%)	70 (%)	7	2.5	40	25
B	70 (%)	100 (%)	---	---	60	30

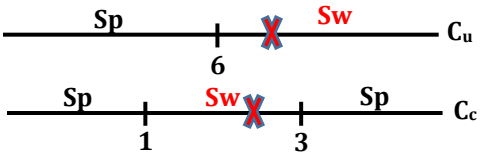
Solution 2.7

Soil classification



- 70% > 50% therefore the soil is Sand

The soil is a sand



- C_c = 2.5
- C_u= 7 > 6 For sand, the conditions are satisfied

Therefore, the sand is a well-graded sand.

- More than12% < 0.08 (30%)

Table 5.8: Classification of granular soils

Definitions			Symbol	Conditions	Names
Gravel	More than 50% of the particles > 0.08 mm have a diameter > 2 mm	Less than 5% of particles < 0.08 mm	GW	$C_u = D_{60}/D_{10} > 4$ and $1 < C_c = (D_{30})^2/D_{10}D_{60} < 3$	Well-graded clean Gravel
			GP	One of the GW conditions is not satisfied	Poorly-graded clean Gravel
		More than 12% of particles < 0.08 mm	GM	Atterberg limits below the A-line	Silty Gravel
			GC	Atterberg limits above the A-line	Clayey Gravel
Sand	More than 50% of the particles > 0.08 mm have a diameter < 2 mm.	Less than 5% of particles < 0.08 mm	SW	$C_u = D_{60}/D_{10} > 6$ and $1 < C_c = (D_{30})^2/D_{10}D_{60} < 3$	Well-graded clean Sand
			SP	One of the SW conditions is not satisfied	Poorly-graded clean Sand
		More than 12% of particles < 0.08 mm	SM	Atterberg limits below the A-line	Silty Sand
			SC	Atterberg limits above the A-line	Clayey Sand
When the fine particle content (< 0.08 mm) is between 5% and 12%, a dual symbol is used. For example: SW-SM					

