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MASTER THESIS

# THE INFLUENCE OF BENTONITE TREATMENT ON SOIL HYDRAULIC PROPERTIES: APPLICATION TO THE CAP COVER OF A FRENCH DISPOSAL FOR NUCLEAR WASTE OF LOW ACTIVITY

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# TABLE OF CONTENTS

ABSTRACT	9
CHAPTER 1	
RADIOACTIVE WASTE MANAGEMENT AND CAP COVER SYSTEM	
IN NEAR SURFACE DISPOSAL	11
1.1 RADIOACTIVE DECAY	11
1.2 CLASSIFICATION OF RADIOACTIVE WASTE	14
1.2.1 Classification in France	18
1.2.2 Classification in Italy	19
1.2.3 Classification in the United States	20
1.2.4 Classification in Japan	21
1.3 DISPOSAL FACILITIES	22
1.3.1 Objectives and safety means of disposal facilities	22
1.3.2 Disposal methods	23
1.4 CAP COVER SYSTEM IN NEAR SURFACE DISPOSAL	25
1.4.1 Cap cover system functions and composition	26
1.4.2 Cap cover system for radioactive waste in France	28
1.4.3 Clay layer compaction requirements	29
CHAPTER 2	
SATURATED AND UNSATURATED HYDRAULIC CONDUCTIVITY	33
2.1 SATURATED AND UNSATURATED SOILS	33
2.1.1 Water content and degree of saturation	35
2.1.2 Pore-water pressure in saturated media	36
2.1.3 Adsorption and capillarity in soils	37
2.1.4 Suction definition	40
2.1.5 Conclusions on pore-water pressure in saturated and unsaturated media	43
2.2 SATURATED HYDRAULIC CONDUCTIVITY	44

2.2.1 Definition hydraulic conductivity for saturated media	44
2.2.2 Permeability and Hydraulic conductivity	46
2.2.3 Methods for measuring K <sub>SAT</sub>	47
2.3 UNSATURATED HYDRAULIC CONDUCTIVITY	49
2.3.1 Definition of hydraulic conductivity for unsaturated media	49
2.3.2 The Soil Water Retention Curve (SWRC)	50
2.3.3 The Van Genuchten model	53
CHAPTER 3	
BENTONITE TREATMENTS FOR DISPOSAL PASSIVE BARRIER	55
3.1 CLAYS COMPOSITION AND PROPERTIES	55
3.1.1 Bentonite characteristics	57
3.2 EFFECT OF BENTONITE TREATMENTS ON SOIL PROPERTIES	58
3.2.1 Atterberg's limits	58
3.2.2 Methylene blue adsorption test	60
3.2.3 Compaction properties	61
3.2.4 Hydraulic conductivity	64
3.3 CONCLUSIONS ON BENTONITE TREATMENTS	68
CHAPTER 4	
THE CSM, THE CHARACTERISATION OF THE SOILS OF THE CSM	
AND THEIR BENTONITE TREATMENTS	69
4.1 THE CENTRE DE STOCKAGE DE LA MANCHE (CSM)	69
4.1.1 History of the CSM	
4.1.2 The cap cover system of the CSM	72
4.1.3 The long term cap cover system	74
4.2 THE TYPE MATERIALS AND THEIR TREATMENTS	75
4.2.1 Definition of the type materials: T12 and T3	75
4.2.2 The bentonite treatments	76
4.3 CLASSIFICATION OF THE SOILS	76
4.3.1 Granulometric curves	77

4.3.2 Methylene blue adsorption test (VBS)	80
4.3.3 Atterberg limits	82
4.3.4 Classification of the samples	86
4.4 SOLID UNIT WEIGHT	88
4.5 PROCTOR TEST	89
4.6 SUMMARY OF THE SAMPLES CHARACTERISATION	
CHAPTER 5	
INFLUENCE OF THE TREATMENTS ON HYDRAULIC CONDUCTIVITY	
IN SATURATED CONDITIONS	
5.1 Oedo-permeameter test	
5.1.1 Samples preparation	
5.1.2 Description of the method	
5.1.3 Results	
5.2 Correlations between $K_{\text{SAT}}$ and soil properties	100
5.2.1 Untreated soils	101
5.2.2 Treated soils	104
5.3 SUMMARY OF RESULTS	107
CHAPTER 6	
INFLUENCE OF BENTONITE TREATMENTS ON SOIL	
IN UNSATURATED CONDITIONS	109
6.1 DESCRIPTION OF THE HANGING COLUMN TEST	109
6.2 PREPARATION OF THE SAMPLES	113
6.2.1 Compaction with Proctor procedure	115
6.2.2 Retrieving of the sample with normal compression	116
6.2.3 Measure of samples mass, height and volume	117
6.3 PRELIMINARIES TO THE TEST	118
6.3.1 Calculation of samples dry masses and dry unit weights	119
6.3.2 Calculation of water content parameters	119
6.4 THE TEST PROCESS	120

6.4.1 Preparation of the sandbox	120
6.4.2 Saturation phase	122
6.4.3 Suction phase	123
6.4.4 Temperature and humidity survey	124
6.4.5 Dry samples masses and filters masses	125
6.5 PROBLEMS ENCOUNTERED AND CORRECTIONS	125
6.5.1 Reference level for the zero suction	126
6.5.2 Overestimation of water content	126
6.5.3 Assessment of filter mass during the experiment	127
6.5.4 Swelling of the samples	128
6.5.5 Fine material loss from the samples	129
6.5.6 Presence of mushrooms on the sample upper surface	130
6.5.7 Calculation of the degree of saturation S	130
6.6 APPLICATION OF THE VAN GENUCHTEN MODEL	133
6.6.1 The experimental Soil Water Retention Curve	134
6.6.2 The Van Genuchten model parameters	135
6.7 RESULTS	137
6.7.1 Estimation of the solid unit weight	138
6.7.2 Soil Water Retention Curves	140
6.7.3 Observations on swelling and shrinkage	142
6.7.4 Estimation of the hydraulic conductivity	145
6.8 CONCLUSIONS ON THE HANGING COLUMN TEST	149
CHAPTER 7	
SOILS SENSITIVITY TO EROSION	151
7.1 PRESENTATION OF THE PROBLEM	151
7.1.1 Possible impact on the behaviour of the landfill soil cover	153
7.2 INTERNAL AND EXTERNAL EROSION	154
7.3 INTERNAL STABILIY GEOMETRIC CRITERIA	155

7.3.1 Kezdi's criterion	156
7.3.2 Kenney and Lau's criterion	158
7.3.3 Burenkova's criterion	160
7.3.4 Liu's criterion	162
7.3.5 Mao's criterion	163
7.3.6 Conclusions on the internal stability criteria	165
7.4 EXTERNAL EROSION CRITERIA	166
7.4.1 Analysis of the untreated materials	167
7.4.2 Effect of bentonite treatments	169
7.4.3 Conclusions on the external erosion criteria	171
7.5 CONCLUSIONS ON SOILS SENSITIVITY TO EROSION	172
CONCLUSIONS	173
ATTACHMENT	175
BIBLIOGRAPHY	189
ACKNOWLEDGEMENTS	197

## ABSTRACT

This work focuses on the influence of bentonite treatment on the properties of two soils (called T12 and T3) used to realise the clay layer of the cap-cover system of a French near-surface disposal facility for nuclear waste of low radioactivity: the *Centre de Stockage de la Manche* (CSM).

The main aim of the bentonite treatment is to reduce the hydraulic conductivity of the clay layer in order to provide a long-term impervious barrier.

In chapter 1, the waste management for radioactive waste will be introduced referring in particular to the documents provided by the International Atomic Energy Agency (IAEA). After presenting the radioactivity problem in waste facilities, the radioactive waste classification and the possible disposal methods will be presented. The dissertation will focus on the characteristics required by the cap-cover system of near-surface disposals. In chapter 2, the parameters used to define soils in unsaturated and saturated conditions

will be presented: water content measures, capillarity, pore-water pressure and suction. In particular, the differences between hydraulic conductivity in saturated and unsaturated conditions will be discussed.

In chapter 3, after defining clay and bentonite properties, the dissertation will focus on the effect of bentonite treatments on the soil properties. Therefore, some studies about the bentonite influence on Atterberg limits, VBS value, compaction characteristics and hydraulic conductivity will be considered.

In chapter 4, the CSM and the soils (T12 and T3) to be analysed and treated will be presented. The tests, performed to characterise and classify the raw soils and the 5%, 7%, 12% treatments, will be presented.

In chapter 5, the hydraulic conductivity in saturated conditions will be calculated with the rigid wall oedo-permeameter with variable hydraulic charge and some empirical correlations. The discussion will focus on the differences between the different approaches and on the effect of bentonite on the hydraulic conductivity decrease.

In chapter 6, the unsaturated conditions will be considered. In order to build the Soil Water Retention Curve (SWRC) the hanging column test and the Van Genuchten model will be

used. The Van Genuchten model will be applied also to obtain the variation of hydraulic conductivity with suction and, hence, of the water content.

In chapter 7, the effect of bentonite content on the soil sensitivity to erosion will be analysed. The urge to analyse this aspect is due to the fine particles loss observed during the hanging column test. Internal and external erosion criteria will be applied in order to understand the nature of the phenomenon.

# CHAPTER 1 Radioactive waste management and Cap Cover System in near surface disposal

In this chapter, the regulations about radioactive waste management will be presented. After introducing the radioactive decay process (paragraph 1.1), the radioactive waste classification (paragraph 1.2) and its final disposal (paragraph 1.3) will be described, referring to the standards provided by the International Atomic Energy Agency (IAEA). Since the material object of study in the following chapter comes from a French near surface disposal, in paragraph 1.3 the dissertation will focus on these kind of facilities referring, in particular, to the characterisation of the cap cover system.

## **1.1 RADIOACTIVE DECAY**

The term radioactive decay (or radioactivity) is used to define all the atomic and nuclear processes through which an instable atomic nucleus decays into a lower energy nucleus in order to achieve stability (IAEA, 2009b). This process always takes place with a release of energy in the form of radiations (atomic particles) implying, sometimes, chemical transformation and charge loss (Capecchi, 2013).

It is possible to distinguish four groups of radiations (Knoll, 2010):

- 1. Fast electrons (β rays),
- 2. Heavy charged particles (α rays, protons, fission products),
- 3. Electromagnetical radiation (X rays and γ rays),
- 4. Neutrons.

Each category is characterised by different properties and degree of danger that are summarised in table 1.1 (Department of the Army, 1993).

Name	What is it?	Source	Energy and Speed	Range in Air	Range in Tissue	Shielding Required	Biological Hazard
α Alpha particle	Helium Nucleus	Decay of Uranium and plutonium	Energy varies: speed varies from 1/20 to 1/10 speed of light	5 cm	Cannot penetrate the epiderms	None	None unless ingested or inhaled in sufficient quantities
β Beta particle	High-speed electron $\Theta$	Decay of fission products and neutron induced elements	Varies	5 m	Several layers of skin	Stopped by a few cm or moderate clothing	Superficial skin injury
γ Gamma ray	Electro- magnetic energy	Decay of fission products and neutron induced elements	Energy varies: travels at the speed of light	Up to 500 m but energy dependent	Very penetrating, but energy dependent	Dense material, such as concrete, steel plate, earth.	Whole body injury, many casualties possible
Neutron	Uncharged particle	Fission and fusion reactions	Varies	Less than gamma ray but energy dependent	Very penetrating, but energy dependent	Hydrogeous materials, such as water or damp earth	Whole body injury, many casualties possible

In the disposal facilities for radioactive waste is of fundamental interest to prevent the radiations from spreading in the environment. For this purpose, it is necessary to choose proper barriers in order to reduce the radiation intensity and stop its transmission. This operation is commonly called shielding.

In table 1.1 and figure 1.1 (JAEA, 2014), the shielding requirements for each category of radiation are summarised.

Since  $\beta$  rays and  $\alpha$  rays are composed by charged particles, they interact with the environment and, hence, their energy decreases rapidly in short spaces. They are easily blocked by thin barriers.

X rays,  $\gamma$  rays and neutrons do not have electric charge and can be absorbed only by collision between atoms. Therefore, these radiations are able to cover long distances and, hence, penetrate in the human body. In order to block these emissions, a thicker barrier is required.



Figure 1.1

In order to choose the most appropriate material to realise the barrier, the concept of halfvalue layer (HVL) is used. HVL is the thickness of the material at which the radiation intensity is reduced by one-half (ENS, 2013).

In table 1.2 (De Rose, 2003), some HVL values are displayed for different materials. Lead, steel, iron and concrete are commonly used as shielding materials. The halving mass indicates the mass of material necessary to cut radiation by 50%.

