Democratic and Popular Republic of Algeria

Ministry Higher Education and Scientific Research

Djillali LIABES University, Sidi Bel-Abbès

Faculty of Technology

EBST Department

2nd year Civil Engineering and Public Works

Courses and application exercises of Soil mechanics

Intended for students in the 2nd yearth Year of Civil Engineering and 3th Hydraulics License Year

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Academic year 2025-2026

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FOREWORD

This handout is intended for students in the 2nd yearth Bachelor's degree (S4) of Civil Engineering and 3rd year license (S5) in Hydraulics. It covers the official program selected within the framework of the standardization of the educational program of the license. Its objective is to provide the necessary bases for understanding the behavior of soils in order to be able to design, build and control the structures with which the soil has a significant interaction. It also presents the fundamental elements of the subject "Soil Mechanics I". It does not aim to replace the many works and documents that exist in the literature, but constitutes an overview and a general synthesis allowing students to understand the broad outlines and fundamental principles of the subject.

The handout includes everything needed to perform the practical calculation in terms of principles, methods, theory, formulas, tables and charts.

In this context, the course is organized into four chapters, where the first one deals with an introduction to soil mechanics, the second chapter deals with the calculation of the physical properties of soils, as well as soil identification tests and their classification. The third chapter deals with the mechanical stabilization of soils by the compaction technique and its theory of application and finally the last and fourth chapter is devoted to soil hydraulics which presents the different methods and theories of calculating the effect of water in soils.

Each chapter presents the course accompanied by a series of solved exercises proposed for the students. Note that the availability of the handout should in no way discourage the student from attending the oral course, because a written document can never replace the masterful learning of the teacher.

Students who wish to deepen their knowledge in this module can always consult the documents and works in the list of references, proposed at the end of this document. Finally, since this is the first version of the document, I would be grateful to the reader for his corrections of the writing, his remarks, as well as his suggestions.

Chapter 1
Introduction to soil mechanics

Chapter 1: Introduction to soil mechanics

1.1 Introduction

Soil mechanics is the branch of engineering that studies the behavior of soils under different loading and environmental conditions. However, soil is considered to be the weathered material in the upper layers of the earth's crust. The unweathered material in this crust is referred to as rock and its mechanics is the discipline of rock mechanics.

Soil mechanics has become a distinct and separate branch of engineering mechanics, because soils have several properties that distinguish them from other engineering materials.

Its development has also been stimulated, of course, by the wide range of application of soil engineering in the sectors of Civil Engineering, Public Works and Hydraulics, since all structures require a solid foundation and must transfer their loads to the ground.

The most important properties of soils will be briefly described in this chapter. In the following chapters, they will be treated in more detail, focusing on quantitative analysis methods.

1.2 Purpose of soil mechanics (history and field of application)

Soil mechanics is the study of the mechanical, physical and hydraulic properties of soils, with a view to their application to construction. It studies in particular the behavior of soils in terms of their resistance and deformability aspects.

From laboratory and in situ tests, soil mechanics provides builders with the data necessary to study civil engineering, public works and hydraulic works and ensure their stability according to the soils on which they must be founded, or with which they will be built; this both during the progress of the works, and after completion and commissioning of the constructions.

1.3 History of soil mechanics

Since ancient times, man has used soil as a building material. However, it is generally accepted that the science of soil mechanics began with the work of Terzaghi in the early 20th century. The history of foundation works and soil mechanics from antiquity to the year 1700 is described by Kérisel (1985), while the history of foundation works between 1700 and 1927 is summarized by Skempton (1985) and finally the history of modern soil mechanics, between 1927 and 1985 is summarized by Peck (1985) and Briaud (2013).

The evolution of soil mechanics and the development of its major theories can be followed in the table1.1.

Table 1. 1.History of soil mechanics

Period	**Author**	**Theory**
18ème siècle	Coulomb	Shear strength
19ème siècle	Collin	Breakage in clay slopes
	Darcy	Water flow inside sand
	Rankine	Earth pressure on retaining walls
	Gregory	Horizontal drainage, compacted embankment with buttress to stabilize the slope of railway trenches
20ème siècle	Atterberg	Consistency limits of clay
	Terzaghi	First modern textbook of soil mechanics
	Casagrande	Liquidity Limit Tests

1.4 Areas of application

The fields of application of soil mechanics are numerous and varied. They concern, in general, the professions:

public works,

of the building.

hydraulics

In fact, the field of application of soil mechanics includes:

- natural environments such as slopes (landslide problems) and the banks of watercourses or reservoirs.

- soil works, where soil is the basic material, are also:

- embankments (roads, railways, dams, earth basin dikes, platforms maritime….) ; the debris (slopes, canals, basins, etc.).

- Mixed structures, where the soil is involved in relation to another material, such as concrete or steel. The anchoring conditions in the soil are often essential for structures such as:

- retaining walls (concrete, reinforced earth, geotextile reinforced soil, etc.);

- sheet piles used in canals, ports, urban constructions, etc.;

- diaphragm walls (waterproofing or support function)

- the foundations of works or buildings, In the study of foundations, the soil and the structure do not constitute a mixed whole, but two sets whose interactions must be understood. Soil mechanics ditinguish:

-surface foundations (bases or rafts);

-deep foundations (piles, wells, bars).

We can thus understand the importance of geotechnics, which aims to study the mechanical behavior of the soil, regardless of the practical conditions of use. Soil recognition will allow the engineer or technician to specify the possible use or not of a soil for a specific structure.

1.5 Disciplines of soil mechanics

Soil mechanics is the science that brings together all the knowledge and techniques that enable objectives to be achieved in several disciplines, including those using:

1.5.1 Terrain geology

The study of the geology of the terrain is of great importance. Indeed, it allows us to identify the different layers of the soil, their thicknesses and their dips, as well as the possible presence of underground water tables. On the other hand, the geological study of the layers present gives qualitative descriptions of the soil, answers some questions related to the history of the deposit and allows to guide the preliminary research.

1.5.2 Physicochemical characteristics

The study of these characteristics has shown great utility for prediction or interpretation soil behavior. The majority of these properties are determined by laboratory or in situ tests.

1.5.3 Hydraulic study

In the presence of water, the study of the permeability of the different layers of the soil is essential to estimate the resistance of the soil in the most unfavourable conditions and the risk of possible landslides. Determining the groundwater level and studying the flow regime allows the choice of pumping and drainage equipment. Determining the chemical nature of the groundwater also allows the method of sealing buried structures to be predicted.

1.5.4 Mechanical characteristics

The analysis of the mechanical behavior of soils is based on the conclusions of the previous disciplines, as well as on laboratory or in situ tests. This discipline makes it possible to determine the resistance of the soil and its bearing capacity and consequently the choice of the foundation method and the dimensions of the buried elements. Finally, this discipline makes it possible to quantitatively predict the deformation or settlement of the soil under the load of the structure.

1.6 Definition of soils

Soils are natural materials formed by the **weathering and decomposition of rocks** under the action of physical, chemical, and mechanical processes over time. They constitute a **complex and heterogeneous system** that plays a fundamental role in life on Earth. Soils act as a **support for structures and vegetation**, a **filter for water**, and a **habitat for living organisms**.

Soils are characterized by three main types of properties:

- **Physical properties**: such as grain size distribution (texture), structure, density, and porosity, which influence their mechanical behavior.
- **Chemical properties**: related to the mineral composition and the interactions between particles and fluids.
- **Biological properties**: due to the presence of microorganisms, plant roots, and soil fauna, which contribute to soil evolution.

From a geotechnical point of view, soils include all materials located at the surface of the Earth's crust, such as:

- **Granular soils** (e.g., sands and gravels),
- **Cohesive soils** (e.g., clays),
- **Organic soils** (e.g., peat).

1.6.1 Soil composition

A soil is composed of:
- distinct solid grains of different sizes derived from rocks,
- water,
- air.

Water and air occupy the pores, improperly referred to as voids. So, paradoxically, voids contain water and air. So, a natural soil in place is made up of solid grains bathed in :
- either water ;
- or air ;

- or a combination of these two fluids.

It is therefore, in the most general case, a complex of three phases:

-the solid phase, composed of grains which together constitute the skeleton of the soil;

-the liquid phase, representing water partially filling the voids existing between the particles;

-the gaseous phase, consisting of a mixture of air and water vapor, which occupies the remaining voids.

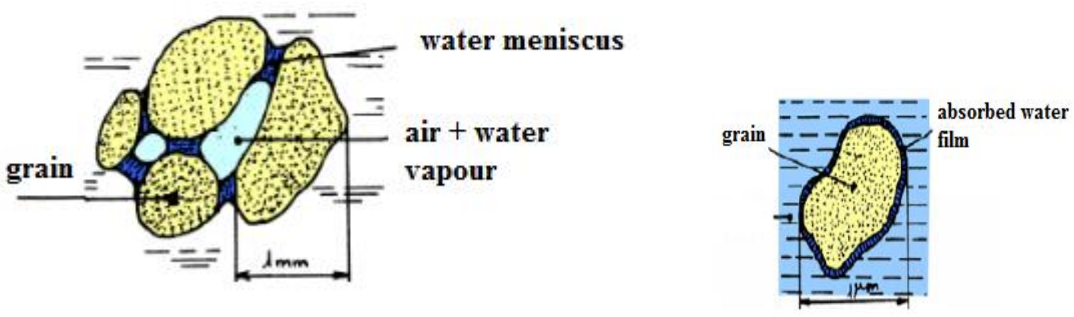


Figure 1. 1. Origin of soil.

1.6.2 Origin and formation of soils

Soil is an assembly of mineral or organic particles of different sizes and shapes. It mainly results from the alteration of rocks under chemical, physical, and mechanical processes, as well as from the decomposition of living organisms.

Soil formation is an incomplete stage in the formation of sedimentary rocks, involving erosion, water action, temperature variations, and transport by natural agents. Two main origins are distinguished: residual soils formed in place and transported soils deposited after transport, the latter generally presenting more complex geotechnical problems.

Soil diversity depends on several factors such as the parent rock, climate, vegetation, relief, and human activity. Each construction site therefore has different physical and mechanical characteristics, which explains the importance of geotechnical investigations.

The most common soil types are sands and gravels (permeable), clays (cohesive), marls (mixture of clay and carbonates), silts (intermediate grain size), peat (organic soils), and muds (highly compressible soils).

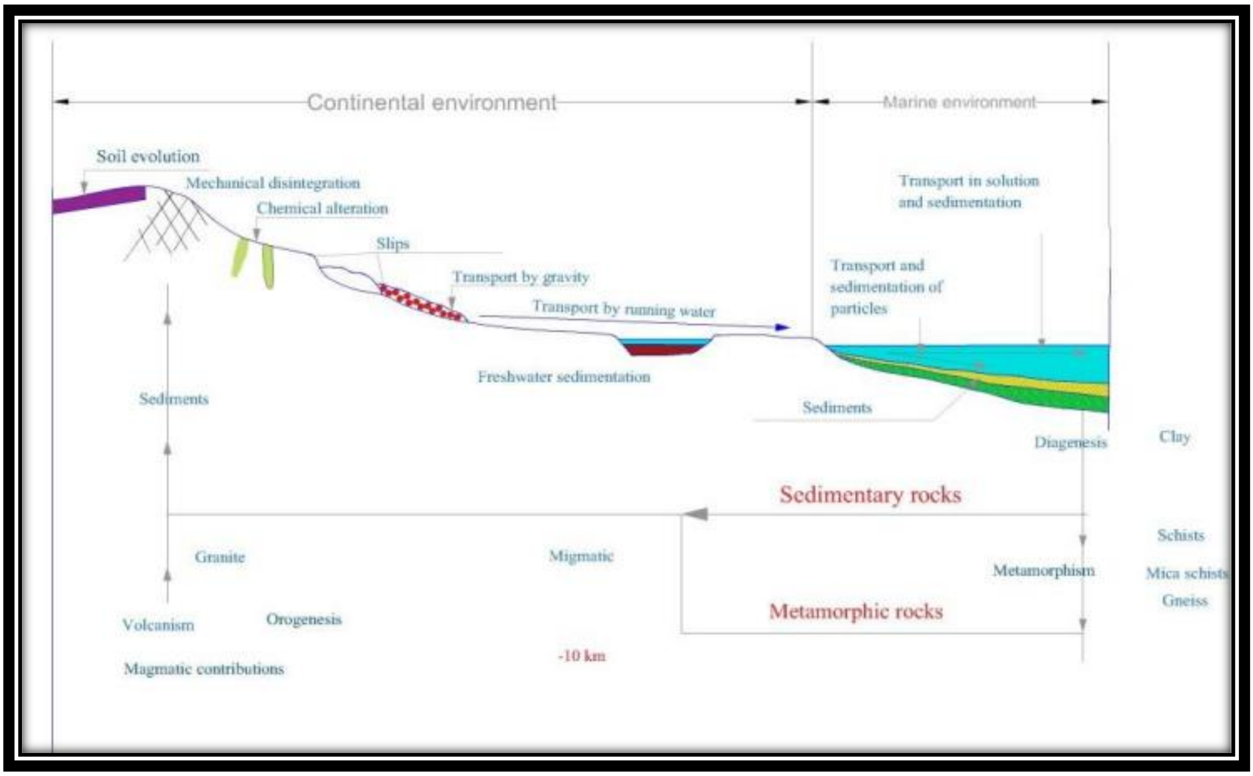


Figure 1. 2. Origin and formation of soils.

1.7 Soil structure

The structure of a soil reflects the size and the way in which the earth particles are arranged in relation to each other. They can assemble into aggregates, of variable sizes and shapes (Figure 1.3).

The structural stability of a soil expresses the greater cohesion of assembly of the particles in the aggregates.

Soil is a material made up of particles. The dimensions of these particles can be uniform or varied, ranging from 10 cm pebbles to fine particles of less than a micron. In addition to grain size, particles have other characteristics, such as shape, texture and elementary structure.

Soil structure has a major influence on soil behavior. A well-structured soil has many living spaces, storage spaces and passages (for use by water, gases, nutrients, roots and a wide range of organisms). A poorly structured soil is much less endowed and much less productive.

In practice, there are two main categories of soil which have quite different properties, namely coarse soils and fine soils.

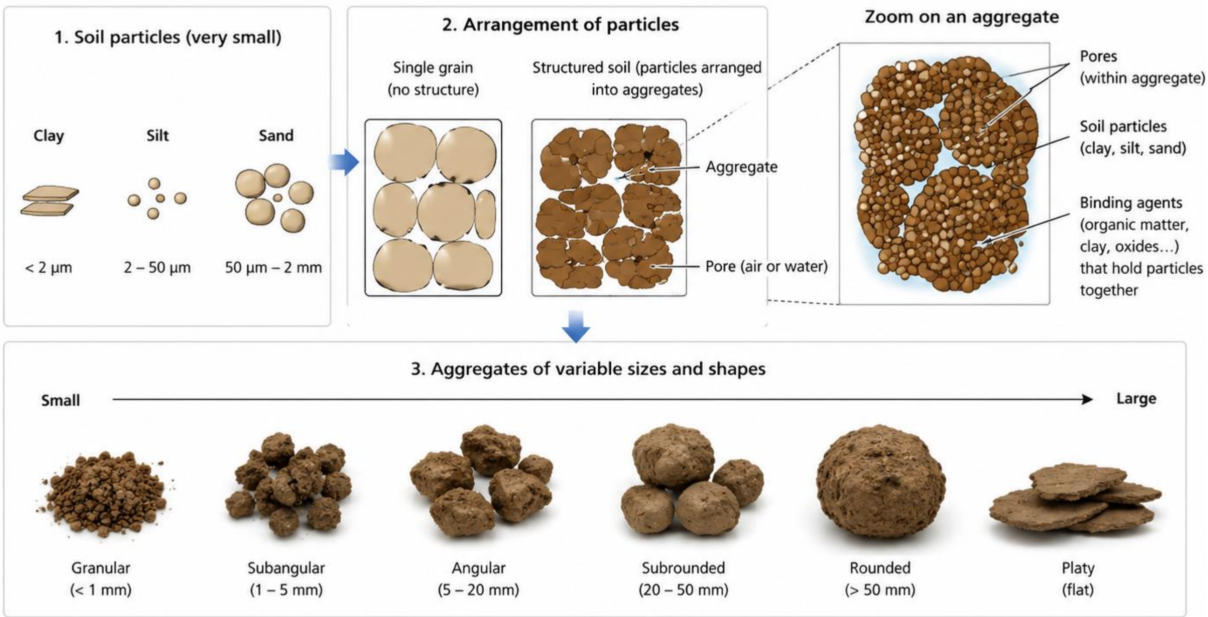


Figure 1. 3. Soil Structure: Arrangement of Particles and Formation of Aggregates**.**

1.7.1 Coarse soils

Coarse soils are those for which the geotechnical characteristics are determined by forces of volume or gravity. They are generally powdery. They are mainly defined granulometrically. Powdery soils (sand, gravel, pebbles and blocks) are mainly composed of silica (quartz), limestone, or other rocks or inert materials.

The particles are relatively large (dimensions greater than 80 μm),are visible to the eye bare, hence the common name of grainy soil. In this type of soil, the predominant forces between the grains are those of weight and the behavior of this soil depends essentially on its state of density.

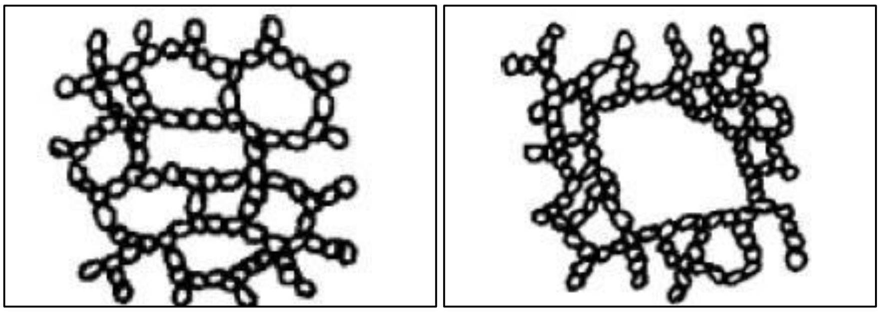
1.7.2 Fine soils

Fine soils are those that have a grain size d < $2\mu m$. The particles remain stuck together to each other. This soil is characterized by the presence of a strong cohesion between the particles, where it has the appearance of a solid and does not disintegrate under the effect of gravity. The particles are formed by a stack of sheets that have the shape of platelets. This category of particles includes, among others, clay, marl, mud, peat and silt. The fine soil is not very permeable and its behavior changes over time under the effect of overload (consolidation phenomenon).

In soil mechanics, clays are defined as soils with an average diameter of less than 2µm. A fine or clayey texture corresponds to a chemically rich, impermeable and poorly aerated soil. It is therefore a subgroup of fine soils.

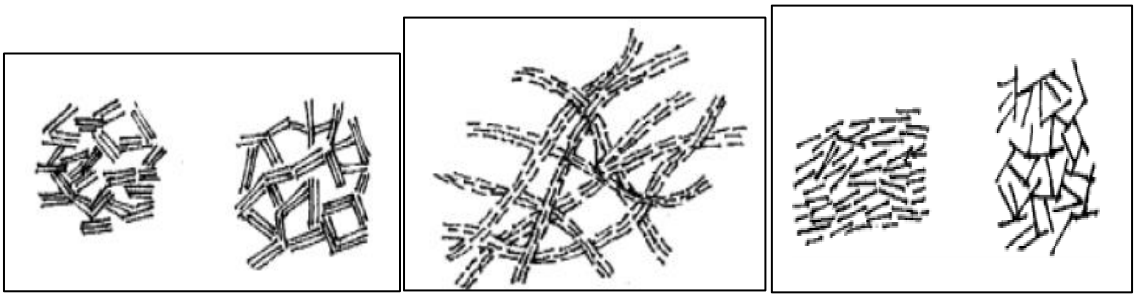
Clays come from the decomposition and chemical alteration of rocks, particularly silicate minerals, such as feldspars, mica, pyroxenes, etc. The most common types of clays are: kaolinite, montmorillonite and illite.

In clays, honeycomb and flocculent structures can be found which are less resistant (Fig 1.4) as they can arrange themselves in several ways (Fig 1.5).



honeycomb structure flaky structure

Figure 1. 4.Arrangement of fine-grained soils.*[Benbakheti, 2023]*



Grouping arrangement of clay platelets *Tangle of clay clusters* *Clay plate arrangement*

Figure 1. 5. Different arrangements of clay platelets**.**

1.8 Conclusion

In this chapter, we have presented a brief history of soil mechanics with its different fields and disciplines. Then we discussed the origin and formation of soils, as well as their structure.

Chapter 2
Identification and classification of soils

Chapter 2: Identification and classification of soils

2 Introduction

Soil classification consists of studying the soil according to its characteristics physical, chemical, mineralogical and geotechnical. Whatever the intended use of a soil, it is important to know its nature, composition and the distribution of grains of different sizes that compose it. These properties are determined by simple and rapid tests, called "*identification tests*".