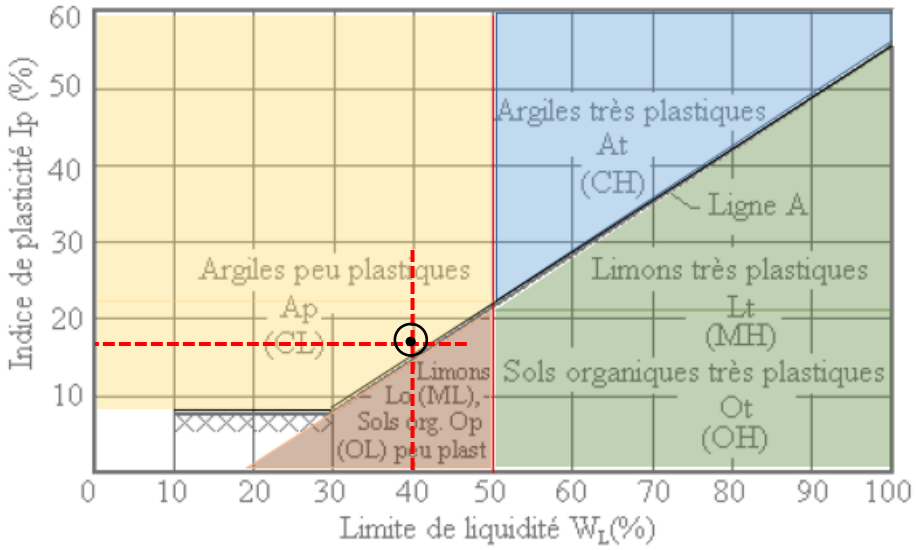


Figure 5.8: Casagrande Chart for Fine Soil Classification

- After the Casagrande chart

Therefore, the soil is a Clayey Sand



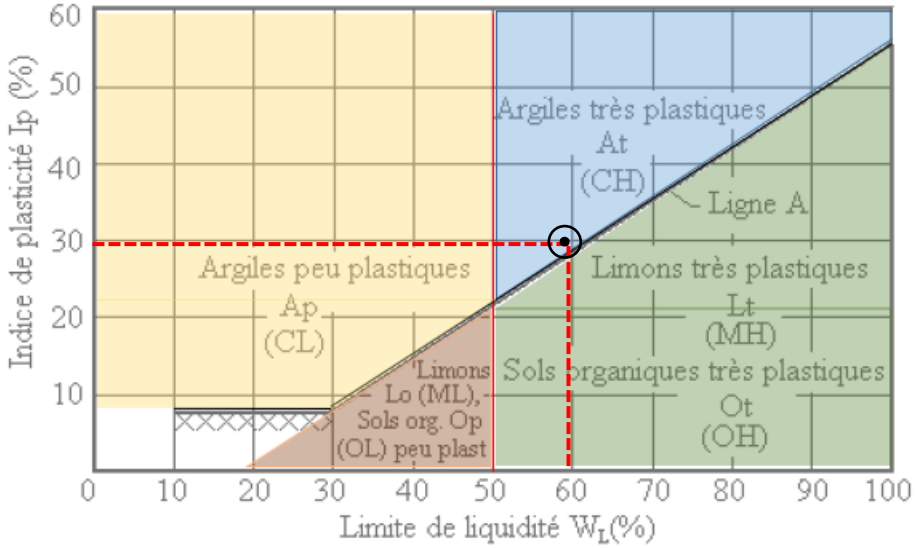


Figure 5.9: Casagrande Chart for Fine Soil Classification

Highly plastic clay (CH).

3. Soil Compaction

Exercise 3.1

The following results were measured during a Proctor test using a standard mold with a volume of 0.96 dm^3. The mass of the mold is 1034 g.

Table 5.9: Soil sample

Mass of a soil sample (g)	**6.65**	**6.12**	**5.02**	**5.18**	**5.20**	**4.77**	**4.74**
Dry mass of the sample (g)	6.03	5.51	4.49	4.60	4.59	4.18	4.12
Mass of compacted soil with the mold (g)	2821	2864	2904	2906	2895	2874	2834

1. Plot the **Proctor compaction curve** and determine:
   - the **maximum dry density**
   - the **optimum water content**
2. Calculate the **water content at 100% saturation (Sr = 100%)** corresponding to the **maximum dry density**, given that:

G_s = 2,67, $\gamma_s = 26.7$ kN/m³, et g = 10 m/s².

Solution 3.1

Table 5.10: Solution Exercise 3.3

Mass of a soil sample (g)	6.65	6.12	5.02	5.18	5.20	4.77	4.74
Dry mass of the sample (g)	6.03	5.51	4.49	4.60	4.59	4.18	4.12
Mass of water (g)	0.62	0.61	0.53	0.58	0.61	0.59	0.62
Water content w (%) = Ww / Ws	10.28	11.07	11.80	12.60	13.28	14.11	15.04
Mass of soil with compaction mold (g)	2821	2864	2904	2906	2895	2874	2834
Mass of the mold (g)	1034						
Mass of soil (g)	1787	1830	1870	1872	1861	1840	1800
Mass of soil (kg)	1.787	1.830	1.870	1.872	1.861	1.840	1.800
Volume of the mold (m^3)	0.96×10^{-3}						
Bulk density ρ(kg/m³)= M_{sol}/V_{moule}	1861.46	1906.25	1947.92	1950	1938.54	1916.67	1875
Dry density ρ_d (kg/m³)= ρ /(1+w)	1687.94	1716.26	1742.32	1731.80	1711.28	1679.67	1629.87
Dry unit weight γd (kN/m³)	**16.879**	**17.1626**	**17.4232**	**17.318**	**17.1128**	**16.7967**	**16.2987**