Material	HVL [cm]	Density [g/cm³]	Halving Mass [g/cm²]
Lead	1	11.3	12
Concrete	6.1	3.33	20
Steel	2.5	7.86	20
Packed soil	9.1	1.99	18
Water	18	1	18
Wood	29	0.56	16
Air 15000		0.0012 18	

Table 1.2

The radioactivity is a process occurring naturally on earth. Moreover, it can be produced artificially in many fields such as medicine, food preservation, archaeology, etc. (Capecchi, 2013).

The environment is considered contaminated when a deposition of radioactive substances is observed on surfaces, solids, liquids or gases (including the human body), in which their presence is unintended and undesirable (IAEA, 2007).

Considering the waste management context, containment and isolation of radioactive waste through durable and effective barriers is an important concern not only for countries involved in the nuclear energy production but also for the rest of the world.

## **1.2 CLASSIFICATION OF RADIOACTIVE WASTE**

The classification of radioactive waste is not homogeneous in the world and it may be defined at different levels (international level, national level, operator level) according to the purpose of the classification. Moreover, the development of different schemes according to physical, chemical and radiological properties has led to different terminologies, which could cause confusion, especially at the international level, in the radioactive waste management.

The International Atomic Energy Agency (IAEA) provides the guidelines of the classification of radioactive waste.

In the General Safety Guide No. GSG-1 a comprehensive range of waste classes has been defined giving also some general boundary conditions in order to distinguish one class from the other. The guide underlines that detailed quantitative boundary conditions must be developed by each country considering a wider range of parameters. Moreover, the guide underlines that in cases when there is more than a facility in a country, the quantitative boundaries between the classes may differ for each disposal facility in accordance with scenarios and geological, technical and safety parameters (IAEA, 2009b). The classification schemes for radioactive waste may be developed using an approach related to safety conditions, regulatory aspects or engineering processes necessary for the waste management.

The different typologies of classification of the radioactive waste are summarized in the following scheme:

- Qualitative classification. It is based on the general characteristics of the radioactive waste. In particular, it is possible to develop different qualitative classifications according to:
  - the origin of the waste;
  - the physical state of the waste (solid, liquid, gaseous);
  - the activity levels.
- Quantitative classification. It provides numerical values to distinguish the waste classes. The first step to develop this kind of classification is to decide the purpose of the classification considering the type of waste, the activity under consideration, the processing options available, the safety objectives, the socio-economic factors and the regulatory and technical constraints. The second step consists in choosing the parameters to use for evaluating numerical values as limits for each class of waste.

The IAEA (2009b) uses essentially two parameters for its classification:

- Activity content;
- Half-life.

The <u>activity content</u> designates the radioactivity level of the radioactive elements (radionuclides) contained in the waste (Verstaevel, 2013). The level of radioactivity can be expressed in Becquerels (Bq) per unit of weight. The Becquerel is the unit of the radioactive decay; an activity of one Becquerel means that one disintegration occurs approximately every second (IAEA, 2009b).

The <u>half-life</u> of a radioactive isotope is the time taken for it to decay to half of its amount of radioactivity as measured at the beginning of the time period considered. It can be expressed in years, minutes or seconds.

In the graphic of figure 1.2 (Verstaevel, 2013), it is possible to see the evolution of the activity content with time. The relation between the two parameters is exponential. The half-life corresponds to the value of time at which the activity level is equal to  $A_0/2$  where  $A_0$  is the initial activity content.



Figure 1.2

The IAEA (2009b) distinguishes 6 classes of waste:

- Exempt waste (EW);
- Very short lived waste (VSLW);
- Very low level waste (VLLW);
- Low level waste (LLW)
- Intermediate level waste (ILW)
- High level waste (HLW)

The <u>exempt waste</u> (EW) is characterized by a concentration of radionuclides, which is so small that it does not require any specific provision for the control and the radiations. Hence, this kind of material can be disposed in conventional landfills or recycled. The <u>very short-lived waste</u> (VSLW) contains radionuclides characterized by a very short half-life but with an activity concentration above the authorized level. In general, this kind of waste undergoes storage for decay until the activity is minor of the authorized level. After the VSLW re-enters into the clearance zone, it moves to the class of the EW and hence it can be managed as a conventional waste.

The <u>very low-level waste</u> (VLLW) is characterized by a level of activity that still does not require high levels of isolation and containment. This type of waste can be disposed in surface landfill facilities with limited regulatory controls.

The <u>low-level waste</u> (LLW) is above the clearance level but it contains only limited amounts of long-lived radionuclides. In general, this kind of waste can include short-lived radionuclides at high level of activity concentration and long lived radionuclides but only at low level of activity. The LLW needs to be stocked into facilities, which provide isolation and containment for limited periods. This type of waste is still suitable for near surface disposal but there are different design options for the realization of the facility since the LLW could be characterized by a wide range of activity levels.

The <u>intermediate level waste</u> (ILW) is characterized by the presence of long-lived radionuclides in quantities, which require a greater level of isolation and containment from the environment. In this case, the disposal in the surface facility is not adequate to reach the needed containment. Therefore, the disposal of the ILW has to be done into facilities at a depth generally between a few tens and a few hundreds of metres. Nevertheless, this kind of waste generally does not need particular provision to prevent heat dissipation during the operations of storage and disposal.

The <u>high-level waste</u> (HLW) is characterized by high levels of activity concentration. In this kind of waste, the radioactive decay process generate quantities of heat, which have to be considered in the design of the facility. The disposal of the HLW has to be done into stable geological formations at a depth of several hundred metres below the surface.

In figure 1.3, the concept of the IAEA classification is displayed. Considering the vertical axis, it is possible to notice that the higher the level of activity content the greater the urgency to provide to an adequate isolation from the biosphere. Considering the horizontal axis, it is possible to assert that, over a certain value of activity content, the higher the half-life the greater the need to isolate the waste.



Figure 1.3

#### **1.2.1 Classification in France**

In France, the management of radioactive wastes are regulated by the Andra (*Agence nationale pour la gestion des déchets radioactifs*). The classification is based on the same parameters used by the IAEA: the activity content and the half-life.

Concerning the activity content, it is possible to distinguish four classes of waste:

- Very low level (VLL) from 1 to 100 Bq/g;
- Low level (LL) from 100 to 100 000 Bq/g;
- Intermediate level (IL) from 100 000 to 1 000 000 Bq/g;
- High level (HL) more than 1 000 000 Bq/g

Concerning the half-life, it is possible to identify three different classes of waste:

- Very short lifetime (VSL) half-life < 100 days;
- Short lifetime (SL) half-life  $\leq$  31 years;
- Long lifetime (LL) half-life  $\geq$  31 years.

In figure 1.4, the French classification is summarized.



#### **Classification - France**

Figure 1.4

#### 1.2.2 Classification in Italy

In Italy, the organism of reference for the management of radioactive waste is the Ispra (*Istituto superiore per la protezione e la ricerca ambientale*). The classification is regulated by the Technical Guide n°26 by ENEA-DISP (now ISPRA) published in 1987. Even though this norm is not up to date, it is still the norm of reference for the classification of the radioactive wastes. The radioactive waste is classified into three categories:

- Category I: waste characterized by radioactivity decay of months or few years (mainly hospital and research waste).
  The disposal occurs according to general waste regulations.
- Category II: waste characterized by decay to radioactivity level of few hundreds Bq/g in few centuries and by a content of long-life radionuclides with an activity level minor than 3700 Bq/g in the already conditioned waste.
  The disposal occurs near the surface. In particular, the normative provides specific values of acceptance for shallow land disposal.
- Category III: waste characterized by decay to few hundreds of Bq/g in few thousands of years and by a content of long-life radionuclides with an activity level superior to 3700 Bq/g in the already conditioned waste.
  The disposal occurs in deep geological formations.





Figure 1.5

#### 1.2.3 Classification in the United States

In the United States the management of radioactive wastes is mostly regulated by the NRC (Nuclear Regulatory Commission), in particular for what concerns the LLW, HLW, the use of uranium and thorium, special nuclear material and by-product material (material that becomes radioactive into a reactor) (Tonkay, 2005).

The DOE (Department of Energy) manages and regulates specific activities within the federal government for energy research and national defence purposes. These activities are subjected to different regulations than the ones managed by the NRC (Tonkay, 2005). Therefore the classification of radioactive waste is distinguished into two branches (figure 1.6, Tonkay, 2005) depending if the waste belongs to a commercial entity (NRC) or to a governmental entity (DOE). In the first case the reference is the NRC regulation while in the second case it is DOE regulation and in particular the Order 435.1, Radioactive Waste Management.



Figure 1.6

The categories of the classification are the following:

- High level waste (HLW)
- Low-level waste (LLW). According to the NRC classification (NRC,2014), the LLW are further classified in the following classes based on hazard, disposal and waste form principles:
  - Class A wastes of this class are segregated from the other wastes in the disposal site; they have to satisfy the minimum requirements defined by the (NRC, 2014, Title 10, §61.56a).
  - Class B wastes of this class must meet both the minimum requirements (NRC, 2014, Title 10, §61.56a) and the stability requirements (NRC, Title 10, §61.56b).
  - Class C wastes of this class must meet both the minimum requirements (NRC, Title 10, §61.56a) and the stability requirements (NRC, 2014, Title 10, §61.56b) and need special measures at the disposal site to prevent inadvertent intrusion.
  - Greater than class C (GTCC) wastes of this class in general are not acceptable for the near-surface disposal; therefore, it is necessary to take specific measures for their disposal. In absence of specific measures, these wastes are disposed into geological formations.
- Transuranic waste (TRU Waste)
- By-product

#### 1.2.4 Classification in Japan

In Japan, the classification of radioactive waste is provided by the JAEA (Japan Atomic Energy Agency). The radioactive waste is classified in two main categories according to the activity level (JAEA, 2014):

- High-level waste (HLW) wastes that after the recovery of uranium and plutonium in Reprocessing facilities still have high activity contents; these wastes are disposed in deep geological formations. NUMO (Nuclear waste management organization of Japan) is the responsible for the geological disposal of HLW.
- Low level waste (LLW) this category includes the following subcategories:

- Waste from power reactors:
  - Relatively higher radioactive waste (under surface disposal);
  - Relatively lower radioactive waste, (near surface disposal, generally concrete pit type);
  - Very low level radioactive waste, (near surface disposal, generally trench type).
- Waste containing transuranic nuclides (TRU Waste) (geological disposal, sub-surface disposal, surface disposal or near surface disposal);
- Uranium Waste (geological disposal, sub-surface disposal or near surface disposal).

Waste under the clearance level is reused or disposed as general wastes.

## **1.3 DISPOSAL FACILITIES**

As the classification of the radioactive waste, the disposal methods may change from country to country but the IAEA provides the general guidelines in its guides. As stated by the IAEA, "the disposal of radioactive waste represents the final step in its management, and disposal facilities are designed, operated and closed with a view to providing the necessary degree of containment and isolation to ensure safety. The fundamental safety objective is to protect people and the environment from harmful effects of ionizing radiation" (IAEA, 2012).

The word "disposal" is used to refer to "the emplacement of radioactive waste into a facility or a location with no intention of retrieving the waste" (IAEA, 2012) while the word "storage" is used to refer to "the retention of radioactive waste in a facility or a location with the intention of retrieving the waste" (IAEA, 2012). Both the typologies of facilities have to be designed to isolate the waste from the biosphere and hence to ensure an appropriate level of safety.

#### 1.3.1 Objectives and safety means of disposal facilities

The main objective of disposal is to provide containment and isolation of the waste. The aim of <u>containment</u> is to protect the environment from the waste until radioactive decay has reduced significantly the hazard of the waste. The aim of *isolation* is to retain the waste and to keep its hazard away from the accessible biosphere and from people through physical separation, which restricts the mobility of long-lived radionuclides makes human access difficult.

The means to reach adequate levels of containment and isolation of the radioactive waste depend on the hazard of the waste and, hence, on the typology of disposal chosen. Containment and isolation can be provided by active or passive means.

<u>Active means</u> consist essentially in the monitoring and the surveillance of the disposal facility in order to prevent unauthorised or involuntary access to the waste and in general all disturbance of the facility (IAEA, 2014).

<u>Passive means</u> mainly consist in durable physical barriers which contain and isolate the waste from the biosphere and make inadvertent intrusion more difficult (IAEA, 2014). The design of the barrier differs according to the typology of the disposal (geological or near surface), the waste characteristics and the Country regulations.

In general, the safety of the facility must be mainly provided by passive means in order to minimise the measures that have to be taken after the closure of the facility (IAEA, 2011).