We classically distinguish two main categories of identification tests:

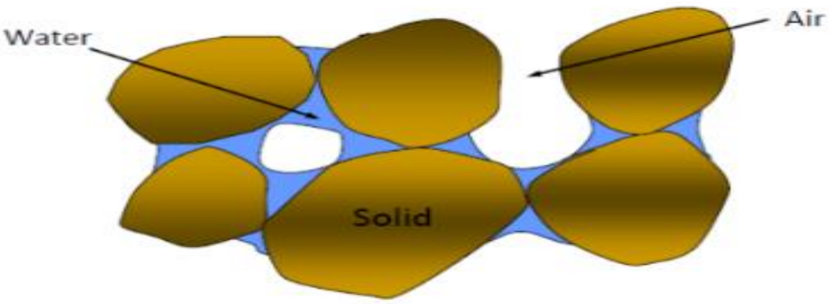
- tests that respond to the arrangement and distribution of phases (solid skeleton, water, air). These tests characterize the *soil condition* and can only be carried out on intact samples;
- tests that reflect the properties of soil particles and the intensity of their bonds with water. These tests characterize the nature of the soil and are carried out on intact or reworked samples.

2.1 Physical characteristics

The soil is mainly composed of pebbles, sand, silt, clay, limestone, humus, water and a mixture of various gases.

It is generally made up of three phases: solid, liquid and gas (Fig 2.1).

- The solid phase is a phase insoluble in water, which is divided into two parts, mineral and organic. The solid phase is characterized by the description of its elementary particles (dimensions, shapes, surface states, chemical and mineralogical natures) and their arrangement.
- The liquid phase is made up of water. This is mainly due to the nature of the ions present.
- The gas phase is generally air, but can be of a different nature under certain conditions.



A three – phase material

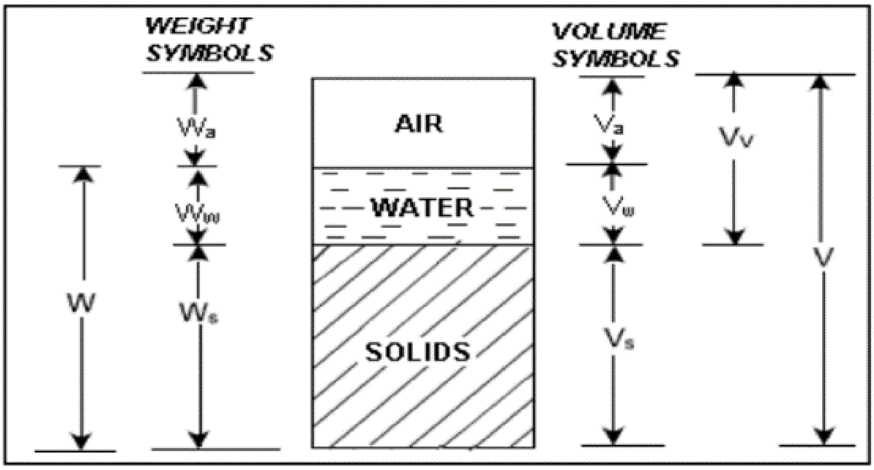


Figure 2. 1. Elementary volume of a soil – weight and volumes of the different phases.

Total volume V is the sum of solid volume V_s and void volume V_v :

$$V = V_s + V_v$$

The volume of the voids can be occupied either by water V_w (*w* = water) and/or air V_a :

$$V_v = V_w + V_a$$

$$\text{So : } V = V_s + V_v = V_s + V_w + V_a$$

In the same way, we define the weight W (or mass M) of a soil sample as the sum of the weight of the solid W_s and the weight of the water W_w (the weight of the air is considered to be zero).

$$W = W_s + W_w$$

$$W_a = 0$$

From the different volumes and masses, we define the different volumetric weights (or volumetric masses or densities).

Conventional notations:

W : total weight of soil

W_s : weight of solid particles

W_w : weight of water

V : Total volume (apparent)

V_s : volume of solid particles

V_v : volume of voids between particles

V_w : volume of water

V_a : volume of air

2.2 Dimensional parameters (volumetric weight)

2.2.1 Density (wet density, apparent density)

It is the weight of material (liquid phase + solid phase) contained in a unit of volume sample. This is the "natural" or "in place" specific weight of the material. It is presented by the symbol or better by γh (h= humid):

$$\gamma_h = \frac{W}{V} = \frac{W_s + W_w + W_a}{V_s + V_w + V_a} \frac{\text{KN}}{m^3}$$

2.2.2 Specific weight of grains (volumetric weight of solid grains)

It is the weight of the material constituting the solid skeleton of the soil in a unit of volume. represents it by the symbol γs, which is defined by:

$$\gamma_s = \frac{W_s}{V_s}$$

Sand and clay = 25 to 28 KN/m^3

2.2.3 Specific weight of water (volumetric weight of water)

$$\gamma_W = \frac{W_W}{V_W} \quad \left(= \frac{10\text{KN}}{m^3}\right)$$

2.2.4 Planed specific weight (planed volumetric weight)

When the ground is completely submerged (for example when it is located below the level of a groundwater), it is appropriate to separate the mechanical effects of water and submerged soil. It is represented by the symbol γ' ;it is equal to the specific weight of the soil considered, assumed to be saturated γ_{sat} , reduced by the specific weight of the interstitial water γ_W.

$$\gamma' = \gamma_{sat} - \gamma_W$$

NB:The use of volumetric weights instead of the density masses ρ, allows to avoid the introduction of the acceleration of gravity g ($\gamma=\rho g$).

We will sometimes encounter the notion of density of a soil in relation to water. The notation to be used will then be for e~~xa~~mple $\left(\frac{\gamma_d}{\gamma_W}\right)$ for dry density and $\left(\frac{\gamma_h}{\gamma_W}\right)$ ~~f~~or wet density.

2.3 Adimensional parameters

These are very important parameters and essentially *variables.* They characterize the state in which the soil is located, that is to say the state of compactness of the skeleton, as well as the quantities of water and air contained in the soil.

2.3.1 The void ratio e

The void ratio is the ratio of the volume of voids V_v to the volume of solid grains V_s expressed in %:

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

The void ratio of a soil can be greater than 1: $0.10 \leq e \leq 5$ (extreme case of Mexico clays: e > 13)

2.3.2 Porosity n

Porosity n is the ratio of the volume of voids relative to the total volume of the soil:

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

The porosity of a soil is always less than 1: $0 \leq n \leq 1$

Sand: n = 0.25 to 0.5, clay: n = 0.20 to 0.80.

2.3.3 Water content ω

Water content ω of a soil is the ratio of the weight of free water Ww by weight of solid grains Ws for a given volume of soil.

The water content is determined either in the laboratory or in situ by drying, microwave oven, heating, burning, or chemical reaction (Speedy device).

In this definition, it is agreed to take into account the water that evaporated after drying the soil in an oven at 105°C until the sample weight is constant. It is expressed as a percentage:

$$\omega = \frac{W_w}{W_s} \times 100 \qquad \%$$

The water content of a soil can exceed 100%.

This is an essential characteristic, water playing a determining role in the mechanical properties of a soil.

2.3.4 The degree of saturation Sr

The degree of saturation Sr is the ratio of the volume occupied by water (volume of water) Vw to total void volume Vv.

It indicates the amount of water contained in the soil (to what extent the voids are filled with water).

It is also expressed as a percentage:

$$S_r = \frac{V_w}{V_V} \times 100 \quad \%$$

- When the soil is dry Sr = 0.
- When the soil is saturated with water Sr = 1.

2.3.5 Relative density

The density index or relative density is defined as follows:

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

or : Id: relative density or Density Index. e_{min}: is the minimum voids index corresponding to the most compact state. e_{max}: is the maximum voids index corresponding to the loosest state. e: is the voids index of the soil in place.

It is determined by strictly standardized laboratory tests:

- For loose soil: Id is close to 0. For a tight.
- Or dense soil: Id is close to 1.

It should also be noted that in the case of sands, we always have: $0.40 \leq Id \leq 1$

The density index quantifies the state of compactness of a gritty soil (dense or loose soil).

The density index is used to quantify the state of compactness of a granular soil (dense soil or loose soil).

2.4 Relationship between parameters

The parameters defined above are not independent. We would need to calculate some of them from the measurement of the other parameters. The most important and commonly used relationships are given as follows:

$$n = \frac{e}{1+e} \qquad (1)$$

$$e = \frac{n}{1-n} \qquad (2)$$

Water content is easily measurable in the laboratory, whereas determining the degree of saturation requires knowledge of the void ratio on the one hand and the density of the solid constituent on the other. The relationship between these quantities is defined as follows:

$$\omega = \frac{e \cdot S_r \cdot \gamma_w}{\gamma_S} \qquad (3)$$

For saturated soils, equation (3) can be written in the following form, which is very useful in many soil mechanics problems:

$$e = \omega \cdot \frac{\gamma_s}{\gamma_w} \qquad (4)$$

These relationships allow us to give the expression of the volumetric weight and the dry volumetric weight, neglecting the weight of the gas phase:

$$\gamma_h = \gamma_s(1-n) + S_r \cdot n \cdot \gamma_w \qquad (5)$$

$$\gamma_d = \gamma_s(1-n) \qquad (6)$$

Or

$$\gamma_d = \frac{\gamma_s}{1+e} \qquad (7)$$

Therefore, for saturated soils only, it is possible to rewrite expression (5) as :

$$\gamma_{sat} = \gamma_d + n \cdot \gamma_w \qquad (8)$$

For unsaturated soils, by eliminating e in equations (3) and (7), we obtain:

$$S_r = \frac{\omega}{\frac{\gamma_w}{\gamma_d} - \frac{\gamma_w}{\gamma_s}} \qquad (9)$$

We will also point out the expression useful in practice:

$$\gamma_h = \gamma_s \frac{1+\omega}{1+e} \qquad (10)$$

Table 2.1 brings together the relationships cited above:

Table 2. 1. Relationships between parameters.

Parameters	Definitions	N	E	γ	γ_d
Water content ω (%)	$\omega = \frac{W_w}{W_s}$	$\omega = \frac{n \cdot S_r \cdot \gamma_w}{(1-n)\cdot\gamma_s}$	$\omega = \frac{e \cdot S_r \cdot \gamma_w}{\gamma_s}$	$\omega = \frac{\gamma}{\gamma_d} - 1$	$\omega = \frac{\gamma}{\gamma_d} - 1$
Porosity n	$n = \frac{V_v}{V} = \frac{V_a + V_w}{V}$	—	$n = \frac{e}{1+e}$	$n = 1 - \frac{\gamma}{(1+\omega)\cdot\gamma_s}$	$n = 1 - \frac{\gamma_d}{\gamma_s}$
Void index e	$e = \frac{V_v}{V_s} = \frac{V_a + V_w}{V_s} = \frac{V - V_s}{V_s}$	$e = \frac{n}{1-n}$	—	$e = \gamma_s \cdot \frac{(1+\omega)}{\gamma} - 1$	$e = \frac{\gamma_s}{\gamma_d} - 1$
Apparent unit weight γ (KN/m^3)	$\gamma = \frac{W}{V} = \frac{W_s + W_w}{V_s + V_w + V_a}$	$\gamma = (1-n)\cdot(1+\omega)\cdot\gamma_s$	$\gamma = \frac{1+\omega}{1+e}\cdot\gamma_s$	—	$\gamma = (1+\omega)\cdot\gamma_d$
Weight volumetric apparent dry γ_d (KN/m^3)	$\gamma_d = \frac{W_s}{V} = \frac{W_s}{V_s + V_w + V_a}$	$\gamma_d = \gamma_s \cdot (1-n)$	$\gamma_d = \frac{\gamma_s}{1+e}$	$\gamma_d = \frac{\gamma}{1+\omega}$	—
Weight volumetric of the grains γ_s (KN/m^3)	$\gamma_s = \frac{W_s}{V_s}$	$\gamma_s = \frac{\gamma}{(1-n)\cdot(1+\omega)}$	$\gamma_s = (1+e)\cdot\gamma_d$	$\gamma_s = \frac{\gamma}{(1-n)\cdot(1+\omega)}$	$\gamma_s = \frac{\gamma_d}{(1-n)}$

2.5 Density by hydrostatic weighing

Hydrostatic weighing is a method used to determine the density of solids and liquids. It is based on the principle of Archimedes' thrust.

The principle of this method is to weigh the soil sample in water. We know that the mass of a solid immersed in water is equal to its mass minus the mass of the displaced water by the immersion of this last “principle of Archimedes’ thrust”.

The steps followed for the determination of density by hydrostatic weighing are as follows:

1. Take a soil sample and weigh it
2. Turn on the resistance to melt the paraffin.
3. Coat the sample with paraffin (Care should be taken to ensure that the entire sample is covered with paraffin).

4. Weigh the paraffin sample.
5. Check that the scale is balanced (the weight of the basket must be balanced before weighing in water).
6. Place the paraffin sample in the basket provided for weighing.
7. Weigh the waxed sample in water

$$\rho = \frac{m}{\dfrac{m_p - m_p^{'}}{\rho_w} - \dfrac{m_p - m}{\rho_p}}$$

where :

m : the sample mass in open air;

m_p : the mass of the paraffin sample in open air (sample + paraffin)

$m_p^{'}$: the mass of paraffin sample immersed in water (sample + paraffin – water moved)

ρ_p : kerosene density = 880 to 900 kg/m^3 ;

ρ_w :the density of water-= 1000 kg/m^3

Figure 2. 2. Determination of density by hydrostatic weighing.

2.6 Granulometric characteristics
2.6.1 Granulometric analysis
Granulometric analysis makes it possible to determine the distribution of the constituent particles of a soil by size class, by measuring by weighing the relative importance of these classes of grains of well-defined dimensions. These different classes have the names indicated in Table 2.2: pebbles (large elements), gravel, sand, silt, clay.
Granularity is further defined as the distribution of the average particle size of the soil, expressed as a percentage of the total mass of the material.

Table 2. 2. Classification of Soil Particles According to Their Average Particle Diameter.

Average Particle Diameter	**> 200 mm**	**20–200 mm**	**2–20 mm**	**0.2–2 mm**	**0.02–0.2 mm**	**2–20 µm**	**< 2 µm**
Classification	Pebbles	Gravel	Coarse Sand	Fine Sand	Silt	Fine Silt	Clay

Two laboratory tests can be used to establish the granulometry of soils (Fig 2.3):
- granulometric analysis by sieving;
- granulometric analysis by sedimentation.

2.6.1.1 Granulometric analysis by sieving NF P 94-056
When the particles are larger than 80 µm, the laboratory operation is a simple sieving. The particle size analysis is one of the simplest laboratory tests to perform (Fig 2.3). The first step is always to pass the sample through an oven at 105°C until its weight has become constant. The purpose of this operation is to determine the exact weight of the sample. The sieving itself is then carried out: either,*dry*; for materials with low cohesion such as sand and gravel ; or underwater, when dealing with silty or clayey soil .
The soil must then be soaked for a sufficient time to break up the clods and agglomerates of earth and stones. This process can take a few minutes to several hours.
A bottom is placed at the bottom of the sieve column, which is superimposed from top to bottom in decreasing order of opening. Then, a representative sample of soil is passed through this sieve column, starting by filling the top one (Fig 2.3) (which is closed with a flat lid once filled, to avoid the loss of particles during agitation).

The larger particles therefore remain trapped on the higher sieves, while the finer particles move towards the lower sieve.

Figure 2. 3. Apparatus for particle size analysis by sieving.

Figure 2. 4. Different phases of granulometric analysis by sieving.

The weight W of the sample subjected to the test is a function of the maximum dimension D of the largest elements; we usually take:

$$200 \cdot D < W < 500 \cdot D$$

This weight limitation aims to:

- on the one hand, to carry out the test on a sufficiently large and representative fraction from the ground;
- on the other hand, to carry out successive sieving with a limited quantity of elements in each of the sieves used.

The sieve column is then subjected to mechanical or manual agitation for 10 minutes or until the masses retained on each sieve become constant. It is subjected to horizontal and vertical movements so as to leave the soil sample constantly in contact with the surface of the sieves and thus increase the efficiency of the sieving. These movements must be rapid, 120 jolts per minute and with an average amplitude of 70 mm. The quantity of material retained by a sieve is called the *refusal* or the *retained*, while the amount of material that passes through a sieve is called the *sieved* or the *passing by*.

Once the sieving is finished, the full sieves are removed, one by one, starting with the largest one, and then the contents are transferred to a large container that has been previously tared. All this, carefully emptying the sieve without losing any material, after stirring its contents with a brush and without forcing any element to pass through one of the sieves. At the end, the weighings are carried out.

The weights of the sieves are compared to the total weight of the sample and calculated as a percentage of this weight.

By designating by:

- Rj rejects;
- Ti sieves.

At the nth sieve of a sieving column :

$$T_{n-1} = R_n + T_n$$

Hence, the sieve of the nth sieve is :

$$T_n = T_{n-1} - R_n$$

Finally, we express the *particle size distribution of a soil*, using the size of the openings of each sieve and the percentage of passage of each. Thus, instead of indicating the quantity of particles having a certain diameter, the particle size gives the proportion of particles, whose diameter is less than the size of the openings of each sieve, that is to say the proportion of particles which have passed through each sieve.

The results are plotted in a semi-logarithmic graph, where they construct a *particle size curve* (Fig 2.5).

This form of presentation allows you to visualize the grain size and compare soils.

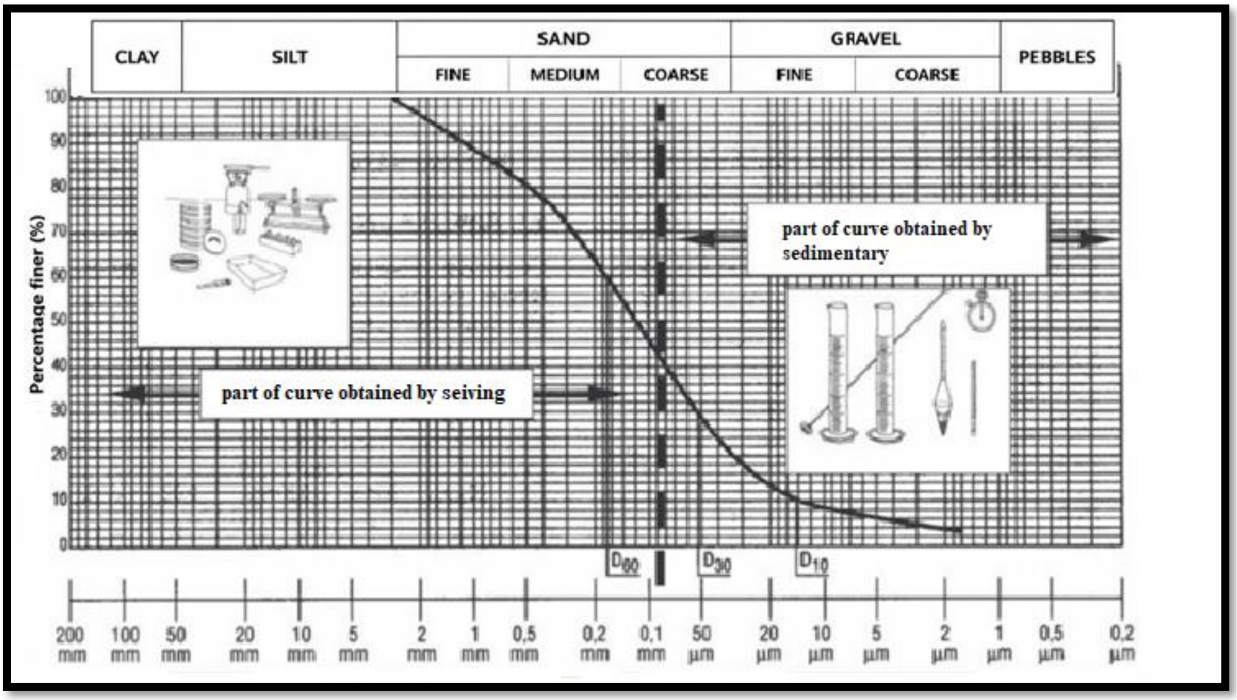


Figure 2. 5. Grain size curve**.**

2.6.1.2 Granulometric analysis by sedimentometry NF P 94-057

When the particle size is less than 80 µm, sieving is no longer possible, because below 63 µm, the sieves become clogged and deformed and therefore the use of the sieves is physically impossible. We then resort to the *sedimentometry*.

It is physically impossible to perform a sieving granulometric analysis for particles with an equivalent diameter of less than 0.08 mm. In such cases, a sedimentation granulometric analysis is used instead to estimate the granulometric distribution of silt particles, particularly clay particles. This test is based on the relationships (established by the English physicist GG Stokes in 1850) between the speed of fall of a sphere in a liquid, the diameter of the sphere, the relative density of the sphere and the liquid, and the viscosity of the liquid.

These relationships show that the speed of fall of a sphere is directly proportional to the square of its diameter (the larger the sphere, the higher its speed of fall).

Sedimentometry is a test that complements the granulometric analysis by dry sieving of soils. It is a test of decantation by gravity of the grains of a sample suspended in a viscous liquid. It applies to an element with a diameter of less than 80 µm.

The principle of the test is based on the fact that in a liquid medium at rest, the settling speed of fine to very fine grains is a function of their size. Stokes' law gives, in the case of spherical grains of the same density, the relationship between the diameter of the grains and their sedimentation speed. By convention, this law is applied to the elements of a soil to determine the equivalent particle diameters.

The particles passing through the 80 μm sieve during the sieving granulometric analysis are collected and suspended in water with a deflocculant (product to separate the particles and prevent them from sticking together) added. The particles sediment at different speeds in relation to their size. Using a densimeter immersed in the solution, the change in the density of the latter is measured as a function of time and immersion depth. The weight distribution of the particle size is calculated from these data.

Figure 2. 6. Granulometric analysis by sedimentometry.