0.96 dm^3 = 0.00096 m^3

- The Proctor curve

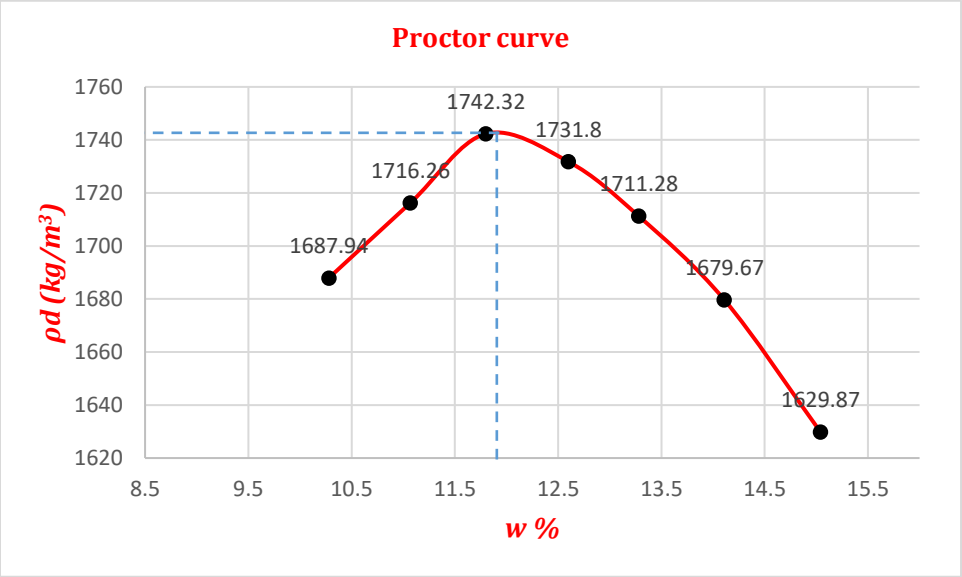


Figure 5.10: Proctor curve

- Maximum dry density and optimum water content

ρ_d =1742.33 Kg/m³

***w* =11.80 %**

• Water content at full saturation

Degree of saturation: 100%

$$S_r = 1$$

$$G_s = 2.67$$

$$\gamma_d = 17.4233$$

$$\gamma_s = 26.7$$

$$\gamma_d = \frac{\gamma_s}{\left(1 + w\frac{G_s}{S_r}\right)}$$

$$w = \frac{S_r}{G_s}\left(\frac{\gamma_s}{\gamma_d} - 1\right) = 0.1994$$

***w* =19.94 %**

Exercise 3.2

In a compaction test, the results shown in the table below were obtained.

Table 5.11: Compaction test results

Water content (%)	Wet unit weight (kN/m³)
	25 blows / layer
11.2	20.40
12.9	21.28
14.0	21.63
16.3	21.44
18.8	20.93
20.5	20.48

1. Plot the curve of dry density versus water content for the given results. Determine the maximum dry density and the corresponding optimum water content.
   Assume that $G_s = 2.7$ and $g = 9.81\ m/s^2$.
2. Calculate the degree of saturation corresponding to the maximum dry density and determine the water content of this material at full saturation without any change in volume.
3. Plot the curves corresponding to degrees of saturation of 90%, 95%, and 100%.

Solution 3.2

Table 5.12: Solution Exercise 3.4

w (%)	γ_h (kN/m³)	$\gamma_d = \frac{\gamma_h}{1+w}$ (kN/m³)	$\rho_d = \frac{\gamma_d}{g}10^3$ (kg/m³)
11.2	20.40	18.34	1869.52
12.9	21.28	18.84	1920.48
14.0	21.63	18.97	1933.74
16.3	21.44	18.43	1878.69
18.8	20.93	17.61	1795.10
20.5	20.48	16.99	1731.91