#### 1.3.2 Disposal methods

It is possible to distinguish the disposal in two main typologies:

- Deep geological disposal
- Near surface disposal

The <u>geological disposal</u> is used the radioactive waste representing a significant hazard for the biosphere for long time periods and, hence, not suitable for a disposal in a conventional landfill or in a near surface facility. Therefore, the geological disposal has been recommended as a long-term management solution for intermediate level (ILW) and high level (HLW) radioactive waste (IAEA, 2011).

The geological disposal consists in disposing the solid radioactive waste in an underground facility positioned in a stable geological formation in order to contain and isolate the waste from the biosphere for long periods.

The containment can be provided by durable packaging of waste, engineered barriers and host geological formation.

The isolation is provided by the host geological formation itself.

The overall performance of the facility relies essentially on the properties such as retention

capability, low permeability and homogeneity of the geological formation, which must delay the contact of the radioactive waste with the biosphere until its impact is not more hazardous than the naturally occurring radioactivity (Andra, 2011).

The geological disposal is a method still under study nowadays. Therefore, the knowledge on the phases for the development of this kind of facility, in particular the closure phase, is still limited.



A scheme of a deep geological disposal is displayed in figure 1.7 (VAE, 2008).

Figure 1.7

The <u>near surface disposal</u> consists in the emplacement of solid radioactive waste in earthen trenches or in ground-engineered structures above or just below the ground surface, with a maximum depth up to a few tens of metres. This typology of disposal is suitable for the disposal of VLLW and LLW (IAEA, 2014).

In order to guarantee a high level of safety, the main concern is to provide containment and isolation of the facility through appropriate engineered barriers.

The engineered barriers include the design of waste form and packaging, the barriers used during the operative period and, when the facility is closed, and the cap-cover system for the long-term protection.

The engineered barriers provide containment of the radionuclides associated with the

waste until radioactive decay has significantly reduced the hazard posed by waste (IAEA, 2014).

The engineered barriers can be based on the chemical barrier or the physical barrier concept.

The chemical containment relates primarily to the retardation of the migration of radionuclides by reduction of their solubility and by sorption of radionuclides onto a substrate material. Hence, it consists generally in the use of cementitious waste forms. The physical containment relates to the prevention of radionuclide migration by means of barriers characterised by a low permeability.

In most environments, prevention and limitation of ingress of water, coupled with chemical containment, are key determinants of the safety of near surface disposal.



A scheme of a near surface disposal is displayed in figure 1.8 (VAE, 2008).

Figure 1.8

### 1.4 CAP COVER SYSTEM IN NEAR SURFACE DISPOSAL

The soils object of study come from the cap cover system of the *Centre de Stockage de La Manche* (CSM), a near surface disposal facility. The CSM will be described in detail in chapter 4, while, in this paragraph, the attention will be focused on the general dispositions adopted for near surface disposal systems, referring, in particular, to the cap cover system requirements.

The dissertation will follow the IAEA guidelines for the general matters and the French guidelines for the cap cover system characterisation.

#### 1.4.1 Cap cover system functions and composition

The cap cover system of the near surface disposal consists in a multi-layer barrier, in which each layer has specific functions and characteristics. It is placed on the near surface disposal when its exploitation is finished.

The conception of the cap cover system varies in function of the legislation of the country in which the near surface disposal is. However, some aspects have general value regardless of the particular situation of the disposal.

According to IAEA (2004), the cap cover system of a facility has the following functions:

- Avoid the penetration of water in the facility,
- Avoid intrusions perpetrated by plants, animals and humans,
- Avoid the dispersion of radon,
- Limit the radiations,
- Avoid the superficial erosion.

The factors to take into account for the cap cover design are the following:

- Nature of the waste,
- Cover geometry (layers thickness and slope),
- Configuration of the site,
- Materials availability,
- Climatic conditions (rain, erosion, freeze-thaw cycle, etc.),
- Future of the site.

In figure 1.9 (IAEA, 2003), a schematic disposition of the cap cover system in the near surface disposal is displayed.



Figure 1.9

In order to satisfy the objectives and the constraints, the cap cover system consists generally in five different layers, which are from the bottom to the top:

- Support layer above the waste in order to provide a regular base for the other layers,
- Low permeability layer in order to avoid water infiltrations in the waste and leaks from the waste,
- Drainage layer, which collects water not drained away through runoff,
- Protective layer against human, animal and plant intrusions and against climatic cycles (freeze-thaw and wetting-drying),
- Superficial layer, which integrates the site in the environment and protects the cover system from erosion and the climatic agents.

In figure 1.10 (Van Impe, 1998), different configurations of the cap cover system used in different countries and for different waste typologies are displayed.

Generally, the impermeable function is performed by a low permeability clay layer, often associated to geosynthetics with barrier function (geomembranes).



Figure 1.10

#### 1.4.2 Cap cover system for radioactive waste in France

According to the French norm, the design of the cap cover system for radioactive waste disposal has to satisfy the prescriptions of the decree of 30<sup>rd</sup> December 2002, regarding the disposal for dangerous waste. In particular, article 25 gives the geometric and hydraulic characteristics required by the cap cover.

According to this article, the cap cover system must be composed by the following layers, from the bottom to the top:

- Draining layer above the waste,
- Layer of 1 m characterised by low hydraulic conductivity (K<10<sup>-9</sup>m/s),
- Impermeable barrier realised with a geosynthetic, usually a geomembrane,
- Draining layer of 0,50 m (K>10<sup>-4</sup>m/s),
- Vegetative cover of 0,30 m.

In figure 1.11, the cap cover system prescribed by the decree is displayed.



Figure 1.11

#### **1.4.3 Clay layer compaction requirements**

As seen in figure 1.10, in the near surface disposal for dangerous waste of Europe and USA, the impermeable and protective function is always entrusted to a low permeability clay layer. Hence, in this paragraph, the clay layer requirements will be exposed. The clay layer has to be characterised by a low hydraulic conductivity (K<10<sup>-9</sup> m/s), by good mechanical resistance (high shear strength) and a low shrinkage limit. Given the soil, these characteristics depend on the compaction conditions *in situ* and, hence, on the water content and on the energy of compaction.

The Technical Committee 5 of the International Society for Soil Mechanics and Foundation Engineering (ISSMFE) has proposed the ideal compaction characteristics for the clay layer installation.

Considering a fixed energy of compaction, the graphic in figure 1.12 (Camp, 2009) shows the areas corresponding to good resistance and hydraulic conductivity values for a cap cover system in function of the water content used in the installation.



Figure 1.12

Limit 1 corresponds to the optimum water content line and represents the inferior limit of the acceptable zone in terms of low hydraulic conductivity.

Limit 2 corresponds to the maximum water content allowed not to have cracking due to shrinkage. This limit was determined by Daniel and Wu (1993) studying the evolution of volume deformation in function of the soil water content.

Limit 3 and limit 4 are respectively the maximum water content and the minimum dry unit weight, at which the soil has a compression resistance of 200 kPa at least (Daniel and Wu, 1993). Hence, the shear resistance increases with the increase of the dry unit weight and the decrease of the water content.

The intersection of the defined zones correspond to the compaction conditions and, hence, the couples (w,  $\gamma_d$ ), for which hydraulic conductivity, shrinkage resistance and shear resistance are characterised by acceptable values.

Referring to the French guidelines, the *Bureau de Recherches Géologiques et Minières* (BRGM) has defined the compaction domain in order to have a clay layer with acceptable hydraulic and mechanical characteristics. The domain is shown in figure 1.13 (BRGM, 2001).



Figure 1.13

According to BRGM, the water content to use for the installation and the compaction of a clay layer for a cap cover system must be comprised between  $w_{OPT}+2\%$  and  $w_{OPT}+6\%$ . This indication will be used for the compactions with the standard Proctor procedure performed to prepare the soil samples to be tested with the oedo-permeameter (chapter 5) and the hanging column test (chapter 6).

# **CHAPTER 2**

# Saturated and unsaturated hydraulic conductivity

In this chapter, the soil behaviour in saturated and unsaturated conditions will be described.

In paragraph 2.1, the parameters necessary to define saturated and unsaturated soils will be introduced.

In paragraph 2.2 and 2.3, the hydraulic conductivity in saturated and unsaturated conditions, respectively, will be defined.

## 2.1 SATURATED AND UNSATURATED SOILS

A soil is saturated when all its porosities are filled by water. Hence, a saturated soil can be considered a biphasic material composed by a solid phase (soil particles) and a liquid phase (water filling the pores).

A soil is unsaturated when its porosities are not completely filled by water. Hence, an unsaturated soil can be considered a multiphasic material composed by a solid phase (soil particles), a liquid phase (water partially filling the pores) and a gaseous phase (air filling the rest of the pores).

Unsaturated soils form the largest category of soils present in nature and do not adhere in behaviour to the classical saturated soil mechanics (Fredlund et al., 2011). Due to its multiphasic nature, the analysis of unsaturated soils behaviour is more complicated than saturated soils ones. The physical behaviour of unsaturated soil is usually formulated with differential equations that must be solved with a numerical approach. These approaches and the models used to describe the material behaviour play an important role in solving problems related to unsaturated soils (Fredlund et al. 2011).



Figure 2.1

The ground surface climate is an important factor that controls the depth of the groundwater table and, hence, the thickness of the unsaturated zone. In figure 2.2, a schematic representation of the subdivision between the saturated and unsaturated zone in the environment is represented (Fredlund et al., 2011).



Figure 2.2

Since the unsaturated zone includes also almost saturated soil (the capillary zone immediately above the water table), the correct word to label it is "vadose zone" (vadose is the Latin word for shallow). However, in geotechnical engineering, the term unsaturated zone has become more common to refer to the part of the ground soil subjected to negative pore-water pressures (Fredlund et al., 2011).

In order to describe and to study unsaturated soils, it is necessary to define the soil water content and the pore-water pressure.

In the following paragraphs, the parameters used to define the water content and the porewater pressure in saturated media and unsaturated media will be defined.

#### 2.1.1 Water content and degree of saturation

The soil water content is the quantity of water contained in the soil porosities. It is expressed as a ratio in massic (gravimetric) or volumetric basis.

The following equation defines the gravimetric water content:

$$w = \frac{m_w}{m_d} = \frac{m_h - m_d}{m_d} \tag{2.1}$$

Where  $m_w$  (g) is the water mass in the sample calculated as the difference between the soil sample mass  $m_h$  (g) and the dry sample mass  $m_d$  (g).

On the other hand, the volumetric water content is defined by the following equation:

$$\theta = \frac{V_w}{V_{tot}} = \frac{V_w}{V_s + V_v} \tag{2.2}$$

Where  $V_w$  (cm<sup>3</sup>) is the water volume in the sample and  $V_{tot}$  (cm<sup>3</sup>) is the total volume of the sample, which consists in the sum of the solid soil particles volume  $V_s$  (cm<sup>3</sup>) and the voids volume  $V_v$  (cm<sup>3</sup>) filled by water and/or air.

Volumetric water content can be expressed in function of the gravimetric water content with the following expression:

$$\theta = w \cdot \frac{\gamma_d}{\gamma_w} \tag{2.3}$$

Where  $\gamma_d$  is the dry soil unit weight (kN/m<sup>3</sup>) and  $\gamma_w$  is the water unit weight (kN/m<sup>3</sup>). The effective saturation degree of a soil, also called normalised water content, is a dimensionless value defined by Van Genuchten (1980) with the following equation:

$$S = \frac{\theta - \theta_r}{\theta_{sat} - \theta_r} \tag{2.4}$$

Where  $\theta$  is the volumetric water content of the sample,  $\theta_r$  is the residual volumetric content of the sample and  $\theta_{sat}$  is the saturated volumetric water content, which, hence, is equal to the soil porosity *n*.

The value of S ranges from 0 % (dry conditions) to 100 % (saturated conditions). The residual volumetric water content can be measured experimentally by determining the water content on very dry soil. Nevertheless, since this measurement is not made routinely,  $\theta_r$  value can be extrapolated from soil retention data as the lowest water content value at which the gradient d $\theta$ /ds (where s is the suction) becomes zero. However, the rigorous calculation of  $\theta_r$  is unimportant for most practical field problems (Van Genuchten, 1980).

#### 2.1.2 Pore-water pressure in saturated media

Since soil is a multiphasic material, it is necessary to define a relationship, which describes the interaction between the soil particles and the water filling the voids.

Considering a saturated soil element subjected to forces, Terzaghi distinguishes two kinds of pressures: the effective stresses and the pore-water stresses.

Effective stresses are transmitted through contact between the soil particles. A soil compression behaviour and shear strength depend on them.

Pore-water stresses are transmitted through water filling the voids and are also called neutral stresses (Colombo, 2004).

Effective stresses can be calculated as a difference between the total stress, which depends on the total weight (soil + water), and the neutral pressure (Terzaghi, 1948). Hence, it is possible to define effective stress principle with the following equation:

$$\sigma' = \sigma - u \tag{2.5}$$

Where  $\sigma$ ' is the effective stress (Pa),  $\sigma$  is the total stress (Pa) and u is the neutral stress (Pa).