Stockes' law expresses the relationship between the settling speed and the diameter of a spherical particle. It is written in the following form:

$$V = \frac{\gamma_s - \gamma_w}{18\mu} d^2$$

Where:

- V: Settling velocity of the particle.
- d: Diameter of the particle.
- γs: Specific weight of the particle.
- γw: Unit weight of the liquid used.
- μ: Dynamic viscosity of the liquid.

The test specimen is taken from the fraction of the soil sample, passing through the 80-m sieve, collected in the bottom arranged at the bottom of the sieve column, during the granulometric analysis by sieving.

After steaming until completely dry, the dry material is disintegrated and mixed well, in order to obtain a homogeneous mixture.

The approximate dry mass of the sample with which the granulometric analysis by sedimentometry is carried out is between 40 g and 100 g.

When the sample is ready, it is incorporated into a deflocculating solution. The finer grains normally tend to flocculate in the suspension, that is, to adhere to each other and settle together.

So, to prevent flocculation of grains, a dispersing agent is added to all samples. An amount of 125 cm3of a deflocculating solution of sodium hexametaphosphate dosed at 40 g/L, i.e. 5 g of sodium hexametaphosphate in 125 cm3of distilled or demineralized water, is usually sufficient to disperse most soils.

After the prepared dry sample is placed in a container, distilled or demineralized water is added to it until it is submerged and then the 125 cm3deflocculating solution is added.

In order to achieve complete saturation allowing the dispersing agent to act under ideal conditions, the soil sample is left to soak overnight (24 hours) or until all clods have disintegrated.

After the addition of the dispersing agent and the calibration of the settling cylinder and the densimeter, the actual test follows the following steps:

1.- Determine the correction factors for the dispersing agent Cdand meniscus Cm.

2.- Measuring the specific gravity of grains Gs(-s).

3.- Transfer the soaked sample from the evaporation bowl to the stirrer bowl, washingany residue from the bowl with distilled or demineralized water.

Then, add the distilled water to the bowl of the appliance up to 5 to 8 cm below the upper edge of the bowl, so that it does not overflow when mixing (Fig 2.6). Place the bowl in the agitator and disperse the solution for 1 to 10 minutes.

4.-Pour all the suspension thus obtained into a one-litre sedimentation cylinder and add, if necessary, distilled or demineralized water up to 1000 mL.

5.-About 1 minute before the start of the test, take the cylinder with one hand and cap it with the palm of the other hand or a suitable stopper, then shake the suspension vigorously for a few seconds by inverting it from bottom to top several times, in order to mix the sediment from the bottom with the uniform suspension. Sometimes, to dislodge the sediment from the bottom of the cylinder, it is necessary to use a hand stirrer. Maintain a uniform suspension until the start of the test.

6.-Place the cylinder in a stable, vibration-free location, if possible in a bath. at constant temperature. Immerse the hydrometer gently into the suspension, 20 to 25 seconds before any reading. Insert and remove it very gently to avoid creating turbulence in the suspension.

7.-The stopwatch being started at the end of agitation, at the exact moment when the cylinder was deposited, which defines the beginning of sedimentation, record the corresponding reading at the top of the meniscus formed by the suspension on the stem of the hydrometer, after 1 and 2 minutes. Immediately after recording the 2-minute reading, carefully remove the hydrometer from the suspension and insert it into clean water. If the hydrometer is left in the soil suspension for a long time, the material will cling or adhere to the bulb, causing significant errors in the reading. Reinsert the hydrometer into the suspension and record the readings after 4, 15, 30, 60, 120, 240, and 1240 minutes.

8.-At the end of 2 minutes and after each reading, record the water temperature. The Changes in the temperature of the suspension during testing affect the results. Temperature variations can be minimized by moving the suspension away from heat sources, such as radiators, sunlight, or open windows.

The corrected reading of the densimeter is given by:

$$R' = R_t + C_m$$

Or

- R_t : Hydrometer reading at time ttt.
- Cm : Meniscus correction factor.

The drop height is calculated by linear interpolation of the corrected reading R'. For example, when 20≤R′≤30 g / ,H corresponding to R' is:

$$H = h_{20} + (h_{30} - h_{20}) \cdot \frac{R' - 20}{30 - 20}$$

Such that:

- h20 and h30 correspond to the graduations 20 and 30 g/L, respectively, as defined during the hydrometer calibration.

The corrected fall height HRis defined by taking into account the rise in water level when the hydrometer is submerged, i.e.:

$$H_R = H - \frac{V_b}{2A}$$

Or

- Vb Volume of the hydrometer bulb.
- A: Cross-sectional area of the sedimentation cylinder.

$$A = \frac{\pi}{4} \cdot D_C^2 \qquad D_C - \text{cylinder diameter}$$

According to the Stokes equation, the particle diameter D (mm) is:

$$d = \sqrt{\frac{18 \cdot \eta}{\gamma_s - \gamma_w} \cdot \frac{H_R}{t}}$$

with:

t: Time after the start of sedimentation (min).

γ_s: The volumetric weight of solid soil grains (g/cm3).

γ_w: The specific weight of water at temperature T (g/cm3).

η: The viscosity of water at water temperature (g/cm.s).

H_R: The corrected fall height (cm).

The percentage p by weight of particles with a diameter less than D corresponding to R' is given by:

$$p = \frac{0{,}6226}{W_0} \cdot (R' - C_d + m) \cdot \frac{\gamma_s}{\gamma_s - \gamma_w} \cdot 100 \qquad (\%)$$

Or :

W0: being the dry weight of soil per liter of suspension (g/L).

Cd: being the correction factor for the dispersing agent (g/L).

m: being the correction factor determined by the equation:

$$m = 1000 \cdot [0{,}99823 - \rho_w - 0{,}000025 \cdot (T - 20)]$$

Or :

T - being the water temperature (°C).

ρ_w - being the density of water (g/cm^3) at temperature T.

2.6.1.3 Granulometric curve

The granularity is expressed by the granulometric curve (called cumulative, of distribution of grains) which is the curve of the cumulative sieves (in %) as a function of the sieve dimensions (semi-logarithmic scale), obtained from the results of the sieving and sedimentometry tests (Fig 2.5).

It should be noted that a grain size curve allows us to identify the types of soils that make up the sample being analyzed. Looking at the grain size curve in Figure 2.7, we can see that the sample is made up of gravel, sand, silt and clay.

The more detailed curve shows the respective proportions of each soil type, expressed as a percentage.

Based on the granulometric analysis, two coefficients have been defined which allow a rapid approach to the geotechnical qualities of a soil (for example its permeability):

The Hazen *uniformity coefficient* ; The *coefficient of curvature*.

The particle size curve remains the most reliable way to represent particle size. The uniformity coefficient allows us to express the spread of the particle size curve. It is determined by the following ratio (Fig 2.5):

$$C_U = \frac{d_{60}}{d_{10}}$$

Or:

d_{60}: represents the opening of the sieve through which 60% of the grains pass.

d_{10}: represents the opening of the sieve through which 10% of the grains pass.

The diameter of d_{10} is called: *effective diameter*.

In general, five classes of granulometry are recognized:

Uniformity coefficient	**Granular class**
$C_u \leq 2$	very tight grain size
$2 < C_u \leq 5$	tight grain size
$5 < C_u \leq 200$	semi-spread nulometry
$20 < C_u \leq 200$	Granulometry
$200 < C_u$	very spread grain size

A uniformity coefficient of 1 indicates a soil composed of particles having the same equivalent diameter. The uniformity coefficient of beach sand, for example, normally varies from 2 to 3. That of clayey gravel can vary from 25 to 1000. Soils with a uniformity coefficient of less than 5 are often referred to as uniform.

The curvature coefficient allows us to write the shape of the granulometric curve between the diameters d_{10} and d_{60}. It is defined by the following equation (Fig. 2.5):

$$C_C = \frac{d_{30}^2}{d_{10} \cdot d_{60}}$$

Or

d_{30}: represents the opening of the sieve through which 30% of the grains pass.

When the value of the curvature coefficient is between 1 and 3 and the uniformity coefficient is greater than 4 for gravel and 6 for sand, the granulometric curve descends in a fairly regular manner, thus indicating the presence of a wide variety of diameters.

In such a case, the granulometry is said to be well graduated.

When the value of the curvature coefficient exceeds 3, the grain size curve begins to take the form of a downward-oriented trough; the higher the value, the greater the trough becomes more pronounced. If this value is less than 1, the trough of the curve tends to be upwards.

A curvature coefficient that is too large or too small indicates the absence of certain diameters between the diameters of d_{10} and d_{60}: the granulometry is said to be poorly graduated.

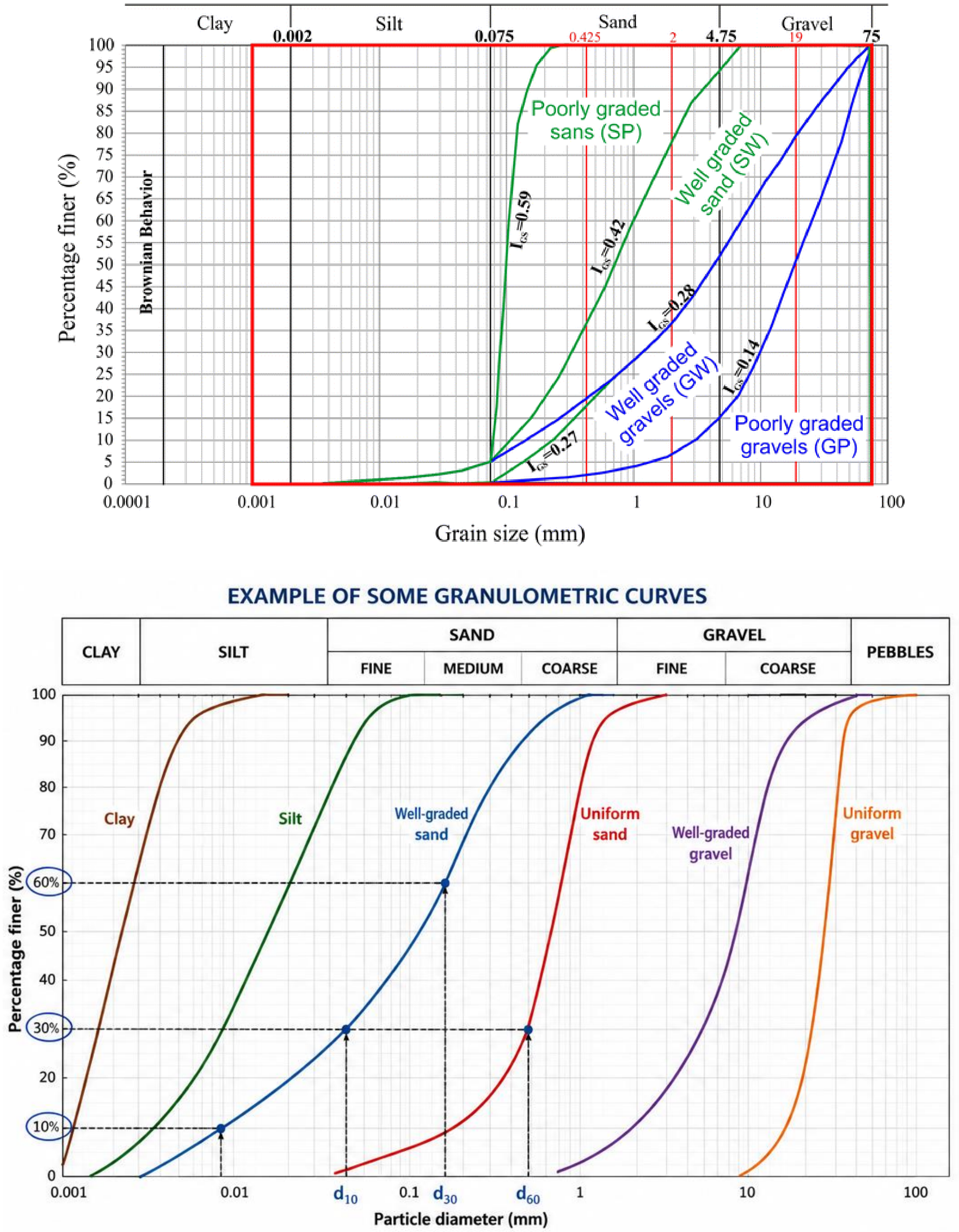


Figure 2. 7. Example of some granulometric curves.

2.7 Sand equivalent test (NF P 18-598)

The sand equivalent test is used to determine the proportion of clay-like fines present in a sand sample. It provides an assessment of the sand quality and, consequently, its degree of cleanliness.

The test consists of separating the sand particles from the fine clay and silt particles. In the presence of a flocculating solution, the fine particles remain suspended and accumulate above the sand layer in a graduated cylinder, allowing the relative proportions of sand and fines to be measured.

The mass of material required for the test is **120 g for dry sand** and **120(1 + w) g for wet sand**, where **w** is the water content expressed as a decimal.

2.7.1 Test procedure

Take a vertical graduated test tube and pour the washing solution up to the mark. lower. Then carefully pour one of the wet samples (120 (1+w) g) into the same test tube using a funnel. Repeatedly tap the base of the test tube firmly on the palm of your hand to expel air bubbles and encourage the sample to wet. Leave to stand for ten minutes.

At the end of this ten-minute period, stopper the test tube with the rubber stopper, then fix the test tube on the stirring machine and operate the machine. Subject the test tube to90 cycles ± 1 stirring cycle in 30 s ± 1 s.When finished, return the test piece to the upright position on the test table.Then remove the rubber stopper and rinse it above the test tube with the washing solution. By lowering the washing tube into the test tube, rinse the walls of the test tube with the washing solution and push the tube to the bottom of the test tube.

Bring up the clay elements, while keeping the test tube in a vertical position, by proceeding as follows: Lower the washing tube by rotating it between your fingers, which allows you to wash the inside walls of the test tube. When the liquid level reaches the upper reference mark, stop the flow and raise the washing tube (without dragging fine particles) so that the liquid level remains at the reference mark. Leave the contents of the test tube to stand for20 min ± 10 s. The sand sinks very quickly (heavy particles)

while the flocculated fines settle more slowly on top. The raw sand can thus be visibly separated from the fines.

A first visual measurement (Esv) is carried out using the graduated ruler by measuring the height h1 of the upper level of the flocculate relative to the bottom of the test tube. Also measure the height h'2 of the upper level of the sedimented part relative to the bottom of the test tube. A second measurement by piston (Es) is carried out by gently lowering the calibrated piston into the test tube until it rests on the sediment.

During this operation, the sliding sleeve rests on the test piece. When the base of the piston rests on the sediment, the sliding sleeve must be locked on the piston rod. Insert the ruler into the notch of the sleeve, bring the zero up against the lower face of the piston head. Read the height of sediment h_2 at the upper edge of the sleeve.

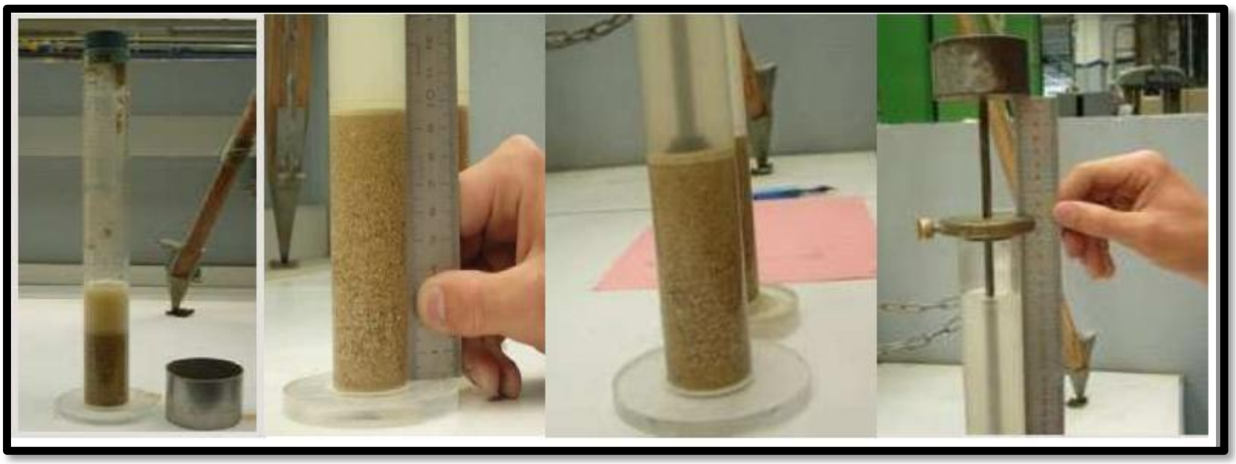


Figure 2. 8. Different stages of the sand equivalent test.

Two measures:

h1: height of sand + flocculate

h2: height of sediment (sand) using a piston.

The **sand equivalent (SE)** is defined as the ratio of the height of the sand layer to the total height of the sand and clay suspension, expressed as a percentage:

$$Es = \frac{h_s}{h_c} \times 100 \qquad \%$$

where:

- h_s = height of the sand layer;
- h_c= height of the clay suspension above the sand layer.

For the production of good-quality concrete, the sand equivalent should generally be: SE>70

and preferably: SE>80 to ensure a low content of clay and harmful fines.

Table 2. 3. Interpretation of Sand Equivalent Values.

Sand Equivalent (%)	Soil Quality
< 25	Clayey soil
25 – 50	Plastic silty or clayey soil
50 – 70	Slightly plastic soil
> 70	Clean sand
> 80	Very clean sand suitable for high-quality concrete

2.8 Methylene blue test NF P 94-068

The test consists of measuring by dosage the quantity of methylene blue adsorbed by the material suspended in water. This test (NF P 94-068) is an indirect measurement of the specific surface area of solid grains by absorption of a methylene blue solution until saturation. The principle of the test consists of constantly stirring a mixture [sample + water] and then introducing increasing quantities of methylene blue in successive doses, until the clay particles are saturated. An excess then appears which marks the end of the test and which is detected by the stain test. The latter consists of forming with a drop of the suspension on standardized filter paper, a stain which is a deposit of soil colored blue, surrounded by a colorless wet zone. The excess of blue results in the appearance in this zone of a light blue halo. The test is then positive.

2.9 Consistency of fine soils (Atterberg limits) NF P 94-051.

Consistency, representing the state of firmness of fine cohesive soils, is linked to the forces of cohesion between particles, which has a great influence on their resistance to deformation. The distance (voids) between fine particles is generally occupied by water (saturated state). Thus, the greater the water content, the further the soil particles are from each other. Consequently, the cohesion between them decreases, the consistency becomes soft and the soil deforms easily under loading. Conversely, a low water content allows the particles to come closer to each other, which increases the cohesive forces between them. Indeed, fine soils after drying (dry state) have a hard consistency (solid state) and therefore develop a very high resistance.

The Atterberg limits represent the water contents that delimit the four states of consistency, namely, the solid state (without shrinkage), the semi-solid state (with shrinkage), the plastic state and the liquid state. These limits, which are expressed as a percentage, are: the liquidity limit (w_L), the plastic limit (w_P) and the withdrawal limit (w_S).

The liquidity limit w_L is the water content of a reworked soil at the transition point between the liquid and plastic states (it separates the liquid state from the plastic state). It can reach 100% in the case of certain clays, but generally it does not exceed 100%.

The plastic limit w_P is the water content of a reworked soil at the transition point between plastic and semi-solid states. At this water content, the soil loses its plasticity and cracks by deforming when subjected to low loads. This water content varies from 0% to 100%, but is generally less than 40%.

The withdrawal limit w_S is the limiting water content that separates the semi-solid state from the solid state.This is the maximum water content that the soil can hold without changing volume.

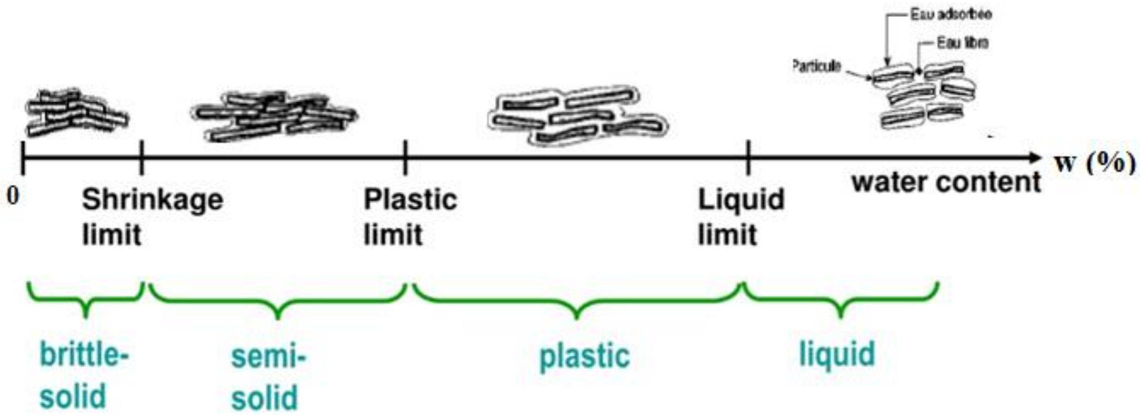
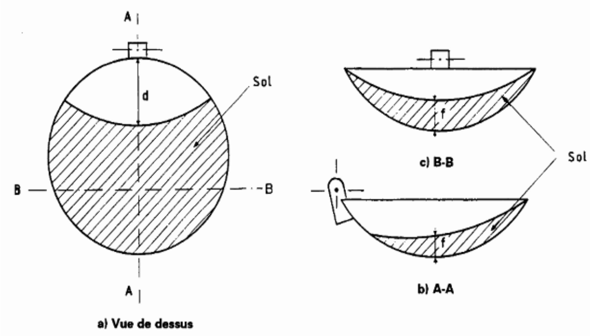


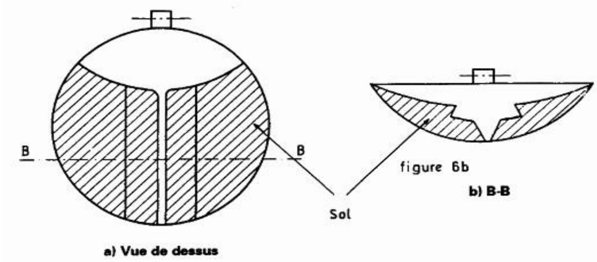
Figure 2. 9. Different states of consistency.