- The curve

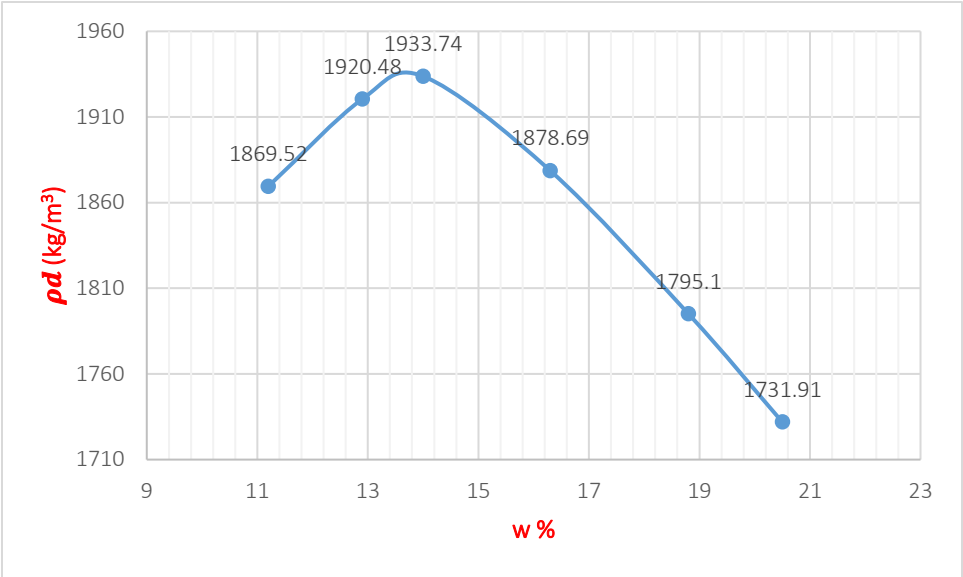


Figure 5.11: Curve of dry density versus water (Proctor curve)

According to the curve, we have:

$$\boldsymbol{\rho_{d\,(max)} = 1933.74\ (kg/m^3)\ w = 14\ \%}$$

- Calculation of the void ratio and the degree of saturation.

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

$$\boldsymbol{e} = G_s(1+w)\frac{\rho_w}{\rho} - 1$$

$$\boldsymbol{e} = 2.7(1+0.14)\frac{1000}{1933.74} - 1$$

$$e = 59.17\ \%$$

$$s_r = \frac{wG_s}{e}$$

$$s_r = \frac{0.14\ x\ 2.7}{0.5917}$$

$$s_r = 63.88\ \%$$

- Calculation of the water content after saturation: **Sr = 1**

$$s_r = \frac{wG_s}{e}$$

$$w = \frac{e\, s_r}{G_s}$$

$$w = \frac{0.5917\ x\ 1}{2.7}$$

$$\boldsymbol{w = 21.91\ \%}$$

- Saturation curves

S_r = 90 % therefore A= 10%

$$\boldsymbol{\rho_d = \frac{G_s(1-A)}{1 + G_s * w}\rho_w}$$

Table 5.13: Saturation (Sr = 90 % therefore A= 10%)

$\boldsymbol{w}$ **(%)**	$\boldsymbol{\rho_d = \frac{G_s(1-0.1)}{1 + G_s * w}10^3}$ **(kg/m³)**
11.2	1865.78
12.9	1802.26
14.0	1763.42
16.3	1687.38
18.8	1611.8
20.5	1564.20

S_r = 95 % therefore A= 5 %

Table 5.14: Saturation (S_r = 95 % therefore A= 5 %)

$\boldsymbol{w}$ **(%)**	$\boldsymbol{\rho_d = \frac{G_s(1-0.05)}{1 + G_s * w}10^3}$ **(kg/m³)**
11.2	1969.44
12.9	1902.39
14.0	1861.39
16.3	1781.12
18.8	1726.11
20.5	1651.11