Thanks to equation 2.5, it is possible to consider separately the solid phase (soil particles) and the liquid phase (water filling the voids) of a saturated medium.

Therefore, despite the presence of the soil particles, the pore-water pressure can be calculated with the classic hydraulic relation regardless of the solid phase.
In order to calculate the hydraulic head, the Bernoulli's equation for incompressible fluids and steady state flow can be used. Neglecting the inertial head and the transformation of mechanical energy into thermal energy, the Bernoulli's principle can be expressed in the following form (Vicaire, 2006)

$$h = z + \frac{u_w}{\gamma_w} + \frac{v^2}{2g} \tag{2.6}$$

Where h is the hydraulic head of the point P in respect of a horizontal arbitrary datum (m), z is the elevation of point P in respect of the reference plan (m),  $u_w$  is the pore-water pressure in the point P (Pa),  $\gamma_w$  is water unit weight (kN/m<sup>3</sup>) and v is the groundwater velocity in the point P (m/s).

The sum of the first and the second term of equation 2.6 corresponds to the piezometric head.

The third term is the kinetic head and can be neglected because of the small values of velocities in porous media.

Setting z=0, equation 2.6 can be simplified:

$$h = \frac{u_w}{\gamma_w} \tag{2.7}$$

Hence, it is finally possible to calculate the pore-water pressure from equation 2.7:

$$u = \gamma_w \cdot h \tag{2.8}$$

In hydrostatic conditions, the pore-water pressure increases linearly from the surface table (z=0) to the bottom of the aquifer (z=H). Hence, in saturated media, the pore-water pressure is always a positive value.

After having determined the water table position in the soil, the calculation of the porewater pressure distribution under the water table can be easily determined.

#### 2.1.3 Adsorption and capillarity in soils

In unsaturated soils, water is not submitted only to gravitational force but also to adsorption and capillarity forces. In the porous media, these forces cannot be separated and their joint effect is known as the soil-water interaction (Vicaire, 2006).

Adsorption is due to electrostatic forces, which depend on the chemical structure of the soil particles and, the soil voids dimensions and the water content. These forces are able

to retain water molecules and cations in the capillarity between the soil particles. These retaining forces decrease with the increase of the water content. When soil is less humid, its avidity for water is greater and water is strongly retained into the voids. When the soil is more humid, the retaining forces decreases and water is less retained. In saturation conditions, these forces are equal to zero.

Water adsorption forces are particularly strong in clays because of their huge specific surfaces, while for granular soils are less relevant.

Capillarity is the ability of a liquid to flow in narrow spaces in opposition to gravity. In order to understand this phenomenon, it is useful to analyse the capillary rise of water in a glass tube with a small diameter, as shown in figure 2.3 (Facciorusso et al., 2011).



Figure 2.3

It is possible to notice that the separation surface between water and air (meniscus) is concave. The separation surface behaves as an elastic membrane in equilibrium subjected to water and air pressures. The meniscus is concave because the atmospheric pressure is higher than the water pressure (Facciorusso, 2011).

Therefore, considering figure 2.3, point 1 and point 2 are characterised by atmospheric pressure, conventionally set as a reference and, hence, equal to zero.

In the glass tube, water pressure decreases linearly under the atmospheric pressure. The minimum value is reached at point 3 and can be calculated with the following expression:

$$u_w = -\gamma_w h_c \tag{2.9}$$

Where  $u_w$  is the water pressure (Pa) and  $h_c$  is the water rise height (m). Referring to the situation displayed in figure 2.3,  $h_c$  is calculated with the following expression:

$$h_c = \frac{2T}{r\gamma_w} \cos\alpha \tag{2.10}$$

Where T is the superficial tension on the membrane (Pa), r is the glass tube radius (m),

 $\alpha$  is the contact angle of the meniscus with the tube (rad).

In table 2.1 (modified from Weast et al., 1982), the variation of water density  $\rho_w$  and superficial tension T with the temperature is displayed.

Temperature [°C]	ρ <sub>w</sub> [g/cm³]	T [N/m]		
0	0,9998	0,0756		
10	0,9997	0,0742		
20	0,9982	0,0728		
25	0,9970	0,0720		
40	0,9922	0,0696		
Table 2.1				

Considering a contact angle equal to zero ( $\alpha$ =0°) and a water temperature of 20°C, the variation of the water rise height h<sub>c</sub> with the glass tube radius r is displayed in figure 2.4.



Figure 2.4



Figure 2.5

Considering a column of soil, (figure 2.5) the water rises from the water table through the canals formed by the soil voids. The water rise height depends on several factors (Facciorusso, 2011):

- Tortuosity of the canals,
- Rugosity of the canals,
- Dimensions of the voids,
- Soil nature.

The irregularities existing in the pores distribution and in the particles structure do not allow to develop a theory based only on capillarity or on microscopical properties. For this reason, a macroscopic point of view is needed and the definition of suction is introduced.

#### 2.1.4 Suction definition

Suction definition is closely related to the pore structure of the material. In fact, the pore structure determines how the soil works for the transport of water through the connected pore spaces (Dexter et al, 2007).

The soil porosity distribution depends on the hierarchy of the particles in the soil structure. In general, it is possible to distinguish the following groups of soil particles, listed in hierarchical order (Dexter et al., 2007):

1. Primary particles,

- 2. Micro-aggregates,
- 3. Aggregates,
- 4. Clods or bulk soil.

The mean size of the pores separating the compound particles of progressively higher levels are themselves progressively bigger. The total porosity consists in the following contributes (Dexter et al., 2007):

1. Residual porosity.

It corresponds to the smallest voids of the pore distribution. They are filled by the residual water (paragraph 2.1.1).

2. Matrix porosity.

It corresponds to the pore space between individual soil mineral particles.

3. Structural porosity.

It corresponds to the pore space between the micro aggregates too. In general, these pores are mainly composed of micro-cracks, which have an important role in transport processes.

4. Macro porosity.

It corresponds to the largest porosities in the soil structure.

Since residual porosity and macro porosity are respectively too small and too large to be characterised in standard water retention experiments, they are not considered for suction definition.

<u>Total suction  $\Psi$  characterises the unsaturated soil tendency to attire water (Facciorusso,</u> 2011). It is defined with the following equation:

$$\Psi = s + \pi \tag{2.11}$$

Where s is the matric suction (Pa) and  $\pi$  is the osmotic suction (Pa).

<u>Matric suction s</u> is defined as the difference between the air pressure  $(u_a)$  and the water pressure  $(u_w)$  into porosities of an unsaturated soil:

$$s = u_a - u_w \tag{2.12}$$

Where  $u_w$  is always lower than  $u_a$  due to the capillarity effect (paragraph 2.1.3) In natural conditions,  $u_a$  is equal to the atmospheric pressure and, hence, can be set as the reference and neglected. <u>Osmotic suction  $\pi$ </u> is due to the presence of dissolved cations in the interstitial water and, hence, to the electro-chemical potential difference between interstitial and free water (adsorption effect, paragraph 2.1.3). Osmotic suction is variable with dissolved cations content of water and is present in saturated soils too.

In geotechnical engineering, osmotic suction is generally neglected and the problems of the unsaturated soils are referred to matric suction variations. This approximation can be justified observing the graphic in figure 2.6 (Fredlund et al., 2011).



Figure 2.6

In figure 2.6, total suction, matric suction and osmotic suction variations in function of the gravimetric water content of a clay are displayed. It is possible to notice that the osmotic suction remains almost constant with the water content variation. Therefore, fixed a water content variation  $\Delta w$ , total suction variation  $\Delta \Psi$  is equal to matric suction variation  $\Delta s$ .

In order to measure *in situ* suction, tensiometers can be used.

A tensiometer consists in a tube with a ceramic porous tip and a water reservoir at the other extremity. After inserting the instrument into the soil, water flows through the ceramic

tip, determining a depression in the reservoir. The depression, corresponding to the suction value, can be measured with a manometer (Facciorusso, 2011).

#### 2.1.5 Conclusions on pore-water pressure in saturated and unsaturated media

Considering only matric suction variations to describe unsaturated soil behaviour, the hydraulic head for both saturated and unsaturated soils can be expressed with equation 2.6, reported underneath neglecting the kinetic term:

$$h = z + \frac{u_w}{\gamma_w} \tag{2.13}$$

Where:

 $\frac{u_{w}}{\gamma_{w}} > 0 \quad \text{for saturated soils,}$  $\frac{u_{w}}{\gamma_{w}} < 0 \quad \text{for unsaturated soils.}$ 

In figure 2.7 (Fredlund et al., 2011), the hydraulic head for saturated condition (Point A) and unsaturated condition (Point B) is represented.





In figure 2.8 (Vicaire, 2006), the typical profile of pore-water pressures distribution in the soil is displayed. While the pore-water pressure distribution under the water table is not affected by variations, the pore-water distribution in the vadose zone depends on the soil water content and, hence, the climatic conditions.

When the water content decreases due to excessive evaporation (drying phase) suction increases, when the water content increase due to a rainfall (wetting phase) the superficial

soil tends to saturated conditions and, as a consequence, suction decreases. Theoretically, considering moisture equilibrium and no water movement in the vadose zone, suction is in equilibrium with the water table. This situation could be schematised with an imaginary infinite impervious membrane on the ground surface (Vicaire, 2006).



Figure 2.8

## 2.2 SATURATED HYDRAULIC CONDUCTIVITY

In this paragraph, the hydraulic conductivity for saturated media ( $K_{sat}$ ) will be introduced. In particular,  $K_{sat}$  will be defined from the Darcy's law (paragraph 2.2.1) stressing the conceptual difference with the medium permeability (paragraph 2.2.2) and introducing some methods for its measure (paragraph 2.2.3).

#### 2.2.1 Definition hydraulic conductivity for saturated media

Studying the water laminar flow through horizontal sand layers, Darcy found that apparent velocity through porous media is directly proportional to the hydraulic loss and inversely proportional to the path followed by water. Later he verified that this relation is also valid

for clays and, in general, for all soils (Colombo, 2004).

These concepts are summarised by Darcy's law:

$$v = K_{sat} \cdot i \tag{2.14}$$

Where v is the flow velocity (m/s),  $K_{sat}$  is the hydraulic conductivity of the saturated porous medium (m/s) and i is the hydraulic gradient (adimensional) which corresponds to the piezometric line slope.

The hydraulic gradient is calculated with the following equation:

$$i = \frac{\Delta H}{L} \tag{2.15}$$

Where  $\Delta H$  is the hydraulic loss (m) and L is the shortest path followed by water (m).

K<sub>sat</sub> is influenced by several factors (Colombo, 2004):

- Soil particles dimensions and granulometric distribution,
- Soil particles nature and mineralogy,
- Particles disposition into the soil,
- Material density and compaction properties.

Moreover, the *in situ* effects like soil stratification and organic deposits should be considered too.

In table 2.2, values of  $K_{sat}$  according to the soil typology are displayed (Colombo, 2004).

K <sub>sat</sub> (m/s)	1	10 <sup>-1</sup>	10 <sup>-2</sup>	10 <sup>-3</sup>	10-4	10 <sup>-5</sup>	10 <sup>-6</sup>	10 <sup>-7</sup>	10 <sup>-8</sup>	10 <sup>-9</sup>	10 <sup>-10</sup>
	Pervious				Semi-pervious			Impervious			
		Gravel		Clean sand and gravel mixed with clean sand		Fine sa inorg mixed cla	Fine sand, organic and inorganic silt, sand mixed with silt or clay, clay deposits		Homogeneous clays		

Table 2.2

 $K_{sat}$  is characterised by a huge range of variability and becomes higher with the increase of soil particles dimensions.

#### 2.2.2 Permeability and Hydraulic conductivity

Even if permeability and hydraulic conductivity are often used as synonyms, they are not defined in the same manner.

The permeability (k) is the capacity of a porous media, in general a soil, to let a fluid pass through its voids. It considers all the characteristics of the fluid flow into the porous media with the exception of the fluid viscosity. The permeability is measured in m<sup>2</sup> and can be called geometric permeability or intrinsic permeability.

The hydraulic conductivity provides a measure of the ease with which a fluid moves into a porous media. The hydraulic conductivity is measured in m/s<sup>2</sup> and can be called coefficient of permeability.

The relation between permeability and hydraulic conductivity in saturated conditions is the following (Colombo, 2004):

$$k = \frac{\mu}{\rho g} K_{sat} = \frac{\mu}{\gamma} K_{sat}$$
(2.16)

Where  $\mu$  is the dynamic viscosity of the fluid in kg/(ms),  $\rho$  is the fluid density in kg/m<sup>3</sup> and  $\gamma$  is the unit weight of permeant in N/m<sup>3</sup>.