The procedure for determining the liquidity limit is as follows:

1. Take about **200 grams** of soil previously sieved at **0.4 mm** sieve wet and dried.
2. Knead the entire mixture until you obtain a smooth, almost fluid paste.
3. Take a portion of the dough and spread it in the Casagrande appliance bowl using the spatula. (see figure below).

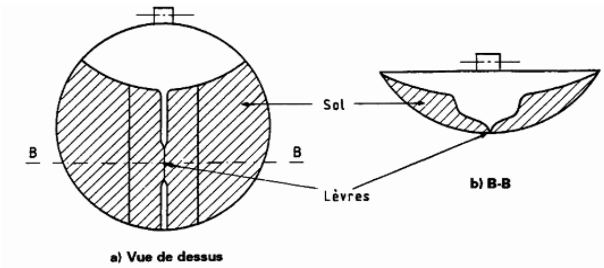


4. Make a groove in this dough so as to divide it in two. The grooving tool should be held perpendicular to the cup with its beveled part facing the direction of movement. (see figure below)



5. Subject the cup and the material it contains to repeated shocks at a rate of **2 strokes per second.**

6. Note the number of strokes (N) so that the groove closes on1 cm appreciated by eye. (see figure below)



7. Collect both sides of the lips where they closed approximately5 grams soil to determine its water content.
8. Re-homogenize the soil and dry it a little then repeat the operations 3 to 4 times. At least three attempts are required with a number of strokes N croissants and preferably well spread between 15and 35.

   The procedure for determining the plastic limit is as follows:

1. Take a little material and form a small ball.
2. Roll this ball by hand on the marble plate so as to obtain a stick.
3. Three cases can arise:

The finished stick begins to crack as soon as it reaches a length of 10 cm and a diameter of 3 mm. In this case, the soil is at the plastic limit and it must be measured (it is a water content).
The soil is still fluid and you can't make the stick. You need to dry the material a little.
The stick starts to crack too early, the material is dry. It needs to be moistened a little.
At least two to three tests should be performed for the plastic limit.

Calculation of the liquidity limit w_L

$$w_L = \omega_n \left(\frac{N}{25}\right)^{0,121}$$

ω : Water content corresponding to the number of strokes N. We will take the average of the three tests.

Calculating the Plasticity Limit w_P

The average of the two tests will be taken.
The plasticity index (Ip) is the difference between the liquid and plastic limits. This index defines the extent of the plastic domain.
It is therefore expressed by the following relation:

$$I_p = w_L - w_p$$

Depending on the value of their plasticity index, soils can be classified as shown in Table 2.3 below:

Table 2. 4. Degree of plasticity of soils.

Plasticity index	Degree of plasticity
$0 < I_P < 5$	Non-plastic soil (the test loses its meaning in this area of values)
$5 < I_P < 30$	Low plastic soil
30 < IP< 50	Plastic floor
50 < IP	Very plastic soil

Liquidity Index I_L allows you to quickly know if a soil is in a liquid, plastic, semi-solid or solid state.

To establish the liquidity index of a soil, we compare its natural water content (ω) to its plasticity and liquidity limits, i.e.:

$$I_L = \frac{\omega - \omega_P}{\omega_L - \omega_P} = \frac{\omega - \omega_P}{I_P}$$

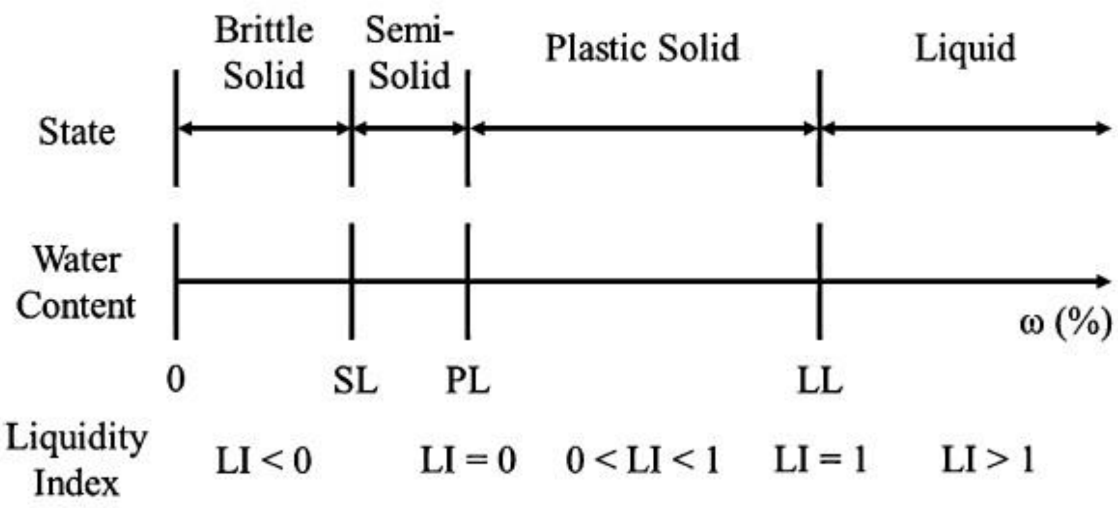


Figure 2. 10.*Atterberg Limits Liquidity Index.*

The consistency index Ic:

Comparing the natural water content ω of a soil and the Atterberg limits gives an idea of the state of a clay which can be characterized by its consistency index.

It characterizes the state of a soil with natural water content ω, is defined by:

$$I_C = \frac{\omega_L - \omega}{\omega_L - \omega_P} = \frac{\omega_L - \omega}{I_P} = 1 - I_L$$

Table 2. 5. Order of magnitude.

Consistency index	Soil condition
Ic>1	Solid
0 < Ic< 1	Plastic
Ic< 0	Liquid

2.10 Casagrande Method: "graphical method by linear regression" *(determination by cup – standard NF P 94-051).*

To determine the liquid limit, a layer of the material is spread on a cup in in which a groove is traced using a V-shaped instrument.

Similar shocks are applied to the cup, counting the number of shocks required to close the groove by 1 cm. We then measure the water content of the dough.

The relationship between the number of shocks N and the water content *w*(%), is a straight line in coordinates semi-logarithmic. After five tests which must be regularly staggered between 15 and 35 strokes, the most representative line is then drawn from the experimental points. The liquid limit is the water content which corresponds to a closure in 25 shocks.

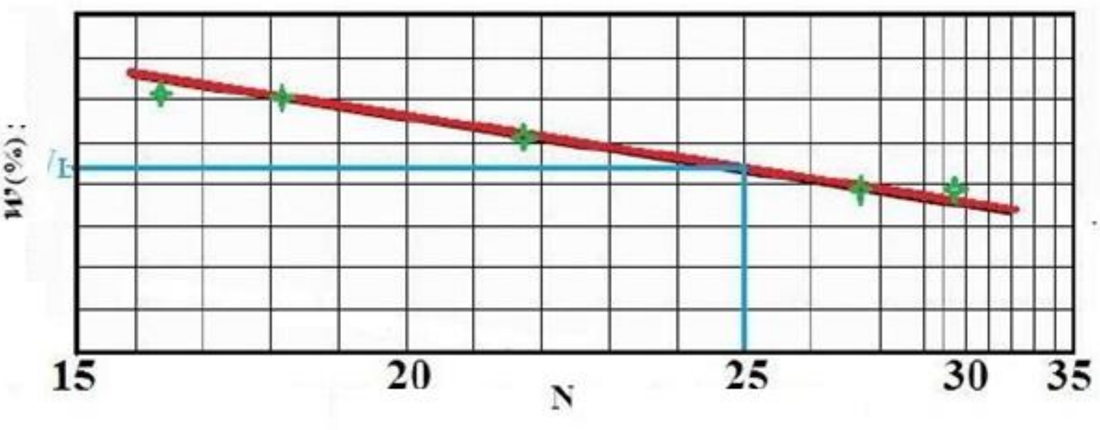


Figure 2. 11.*Curve representing the relationship between water content and number of strokes*.

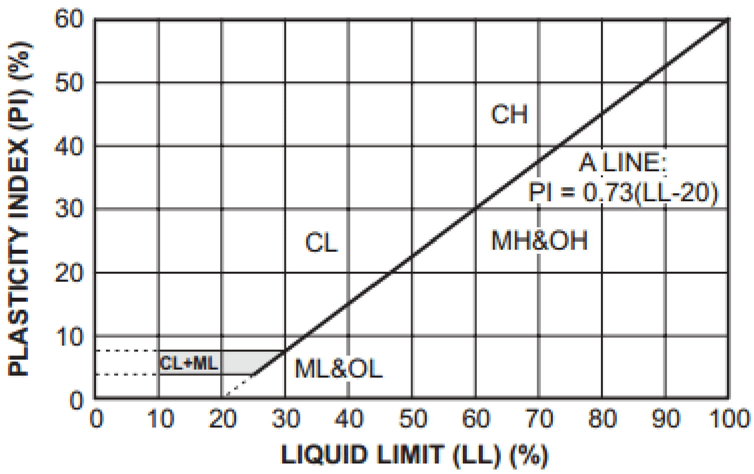


Figure 2. 12. Plasticity chart.

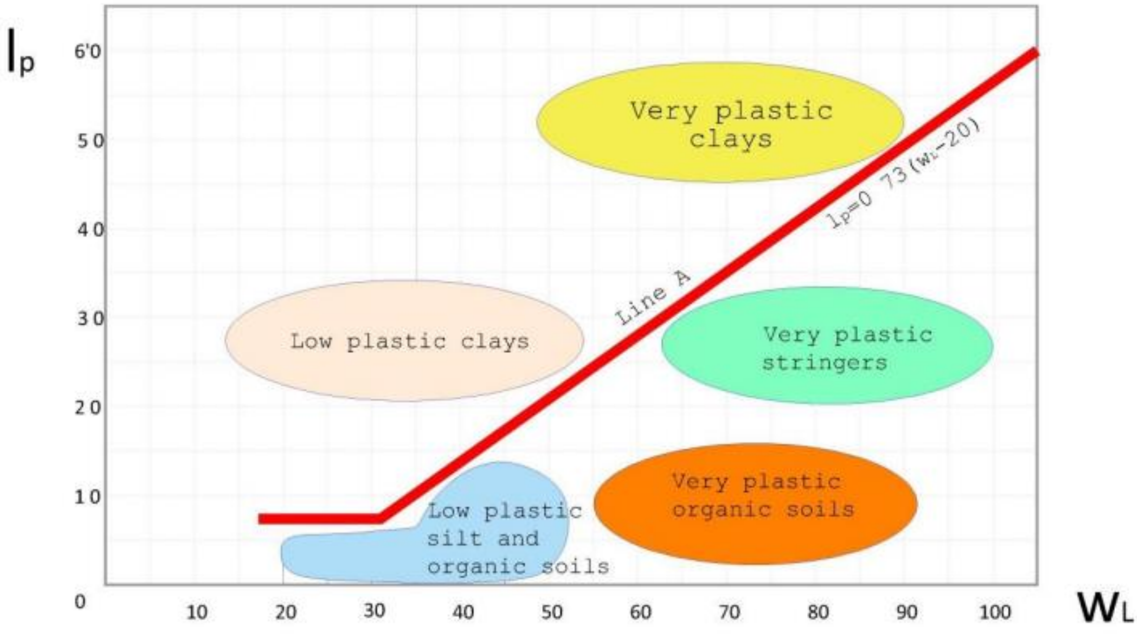


Figure 2. 13. LPC classification of fine soils in the laboratory. Plasticity diagram**.**

2.11 Clay activity

The activity of a clay (AC) is equal to the ratio between its plasticity index IP (%) and the clay content (%) present in a given soil:

$$A_C = \frac{I_P\ (\%)}{\text{clay content } (\%)}$$

It is noted that the activity of a clay is constant and that each type of clay has its own activity. It is therefore a characteristic of the soil which depends mainly on its mineralogical nature. Table 2.5 describes the activities of the most common clay particles:

Table 2. 6. Activity of different clay minerals.

Type of mineral	Activity Ac
Quartz	0
Calcite	0.2
Mica (muscovite)	0.2
Allophane	0.5 -1.2
Attapulgite	0.5 -1.2
Halloysite (hydrated)	0.1

Halloysite (dehydrated)	0.5
Kaolinite	0.3 - 0.5
Illite	0.5 - 1.3
Calcium montmorillonite	1.5
Sodium montmorillonite	4-7

2.12 Geotechnical classification of soils

In soil mechanics, there are several systems for classifying soils with advantages and disadvantages.

Geotechnical classification consists of identifying a soil using quantitative measurements and giving it a name, in order to link it to a group of soils with similar characteristics. Soil classifications aim to compare soils with each other. Their aim is to classify soils into families with the same or very similar geotechnical characteristics.

2.12.1 USCS classification and LCPC classification

The classification *USCS* (*United Soil Classification System*) established by Casagrande which serves basic to the classification adopted in France by the *LCPC* is very elaborate and precise. For coarse soils, the classification is based on the description of the material and essentially on its granulometry; Figure 2.13 groups the different categories of coarse soils according to the USCS classification.

For fine materials it is based on the Atterberg limits. Figure 2.15 depicts the Casagrande plasticity diagram used to classify fine soils according to the limits from atterberg w_L And *Ip*.

The USCS classification (*United Soil Classification System*) uses the results of three tests (sieving, sedimentometry and Atterberg limits).

From the mass fractions of soils with diameters of 0.08 mm and 2 mm, we distinguish:

The fine soils: more than 50% of the elements have a diameter less than 80 µm (*Casagrande diagram*);

The coarse-grained soils: more than 50% of the elements have a diameter greater than 80 µm (*USCS Organization Chart*).

The sand sand the serious are determined from the mass fraction corresponding to the diameter of 2 mm.

When less than 5% of the elements < 0.08 mm, conditions on the uniformity coefficient Cu and the curvature coefficient Cc, inform us on the graduation of the sands or gravels .*For example: Gb, Gm, Sb, Sm.*

When more than 12% of the elements < 0.08 mm, the position of the point *(WL, Ip)*on the plasticity diagram informs us about the presence of clay or silt in the sand or gravel.

For example: SA, SL, GA, GL

When the fine particle content < 0.08 mm is between 5% and 12%, the double symbol is used. f*or example: Sb-SL ...*

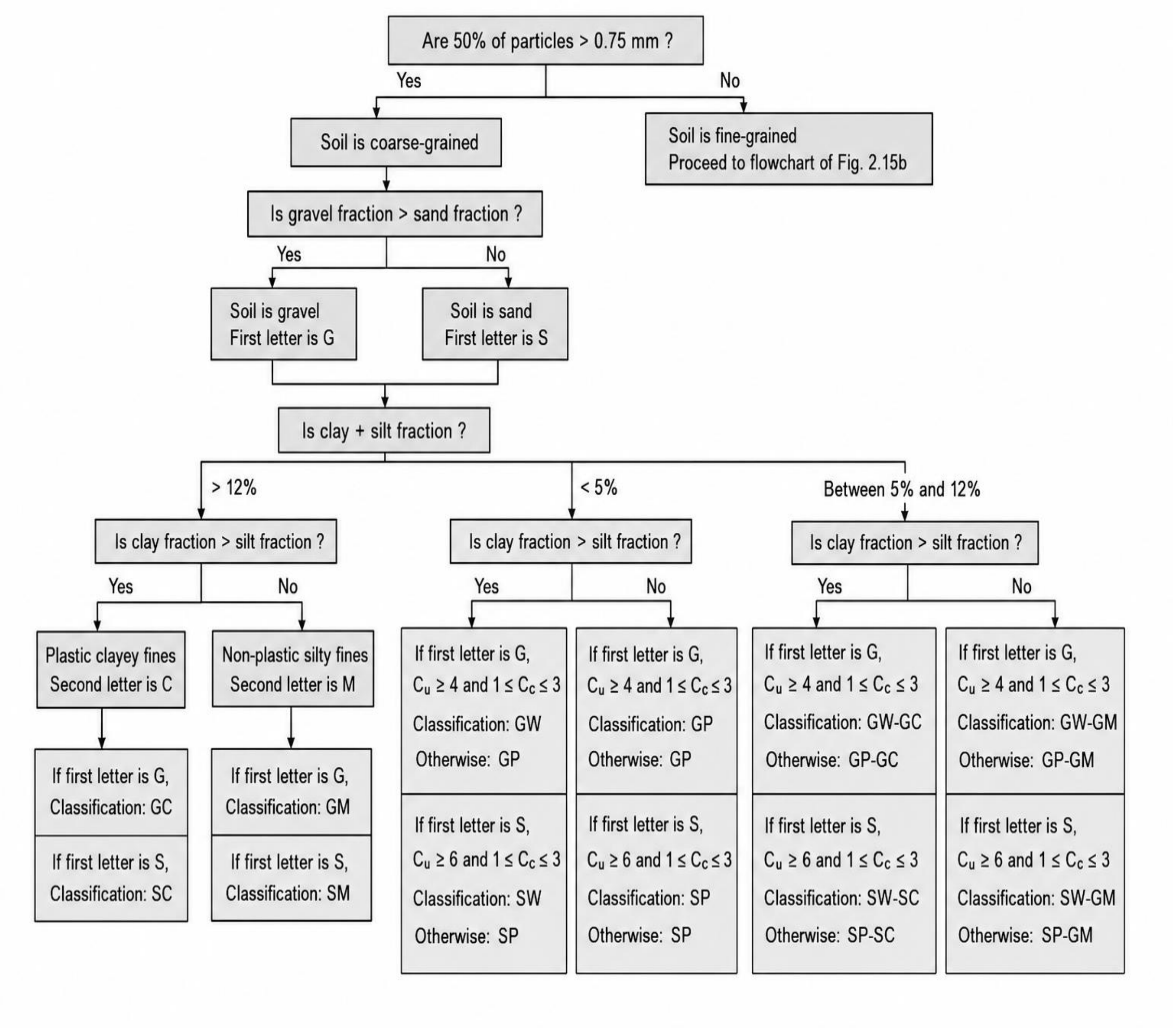


Figure 2. 14. Unified soil classification flowchart for coarse-grained soil.

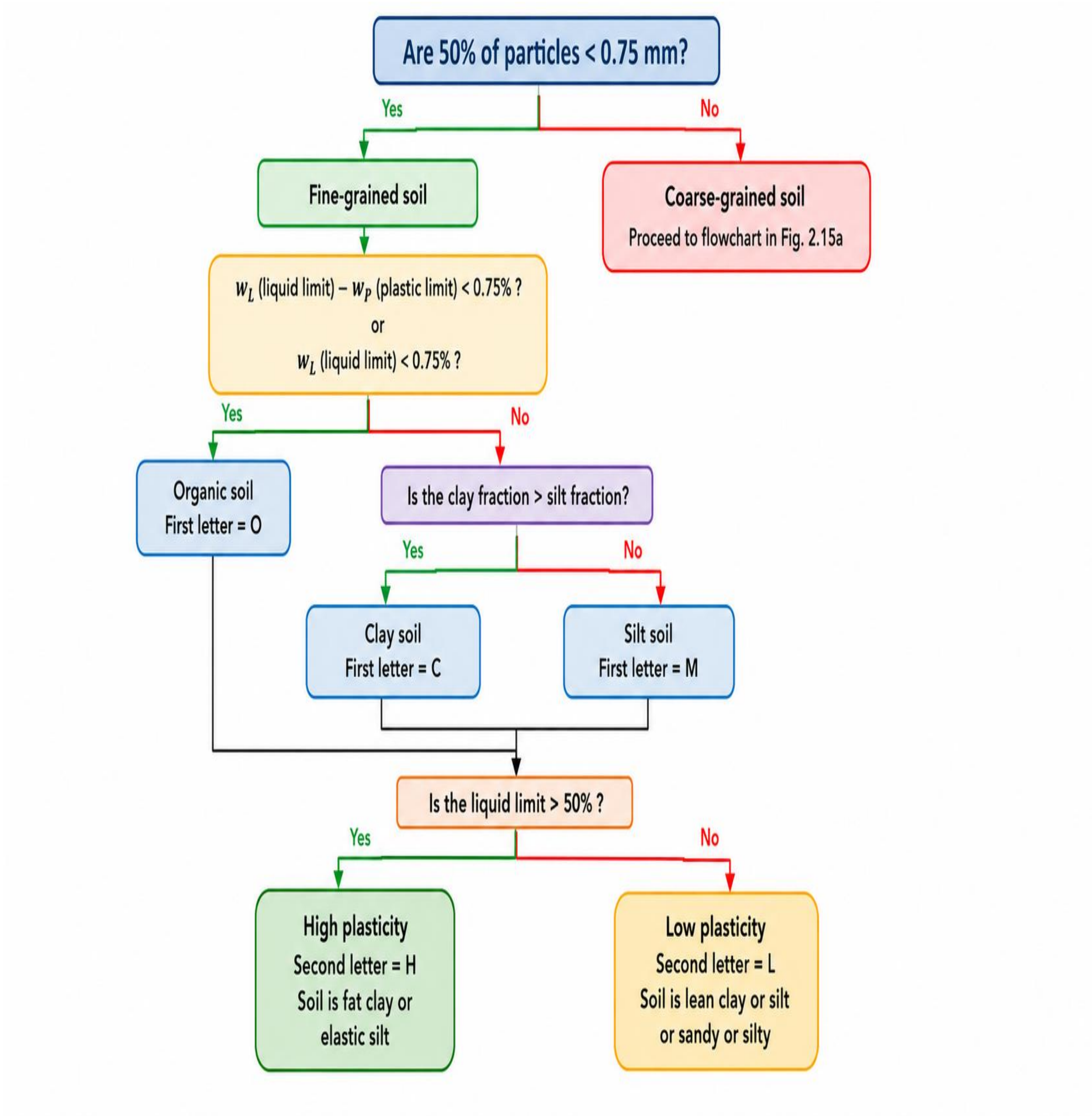


Figure 2. 15. Unified soil classification flowchart for fine-grained soil.