S_r = 100 % therefore A= 0%

Table 5.15: Saturation (Sr = 100 % therefore A= 0%)

w (%)	$\rho_d = \frac{G_s(1-0)}{1+G_s * w}10^3$ (kg/m³)
11.2	2073.09
12.9	2002.52
14.0	1959.36
16.3	1874.86
18.8	1790.92
20.5	1738.01

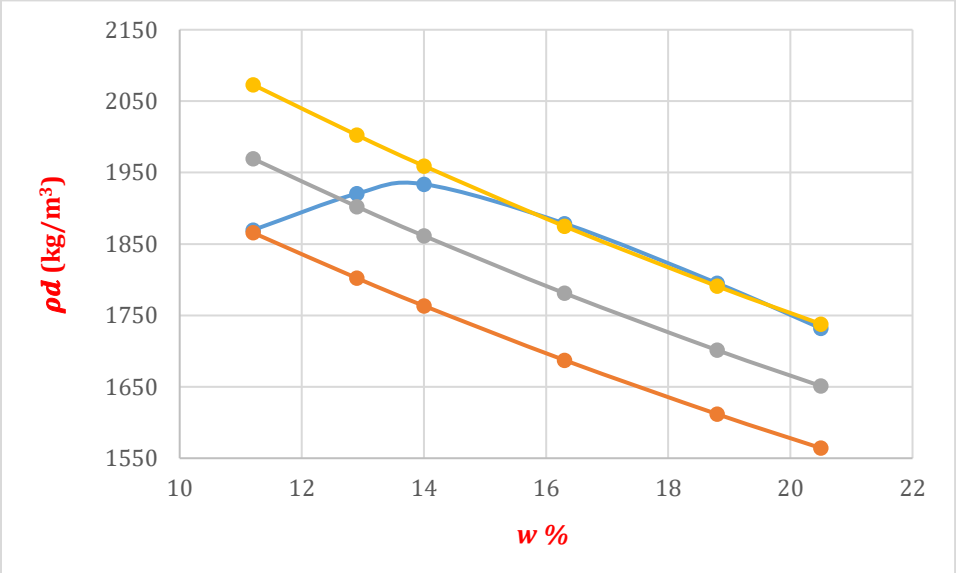


Figure 5.12: Curve of dry density versus water (Proctor curve)

4. Water in the Soil

Exercise 4.1

A test to determine the **coefficient of permeability** of a soil sample is carried out in the laboratory using the **constant head method**.

Sample height: **L = 200 mm**

Hydraulic head difference: **h = 83.2 mm**

Cylinder diameter: **D = 75 mm**

Volume of water: **V = 66.8 ml**

Test duration: **t = 1 min**

- Determine the coefficient of permeability of the tested sample.

Solution 4.1:

$$K = \frac{V.L}{S.\Delta h.t}$$

$$S = \frac{D^2\,\pi}{4}$$

S = 4415.625 mm²
V=66.8 ml = 66.8 x 10^3 mm³
t =60 s

Therefore

$$K = \frac{(66.8 \text{ x } 10^3)\,x\,200}{(4415.625)\text{x}\,(83.2)x(60)} = 0.606\ mm/s$$

Exercise 4.2

A sample of a **cohesive soil** was tested in the laboratory using the **falling head permeability test**.

Initial water level in the tube: **h_1 = 1350 mm**

Final water level in the tube: **h_2 = 550 mm**

Sample length: **L = 140 mm**

Cylinder diameter: **D = 100 mm**

Tube diameter: **d = 5 mm**

Test duration: **t = 3 min**

- Determine the coefficient of permeability of the sample.