In table 2.3 (Kestin et al., 1978), the variations of water density and dynamic viscosity with the temperature are displayed.

Temperature [°C]	ρ <sub>w</sub> [g/cm³]	μ [kg/(ms)]
0	0,999840	0,001792
10	0,999703	0,001307
20	0,998207	0,001002
25	0,997048	0,000890
40	0,992200	0,000653
	T-1-1-0-0	

Table 2.3

In geomechanics the fluid of reference is generally the water at 20°C and at an atmospheric pressure of 100 kPa.

In table 2.4 (Yeda Resa & Dev, 2006), some permeability values in function of the soil typology are displayed.

Coarse gravels	$10^{-4} - 10^{-5} \text{ cm}^2$
Sands and gravels	$10^{-5} - 10^{-8} \text{ cm}^2$
Fine sands, silt, loess	$10^{-8} - 10^{-12} \text{ cm}^2$
Clay, shale, glacial till	$10^{-12} - 10^{-16} \text{ cm}^2$
Sandstone	$10^{-7} - 10^{-13} \text{ cm}^2$
Limestone	$10^{-8} - 10^{-12} \text{ cm}^2$
Weathered chalk	$10^{-6} - 10^{-8} \text{ cm}^2$

Table 2.4

#### 2.2.3 Methods for measuring $K_{\text{SAT}}$

In order to measure the hydraulic conductivity of a saturated soil, different methods can be applied. In general, it is possible to distinguish experimental, empirical and *in situ* measures. Some of these methods are listed hereafter (Colombo, 2004):

- Experimental methods:
  - Oedo-permeameter with constant head,
  - Oedo-permeameter with variable head (falling head test).
- Empirical correlations:
  - Hazen,
  - Kozeny-Carman,
  - Sivakumar,
  - Tavenas.
- In situ measures:
  - Boutwell borehole test,
  - Single ring infiltrometer,
  - Double ring sealed infiltrometer.

Advantages and disadvantages of laboratory testing for measuring the hydraulic conductivity are the following ones (SCS, 1991):

Advantages	Disadvantages
<ul> <li>The water content and the density of the soil can be controlled more easily.</li> <li>Soil samples can be saturated more easily before testing.</li> <li>It is possible to apply high hydraulic gradients in order to obtain clayey soils low hydraulic conductivities in shorter times.</li> <li>Laboratory tests are more economical than field methods, especially for slowly permeable soils.</li> </ul>	<ul> <li>Since laboratory samples have small sizes, it is difficult to model important macro-features of natural soil deposits (drying cracks, alluvional stratification, etc.).</li> <li>Laboratory samples are always disturbed. It is difficult and expensive to obtain undisturbed soil samples.</li> <li>Measure of horizontal hydraulic conductivity is difficult.</li> </ul>

Advantages and disadvantages of field-testing for measuring the hydraulic conductivity are the following ones (SCS, 1991):

Advantages	Disadvantages
<ul> <li>It is possible to test larger samples of soil, which, hence, includes the macro-pores structure too. The obtained values of hydraulic conductivity are more representative of field deposits.</li> <li>Horizontal hydraulic conductivity can be measured more easily.</li> <li>It is possible to measure hydraulic conductivity in granular undisturbed soils.</li> </ul>	<ul> <li>In situ tests are more expensive and need more time to be performed.</li> <li>In situ tests require skilled and experienced personal in order to have reliable results.</li> <li>The equipment needed for performing field test is not readily available.</li> </ul>

In this work, the oedo-permeameter falling-head test and empirical correlations will be applied in chapter 5 in order to obtain the hydraulic conductivity of the soil under analysis in saturated conditions.

In particular, the oedo-permeameter test will be described in paragraph 5.1 and the empirical correlations in paragraph 5.2.

## 2.3 UNSATURATED HYDRAULIC CONDUCTIVITY

In this paragraph, the hydraulic conductivity for unsaturated media (K) will be introduced. As it was done for hydraulic conductivity in saturated media (paragraph 2.2.1), K will be defined from the Darcy's law (paragraph 2.3.1).

Therefore, the method used to obtain hydraulic conductivity variation with suction will be explained introducing the Soil Water Retention Curve (paragraph 2.3.2, 2.3.3 and 2.3.4).

#### 2.3.1 Definition of hydraulic conductivity for unsaturated media

As in saturated media, hydraulic conductivity in unsaturated soils can be defined using Darcy's law, taking into account that K depends on total suction  $\Psi$  (Facciorusso, 2011). In general, instead of total suction  $\Psi$ , the matric suction s is used for the reasons already explained in paragraph 2.1.4.

Darcy's law for unsaturated media can be written as following:

$$v = K(\Psi) \cdot i \tag{2.17}$$

Where v is the flow velocity (m/s),  $K(\Psi)$  is the hydraulic conductivity of the medium (m/s),  $\Psi$  is the total suction (kPa) and i is the hydraulic gradient (adimensional) which corresponds to the piezometric line slope.

The unsaturated hydraulic conductivity  $K(\Psi)$  can be expressed with the following expression:

$$K(\Psi) = K_{sat} \cdot K_r \tag{2.18}$$

Where  $K_{sat}$  is the hydraulic conductivity in saturated conditions (m/s) and  $K_r$  is the relative hydraulic conductivity.

 $K_r$  is adimensional and depends on the suction level and the saturation level. Its value ranges from 0 (residual saturation conditions) to 1 (saturation conditions).

Different authors have proposed some analytical expressions to describe Kr variability in

function of suction or water content. Some of these expressions will be introduced in paragraph 2.3.3.

#### 2.3.2 The Soil Water Retention Curve (SWRC)

The Soil Water Retention Curve (SWRC) defines the relation between matric suction and water content (Fredlund et al., 2011).

The matric suction can be expressed either with pressure s (kPa) or hydraulic head h (m). The water content can be expressed with the gravimetric water content w, the volumetric water content  $\theta$  or the degree of saturation S (paragraph 2.1.1).

The SWRC is usually represented in a semi-logarithmic diagram in which the logarithmic axis is the suction related one.



An example of a SWRC is displayed in figure 2.9 (Fredlund et al., 2011).

Figure 2.9

Considering suction increase, it is possible to distinguish three main zones:

1. Boundary effect zone.

Suction is very low and its increase does not produce a significant decrease of the

water content. This phase ends when the suction corresponding to the air entry value is reached. The air entry value is the suction value at which air bubbles appear in the macropores.

2. Transition zone.

Water content decreases significantly and the liquid phase becomes discontinuous.

3. Residual zone.

For great increments of suction the water content exhibits small decrease. It starts when the suction value corresponding to the residual water content is reached.

SWRC shape is influenced by several factors (Facciorusso, 2011):

- The soil granulometric curve and its compaction characteristics. Fine soils are able to retain more water than coarse soils (figure 2.10, Vicaire 2006).
- The soil compaction conditions (figure 2.11, Vicaire 2006).
- The higher the initial water content the steeper the curve and the higher the AEV.



Figure 2.10



Moreover, drying or wetting conditions must be considered. In fact, when suctions increases, soil follows the main drying retention curve while, when suction decreases, it follows the main wetting curve (figure 2.12, Fredlund et al., 2011).

The main wetting curve does not reach the complete saturation conditions since residual air content remains always entrapped in the soil porosity. Therefore, SWRCs are characterised by hysteresis.



Figure 2.12

Several models have been introduced in order to obtain the SWRC. These models describe the functional relationships between the soil water content (volumetric water content or degree of saturation) and suction.

The main difficulties in modelling are due to the consideration of the porosities and of the hysteresis.

In table 2.5 (Zhou et al., 2005) some proposed models for the wetting and drying phase of water retention curve are presented.

Model name	Model	Pore size distribution $f(\psi)$
Gardner (1956)	$\theta = B/\psi - D$	Constant pore size
Linear function representing water content (Fredlund and Xing, 1994)	$\theta = B - D \psi$	$A/\psi^2$
Brooks and Corey (1964)	$S=B-D\psi^{p}$	$A/\psi^{(m+1)}$
Brutsaert (1966)	$S = \frac{1}{1 + (\psi / a)^n}$	$\frac{n(\psi/a)^{n-1}/a}{[1+(\psi/a)^n]^2}$
Normal distribution	$S = \frac{1}{2} \operatorname{erfc}\left(\frac{\psi - \mu}{\sqrt{2}s}\right)$	Normal distribution
Van Genuchten (1980)	$S = \frac{1}{\left(1 + \left(a\psi\right)^n\right)^m}$	$\frac{mna(a\psi)^{n-1}}{(1+(a\psi)^n)^{m+1}}$
McKee and Bumb (1987) (Boltzman)	$S = \exp(-\psi / B)$	Exponential distribution
Fredlund and Xing (1994)	$S = \frac{1}{(\ln(e + (\psi/a)^{n}))^{m}}$	$\frac{mn(\psi/a)^{n-1}}{a(e+(\psi/a)^n)(\ln(e+(\psi/a)^n))^{m+1}}$
Kosugi (1994)	$S = \frac{1}{2} \operatorname{erfc}\left(\frac{\ln\left(\frac{\psi_{\text{sev}} - \psi}{\psi_{\text{sev}} - \psi_{\text{mode}}}\right) - s^2}{\sqrt{2\pi s}}\right)$	$\frac{\exp\left[\frac{\ln\left(\frac{\psi_{aev}-\psi}{\psi_{aev}-\psi_{mode}}\right)-s^2}{\sqrt{2\pi s}}\right]}{\sqrt{2\pi s}(\psi_{aev}-\psi)}$

Table 2.5

Two experimental methods used to obtain the SWRC are the hanging column test and the pressure plate extractor.

In this work, through the hanging column test the experimental points will be found and the Van Genuchten model will be applied to obtain the analysed soils SWRCs and, hence, the hydraulic conductivity variation with suction.

The detailed procedure will be discussed in chapter 6, while in the following paragraph the Van Genuchten model will be introduced.

#### 2.3.3 The Van Genuchten model

The Van Genuchten model provides a relationship between the saturation degree S and soil suction, using three empirical constants:  $\alpha$ , m and n.

In order to express the suction level, the pressure head h (with positive sign) will be used. The model is defined by the following equation (Van Genuchten, 1980):

$$S = \left(\frac{1}{1 + (\alpha \cdot h)^n}\right)^m \tag{2.19}$$

Parameters m and n are related by the following equation (Van Genuchten, 1980):

$$m = 1 - \frac{1}{n}$$
 (0 < m < 1) (2.20)

Therefore, the parameters that have to be estimated from observed soil retention data are  $\alpha$  and n. The procedure used to obtain these parameters will be explained in chapter 6 paragraph 6.6.2.

Once all the three parameters have been calculated, it is possible to obtain the relative hydraulic conductivity from equation 2.18. Using the pressure head h (m) instead of the pressure value  $\Psi$  (kPa), it can be written as following:

$$v = K(h) \cdot K_r \tag{2.21}$$

According to Van Genuchten, the relative hydraulic conductivity  $K_r$  is calculated with the following equation, which depends on saturation degree S and parameter m:

$$K_r = S^{1/2} \left[ 1 - \left( 1 - S^{1/m} \right)^m \right]^2$$
(2.22)

# **CHAPTER 3**

# Bentonite treatments for disposal passive barrier

As it was outlined in chapter 1, in order to guarantee an appropriate and long-period level of security in a disposal facility, passive means must be used. In order to build the passive barrier characterised by low hydraulic conductivity, the natural material of the site can be used (Verstaevel, 2013).

However, finding in situ materials with appropriate mechanical and hydraulic

characteristics can be very difficult in most cases.

When a fine material with low hydraulic conductivity is not available, a possible solution is the bentonite treatment.

The bentonite increases the soil fine fraction and makes it less permeable.

In this chapter, bentonite characteristics and its effects on soil properties will be discussed.

# **3.1 CLAYS COMPOSITION AND PROPERTIES**

Clays correspond to the  $0/2\mu$ m fraction of a soil or a geological formation. Mineralogically, clays are aluminosilicate formed by two elementary elements (figure 3.1, Barral 2008):

- Tetrahedral elements, SiO<sub>4</sub>.
- Octahedral elements, Al(OH)<sub>6</sub>.



Figure 3.1

These elements are combined in layers according to different sequences, and between these layers, it is possible to find cations or water molecules. Moreover, silicon and aluminium can be substituted by other chemical elements (Ca, Mg, K, etc.). Since clays are formed by thin sheets (7-14 nm), they are characterised by high specific surface (between 5-800 m<sup>2</sup>/g) and, hence, they have important adsorption properties (Verstaevel, 2013). Each sheet has a charge that could be different in intensity and origin. This charge influences clay behaviour, for example swelling and hydratation. The main sources of charge in clay minerals are isomorphous substitution and pH-dependent charges.