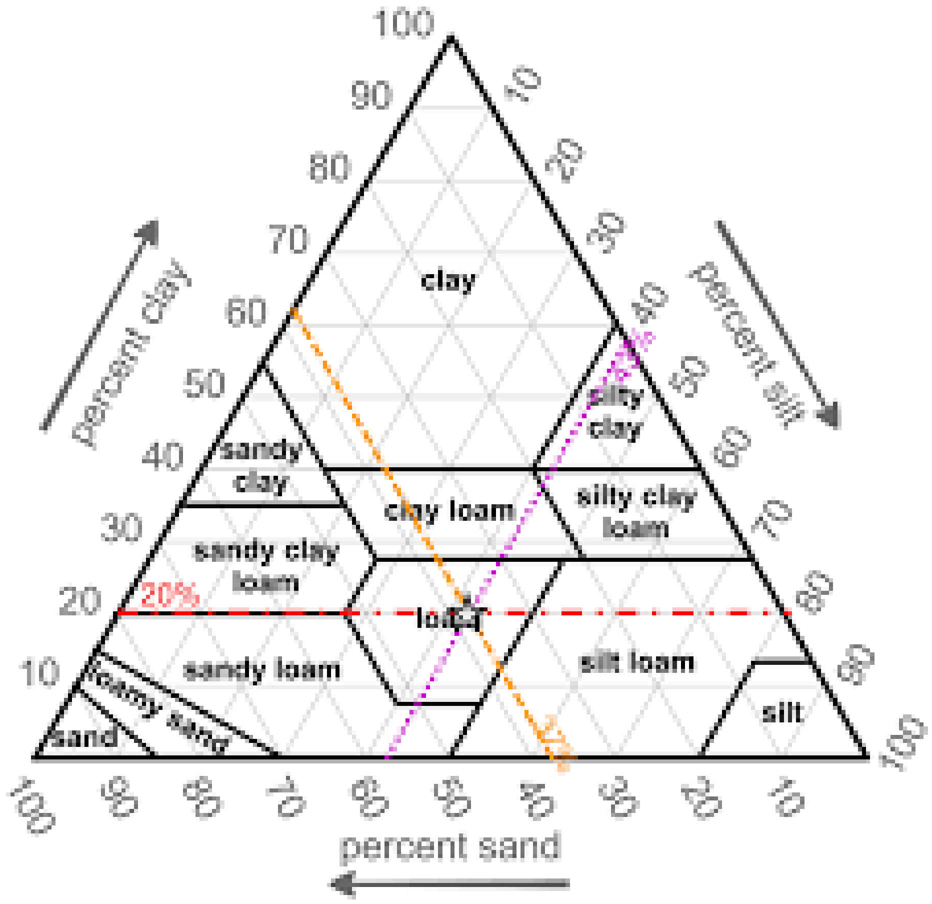


Figure 2. 16. Triangular classification diagram.

2.12.2 GTR classification

The GTR classification (Road Earthworks Guide) allows soils to be classified according tonature , water status and the mechanical behavior. The tests used for this classification are (Sieving, Sedimentometry, Atterberg Limits, Blue Test, Methylene, Proctor, CBR, Los Angeles, Micro-deval in the presence of water, Fragmentability of sands).

Its main categories are summarized in Figure 2.16, as follows:

Category A: fine soils

Category B: sandy and gravelly soils with fines.

Category C: soils with fines and coarse elements.

Category D: soils insensitive to water.

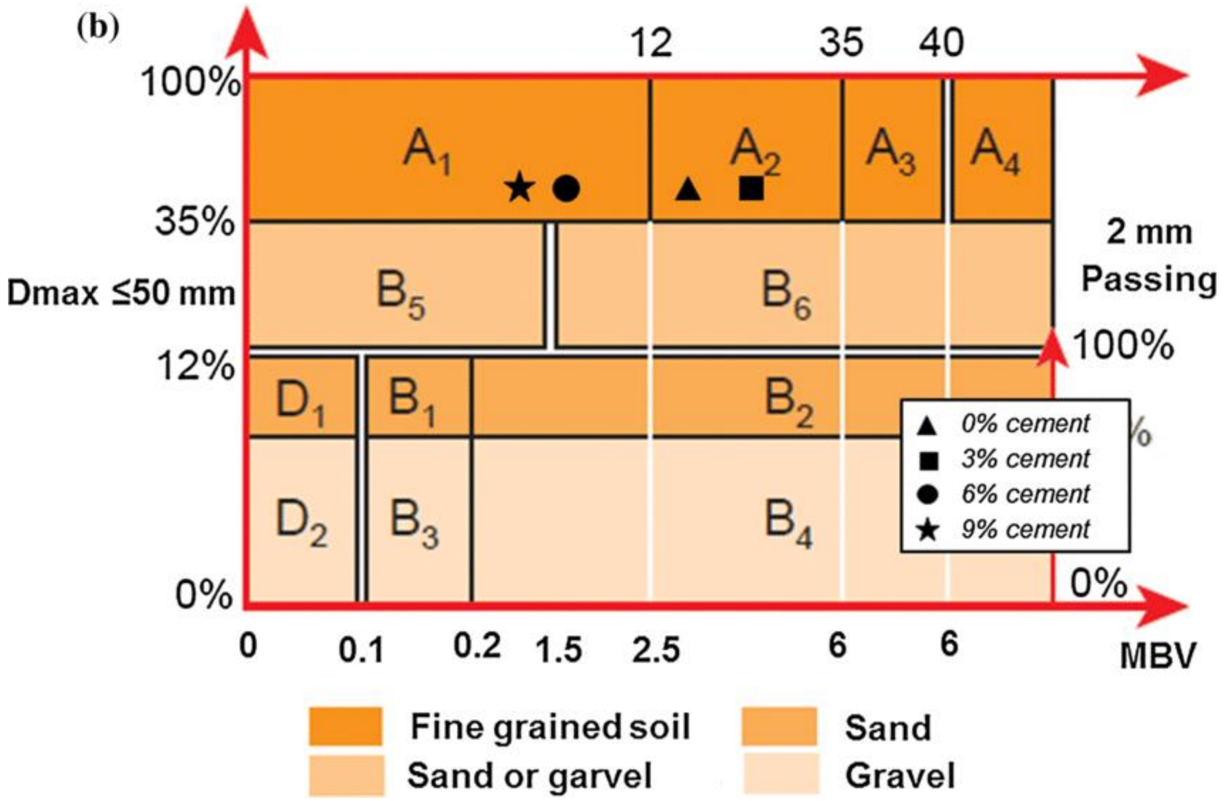


Figure 2. 17. Exemple of fine soil classification flowchart according to the GTR.

2.12.3 Modified LPC Classification

The modified LPC classification differs from the LPC/USCS classification only with regard to the characterization of organic soils, defined as soils containing more than 3% organic matter. Soils with an organic matter content lower than 10% are still classified as fine-grained soils. For soils containing more than 10% organic matter, the classification is based primarily on the degree of decomposition (humification) of the organic fibers, which is determined using the Von Post test.

According to this modified classification system, soils are divided into twenty-two categories:

- **Coarse-grained soils:** Gb, Gm, GA, GL, Sb, Sm, SA, SL;
- **Fine-grained soils:** At, Ap, Lt, Lp;
- **Weakly organic soils:** fO-At, fO-Ap, fO-Lt, fO-Lp;
- **Moderately organic soils:** mO-a, mO-sf, mO-f;
- **Highly organic soils:** tO-a, tO-sf, tO-f.

The symbols used for coarse-grained and fine-grained soils remain identical to those of the LPC/USCS classification. For organic soils, the suffixes **a**, **sf**, and **f** denote **amorphous organic matter**, **semi-fibrous organic matter**, and **fibrous organic matter**, respectively. This classification provides a more accurate description of organic soils by incorporating both their organic matter content and their degree of humification.

Table 2. 7. Classification of grainy soils – LPC/USCS.

more than 50% of elements > 0.08 mm					
Definitions			**Symbols**	**Terms**	**Appellations**
SERIOUS	**More than 50% of elements > 0.08 mm have a diameter > 2 mm**	**less than 5% of elements < 0.08 mm**	**Gb** (GW)	Cu > 4 and Cc between 1 and 3	well-graduated grave
			Gm (GP)	One of the conditions of Gb not satisfied	serious clean poorly graduated
		more than 12% of elements < 0.08 mm	**Gm** (GP)	Atterberg limit below A	silty gravel
			GL (GM)	Atterberg limit above A	clayey gravel
SANDS	**More than 50% of elements > 0.08 mm have a diameter < 2 mm**	**less than 5% of elements < 0.08 mm**	**Sb** (SW)	Cu > 6 and Cc between 1 and 3	well-graduated sand
			Sm (SP)	One of the conditions of Sb not satisfied	poorly graded clean sand
		more than 12% of elements < 0.08 mm	**SL** (SM)	Atterberg limit below A	silty sand
			SA (SC)	Atterberg limit above A	clayey sand
When 5% < % less than 0.08 mm < 12%: a double symbol is used.					

Application

Exercise 1:

A clay sample with a degree of saturation of 90% has a mass of 1420 g. After oven drying, its mass is reduced to 920 g. The specific gravity of the soil solids is Gs=2,7. Given: $g=9,81 m/s^2$.

Determine:

- Water content
- Void ratio
- Porosity
- Saturated unit weight
- Submerged density

Solution:

The weight of the saturated clay sample is: $W_{sat}=1,420 x 9,81=13,93$ N

The weight of the sample after oven drying is: $W_s=0,920x9,81=9,02$ N

The weight of water contained in the sample is therefore: $W_w=13,93-9,02=4,91$ N

Water Content: $W_w/W_s=4,91/9,02=0,55=55\%$
Void Ratio $e=V_v/V_s$
Since the soil is 90% saturated: $V_v=V_w/0,9$
$\gamma_w = \frac{W_w}{V_w} \Rightarrow V_w = \frac{W_w}{\gamma_w} = \frac{4,91}{9,81.10^3} = 0,500.10^{-3}m^3 = 500cm^3$

The unit weight of water is: $G_s = \frac{\gamma_s}{\gamma_w} \Rightarrow \gamma_s = G_s.\gamma_w = 2,7.9,81 = 26,5kN/m^3$

Thus, the volume of water is: $\gamma w=9.81\times10^3 N/m$

The unit weight of solid particles is: $V_s = \frac{W_s}{\gamma_s} = \frac{9,02}{26,5.10^3} = 0,340.10^{-3}m^3 = 340cm^3$

Therefore, the void ratio is:

$$e = \frac{V_v}{V_s} = \frac{V_w}{0.9V_s} = \frac{500}{0.9.340} = \frac{556}{340} = 1,63$$

Porosity

$$\eta = \frac{V_v}{V} = \frac{556}{556+340} = 0,62$$

Saturated Unit Weight : $\gamma_{sat} = \frac{W}{V} = \frac{13.99}{(0,5+0,340).10^{-3}.} = 16.65.\frac{10^3 N}{m^3} = 16.65 kN/m^3$

Submerged Density:

$$\mathbf{d}' = \frac{\boldsymbol{\gamma}'}{\gamma_w} = \frac{\gamma_{sat} - \gamma_w}{\gamma_w} = \frac{\mathbf{16.65 - 9,81}}{\mathbf{9,81}} = \mathbf{0,69}$$

Exercise 2:

Demonstrate the following relationships:

$$n = \frac{V_v}{V_t} = \frac{\frac{V_v}{V_s}}{\frac{V_t}{V_s}} = \frac{\frac{V_v}{V_s}}{\frac{V_s+V_v}{V_s}} = \frac{e}{1+e}$$

$$\gamma_d = \frac{W_s}{V_t} = \frac{\frac{W_s}{V_s}}{\frac{V_t}{V_s}} = \frac{\frac{W_s}{V_s}}{\frac{V_s+V_v}{V_s}} = \frac{\gamma_s}{1+e}$$

$$e = \frac{V_v}{V_s} = \frac{\frac{V_v}{V_t}}{\frac{V_s}{V_t}} = \frac{\frac{V_v}{V_t}}{\frac{V_t-V_v}{V_t}} = \frac{n}{1-n}$$

Exercise 3:

A soil sample has a mass of 129.1 g and a volume of 56.4 cm³. The mass of the solid particles is 121.5 g. The specific gravity of the soil solids is Gs=2,7. Given: g=9,81m/s^2. Determine: : ω ,e, Sr.

Solution:

The weight of the sample is: 0,1291x9,81=1,2668 N

The weight of the solid particles is: 0,1215x9,81=1,1919 N

The weight of water is therefore: 1,2668-1,1919=0,0746 N

1) Water Content

   W_w/W_s=0,0746/1,192=0,063=6,3%

2) Void Ratio $e=V_v/V_s$

   $V_v=V-V_s$

   $V_s = \frac{W_s}{\gamma_s} = \frac{1,1919}{2,7.9,81.10^3} = 4,5.10^{-5} m^3 = 45 cm^3$

   Thus:

   V_v=56,4-45=11,4cm^3

Therefore:

$$e = V_v/V_s = 11,4/45 = 0,253$$

$$e = 0,25$$

3) Degree of Saturation

$$S_r = \frac{V_w}{V_v} = \frac{V_w}{\frac{W_w}{\gamma_w}} = \frac{0,0746}{\frac{0,0746}{9,81.10^3}} = 0,666$$

S_r=67%

<u>Exercise 4:</u>

A sieve analysis performed on a dry soil sample with total mass M=3500 g produced the following results:

Sieve opening (mm)	5	2	1	0,5	0,2	0,1
Retained mass (g)	217	868	1095	809	444	39

1. Determine the percentage passing for each sieve.
2. Plot the grain size distribution curve.
3. Determine the coefficient of uniformity. Conclude.
4. Determine the coefficient of curvature.

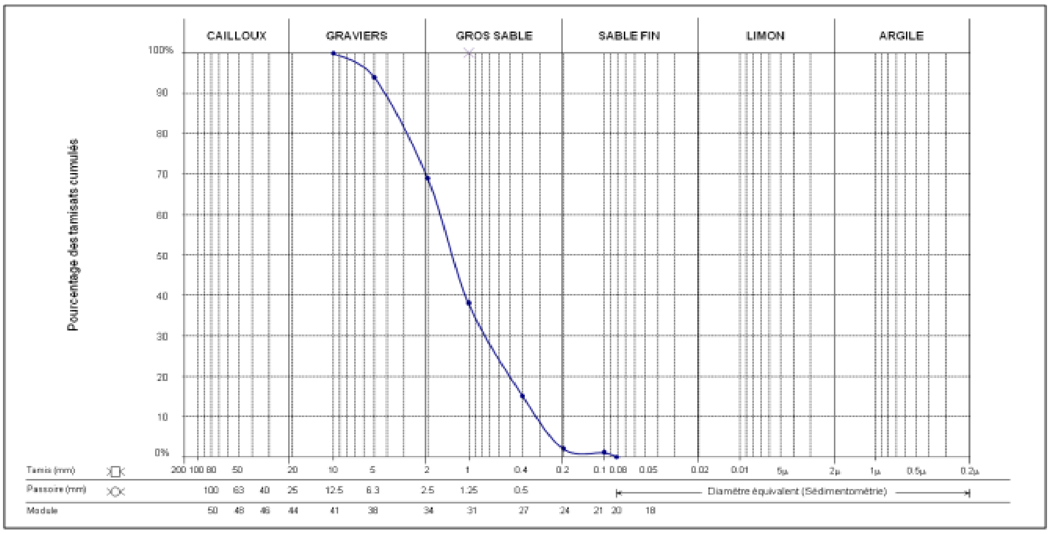
<u>Solution:</u>

1) Percentage Passing

Sieve opening (mm)	12.5	5	2	1	0,5	0,2	0,1
Retained mass (g)	0	217	868	1095	809	444	39
Passing (%)	100	94	69	37.7	14.6	1.92	0.8

Grain Size Distribution Curve

The curve is plotted on semi-logarithmic paper
(sieve size in log scale, percentage passing in arithmetic scale).



Coefficient of Uniformity

$$C_U = \frac{d_{60}}{d_{10}}$$

From the grain size distribution curve:

d_{10}= 0.37 mm, d_{60}= 1.60 mm

$$C_U = \frac{d_{60}}{d_{10}} = \frac{1.6}{0.37} = 4.312$$

Therefore, the soil has a **well-graded (wide-range) particle distribution**.

Coefficient of Curvature

$$C_C = \frac{d_{30}^2}{d_{10} \cdot d_{60}}$$

From the curve: d_{30}= 0.8

$$C_c = \frac{(d_{30})^2}{d_{10}.d_{60}} = \frac{(0.8)^2}{(0.37 \times 1.6)} = 1$$

Exercice 5:

A saturated clay sample has a mass of 1526 g; after oven drying, its mass becomes only 1053g. The solid particles have a specific gravity of: Gs=2.7 given: g=9.81 m/s^2
Calculate: :ω ,e, η, γh, dh .

Solution:

Water Content w :

Mass of water:

$$M_w = M_{sat} - M_s = 1526 - 1053 = 473g$$

$$w = \frac{M_w}{M_s} = \frac{473}{1053} = 0.449$$

$$w = 44.9\%$$

Void Ratio e

For a saturated soil:

$$S_r = \frac{wG_s}{e} = 1$$

$$e = wG_s = 0.449 \times 2.7$$

$$e = 1.21$$

Porosity n:

$$n = \frac{e}{1+e} = \frac{1.21}{1+1.21} = 0.548$$

$$n = 54.8\%$$

Bulk Unit Weight γh:

Since the soil is saturated :

$$\gamma_h = \gamma_{sat} = \frac{G_s + e}{1+e}\gamma_w$$
$$\gamma_w = 9.81 KN / m^3$$
$$\gamma_h = \frac{2.7+1.21}{1+1.21} \times 9.81 = 17.36 KN / m^3$$

Bulk Density (dh):

$$d_h = \frac{\gamma_h}{\gamma_w} = \frac{17.36}{9.81} = 1.77$$

Exercice 6:

Two samples of the same soil are taken.

The first sample is **saturated** (below the water table) and has a unit weight of:

$\gamma sat = 20$ kN/m^3

The second sample is **unsaturated** (above the water table) and has a unit weight of:

$\gamma = 17.5$ kN/m^3

Given:

$\gamma w = 10$ kN/m^3 and the unit weight of the solid particles is: $\gamma s = 26$ kN/m^3

Calculate the degree of saturation Sr of the second sample.

Solution:

Determine the specific gravity of solids

$$G_s = \frac{\gamma_s}{\gamma_w} = \frac{26}{10} = 2.6$$

Determine the void ratio e

$$\gamma_{sat} = \frac{G_s + e}{1+e}\gamma_w$$
$$20 = \frac{2.6+e}{1+e} \times 10$$
$$2 = \frac{2.6+e}{1+e} \times 1$$
$$2(1+e) = 2.6 + e$$
$$2 + 2e = 2.6 + e$$
$$e = 0.6$$

Determine the degree of saturation Sr

For the unsaturated sample:

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

$$17.5 = \frac{2.6 + 0.6S_r}{1,6} \times 10$$

$$1.75 = \frac{2.6 + 0.6S_r}{1,6}$$

$$2.8 = 2.6 + 0.6S_r$$

$$0.6S_r = 0.2$$

$$s_r = \frac{0.6}{0.2} = 0.33$$

$$S_r = 33\%$$

Exercice 7:

A soil sample was placed in an oven for 24 hours and lost **24 g** of its initial mass. Before drying, the soil had a **water content of 4.8%**, a **degree of saturation of 45%**, and a **void ratio of 0.40**. Determine:

1. The initial mass of the sample before oven drying.
2. The total volume of the sample.
3. The porosity.
4. The specific gravity of the soil solids.
5. The wet density and dry density.
6. If the sample is brought to full saturation without changing its volume, determine its new mass.