Solution 4.2:

h_1 = 1350 mm

h_2 = 550 mm

$$K = \frac{s.L}{S.t} ln \frac{h_1}{h_2}$$

$$S = \frac{D^2 \pi}{4}$$

Tube area S = 19.625 mm²
Cylinder area S= 7850 mm²
Time t =3 x 60 = 180 s

Therefore

$$K = \frac{19.625\ x\ 140}{7850\ x\ 3\ x\ 60} ln \frac{1350}{550} = 1.746\ mm/s$$

Exercise 4.3

During a pumping test in a sandy soil (see figure below), the initial groundwater level was 11.20 m above the impermeable layer. While pumping from the main well at a rate of 6.20 m^3/min, the water level dropped by 5.15 m in piezometer 1 and 2.48 m in piezometer 2. Given: R_1 = 4.33 m, R_2 = 45.86 m

- Determine the coefficient of permeability of the soil.

Solution 4.3:

$$K = Q \frac{ln(\frac{R_2}{R_1})}{\pi(H_2^2 - H_1^2)}$$

H_1 = 11.20-5.15= 6.05 m

H_2 = 11.20-2.48= 8.72 m

Q = 6.20 m^3/s =6.20/60=0.1033 m^3/s

Therefore

$$K = 0.1033 \frac{ln(\frac{45.86}{4.33})}{\pi(8.72^2 - 6.05^2)} = 1.97\ x10^{-3}\ m/s$$

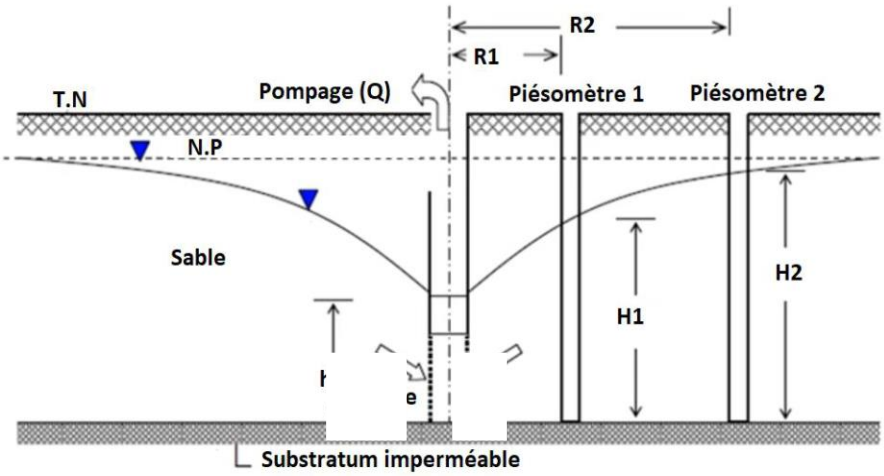


Figure 5.13: Pumping test in a sandy soil

Exercise 4.4

The water level in a thick deposit of very fine sand is 1.20 m below the ground surface. Above the groundwater level, the sand is capillary saturated. The unit weight of the saturated sand is 20.3 kN/m^3.

- What is the effective vertical stress on a horizontal plane located 3.60 m below the ground surface?

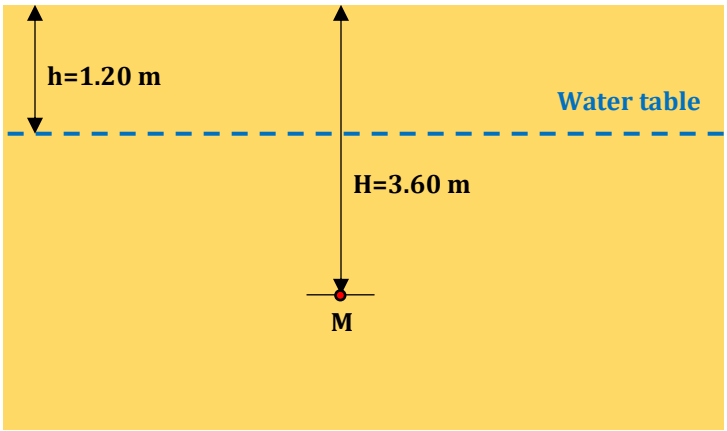


Figure 5.14: Vertical stress with water table

Solution 4.4:

$\gamma_w = 10\ \text{KN/m}^3$

$\mathbf{H = 3.60\ m}$

$\mathbf{h = 1.20\ m}$

$\gamma_{sat} = 20.3\ \text{KN/m}^3$

$$\sigma'_v = \sigma_v - u$$

$$u = \gamma_w(H - h)$$
$$u = 10(3.60 - 1.20)$$
$$u = 24\ \text{KPa}$$

$$\sigma_v = H \text{ x } \gamma_{sat}$$
$$\sigma_v = 3.60 \text{ x } 20.3$$
$$\sigma_v = 73.08\ \text{KPa}$$

$$\sigma'_v = 73.08 - 24$$
$$\sigma'_v = 49.08\ \text{KPa}$$

Exercise 4.5

A steel sheet pile is driven into a soil layer of thickness 8.60 m (see Figure 5.15). The embedment depth of the sheet pile is 6.00 m in the soil and 4.50 m in the water. On the right side of the sheet pile, the water level has been lowered to 0.50 m. The soil permeability is K = 4×10^{-4} cm/s. The second soil layer in the ground profile is impermeable.

- Graphically determine the head loss between two successive equipotential lines.
- Determine the total discharge per unit length of the sheet pile.

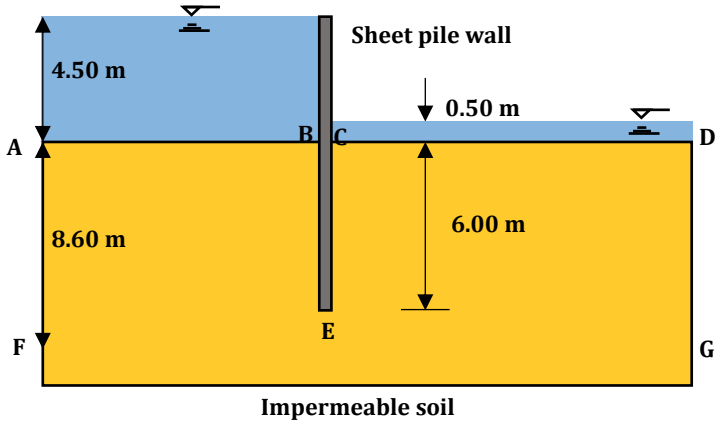


Figure 5.15: Geometry soil and sheet pile wall

Solution4.5:

Conditions for drawing a two-dimensional flow net:

1- Draw at a reasonable scale;

2- Equipotential lines intersect flow lines at right angles.

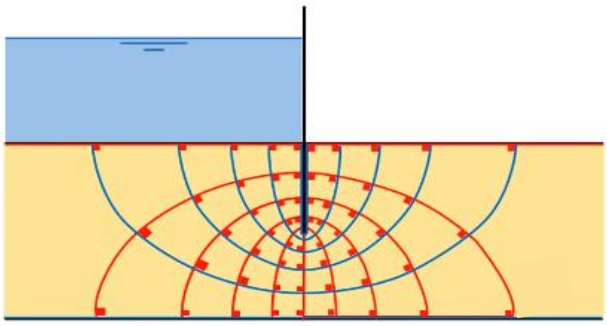
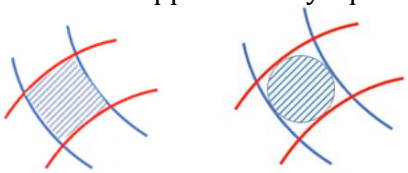


Figure 5.16: Equipotential lines intersect flow lines at right angles

3- The flow elements formed should be approximately square.