Isomorphous substitution consists in the substitution of an element with another in ionic crystals without any change in the structure. This process takes place when the ionic radius of the two elements are almost the same and their charge differs of one unit at the most. It takes place during crystallisation and the developed negative charge is permanent.

pH-dependent charges can be positive or negative depending on the pH of the soil. These charges are variable and are particularly relevant in silicate clays as kaolinite (ENV320, Northern Arizona University).

Clays can be classified into three different main types:

- Kaolinite,
- Illite,
- Smectite.

Kaolinite is formed by a layer of tetrahedral elements and a layer of octahedral elements. Its cation exchange capacity is low and it does not swell very much in contact with water. Illite is formed by the repetition of tetrahedron, octahedron, tetrahedron sequence. As for the kaolinite, its cation exchange capacity is low and, hence, is not characterized by a pronounced swelling when in contact with water.

Smectite is formed by two layers of tetrahedral elements enclosed by octahedral elements. It is characterised by high and permanent negative charge due to the isomorphous substitution. This charge is balanced by exchangeable hydrated cations, in particular Ca2+, Mg2+, Na+ and K+, placed between the clay layers. Smectites have, hence, a high cation exchange capacity. The hydration degree of cations depends on its nature and the relative humidity. When in contact with water, the inter-layer spaces swell (Verstaevel, 2013).

These characteristics are good for clays used in treatments for reducing a soil hydraulic conductivity. Bentonites belong to this group.

#### 3.1.1 Bentonite characteristics

Bentonites are clayey materials mainly composed of a smectite called montmorillonite. Their swelling and permeability properties depend on their cation exchange capacity and, hence, on the following factors:

- Montmorillonite content,
- Concentration of sodium and calcium,
- Fineness of the bentonite,
- Eventual chemical additives used in the mixture.

It is possible to distinguish three types of bentonite named after their dominant element:

- Potassium bentonite,
- Sodium bentonite,
- Calcium activated bentonite.

Potassium bentonite (K-bentonite) is a natural potassium rich illitic clay formed from alteration of volcanic ash.

Sodium bentonite is characterised by excellent colloidal properties. When it is in contact with water it expands, absorbing more water than its dry mass. Thanks to this property, sodium bentonite can be used in several engineering applications (sealant, drilling muds, impervious barriers, etc.).

Calcium bentonite is able to absorb ions in solution. When this bentonite undergoes an ion exchange process (sodium activation), it is able to exhibit many properties typical of sodium bentonites. This process consist in mixing calcium bentonite with a soluble sodium salt and water. After this process, the bentonite acquires swelling properties (Bouazza, 2006).

Sodium bentonites and calcium-activated bentonites can be used to reduce a soil hydraulic conductivity. In table 3.1, the hydraulic conductivity values of some commercial bentonites are shown (Stepniewski et al., 2011).

	Japanese commercial bentonite Kunigel-V1 extracted from Tsukinuno Mine, Japan, mixed with sand	1.10-11-1.10-12 for bentonite content 5- 50%.	Komine 2004, 2010
Bentonite	Bentonite of 92% sodium montmorillonite (by Minarmco SA) added to Chaco-Pampean silt in the amount of 5 and 10%.	3.3·10 <sup>-10</sup> – 5% bentonite 8.5·10 <sup>-11</sup> – 10% bentonite	Francisca & Glatstein, 2010
	Compacted sodium and calcium exchanged bentonite, Gyungsang, Korea	5.4·10 <sup>-12</sup> for Ca bentonite 9.9·10 <sup>-12</sup> for Na bentonite	Ahn & Jo, 2009
	Bentonite compacted (intermediate and modified by Proctor test) different shapes and sizes commercial gravel particles by AquaBlok, Ltd.	6.08·10 <sup>-12</sup> (intermediate Proctor) 5.98·10 <sup>-12</sup> (modified Proctor)	Roberts & Shimanoka, 2008
	Commercial Na-bentonite and Ca- bentonite (Concarde Mining) mixed with crushed, natural zeolites (Etibank-Bigadic, Turkey) at different proportions	5.10-10-8.10-10	Kaya & Dudukan, 2004

#### Table 3.1

However, for a more detailed analysis of the hydraulic conductivity values of bentonites, ageing process *in situ* should be considered. In fact, if cation exchange occurs, sodium cations could be replaced by other cations and this process could affect the hydraulic conductivity (Aldaeef et al., 2014).

### **3.2 EFFECT OF BENTONITE TREATMENTS ON SOIL PROPERTIES**

In this paragraph the effect of bentonite on the treated soils will be discussed referring to the studies by Couradin et al. (2008), Labiod-Aloui et al. (2014), Didier et al. (1996) and BRGM (1992).

#### 3.2.1 Atterberg's limits

Labiod-Aloui et al. (2014) studied the effect of bentonite treatments on a sediment from the Chorfa dam in Algeria, with the purpose of using them to realise an impermeable barrier against the spread out of the "Sebkha of Oran" lake.

In figure 3.2, the granulometry of the Chorfa sediment and of the bentonite used for the

treatment are displayed. In figure 3.3, the evolution of liquid and plastic limit values with the bentonite percentage are displayed.



Figure 3.2



Figure 3.3

The liquid limit ( $w_L$ ), the plastic limit ( $w_P$ ) and the plasticity index (PI) increase with the bentonite content because of the increase of clay content in the mixtures (Labiod-Aloui, 2014). However, it is possible to notice that the sensitivity to Atterberg limits increase is low in this case.

#### 3.2.2 Methylene blue adsorption test

The methylene blue adsorption test is used to find the ionic adsorption capacity of a soil, measuring the quantity of methylene blue necessary to cover the total surface (internal and external) of the clay particles. The adsorption capacity is expressed by the VBS value, referred to the 0/50 mm fraction of the material, or the VB value, referred to the 0/5 mm fraction of the material. The test will be described more precisely in chapter 4, paragraph 4.3.2.

Couradin et al (2008) studied the behaviour of a soil treated with different bentonites and 9 different percentages of treatment calculated on the material dry mass. The raw soil is a clayey sand. The additives used in his study are the following ones:

- Two calcium bentonites (FV, FZ)
- Six calcium bentonites activated with sodium (F9, S, S8, T, V, V6)
- One kaolinite (P).

In figure 3.4, the VB results of Couradin's study in function of the treatment percentage and of the bentonite are shown.

For all the studied bentonites, the relation between VBS and the mixture percentage is linear.

Thanks to linearity, it is possible to control the actual bentonite percentage of the *in situ* treatment, measuring the corresponding *in situ* VBS value.



Figure 3.4

#### 3.2.3 Compaction properties

The compaction properties of a soil can be analysed through the compaction curve ( $\gamma_d$ , w) obtained with Proctor procedure.

In the study already introduced in paragraph 3.2.2, Couradin et al. (2008) analysed the compaction characteristics of the treatments with the standard Proctor procedure according to the French norm NF P 94-093.

In figure 3.5, the results are displayed. The continuous curve is the compaction curve of the raw material (clayey sand - Sm), while the symbols represent the couples ( $\gamma_{d max}$ ,  $w_{OPT}$ ) of each treatment (S, T, V, FV, FZ, S8, F9, P, V6).

It is possible to observe that all the treated samples are characterised by higher dry unit weights.



Figure 3.5

In the study already introduced in paragraph 3.2.1, Labiod-Aloui et al. (2014) obtained the compaction curves of the analysed soils.

Another example of variation of the compaction characteristics of soils treated with bentonite is displayed in figure 3.6 and figure 3.7 (Labiod-Aloui et al., 2014).



Figure 3.7

Also in this case, with the increase of bentonite content, the maximum dry unit weight increases and the optimum water content decreases.

In general, if a soil is mixed with fine material,  $\gamma_{d,max}$  increases and  $w_{OPT}$  decreases. This is due to the bentonite filling the porosities between the coarse particles.

However, it must be considered that the finer the granulometry of a soil, the lower the  $\gamma_{d max}$ 

and the higher the  $w_{OPT}$ , as shown in figure 3.8 (Colombo, 2004).

Therefore, if the bentonite treatment is used on a fine soil, the addition could contribute to loosen the soil structure, leading to a decrease of the dry unit weight.



Figure 3.8

#### 3.2.4 Hydraulic conductivity

The addition of a certain percentage of bentonite diminishes significantly the hydraulic conductivity of a soil, as shown in figure 3.9 (Didier et al., 1996).



Figure 3.9

In figure 3.10, the results by Couradin et al. (2008) are displayed. The analysed soils are always the ones already presented in paragraph 3.2.2: a clayey sand treated with different types of additives (6 calcium-activated bentonites, 2 calcium bentonites, 1 kaolinite). The hydraulic conductivity values were obtained with the rigid wall oedo-permeameter (NF X 30-441) performed on the samples compacted with the standard Proctor procedure at the optimum water content.

The hydraulic conductivity of the raw material is  $1.6 \cdot 10^{-6}$  m/s and the addition of bentonite provokes a consistent decrease of hydraulic conductivity for all the studied treatments. It is possible to notice that in order to reach a hydraulic conductivity of at least  $10^{-9}$  m/s, the necessary percentage of calcium activated bentonites (F9, S, S8, T, V, V6) is minor than the necessary percentage of the natural calcium bentonites (FV, FZ).



Figure 3.10

In 1992 BRGM (Bureau de Recherches Geologiques et Minieres), published a study on the effect of bentonite treatments on a soil characterised by low hydraulic conductivity. All the hydraulic conductivity measures were obtained with the oedo-permeameter test, performed on samples compacted with the standard Proctor procedure at different water contents.

The hydraulic conductivity of the raw material is comprised in the range  $[2 \cdot 10^{-8}, 1 \cdot 10^{-9}]$  m/s. For the treatments, three percentages of bentonite treatments were used: 4%, 7%, 10% of the dry mass. The hydraulic conductivities of the treated materials are comprised in the range  $[1,5 \cdot 10^{-10}, 5 \cdot 10^{-10}]$  m/s.

In figure 3.11, the results of the study are displayed (BRGM, 1992). The lowest hydraulic conductivity was obtained with the 10% treatment, which is the highest bentonite content among the ones analysed by BRGM (1992). However, it is possible to notice that the difference between the hydraulic conductivities obtained with different treatments is not remarkable.



Figure 3.11

In the study already introduced in paragraph 3.2.1, Labiod-Aloui et al. (2014) obtained the variation of the vertical and horizontal hydraulic conductivity in function of the bentonite used for the treatment on the raw soil.

The vertical hydraulic conductivity values were obtained using compaction permeameter (mould with the same dimensions as the Proctor mould). The samples were compacted with the standard Proctor procedure at the optimum water content according to the French

norm NF P 94-093. The upper and lower plates of the Proctor mould were replaced by the compaction permeameter ones in order to allow the water flow through the sample. The saturation phase lasted for a period of about 1,5to 2 months. Samples were considered fully saturated when water was coming out of the water outlet portal of the compaction permeameter equipment.

The horizontal hydraulic conductivity values were obtained using drainage and seepage tank model HM 169. The equipment used for the measure is shown in figure 3.12 (Labiod-Aloui et al., 2014).



1 laboratory trolley, 2 pump, 3 supply tank, 4 measuring gland, evenly distributed over the base of the tank, 5 adjustable overflow pipe, 6 device for injecting the contrast producing medium, 7 pile planking, 8 fourteen tube manometer, 9 water feed, can be positioned as required, 10 tank with viewing window made of special glass

#### Figure 3.12

The results obtained by Labiod-Aloui et al. (2014) are displayed in figure 3.13. The untreated material vertical hydraulic conductivity is  $2 \cdot 10^{-10}$  m/s. The higher the bentonite content of the sample, the lower the hydraulic conductivity.



Figure 3.13

## **3.3 CONCLUSIONS ON BENTONITE TREATMENTS**

The following major conclusions on the effect of bentonite treatments on the geotechnical and hydraulic characteristics of a soil can be drawn:

- The Atterberg's limits values (liquid limit, plastic limit and plasticity index) increase with the increase of bentonite content.
- The VB (and VBS) value increases with the increase of bentonite content.
- Depending on the raw soil fine fractions, the dry unit weight at optimum water content can increase or decrease with the increase of bentonite content.
- The hydraulic conductivity decreases as the bentonite content increases but the efficiency of the treatment depends on the raw soils characteristics.

# **CHAPTER 4**

# The CSM, the characterisation of the soils of the CSM and their bentonite treatments

In this chapter, the *Centre de Stockage de la Manche* (CSM) will be introduced focusing in particular on its history and its cap cover system.

The soils object of this study and the proposed bentonite treatments will be described and characterised through a series of classic geotechnical tests (Granulometry, VBS, Atterberg limit, Proctor, water pycnometer, etc.).