Solution :

Initial mass before drying

Water content:

$$w = \frac{M_w}{M_s}$$

$$M_s = \frac{24}{0.048} = 500g$$

Initial mass: $M_i = M_s + M_w = 500 + 24 = 524g$

Specific gravity of solids

The relationship: $S_r = \dfrac{wG_s}{e}$

$$G_s = \frac{S_r e}{w} = \frac{0.45 \times 0.4}{0.048} = 3.75$$

$$\rho_s = G_s \rho_w$$
$$\rho_w = 1 g / cm^3$$
$$\rho_s = 3.75 g / cm^3$$

$$V_s = \frac{M_s}{\rho_s} = \frac{500}{3.75} = 133.33 cm^3$$
$$V_t = V_s (1+e) = 133.33(1+0.4) = 186.67 cm^3$$

Porosity

$$n = \frac{e}{1+e} = \frac{0.4}{1.4} = 0.286$$
$$n = 28.6\%$$

Dry density

$$\rho_d = \frac{M_s}{V_s} = \frac{500}{186.67}$$
$$\rho_d = 2.68\, g / cm^3$$

Wet density

$$\rho = \frac{M}{V_t} = \frac{524}{186.67} = 2.81 g / cm^3$$

Chapter 3
Soil compaction

Chapter 3: Soil compaction

3 Compaction theory

3.1.1 Introduction

Soil improvement and stabilization methods are diverse and often involve one of the following processes: chemical treatment, injection or mixing of (lime, cement...), thermal treatment, electrical treatment and mechanical treatment, including densification or compaction.

Compaction is one of the oldest methods of soil improvement and stabilization, and is generally simple and easy to master, making it very common and indispensable, especially in road and dam construction.

3.1.1.1 Definition

Compaction is the set of mechanical operations that increase the density of the soil in place.

The purpose of this operation is to tighten the texture of the soil by reducing the air-filled voids (Fig. 3.1):

- reduce the potential for deformation, i.e. eliminate or at least limit settlement;
- increase soil's load-bearing capacity.

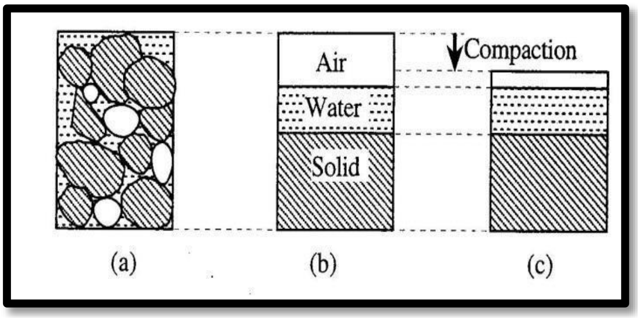


Figure 3. 1. Soil compaction principle.

In fact, as illustrated in Figure 3.1 (a), since the soil is made up of solid grains and voids filled air and water, in the proportions shown in Figure 3.1 (b), compaction, as shown in Figure 3.1 (c), only reduces the air fraction, barely changes the amount of water and has no effect on the volume of solid grains.

Theoretically, the most efficient compaction process should completely eliminate the air fraction. In practice, however, compaction cannot eliminate it entirely, but can reduce it to a minimum, provided appropriate techniques are used.

In 1933, Proctor is credited with having methodically analyzed this phenomenon. Proctor's idea was to demonstrate that there is an optimum water content, which, given the means used, will result in maximum clamping.

This is clearly illustrated in the typical compaction curve in Figure 3.2, where it can be seen that water has a significant effect on soil compaction.

, if we vary the water content of the sample and plot the variation in dry density γd as a function of water content ω, we obtain a bell-shaped curve (Fig. 3.2), known as the Proctor diagram or curve, which has a high point known as the *Proctor optimum* (*P.O.*), the abscissa of which provides the *optimum water content* and the ordinate the maximum dry density sought.

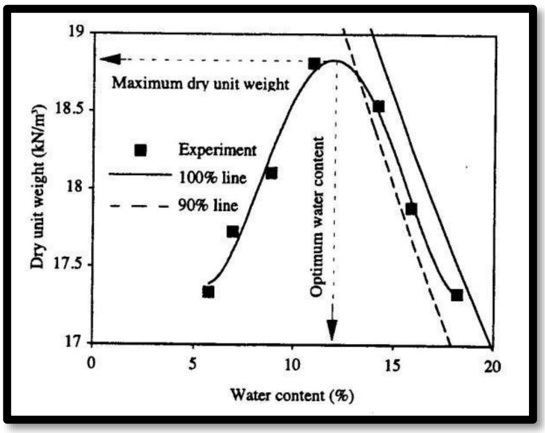


Figure 3. 2. Typical compaction curve.

The weight by volume of compacted soil will therefore be greater than that uncompacted soil. It is this characteristic that enables us to verify the effectiveness of compaction.

Experience has shown that dry density is only a soil characteristic to a certain extent, as it also varies with compaction energy and water content.

3.1.2 Compaction theory (Proctor theory)

Soil compaction, particularly of fine and intermediate soils, depends on :

a)- water content ;

b)- compaction energy;

c)- type of material.

If the water content is high, the water absorbs a large proportion of the compaction energy and takes the place of the solid grains. No compaction is possible.

On the other hand, if the soil is too dry, it won't compact, as water is essential as a lubricant to ensure compaction (better grain arrangement and increased density).

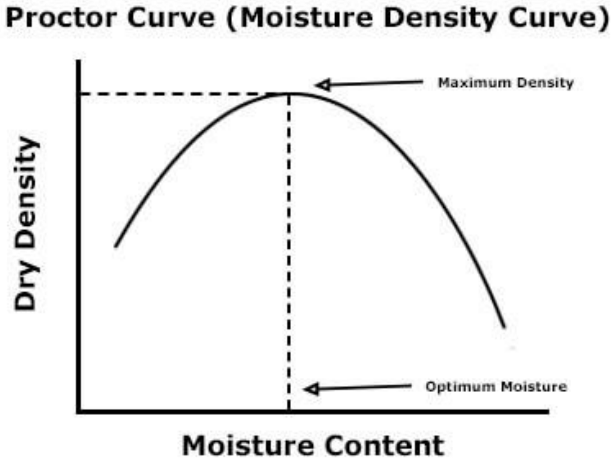


Figure 3. 3. Compaction curve for sand with density2.71.

The change in dry density γ_d with water content depends on the nature of the soil.

They are very flattened for sands (their compaction is therefore little influenced by water content). Materials of this type make the best backfill. The finer the soil, the greater the influence of water content. Clearly, the steeper the curvethe less easily compacted the soil will be, and the more poorly backfilled it will be (Fig 3.4).

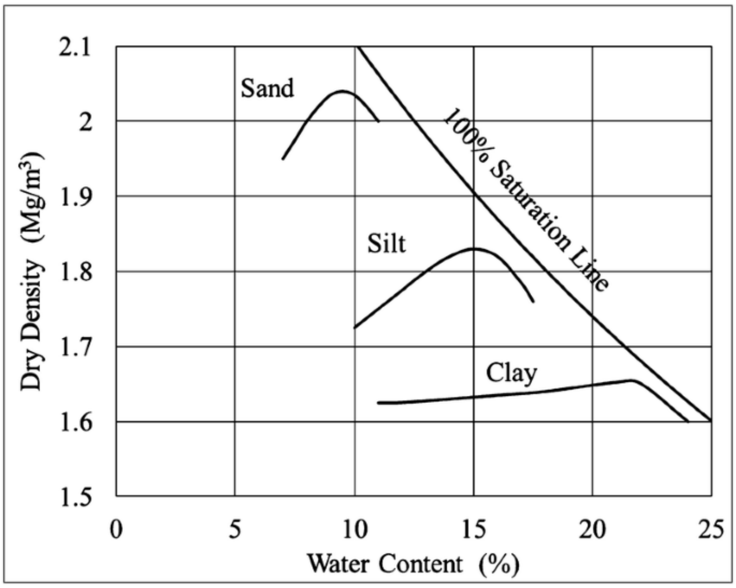


Figure 3. 4. Influence of soil type.

As energy increases, optimum density increases and optimum water content decreases (Fig3.5).

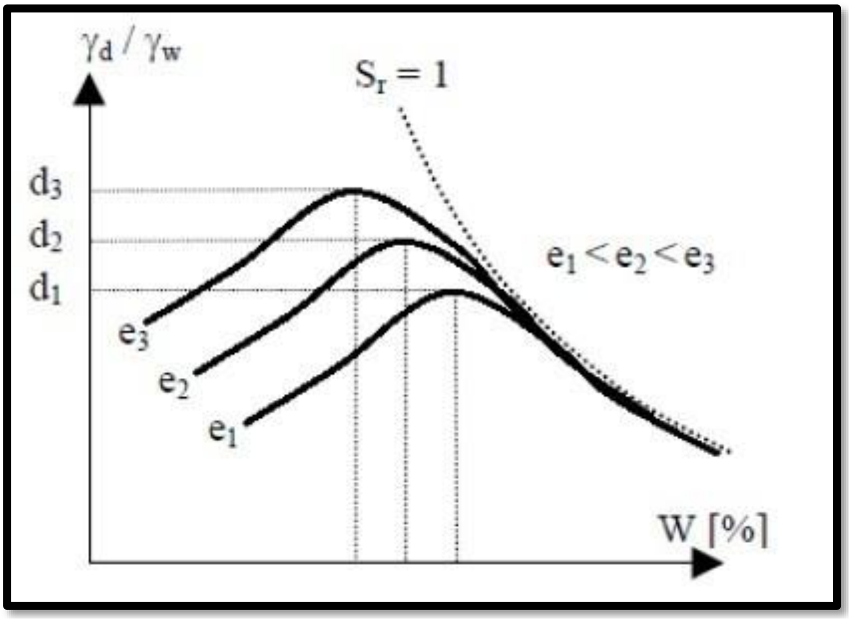


Figure 3. 5. Influence of compaction energy.

3.2 Laboratory compaction tests (Proctor and CBR tests*) (NF P 94-093)*

The Proctor test is a ***laboratory compaction*** test. Its aim is to determine the optimum water content (adequate quantity water) achieve the maximum dry density of the compacted material. It is thus used to define the compaction characteristics of a material, i.e. :

- Optimum water content (w_{opt}).

- Maximum dry density (γd_{max}).

The purpose of *the Proctor test* is to :

- soil evolution during compaction ;
- determine, for a given compaction energy, the water content required to achieve *maximum dry density*.

A material is therefore defined by its grain size. The Proctor test is designed to measure the water content which, after a given compaction of the material, gives a maximum value for the dry density γd.

3.2.1 Test principle

The test consists in compacting a series of samples in a mold according to a well-defined process:

— identical and representative of the soil,

— soaked with increasing water .

Water, which *as a lubricant*, helps the grains to settle and tighten.

The Proctor compaction characteristics of a material are determined by the Normal Proctor Test or the Modified Proctor Test.

The two tests are identical in principle, the only difference being the values of the parameters that define the compaction energy applied.

- The normal (or standard) Proctor test is applied to soils with a maximum diameter of< 5mm using a standardized mold (Proctor mold 10.15 cm in diameter).diameter and 11.7 cm high).
- The Proctor CBR (Californian Bearing Ratio) test is applied to soils with a diameter 5mm < d < 20mm using a CBR mould 15.2 cm in diameter and 15.2 cm

  high.

The purpose of this test is to determine, for a given compaction intensity, the optimum moisture content at which a soil should be compacted to obtain maximum dry density.

1. Normal Proctor test: (recommended for backfill).
2. Modified Proctor test: (recommended for pavement layers).

Table 3. 1. Normal and modified PROCTOR test.

Proctor test	Lady's weight (Kg)	Height of drop (cm)	Number of strokes per layer	Number of layers	Energy from compaction (KJ/dm^3)
normal	2,490	30,5	25 (mold Proctor)	3	0,59
			55 (mold C.B.R.)	3	0,53
modified	4,540	45,7	25 (mold Proctor)	5	2,71
			55 (mold C.B.R.)	5	2,41

3.2.2 Test execution

1. Prepare several kg dry soil.
2. Weigh the empty Proctor mold with its base, without its riser, i.e. $M0$ its mass.
3. Place the riser, securing it to the mold.
4. Moisten 2.5 Kg of this dry soil with 125ml of water and mix to ensure that all the grains are wet.
5. Compact the soil in three layers for the normal Proctor test by applying 25 blows (using the Proctor tamper). Blows should be evenly distributed over the entire surface.
6. After the last layer has been compacted, remove the riser and shave the sample down to the edge of the mold.
7. Weigh the mold plus the compacted soil sample, i.e. $M1$ its mass.
8. Take two samples of the upper and lower parts of the compacted soil to determine their water content.
9. Remove the compacted soil, using the extractor if necessary.
10. For other tests, always moisten 2.5Kg of soil with n times 125ml of water (n: test number).
11. Repeat from step 5.

Figure 3. 6. Percussive compaction in the lab (PROCTOR test).

Figure 3. 7. Proctor test.

Calculation

Dry soil density: $\gamma_d = \frac{M_0 - M_h}{(1+\omega)\times V}$ (Kg/m3)

weight of water W_{water} to be added to the sample in order obtain the desired water content ω_d is determined by the following equation:

$$W_w = \frac{V_{Tot} \times (\omega_d - \omega_0)}{1 + \omega_0}$$

With

V_{Tot} Total sample volume (g)

ω_d : Desired water content (%)

ω_0 : Initial water content (%)

3.3 In situ compaction tests

Compaction involves treating the surface of the soil in relatively thin layers, placed and compacted one on top of the other.

Compaction makes the soil more resistant and improves its bearing capacity. Compaction is achieved by the action of mechanical machines, which act by pressure, vibration or percussion. The compaction machines most commonly used on earthworks sites are :

1. Tire compactors ;
2. Vibratory smooth roll compactors ;
3. Vibratory compactors with tamper feet;
4. Static compactors with pad feet;
5. Vibratory plates ;

-Compaction is produced by the action a heavy mass (compactors, rollers). It moves slowly over the surface of the soil (Fig.3.8);

Figure 3. 8. Roller compactor (2.5t).

In the second case, soil particles are subjected to vibrations produced by a device placed on the surface of the ground, such as a vibrating plate (Fig.3.9) or vibrating rollers (Fig.3.10)

Figure 3. 9. Vibrating plate.

Figure 3. 10. Vibratory roller (1.5 to 3.8 t).

3.4 Conclusion

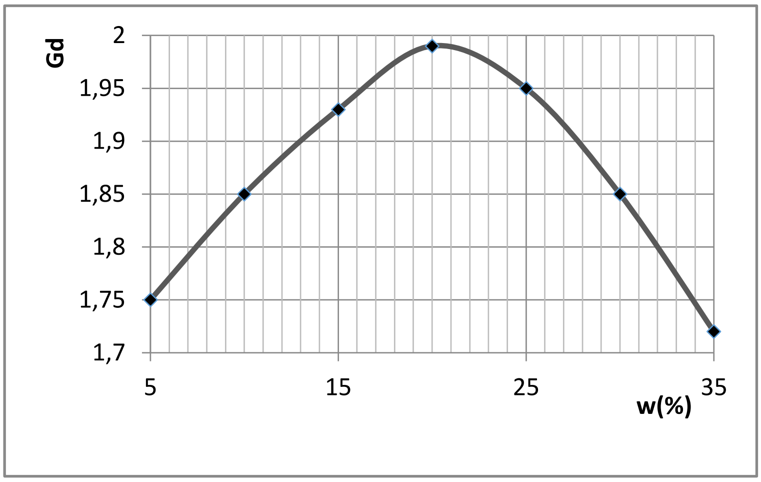
Soil compaction plays a vital role in ensuring the stability and longevity of civil engineering projects. By understanding the principles, factors, and methods of compaction, engineers can optimize soil conditions to meet project requirements. This chapter highlights the importance of proper compaction and provides a foundation for practical applications in the field.

Application

Exercise 1:

The results of a standard compaction test carried out on a soil sample are shown in the figure below:

1. Determine the **Proctor optimum** (optimum moisture content and maximum dry density).
2. Determine the required **water content** and the **volume of water** needed for a section 3.5 m wide, 1 km long, and 30 cm thick.
3. Estimate the number of **10-ton truckloads** required for transporting the embankment material.



Solution:

Proctor Optimum

From the curve, the **Proctor optimum** corresponds to: $\omega=20\%$.

Water Content and Required Water Volume

At the Proctor optimum, the maximum dry density is:1.99.

For the Proctor optimum condition: $\omega=20\%$: $W_w=0,20 \times W_s$

$W_s= G_d.\gamma_w.V_s=1,99 \times 9,81 \times 3,5 \times 1000 \times 0,30=20498$ kN

The weight of water required is: $W_w=0,20 \times W_s =0,20 \times 20498= 4100$ kN

Therefore, the required water volume is:

$$V_w=W_w/\gamma_w=4100/9,81=418 \text{ m}^3$$

So, the required water volume is: 418 m^3

Number of 10-ton Truckloads

Since: 10kN=1t donc 4100kN=410 t

Exercice 2:

The results of a standard compaction test carried out on a soil sample are shown in the figure below:

1. Determine the **Proctor optimum** (optimum moisture content and maximum dry density).
2. The specifications require a **relative compaction of 99%**. Determine the range of water content that satisfies this requirement.
3. Determine the required quantity of water (in volume) for a section 4 m wide, 100 m long, and 20 cm thick.

Solution :

1**) Proctor Optimum**

From the curve, the **Proctor optimum** corresponds to: ω=14%.

2**) Relative Compaction of 99%**

The Proctor optimum corresponds to a maximum dry density of:1.77

Gd (max)=1,77 $\Rightarrow$0,99 Gd=0,99x1,77=1,75$\Rightarrow$$\omega_1$=13% et ω_2=15,5%

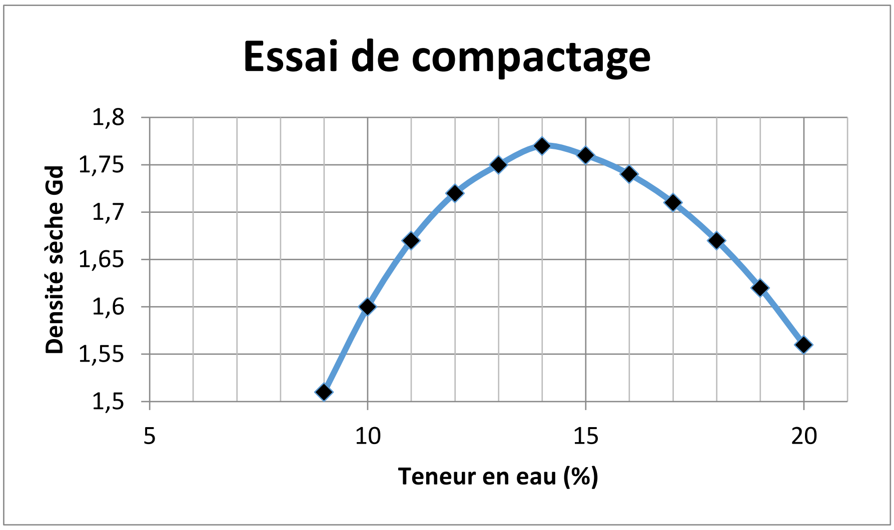
3**) Water Requirement at Proctor Optimum**

For the Proctor optimum:

$\omega=14\%=W_w/W_s$
$\Rightarrow W_w=0,14\text{x } W_s$
$W_s= G_d.\gamma_w.V_s$=1,77x9,81x4x100x0,2=1389,1 kN

W_w=0,14x W_s =0,14x1389,1= 194,5 kN

D'où :V_w=W_w/γ_w=194,5/9,81=19,83 m



Exercice 3:

The results of the modified Proctor test carried out on a road material (Sample A1) are given. Carry out the necessary calculations and plot the curve of the variation of the dry density γ_d depending on the water content (ω %).