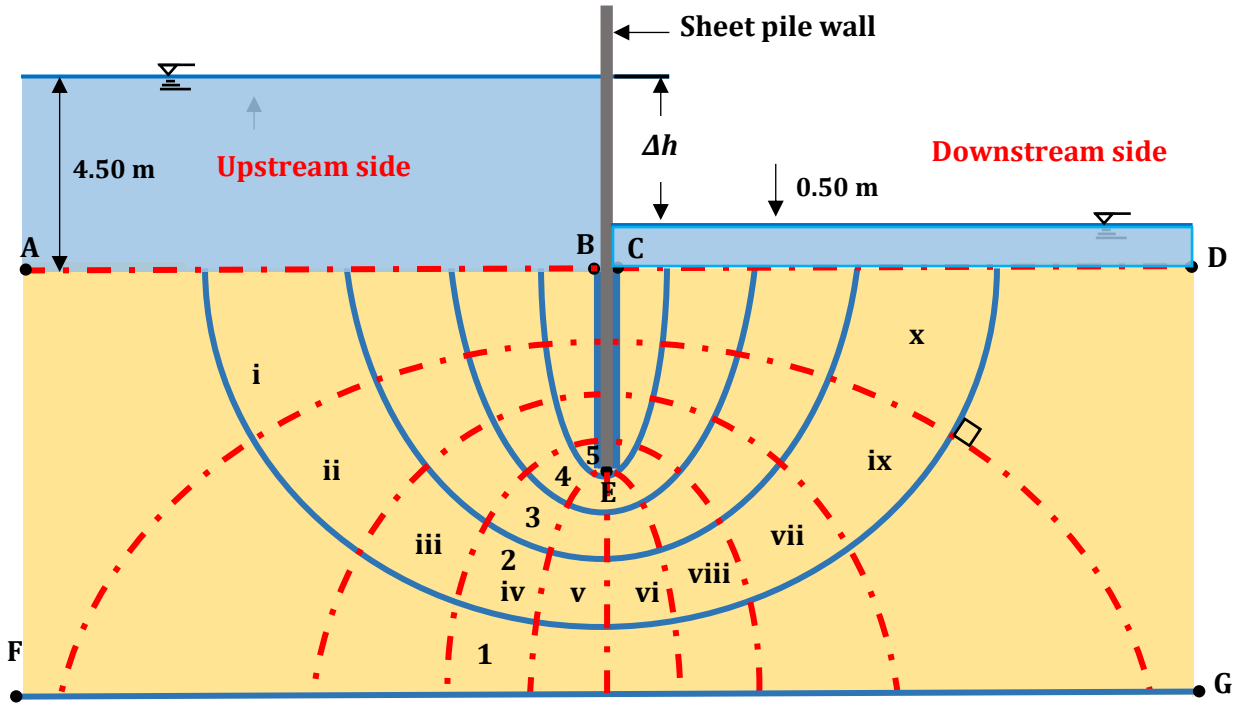


Figure 5.17: Equipotential lines intersect flow lines at right angles

Np = 10 number of equipotential drops

Nt= 5 number of flow channels

Δh = 4 m

K= 4 x 10^{-4} cm/s = 4 x 10^{-6} m/s

The head loss between two successive equipotential lines is:

$$\Delta h' = \frac{\Delta h}{N_p}$$

$$\Delta h = 4.5 - 0.5 = 4\ m$$

$$\Delta h' = \frac{4}{10} = 0.4\ m$$

The discharge of water under the sheet pile per unit time

$$Q = k\Delta h \frac{N_t}{N_p}$$

$$Q = (4 \text{ x } 10^{-6}) \text{ x } 4 \frac{5}{10} = 8 \text{ x } 10^{-6}\ m^3/s$$

Annex

Standardization of Geotechnical Tests

Geotechnical Test	French Standard (NF/EN)	American Standard (ASTM)
Grain Size Analysis	NF EN ISO 17892-4	ASTM D6913
Atterberg Limits	NF EN ISO 17892-12	ASTM D4318
Proctor Compaction Test	NF P94-093	ASTM D698 / ASTM D1557
Permeability Test	NF EN ISO 17892-11	ASTM D2434

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