The results of these tests will be discussed, highlighting the effect of the bentonite treatments on the soil properties.

# 4.1 THE CENTRE DE STOCKAGE DE LA MANCHE (CSM)

The Centre de Stockage de la Manche (CSM) is a surface disposal facility for average and low radioactivity waste characterised by short life. It is located in the department of the French region called *Basse-Normandie* (figure 4.1).

It consists in a surface of 15000  $m^2$  and until the year of its closure, it had received 527225  $m^3$  of average and low radioactivity waste (Verstaevel, 2013). The current appearance of the CSM is displayed in figure 4.2.



Figure 4.1



Figure 4.2

#### 4.1.1 History of the CSM

The CSM was opened in 1969, when it received the first package of radioactive waste, and was closed in 1994 when it arrived at saturation after a total exploitation time of 25 years (Gourden, 1996). During this period, since the technologic progress in radioactive waste management was continuous, it was necessary to adapt and improve the disposal centre. Therefore, the following phases can be distinguished (ANDRA, 2009):

- Initial phase (1969 1979).
- Transitional phase (1979 1983).
- Industrial phase (1983 1994).
- Monitoring phase (1994 2003).
- Long-term monitoring phase (2003 Nowadays).

During the initial phase (1969-1979), the waste was compacted in concrete covered barrels. These barrels were lain down in trenches excavated directly in the ground (figure 4.3). This method was subjected to several problems and, hence, was replaced by a better solution: the storage of the barrels on platforms or in concrete trenches according to the

radioactivity level (figure 4.4). During this phase, the waste conditioning was progressively improved and the controls on the radioactivity level became stricter.

In 1978, 100000 m<sup>3</sup> of waste had already been stocked in the facility (ANDRA, 2009).



Figure 4.3

Figure 4.4

The beginning of the transitional phase (1979-1983) is marked by the creation of a national agency for the radioactive waste management: the ANDRA (*Agence nationale pour la gestion des déchets radioactifs*). This agency began to regulate the procedures of the waste barrels manufacturing, the acceptable radioactivity limits in the facility, the waste conditioning, etc.

During the industrial phase (1983-1994), the waste management was improved introducing a series of regulations with the objective of controlling the radioactivity level (registration of the barrels characteristics: content, dimensions, etc.). The barrels were stored in cells made of reinforced concrete (monolith) (figure 4.5) and with a tumulus disposition (figure 4.6). In particular, the waste that was disposed in the ground during the first phase was retrieved and stored in the tumuli.

The water collecting system was improved separating the superficial water and the infiltration one. At the beginning of the eighties, a facility to collect the infiltration water was built, and, in 1991, the cap-cover system building started (ANDRA, 2009).





Figure 4.5



In 1994, the facility received its last waste package and the monitoring phase started (1994-2003). The cap cover system was complete in 1997. Its main objective is to isolate the radioactive waste from the external environment. The water collecting system was also improved in order to collect superficial and infiltration water (ANDRA, 2009).

The characteristics of the cap cover system will be described in more detail in paragraph 4.1.2.

In 2003, the long-term monitoring phase (2003 - nowadays) started officially. The main objective of this phase is to control the impact of the disposal facility on the environment, to identify eventual problems and, hence, to provide proper resolutions to these problems. This phase will last some centuries (ANDRA, 2009).

#### 4.1.2 The cap cover system of the CSM

The installation of the cap cover system of the CSM took place from 1991 to 1997. It was the first time in the world that a multi-layer cap cover system was used in a disposal facility for nuclear wastes (ANDRA, 2009). The function of the cap cover system is to isolate the radioactive waste stored with the tumulus system or with the monolith system. Therefore, it is possible to distinguish the three following functions (ANDRA, 2009):

• To prevent water from reaching the radioactive waste and, hence, from producing leachate which could contaminate the groundwater and the environment near the facility.
- To protect the stored waste from the ground movements.
- To protect the facility from human, animal and vegetal intrusion.

The cap cover configuration and composition are displayed in figure 4.7.



Figure 4.7

Layer 7 is the vegetative cover consisting of natural untreated soil. It has a protective function against the meteorological agents.

Layer 6 consists of sandy loam with a protective function against human, animal and vegetal intrusion.

Layer 5 consists of a sand layer with draining function. This layer collects the water filtrating from the upper one (layer 6) and conveys it to a drainpipe situated at the lowest point of the layer.

Layer 4 consists of an impermeable bituminous geomembrane with the function of preventing water from seeping to the lower layers.

Layer 3 is equivalent to layer 5. Therefore, it is made of a draining sand layer, which conveys to the drainpipes the eventual water leakage from the superior layers. This layer is equipped with an alarm system, which activates when water leakage is detected. Layer 2 consists in a bituminous geomembrane equivalent to layer 4. Its aim is to collect the possible leakage from the upper geomembrane (layer 4).

Layer 1 is a sandy loam layer covering the radioactive waste. Its function is to give to the cap cover the proper slope and hence, the "factory roof" shape selected in order to facilitate the drainage in layer 5 and layer 3.

In table 4.1 the composition, the function and the thickness of each layer are presented.

LAYER	Material	Function	Thickness
7	Natural soil	Protection against meteorological agents	0,15/0,30 m
6	Sandy Loam	Protection against intrusion	1,00/1,40 m
5	Sand	Drainage	0,20 m
4	Bituminous Geomembrane	Impermeable barrier	5 mm
3	Sand	Drainage	0,20 m
2	Bituminous Geomembrane	Impermeable barrier	5 mm
1	Sandy Loam	Cap cover shape	Variable

Table 4.1

### 4.1.3 The long term cap cover system

In 2012, a retrieving campaign was conducted on the superficial layer (sandy loam). All the samples have been brought to the *Laboratoire d'étude des transferts en hydrologie et environnement* (LTHE) to be classified and studied.

The purpose was the study of the hydraulic conductivity of the superficial part of the cap cover system (layer 7 and layer 6) in order to consider a possible treatment to improve its impermeability (ANDRA, 2012b) and let the CSM go from active protection conditions to passive protection ones (Chapter 1 paragraph 1.3.1).

The treated soil should substitute in the future for half the superficial layer of the cap cover system. This substitution could interest the upper half or the lower half.

In the first case the installation would be much easier, in the second case it would be a more complex operation because of the risk of damaging the geomembrane and the need of removing the upper soil, but more efficient.

In the following paragraphs, after introducing the soil *in situ* and the proposed treatments, the tests performed to characterise these materials will be presented.

# 4.2 THE TYPE MATERIALS AND THEIR TREATMENTS

## 4.2.1 Definition of the type materials: T12 and T3

Since the CSM cap cover system was built in three different steps, the samples collected with the sample taking campaign were distinguished in the following three groups (*tranche*) (Verstaevel, 2013):

- T1 (26 samples) samples collected from the most ancient part of the cap cover system (first step of its building); this group consists of *in situ* excavation material.
- T2 (38 samples) samples collected from the part corresponding to the second step of the cap cover realisation; as T1, this group consists of *in situ* excavation material.
- T3 (36 samples) samples collected from the part corresponding to the last step of the cap cover realisation; this group consists of calibrated material coming from a quarry.

The results of the granulometry, the methylene blue adsorption test and the Atterberg limits, performed for each single sample of the three groups, showed that the first two typologies of material (T1 and T2) have very similar properties. Therefore, since the differences in the properties of T1 and T2 are not significant, a type material was prepared blending the T1 and T2 samples. The obtained type material was called T12. All the T3 samples were mixed too in order to obtain the T3 type material. From now on, the appellations T12 and T3 will be used to indicate the type materials. In figure 4.8 and figure 4.9 the aspect of the 0/5 mm fraction of T12 and T3 respectively are displayed.



Figure 4.8



Figure 4.9

## 4.2.2 The bentonite treatments

The additive chosen for the treatment of type soils T12 and T3 is a calcium-activated bentonite, belonging to the smectites family. The properties of this typology of bentonite have already been described in chapter 3 paragraph 3.1.1.

IMPERSOL <sup>®</sup> Sv					
Colour	Pinkish powder				
Medium swelling	14 ml/g				
Specific weight	2,7 g/cm <sup>3</sup>				
Apparent density	0,7 g/cm <sup>3</sup>				
Passing at 75 μm sieve (dry)	≥ 75 %				
Passing at 75 μm sieve (humid)	≥ 96 %				
Water content	8÷14%				

The characteristics of the bentonite used for the treatments are presented in table 4.2.



In order to study the influence of the addition of bentonite to the T12 and the T3, the following percentages for the treatments were proposed:

- T12 (or T3) + 5% of bentonite  $\rightarrow$  T12 5% (or T3 5%).
- T12 (or T3) + 7% of bentonite  $\rightarrow$  T12 7% (or T3 7%).
- T12 (or T3) + 12% of bentonite → T12 12% (or T3 12%).

The percentage is calculated on the dry mass of the 0/5 mm fraction of the type material (T12 or T3).

The 7% bentonite treatment is the most eligible to be used effectively in the CSM for the long-term cap cover system. However, the three values were considered in order to find correlations between the bentonite content and the soils properties, focusing in particular on the hydraulic ones.

# **4.3 CLASSIFICATION OF THE SOILS**

In this paragraph, the soils introduced in paragraph 4.1 will be characterised with the help of the French norm NF P 11-300 (1992).

Therefore, after presenting the granular distributions, the methylene blue adsorption test and the Atterberg limits respectively in paragraph 4.2.1 paragraph 4.2.2 and paragraph 4.2.3, the classification will be presented (paragraph 4.2.4)

## 4.3.1 Granulometric curves

The granulometric analyses consists in determining the massic distribution of the particles versus their size.

In order to study the distribution of the elements d>80µm the sieving method was used, referring to the French norm NF P 94-056 (1996). The passing at 80µm sieve (0/80µm fraction) was analysed with the sedimentation method referring to the French norm NF P 94-056 (1992).

The main result of the granulometric analyses is the granulometric curve of the material. In figure 4.10, the T12 and T3 granulometric curves are presented. In figure 4.11, the granulometric curves of the 0/5mm fraction are presented.

The 0/5 mm fraction curves were considered too because it is the reference fraction used for the preparation of all the samples studied in this work.



Figure 4.10



Figure 4.11

From the granular distributions, it is possible to calculate two parameters: the uniformity coefficient  $C_U$  and the curvature coefficient  $C_Z$ .

C<sub>U</sub> corresponds to the average slope of the curve with the vertical axis in the mean part:

$$C_U = \frac{d_{60}}{d_{10}} \tag{4.1}$$

Where  $d_{60}$  is the particle diameter (mm) corresponding to the 60% of the passing material and  $d_{10}$  is the particle diameter (mm) corresponding to the 10% of the passing material. When  $C_U$  is in minor than 2, the soil has a uniform granulometry. When  $C_U$  is higher than 2 the soil is characterised by a wide range of particles dimension (Verstaevel, 2013).  $C_Z$  measures the shape of the particle size curve and is calculated with the following equation:

$$C_Z = \frac{(d_{30})^2}{d_{60} \cdot d_{10}} \tag{4.2}$$

Where  $d_{30}$  is the particle diameter (mm) corresponding to the 30% of the passing material. When  $C_Z$  is between 1 and 3 the soil is well graded (Verstaevel, 2013).

Other two parameters used for the soils classification are the fine fraction FF (percentage of passing material at the  $80\mu m$  sieve) and  $d_{max}$  (the maximum dimension of the biggest particles of the soil).

In table 4.3 the values of  $d_{10}$ ,  $d_{30}$ ,  $d_{60}$ ,  $d_{max}$ , FF,  $C_U$  and  $C_Z$  are displayed for the type materials T12 and T3 considering the complete curves and the 0/5 mm fraction curves.

	T12	Т3	T12 (0/5mm)	T3 (0/5mm)
d <sub>10</sub> [mm]	0,00124	0,00899	0,00093	0,00129
d <sub>30</sub> [mm]	0,03429	0,58162	0,01439	0,05092
d <sub>60</sub> [mm]	0,86096	8,34200	0,18594	0,68447
d <sub>max</sub> [mm]	63,00	63,00	5,00	5,00
FF	37,92%	17,15%	50,20%	33,01%
Cu	696,71	927,43	199,21	529,76
Cz	1,11	4,51	1,19	2,93

The T12 results a well-graded soil considering both the complete curve and the 0/5mm fraction ( $C_U$ >2 and 1< $C_Z$ <3).

The T3 results a bad-graded soil if the complete curve is considered ( $C_U$ >2 and  $C_Z$ >3) and a well-graded soil if the 0/5mm fraction is considered ( $C_U$ >2 and 1< $C_Z$ <3).