Determination of soil wet density γ_h	Total wet weight (gr)	7610	7751	7740
	Mold weight (gr)	3920	3920	3830
	Mold volume (cm ^{3})	2104		

Determination of soil water content ω %	Tare number:	537	I7	A3	569	1	2
	Total wet weight (gr)	138.05	126.8	104.71	115.72	109.8	100.31
	Total dry weight (gr)	131.25	120.3	98.25	108.43	99.97	91.22
	Tare weight (gr)	18.63	18.75	19.39	18.22	18.27	18.71

Solution:

- Determination of the wet density of the soil γ_h:

$$* \; Poids\ du\ sol\ humide\ P_h \;=\; Poids\ total\ humide\ - Mold\ weight$$

$$Poids\ du\ sol\ humide\ P_h = 7610 - 3920 = 3690$$

$$^{**}\ Densité\ humide\ \gamma_h = \frac{P_h}{V} = \frac{3690}{2104} = \mathbf{1.75}$$

Weight of wet soil (gr)	3690	3831	3910
Wet density	**<u>1.75</u>**	**1.82**	**1.86**

- Determination of soil water content ω %:

$$\omega 1\ \% = \frac{P\omega}{P_S} = \frac{138{,}05 - 131{,}25}{131{,}25 - 18{,}63} = \frac{6{,}8}{112{,}62} = 6{,}04$$

$$\omega 2\ \% = \frac{P\omega}{P_S} = \frac{126{,}8 - 120{,}3}{[illegible]} = \frac{6{,}5}{101{,}55} = 6{,}40$$

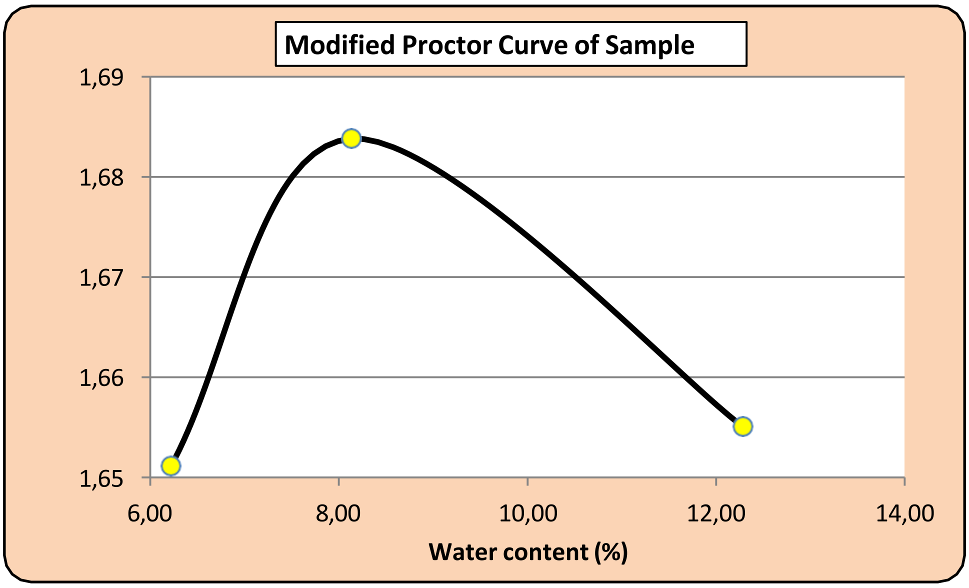
Average: $$w = \frac{w_1 + w_2}{2} = 6.22$$

Water weight (gr)	**6.8**	**6.5**	6.46	7.29	9.83	9.09
Dry soil weight (gr)	**112.62**	**101.55**	78.86	90.21	81.7	72.51
Water content (%)	**<u>6.04</u>**	**<u>6.40</u>**	8.19	8.08	12.03	12.54
Average (%)	**<u>6.22</u>**		**8.14**		**12.28**	

- Determination of the dry density of compacted soil γ_d:

$$\gamma_d = \frac{\gamma_h}{1+w} = \frac{1.75}{1+\frac{6.22}{100}} = 1.65$$

Water content (%)	**6.22**	**8.14**	**12.28**
Dry density	**<u>1.65</u>**	**1.68**	**1.66**


Modified Proctor Curve of Sample
1,69
1,68
1,67
1,66
1,65
6,00
8,00
10,00
12,00
14,00
Water content (%)

Chapter 4
Soil hydraulics

Chapter 4: Soil hydraulics

4 Introduction

Water has a major influence on soil behavior, especially in fine-grained soils. The presence of water in soils often leads to complications. Many engineering failures occur because water has not been properly considered, such as:

- settlements
- landslides
- swelling phenomena

Water plays a very important role in soil behavior in civil engineering and public works. Water is also an important factor in most geotechnical problems.

At the time of construction, water in the soil is often an undesirable element, which must be avoided (e.g. drying out a pit during construction), or permanently drained (e.g. an earth dam or a slope at the limit of stability).

This chapter presents the main characteristics of groundwater and studies the principal hydraulic properties of soils. When this water saturates a soil mass, it forms an underground water table, usually at free surface or sometimes located between two impermeable formations. The behavior of this water in the ground is governed by well-established and relatively simple laws.

The open water attracting a mass of land constitutes :

➢ An underground water table, usually with a free surface: this is the free water table or the water table.

This water occupies the porous medium of an impermeable level at the base of which the hydrostatic pressure is zero. When the surface of the water corresponds to the surface of the ground, the water table is said to be "superficial".

➢ A water table located between impermeable formations: this is the captive water table. waterproof.

Here, the water-filled porous level is isolated between two much less permeable levels (upper and lower).

The piezometric level of the water table is the level reached by the water in a borehole passing through the water table over its entire height (piezometer). All the piezometric levels of a water table together define its "piezometric surface". In the case of a captive water table, the height of a piezometer is higher than that of the water table's impermeable roof, and the water is said to be "charged". In the case of a captive water table, a piezometer's elevation is higher than the impermeable roof of the water table, and the water is referred to as "head". If this head is high, the water may gush to the surface through a borehole, and the water table is referred to as "artesian" or "artesian".

4.1 Darcy's law

The first experiments on permeability were carried out by the researcher Darcy in 1854, who observed that flow per unit area is proportional to head loss and inversely proportional to height.

4.1.1 Basic assumptions

The classic study water flow in soil is based on the following assumptions:

- Interstitial water is incompressible, as are the grains, which are also incompressible.
- Continuity of the liquid phase: the volume entering is equal to the volume leaving through a soil sample.

In other words: $V1=V2$

- The soil is assumed to be completely saturated.
- The water circulating between the grains has viscosity (Newton's fluid), which leads to pressure drops. A perfect fluid has no pressure drop.
- The effect of gravity is taken into account.
- Water flow is said to be permanent, meaning that water velocities at different points on the ground are independent of time.

4.1.2 Water velocity in soil

As water circulates through the soil, it seeps into the interstices between the grains, pores of varying dimensions. The actual water velocity is therefore variable, and cannot be determined in relation to the complexity of the soil texture.

On the other hand, an apparent velocity v_a is defined as follows:

$$v_a = \frac{q}{S}$$

where :

q : flow rate expressed in [m^3 / s].

$$V = S.H$$
$$V_v = S_v.H$$

V and V_v are total volume and void volume respectively.

$$\eta = \frac{V_v}{V} = \frac{S_v.H}{S.H} = \frac{S_v}{S}$$

$$\Rightarrow S_v = \eta.S$$

Assuming that the water flows only in the voids, we define a true mean velocity v' :

$$v' = \frac{q}{S_v} = \frac{q}{\eta.S} = \frac{V}{\eta}$$

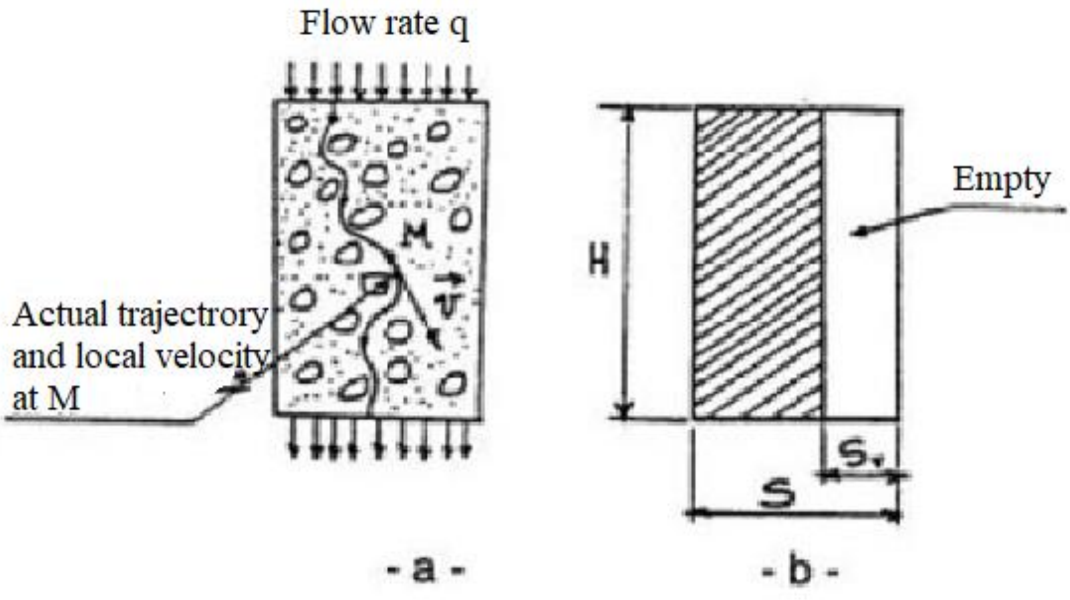


Figure 4. 1. Definition of speed.

4.1.3 Hydraulic load

In hydrodynamics, the term hydraulic load is defined by Bernoulli's law:

$$h = \frac{v^2}{2g} + \frac{U}{\gamma_w} + z$$

with :

v : Velocity of the fluid particle

U: Water pore pressure

z : Particle altitude

g: : The acceleration of gravity

γ_w : The density of the liquid.

The hydraulic load is defined to within one constant.

4.1.4 Hydraulic gradient

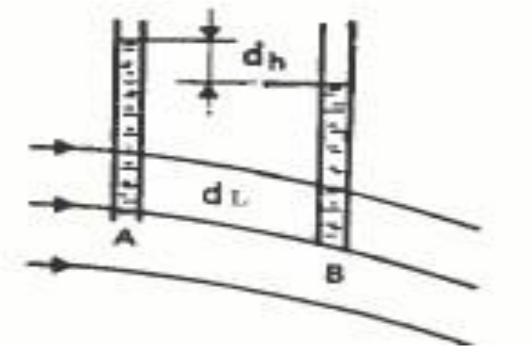


Figure 4. 2. Hydraulic gradient.

The hydraulic gradient i is the variation in load per unit length travelled and is defined by the following ratio between two points A and B :

$$i = \frac{h_A - h_B}{L}$$

$$i = -\frac{dh}{dl} \quad \text{if} \quad i > 0 \qquad (dh < 0)$$

with dh: load variation and dl: distance travelled.

Example:

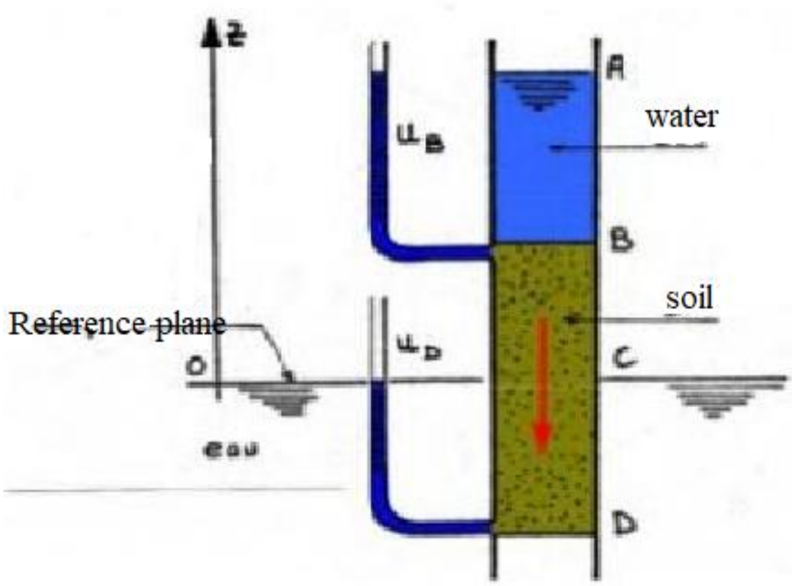


Figure 4. 3. Exemple Hydraulic gradient

Hydraulic loads: At point B (inlet) :

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

At point D (exit) :

$$h_D = \frac{U_D}{\gamma_w} + z_D = CD + (-CD) = 0$$

Between points B and D, there is a head loss The $h_D - h_B = -AC$

hydraulic gradient "i"

$$i = -\frac{h_D - h_B}{BD} = -\frac{(-AC)}{BD} = \frac{AC}{BD}$$

In the particular case of water filtration in soils, flow are very low: For a speed v= 10 cm /s, which is never reached in practice, the value of $\frac{v^2}{2g}$ is only 0.5 mm. As a result, when expressing the hydraulic head, we neglect the term $\frac{v^2}{2g}$ from which :

$$h = \frac{U}{\gamma_w} + z$$

4.1.5 Darcy's Law

DARCY'S LAW is the fundamental law of soil hydraulics, established experimentally by Darcy in 1854, demonstrating that the flow velocity of pore water is proportional to the hydraulic gradient "i" :

$$v = k.i$$

With

k: the permeability coefficient of the material with the velocity dimension. It is expressed in m / s or cm / s .

Table 4.1 gives an approximate range of values for the permeability coefficient k, depending on the type of soil.

Table 4. 1.k permeability coefficient values for different types of soil.

Floors	**Permeability coefficient** cm / s
Gravel	$10^{-1} < k < 10^{2}$
Sand	$10^{-3} < k < 10^{-1}$
Silt and clayey sand	$10^{-7} < k < 10^{-3}$
Clay	$10^{-11} < k < 10^{-7}$
Apparently uncracked rock	$10^{-10} < k < 10^{-8}$

$10^{-6}\, cm / s$ corresponding to a speed of 30 cm per year**.**

k depends on the granulometry of the material, the shape of the grains, their size and their assembly. Rounded grains have a higher permeability than angular ones.

4.1.6 Laboratory permeability determination

Soil permeability (hydraulic conductivity) is the rate at which wate**r flows through soil materials.** It is an essential characteristic across a broad spectrum o**f engineering and earth-science** disciplines. The coefficient of permeability (k) is a constant of proportionality relating to the ease with which fluid passes through a porous medium.

Two general types of permeability test methods are routinely performed in the laboratory: (1) the constant head test method, and (2) the falling head test method. The constant head test method is used for cohesionless and more permeable soils (k>10-4 cm/s) and the falling head test is mainly used for cohesive or less permeable soils (k<10-4 cm/s).

4.1.6.1 Constant load permeability

The **constant head permeability test**, shown in Figure 4.4, is suitable for highly permeable soils such as sands. This test must satisfy the following conditions:

- The soil specimen must contain no more than 10% of particles smaller than 80 µm, and 0% of particles larger than 20 mm.
- Water flow through the soil specimen must be laminar and steady, so that the flow velocity remains proportional to the hydraulic gradient.
- The soil specimen must be fully saturated and must not undergo any volume change during the test.
- The head loss (Δh) must remain constant throughout the test.

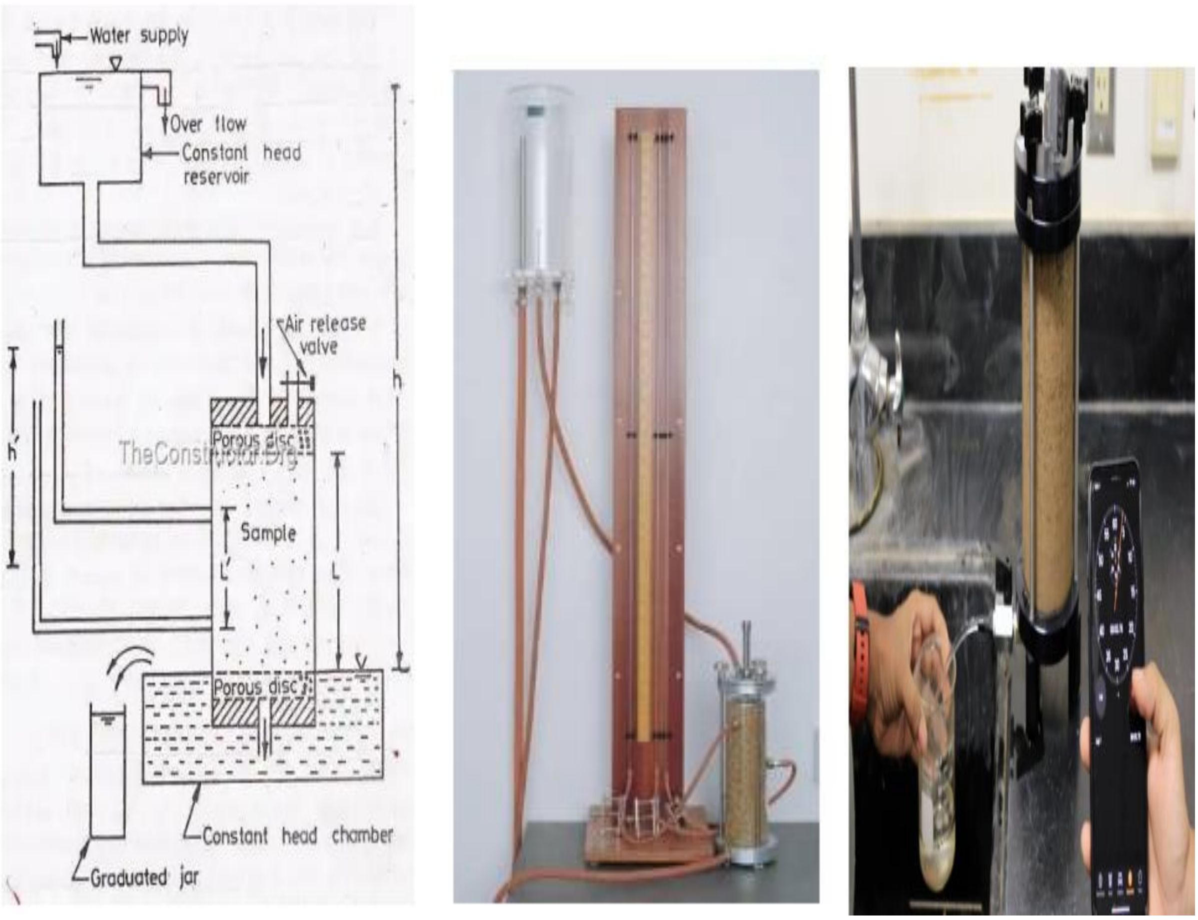


Figure 4. 4. Permeability at constant load.

In this case, the materials are quite permeable.

1. Place the floor in the closed mold.
2. Close both ends of the mold with the two lids.
3. The upper cover allows water to drain off, while the lower cover is connected to a tank and a tube;
4. Bringing the sample to saturation, the mold is filled with water and connected to the graduated tube. In this case, the graduated tube is disconnected and a constant level is maintained in the mould.

   reservoir. The hydraulic gradient i is therefore constant. The discharge velocity is determined by measuring the volume water passing through a sample in a given time.

$$q = v.S = k.i.S = k.\frac{h}{L}.S$$

$$\text{Hence:} \quad k = \frac{Q \times L}{Ath}$$

Where,

k = coefficient of permeability at temperature T, cm/sec.

L = length of the specimen in centimeters

t = time for discharge in seconds

Q = volume of discharge in cm^3 (assume 1 mL = 1 cm^3)

S= cross-sectional area of permeameter

h = hydraulic head difference across length L, in cm of water;

4.1.6.2 Variable load permeability

The soil sample studied is placed in a cylindrical mold between two porous stones to prevent grain displacement, and closed at both ends by lids.

The top cover allows water to drain off. The lower lid is connected to a reservoir and a tube. Bring the sample to saturation, fill the mold with water and connect it to the graduated tube; note that the level drops in the tube. The time "t" between levels h1 and h2 is then measured. These two levels are marked in relation to the level of the sotie fitting.

If "S" is the cross-sectional area of the sample and "s" is the cross-sectional area of the graduated tube, then the flow rate q is equal to :

$$q = S \times V$$

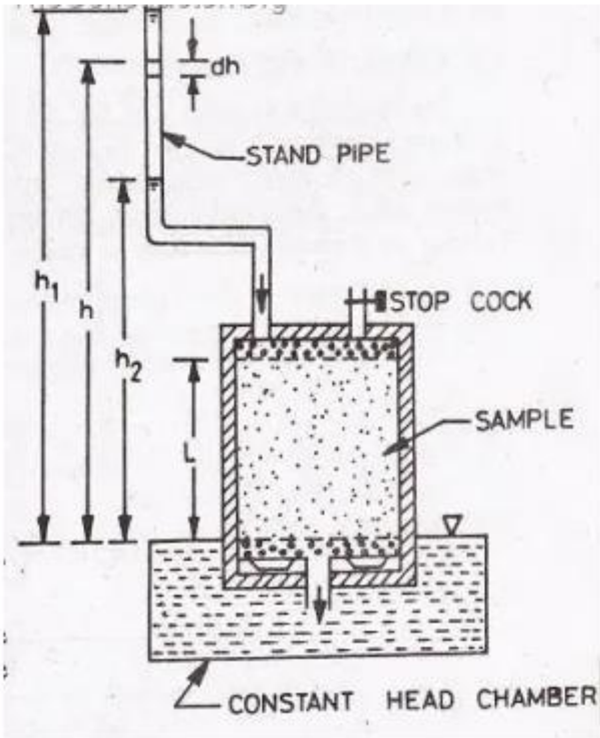


Figure 4. 5. Variable load permeability

Taking permeability into account, the expression for flow rates thus becomes :

$$q = S \times V = k.S.i = \frac{k.S.h}{L}$$

with :
V: flow in [m/s].

S: cross-sectional area of flow in [m²] ;

k: Darcy permeability in [m³/sec] ;

i: hydraulic gradient.

The volume of water passing through the sample is equal to the decrease in the volume of water in the tube:

$$q.dt = -s.dh$$

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

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

after integration, we get :

$$k = \frac{s.L}{S.t}\ln\frac{h_1}{h_2} = \frac{2}{3}\frac{s.L}{S.t}\log\frac{h_1}{h_2}$$

4.1.6.3 In-situ permeability measurement

Permeability measured in the laboratory on soil samples is not representative of reality in the field. In , soil in situ is locally heterogeneous (presence rocks, cavities…..). The permeability measured in the laboratory is always lower than that measured in situ.

In the field, this is achieved by pumping a well drilled under steady flow conditions.

4.1.6.3.1 Dupuit test (NFP 94-130 standard)

Consider a permeable cylindrical soil mass of radius R. The mass rests on an impermeable layer. A well is drilled in the center of the bed. Pumping at flow rate Q causes the water table to fall, as shown in the diagram below. The water table is located at H at the ends of the water table and h in the well.

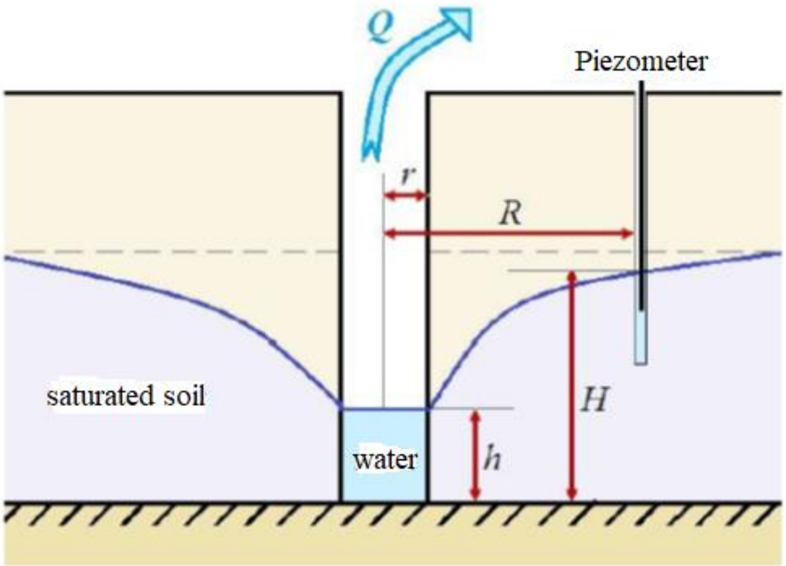


Figure 4. 6. Dupuit test.

In this drawdown test, the permeability coefficient is obtained using the Dupuit formula:

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

Heights h and H are measured in relation to the impermeable bedrock.

4.1.6.4 Permeability of a floor.

When a soil is stratified (i.e. made up of several layers of different types), we define average horizontal permeability coefficients k_H and vertical permeability coefficients k_V

a. Flow parallel to the stratification plane

k_h be the permeability coefficient of the fictitious homogeneous soil

Expressing that :

The head loss is the same for all layers (so the hydraulic gradient i is also the same). The total flow is the sum of the flows in each layer.

We show that we have: $k_h = \frac{1}{H}.\sum_{i=1}^{n} k_i.H_i$

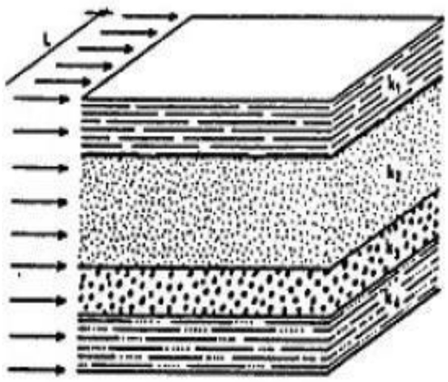


Figure 4. 7. Parallel flow.

b.Flow perpendicular to the stratification plane

Let k_v be the permeability coefficient of the fictitious homogeneous soil. Expressing that :

The total pressure drop is the sum of the pressure drops in each layer.

The flow rate is the same for all layers (so the discharge velocity v is also the same).

We show that we have: $k_v = \frac{H}{\sum_{i=1}^{n} \frac{H_i}{k_i}}$

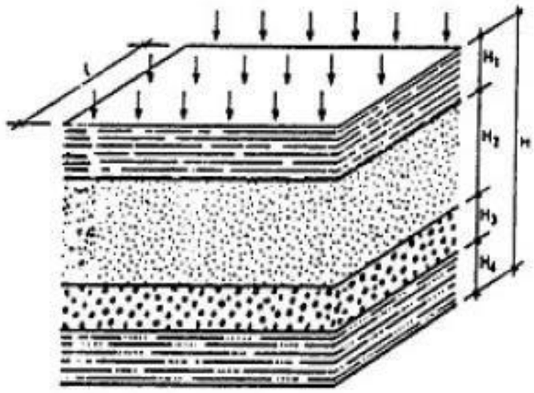


Figure 4. 8. Flow perpendicular to the stratification plane.

4.2 Relationship between the permeability coefficient and certain physical characteristics

Hazen proposed a very simple empirical formula using the effective diameter d_{10} (diameter of the sieve allowing only 10% of the material to pass through).

$$K = 100 d_{10}^2 \quad \text{(cm/s)}$$

This relationship is only valid sand of medium compactness.

Casagrande studied the influence of porosity on the permeability coefficient and gave the following empirical relationship:

$$K = 1{,}4.K_{0.85}.e^2$$

with

$K_{0.85}$: Permeability coefficient when void index is equal to e=0.85 A more complex formula is given by **Terzagui** :

$$K = \frac{C}{\eta}.\frac{(n-0.13)^2}{(\sqrt[3]{1-n})}.d_{10}^2$$

C: Coefficient dependent on uniformity, grain size and shape.

η : Viscosity of the liquid

n : Porosity

d_{10}: Effective diameter.

4.3 Flow : effective stress principle

presence of water in voids influences their mechanical properties. Stresses are transmitted into the soil through the grains and interstitial water. The overall behavior saturated soil is due to the interaction of the mechanical behavior of the liquid and solid phases.

4.3.1 TERZAGUI's postulate

If we apply compression to a soil specimen, stresses are generated and distributed between the solid skeleton and the water. In water, stresses are reduced to interstitial pressure. In the grains, there will be total stresses, as part of the stress is taken up by the water.

$$\sigma^{'} = \sigma_t - U$$
$$\tau^{'} = \tau$$

where: σ and τ are the total normal and tangential stresses respectively.

σ' and' are the effective normal and tangential stresses respectively. and *U* is the fluid pore pressure.

The total normal stress is: $\sigma_t = h.\gamma_{sat}$ if the soil is saturated

Or $\sigma_t = h.\gamma_h$ if the soil is damp

The total effective stress is: $\sigma^{'} = h.\gamma^{'}$ ($\gamma^{'}$ discharged density)

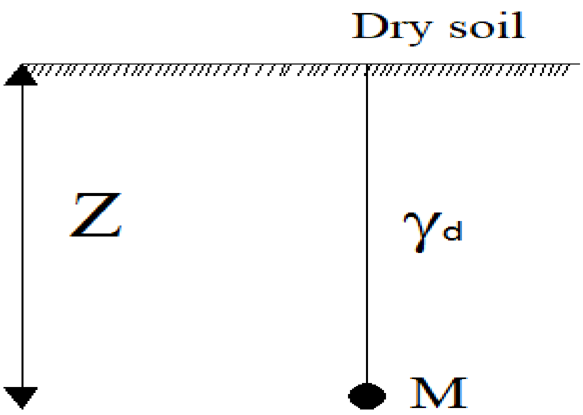
Water pressure is: $U = h.\gamma_w$

As in : $\gamma_{sat} = \gamma^{'} + \gamma_w \Rightarrow \sigma_t = \sigma^{'} + U$

So we have: $\sigma^{'} = \sigma_t - U$

4.4 Examples of the principle of effective constraints

4.4.1 Perfectly dry soil

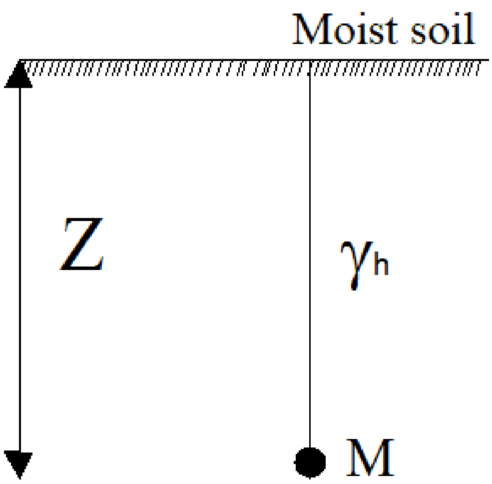


$\sigma_t = \sigma' + U$ and $\sigma' = \sigma_t - U$

$\sigma_t = \gamma_d . z$

$U = \gamma_w . z = 0$ (dry soil) so $\sigma' = \sigma_t = \gamma_d . z$

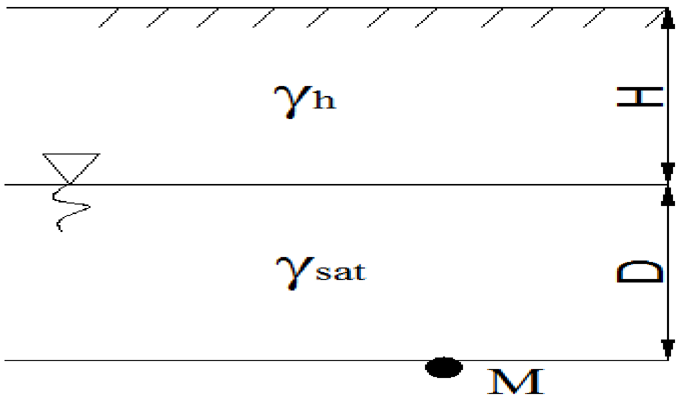
4.4.2 Moist but not saturated soil



$$\sigma_t = \gamma_h . z \qquad \sigma' = \sigma_t - U$$

$$U = \gamma_w . z = 0 \qquad \Rightarrow \sigma' = \sigma_t = \gamma_h . z$$

4.4.3 Presence of a water table



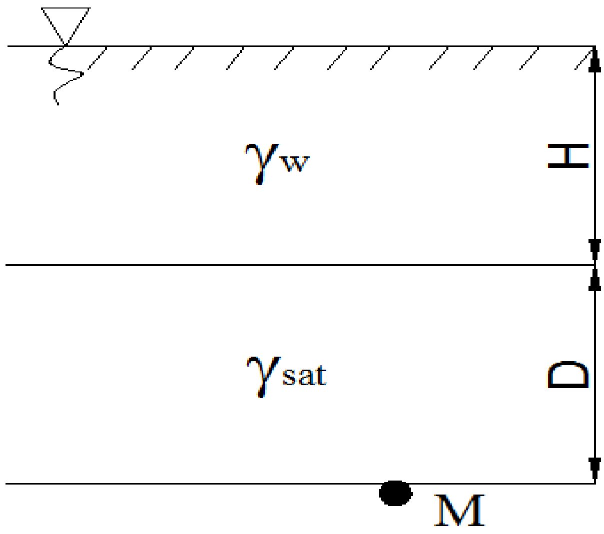
$$\sigma_t = \sigma' + U$$

$\sigma_t = \gamma_h . H + \gamma_{sat} . D$ et $U = \gamma_w . D$

$$\sigma' = \gamma_h . H + \gamma_{sat} . D - \gamma_w . D = \gamma_h . H + (\gamma_{sat} - \gamma_w) . D$$

$$\sigma' = \gamma_h . H + \gamma' . D$$

4.4.4 Soil submerged by water



$\sigma_t = \gamma_h.H + \gamma_{sat}.D$ et $U = \gamma_w.(H + D)$

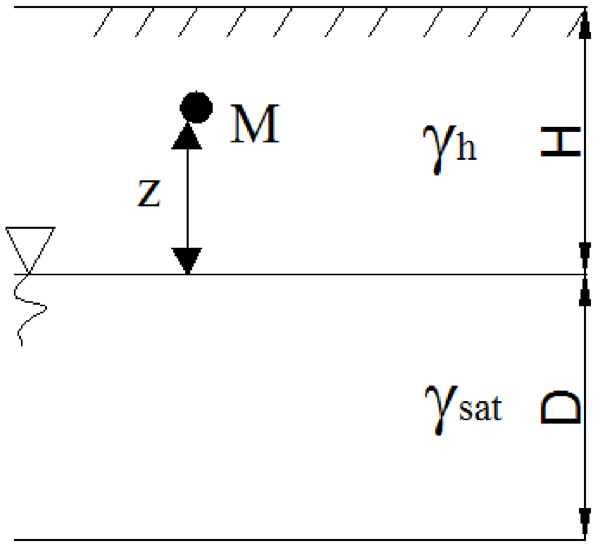
$\sigma' = \gamma_w.H + \gamma_{sat}.D - \gamma_w.H - \gamma_w.D = (\gamma_{sat} - \gamma_w).D$

$\sigma' = \gamma'.D$

Remarque :

U : pore presure

$U_M \succ 0$ if point M is in watertable



$U = \gamma_w.D$ et $\sigma_t = \gamma_h.H + \gamma_{sat}.D$

$\sigma' = \gamma_h.H + \gamma_{sat}.D - \gamma_w.D$

Or again :

$\sigma' = \gamma_h.H + \gamma'.D$

$U_M \prec 0$ if water is capillary.

$U = -\gamma_w.Z$ et $\sigma_t = \gamma_h.H + \gamma_{sat}.D$

$\sigma' = \gamma_h.H + \gamma_{sat}.D - (-\gamma_w.Z)$

Or again :

$\sigma' = \gamma_h.H + \gamma_{sat}.D + \gamma_w.Z$

4.5 Boulance Effect

Under the effect of water circulation, soil particles can migrate to a soil are a coarser. This phenomenon is called boulance.

It can be avoided by making filters consisting of layers of permeable materials of intermediate grain size, or of sheets of suitable geo-synthetics. They are chosen so as to allow the water to flow without entraining particles.

4.6 Fox Effect

Water infiltration under a structure (dam, construction, sheet pile wall, etc.) can cause a phenomenon similar to the phenomenon of boulance when the observed hydraulic gradient reaches a certain critical value. Initially, there is an increase in the flow rate with a progressive entrainment of the fine elements of the soil and very quickly, a general entrainment of the various materials constituting the environment. A privileged water circulation path is then formed through which the arrival of water quickly takes on a catastrophic appearance and which is called Renard. This is therefore characterized by the speed of its formation which can be the cause of significant disorders for all works located nearby (dam failure, building cracking, sheet pile wall overturning, etc.).

The phenomenon of bubbling appears in the case of an upward vertical flow. In the general case of a vertical or non-vertical flow, in a permeable environment, the water can locally reach high speeds likely to entrain fine particles from the soil.

As a result, the soil being made locally more permeable, the speed of the water increases and the phenomenon is amplified. Larger elements will be carried along, while erosion will progress regressively (from downstream to upstream) along a current line. A conduit is formed through which the water rushes in and completely disorganizes the soil. This is the fox phenomenon.

Application
Exercise 1:
Determine the average horizontal and vertical permeability of a stratified soil mass composed of two distinct and superposed clay layers (Soil 1 and Soil 2).

Permeability tests were carried out on samples taken from both layers. Each sample has a length of 30 cm and a diameter of 15 cm. During the test, the hydraulic head decreases from 100 cm to 20 cm.

It is given that a volume of 200 mL is collected within:

- 2 hours for Soil 1,
- 4 hours for Soil 2.

The cross-sectional area of the standpipe is 4 cm².

The thicknesses of the two soil layers are:

H_1=3m H_2=7m

Questions

1. State the type of permeability test (constant head or falling head).
2. Calculate the permeability coefficient kkk for each soil.
3. Determine the average horizontal permeability.
4. Determine the average vertical permeability.

Solution :

1) **Type of Test** Falling-head permeability test (variable head test).
2) **Permeability Calculation**
3) Permeability is calculated using the falling-head test formula.:$k = 2{,}3\frac{a}{A}\frac{l}{T}log\frac{h1}{h2}$

A=πx(7,5²)=176,71 cm².

Soil 1 : $k_1 = 2{,}3\frac{4}{176{,}71}\frac{30}{7200}log\frac{100}{20} = 1{,}516x10^{-4}\ cm/s$

Soil 2 : $k_2 = 2{,}3\frac{4}{176{,}71}\frac{30}{14400}log\frac{100}{20} = 0{,}758x10^{-4}\ cm/s$

4) Determination of the Average Horizontal Permeability KhK_hKh
The average horizontal permeability of a layered soil mass is given by the **weighted arithmetic mean** of the permeabilities of the individual layers:

$$K_h = \frac{\sum K_i H_i}{H} = \frac{(1{,}516x10^{-4}x300) + (0{,}758x10^{-4}x700)}{(300 + 700)}$$

$$= 0{,}985x10^{-4}cm/$$

4) Determination of the Average Vertical Permeability K_v :

The average vertical permeability of a layered soil mass is given by the **harmonic mean** of the permeabilities of the individual layers:

$$K_v = \frac{H}{\sum \frac{H_i}{K_i}} = \frac{1000}{\frac{300}{(1.516x10^{-4})} + \frac{700}{(0,758x10^{-4})}} = 0,892x10^{-4} cm/s$$

<u>Exercise 2:</u>

Constant Head Permeability Test (Sample 1)

A soil sample with a length of 15 cm and a diameter of 8 cm is tested using a constant head permeameter. The head is maintained at 30 cm. It takes 2 min 35 s to collect 200 mL of water. Determine the permeability coefficient and classify the soil.

<u>Solution :</u>

Flow rate:

$q=V/t=250/155 \Rightarrow q=1,29\ cm^3/s$

Permeability coefficient:

$k=qL/Ah=(1,29x15)/(\pi.4^2\ x30) \Rightarrow k=0,0128cm/s=0,13x10^{-1}\ cm/s$

Soil classification: Very permeable soil.

<u>Exercise 3:</u>

A constant head permeability test is performed on a soil sample 22 cm long and 10 cm in diameter. The hydraulic head is maintained at 30 cm. A quarter of the container (800 mL total) is filled after 3 min 20 s.

Questions:

1. Calculate the collected water flow rate.
2. Determine the hydraulic gradient.
3. Calculate the soil permeability kkk.
4. Classify the soil

<u>Solution :</u>

Flow rate:

$q=V/t=0,200.10^{-3}/200 \Rightarrow q=10^{-6}\ m^3/s$

Hydraulic gradient:

i=h/L =30/22 i=1,36

Permeability coefficient:

$k=qL/Ah=(10^{-6}x0,22)/(\pi.0,05^2\,x0,30)\Rightarrow k=9,34.10^{-5}$ m/s

$k=9,34.10^{-7}$ cm/s.

Soil classification: From standard tables:
$10^{-3}<9,34.10^{-7}<10^{-7}\Rightarrow$Silt and clayey sand.

Exercise 4:
A soil specimen has a **height of 25 cm**, a **width of 10 cm**, and a **length of 12 cm**. It is subjected to a **constant hydraulic head of 60 cm**. During a permeability test, a volume of **1.08 liters** of water flows through the specimen in **90 seconds**.

Calculate the coefficient of permeability k in cm/s.

Solution :

Given Data

- Height (flow length): L=25 cm
- Width: b=10 cm
- Length: l=12 cm
- Hydraulic head: h=60 cm
- Water volume collected: Q=1.08 L=1080cm^3
- Time: t=90 s

Calculate the cross-sectional area

$$A = b \times l$$
$$A = 10 \times 12 = 120 cm^3$$
$$Q = k.A.\frac{h}{L}.t$$
$$k = \frac{QL}{Aht} = \frac{1080 \times 25}{120 \times 60 \times 90}$$
$$k = 4.17 \times 10^{-2} cm^2 / s$$

Exercise 5:

Three layers of different soils are stacked horizontally, each having the same thickness. Their coefficients of permeability are respectively:

$2\times10^{-3}\, cm/s$,$10^{-2}\ cm/s$,$6\times10^{-3}\, cm/s$

Calculate the **equivalent horizontal coefficient of permeability** and the **equivalent vertical coefficient of permeability** of the entire soil deposit.

Solution :

Equivalent horizontal coefficient of permeability

$$\frac{\sum_{1}^{n} h_i}{k_v} = \sum_{1}^{n}\frac{h_i}{k_i}$$

$$\frac{\sum_{1}^{3} h_i}{k_v} = \sum_{1}^{3}\frac{h_i}{k_i}$$

$$\frac{3h}{k_v} = \frac{h}{k_1}+\frac{h}{k_2}+\frac{h}{k_3}$$

$$k_v = \frac{3}{\frac{1}{2.10^{-3}}+\frac{1}{10^{-2}}+\frac{1}{6.10^{-3}}}$$

$$k_v = 0.39\times10^{-2}$$

$$k_h\sum_{1}^{n} h_i = \sum_{1}^{n} k_i h_i$$

$$k_h\sum_{1}^{3} h_i = \sum_{1}^{3} k_i h_i$$

$$k_h(h_1+h_2+h_3) = k_1h_1+k_2h_2+k_3h_3$$

$$h_1 = h_2 = h_3 = h$$

$$3k_h.h = h(k_1+k_2+k_3)$$

$$k_h = \frac{k_1+k_2+k_3}{3} = \frac{2.10^{-3}+10^{-3}+6.10^{-3}}{3}$$

$$k_h = 6\times10^{-3}\ cm/s$$

Equivalent vertical coefficient of permeability

$$\frac{\sum_{1}^{n} h_i}{k_v} = \sum_{1}^{n}\frac{h_i}{k_i}$$

$$\frac{\sum_{1}^{3} h_i}{k_v} = \sum_{1}^{3}\frac{h_i}{k_i}$$

$$\frac{3h}{k_v} = \frac{k}{h_1}+\frac{k}{h_2}+\frac{k}{h_3}$$

$$k_v = \frac{3}{\frac{1}{2.10^{-3}}+\frac{1}{10^{-2}}+\frac{1}{6\times10^{-3}}}$$

$$k_v = 0.39\times10^{-2}$$

Exercise 6:

A **falling-head permeability test** was performed on a **sandy clay** specimen. The height of the soil sample is **15 cm**, the cross-sectional area of the soil specimen is **25 cm²**, and the cross-sectional area of the standpipe is **2.3 cm²**.

The water head measured at the beginning of the test and after **24 hours** were **10.4 cm** and **7.02 cm**, respectively.

Determine the coefficient of permeability of the soil.

Solution :

$$k = \frac{a.l}{A.\Delta t} \ln \frac{h_2}{h_1}$$

$$k = \frac{2,3 \times 15}{25 \times 24 \times 3600} \ln(\frac{10,4}{7,02})$$

$$k = 6,278 \times 10^{-6} cm/s$$

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