Regarding the treated soils granulometries, they were not analysed with the sieving and the sedimentation method. However, it is possible to find the fine fraction FF (d<80 $\mu$ m) of each treatment considering that the bentonite consists in particles inferior to 80 $\mu$ m (table 4.2).

The values of FF for each treatment distinguishing between the complete curve and the 0/5mm fraction are displayed in table 4.4.

	T12	Т3	T12 (0/5mm)	T3 (0/5mm)
0%	37,92%	17,15%	50,20%	33,01%
5%	40,88%	21,10%	52,57%	36,20%
7%	41,98%	22,57%	53,46%	37,39%
12%	44,57%	26,03%	55,54%	40,19%

|--|

Because of the bentonite treatment, the fine fraction FF increases. This effect is obviously more visible if the 0/5mm fraction only is considered.

#### 4.3.2 Methylene blue adsorption test (VBS)

The methylene blue adsorption test is used to find the ionic adsorption capacity of a soil, measuring the quantity of methylene blue necessary to cover the total surface (internal and external) of the clay particles. The test is performed on samples obtained from the 0/5 mm fraction as explained in the French norm NF P 94-068.

The adsorption capacity is expressed by the VBS value, from the French *Valeur de Bleu du Sol*, (grams of methylene blue absorbable by 100 grams of dry soil) referred to the 0/50 mm fraction of the material.

VBS is calculated with the following equation (NF P 94-068):

$$VBS = C \cdot VB \quad [0/50 mm] \tag{4.3}$$

Where C is the proportion of the 0/5 mm fraction in the 0/50 mm fraction of the dry material, and VB is the adsorption capacity of the soil (grams of methylene blue absorbable by 100 grams of dry soil) referred to the 0/5 mm fraction.

The VB is calculated from the test results with the following equation:

$$VB = \frac{B}{m_d} \cdot 100 \quad [0/5 \ mm]$$
(4.4)

Where B is the methylene blue mass used in the test (g) and  $m_d$  is the dry mass of the sample (g).

The VB and VBS calculated from the test results are presented in table 4.5.

	T12			Т3				
	0%	5%	7%	12%	0%	5%	7%	12%
VB [0/5 mm]	0,96	2,22	2,88	4,41	0,65	2,04	2,37	3,45
VBS [0/50 mm]	0,72	1,69	2,20	3,40	0,34	1,08	1,27	1,89

Table 4	.5
---------	----

The higher the bentonite percentage, the higher the value obtained from the test. This happens because the bentonite contributes to increase the clay fraction of the starting soils (T12 and T3) making the mix more reactive and capable of absorbing the methylene blue. In fact, the bentonite is the clay with the extreme specific area.

Moreover, it is possible to observe that T12 and all its treatments are characterised by higher VB and VBS values than the corresponding ones of the T3 based soils. In fact, T12 clay fraction (0/2  $\mu$ m) is higher than T3 one and, hence, this makes T12 based soils more reactive.

These considerations can also be deduced observing the graphic in figure 4.12, which represents the VB in function of the bentonite percentage.



Figure 4.12

The two lines plotted in figure represent the linear correlation between VB and the bentonite percentage of the treatment, respectively for T12 based and T3 based soils. It is possible to verify that the linear correlation found by Couradin and others (2008) (Chapter 3 paragraph 3.2.2) is valid for the soils studied in this work. Thanks to the existence of a linear correlation between VB (or VBS) and the bentonite percentage of the treatment, it is possible to make valid hypothesis on the VB (and VBS) values of T12 and T3 treated with bentonite percentages not directly studied with the methylene blue test. Moreover, thanks to linearity, it is possible to control the actual bentonite percentage of the *in situ* treatment, measuring the corresponding *in situ* VBS value.

#### 4.3.3 Atterberg limits

The liquid limit ( $w_L$ ), the plastic limit ( $w_P$ ) and the plasticity index (PI) were found for all the untreated and treated soils referring to the French norm NF P 94-051 (1993). This standard is applied to the soil fraction passing at the 400 µm sieve (0/400µm fraction). The liquid limit  $w_L$  is the soil water content corresponding to the transition point between liquid and plastic behaviour. Experimentally, it corresponds to the water content at which a groove made on a soil placed in the Casagrande cup closes when the cup undergoes 25 repeated drops from a 10 mm height (NF P 94-051, 1993).

The plastic limit  $w_P$  is the soil water content corresponding to the transition point between plastic and solid behaviour. Experimentally, it corresponds to the water content at which a thread of soil breaks apart having a diameter of 3 mm ±0,5 mm (NF P 94-051, 1993). The plasticity index PI is the difference between the liquid limit and the plastic limit:

$$PI = w_L - w_P \tag{4.5}$$

All the three parameters are expressed in percentage format.

The test were performed at least two times for all the analysed soils. In particular, for the untreated soils, the tests were performed on 64 samples for T1+T2 (26 samples of T1 and 38 samples of T2) and on 36 samples for T3.

The results were calculated as average values.

In table 4.6 and figure 4.13, the results are presented.

	T12			Т3				
	0%	5%	7%	12%	0%	5%	7%	12%
WL	31,25%	53,74%	76,00%	110,35%	32,50%	52,87%	63,24%	109,23%
WP	21,87%	22,30%	27,13%	35,10%	22,81%	23,59%	27,70%	33,33%
PI	9,38%	31,44%	48,87%	75,25%	9,69%	29,27%	35,54%	75,90%

Table -	4.6	;
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Figure 4.13

In figure 4.13 the dotted lines correspond to the average values of Atterberg limits obtained from the 64 samples of T1+T2 and the 36 samples of T3. In first place, it is evident that the higher the bentonite content, the higher the Atterberg limits. In particular, it is possible to notice that the liquid limit is subjected to a more



important increase than the plastic limit. Therefore, the resulting plasticity index increases significantly with the bentonite percentage, as it is shown in figure 4.14.

Figure 4.14

In second place, it is possible to notice that the T12 and T3 do not show important differences in the Atterberg limits values with the exception of the liquid limit at 7% of bentonite. However, this exception could be explainable as a consequence of an overestimation of the liquid limit of T12 7%. In fact, the Atterberg limits estimation is always exposed to errors due to the operator's subjectivity and the water content measures. Finally, the GTR (*Guide des Terrassements Routiers*) provides the following criteria to judge the influence of the Atterberg limits on the soil behaviour (GTR, 1992):

- If the 0/400 µm fraction is more than the 50% of the complete granulometric curve and PI is higher than 12%, the results are realistic and can be used to describe the soil behaviour.
- If the 0/400 µm fraction is less than the 35% of the complete granulometric curve and PI is lower than 6%, the results cannot be interpreted.

All the 0/5 mm fractions of the soils analysed in this work are characterised by a 0/400  $\mu$ m fraction forming more than the 50% of the granulometric curve. The PI is always higher than 12% with the exception of the untreated soils, T12 and T3, for which it is between 9% and 10%.

Therefore, according to the GTR indications, the obtained results for the Atterberg limits can be considered sufficiently reliable.

The liquid limit and the plasticity index obtained for each soil, were used to find the corresponding point in the Casagrande's chart.



In figure 4.15, the Casagrande's chart is presented.

Figure 4.15

The soils are always in the field of inorganic clays. Given the bentonite percentage, the positions in the chart of T12 based soils and T3 based ones are very similar. The only exception is the 7% percentage for the reasons already explained previously. It is possible to point out that, increasing the soil bentonite content, the soil plasticity increases consistently:

- T12 0% and T3 0% are low plasticity clays.
- T12 5% and T3 5% are medium plasticity clays.
- T12 7% and T3 7% are high plasticity clays.
- T12 12% and T3 12% are very high plasticity clays.

## 4.3.4 Classification of the samples

The results presented in the previous paragraphs will be used to classify the soils with the use of the French standard NF P 11-300 (1992).

Since all the tests were performed on samples obtained from the 0/5mm fraction of the soils, the soils were classified referring to the same fraction.

The standard classifies the soils according to the following characteristics:

- The granulometric curve, in particular the d<sub>max</sub> (maximum dimension of the soil grains) and the fine material fraction FF (percentage of particles with dimensions inferior to 80 μm).
- The plastic index (PI).
- The methylene blue adsorption value referred to the 0/5mm fraction (VB).

In table 4.7, the NF P 11-300 tables used for the classification are presented. In table 4.8 the properties used for the classification are presented.

	Class	ement selon la nat	ure	Classement selon la nature				
Paramètres de nature Premier niveau de classification	Classe	Paramètres de nature Deuxième niveau de classification	Sous classe fonction de la nature	Paramètres de nature Premier niveau de classification	Classe	Paramètres de nature Deuxième niveau de classification	Sous classe fonction de la nature	
Dmax < 50		<b>VBS ≤ 2,5</b> <sup>(*)</sup> ou I <sub>p</sub> ≤ 12	A <sub>1</sub> Limons peu plastiques, loess, siits alluvionnaires, sables fins peu pollués, arènes peu plastiques		в	tamisat à 80 μm ≤ 12% tamisat à 2 mm ≤ 70% VBS > 0,2	B <sub>4</sub> Graves argileuses (peu argileuses)	
mm	Δ	12 < I <sub>n</sub> < 25 <sup>(*)</sup>	Α.					
Tamisat à 80 μm > 35%	Sols fins	ou 2,5 < VBS ≤ 6	Sables fins argileux, limons, argiles et marnes peu plastiques arènes	Dmax ≤ 50 mm et Tamisat à 80 μm ≤ 35%	Sols sableux et graveleux avec fines	tamisat à 80 $\mu$ m compris entre 12 et 35% tamisat à 2 mm $\leq$ 70%	B <sub>5</sub> Sables et graves très	
		25 < I <sub>p</sub> ≤ 40 <sup>(*)</sup> ou 6 < VBS < 8	A3 Argiles et argiles marneuses, limons très plastiques			vвs < 1,5 ↔ ou I <sub>p</sub> ≤ 12	silicux	
		0 4 7 8 0 2 0				tamisat à 80 µm	B <sub>6</sub>	
		l <mark>p &gt; 40</mark> <sup>(*)</sup> ou VBS > 8	A4 Argiles et argiles marneuses, très plastiques			VBS > 1,5 <sup>(*)</sup> ou I <sub>p</sub> > 12	Sables et graves argileux à très argileux	

Table 4.7

		d <sub>max</sub> [mm]	FF	PI	VB	
T12 0/5mm	0%	5,00	50,20%	9,38%	0,96	A1
	5%	5,00	52,57%	31,44%	2,22	A3
	7%	5,00	53,46%	48,87%	2,88	A4
	12%	5,00	55,54%	75,25%	4,41	A4
	0%	5,00	33,01%	9,69%	0,65	B5
T3 0/5mm	5%	5,00	36,20%	29,27%	2,04	A3
	7%	5,00	37,39%	35,54%	2,37	A3
	12%	5,00	40,19%	75,90%	3,45	A4

Table 4.8

Hence, the 0/5mm fractions of the studied soils can be classified as following:

- T12 0% Low plasticity silt (A1).
- T12 5% High plasticity silt (A3).
- T12 7% High plasticity clay (A4).

- T12 12% High plasticity clay (A4).
- T3 0% Sand with high silt content (B5).
- T3 5% High plasticity silt (A3)
- T3 7% High plasticity silt (A3).
- T3 12% High plasticity clay (A4).

## **4.4 SOLID UNIT WEIGHT**

The solid unit weights  $\gamma_s$  (kN/m<sup>3</sup>) of the soils were obtained with the water pycnometer method. The principles and the procedure of this method are described in the French standard NF P 94-054 (1991).

The test was performed on the untreated type materials T12 and T3 and on the bentonite used for the treatment. The obtained values of  $\gamma_s$  are displayed in table 4.9.

	T12	Т3	Bentonite			
γ <sub>s</sub> [kN/m³]	26,79	27,36	26,34			
Table 4.9						

The $\gamma_{s}$ of the treated T12 soils was calculated with the following equation in function of the
treatment percentage x%, the solid unit weight of T12 ( $\gamma_{s T12}$ ) and the solid unit weight of
the bentonite (γ <sub>s bento</sub> ):

$$\gamma_{s\,x\%} = \frac{W_{s\,TOT}}{V_{s\,TOT}} = \frac{1 + x\%}{\left(\frac{1}{\gamma_{s\,T12}} + \frac{x\%}{\gamma_{s\,bento}}\right)} \tag{4.8}$$

The equation for calculating the  $\gamma_s$  of T3 treated soils is analogous to the equation 4.8.

The obtained values of  $\gamma_s$  for the treated soils are displayed in table 4.10.

In figure 4.16, the  $\gamma_s$  in function of the bentonite percentage is represented.

	T12			ТЗ				
	0%	5%	7%	12%	0%	5%	7%	12%
γ <sub>s</sub> [kN/m³]	26,79	26,77	26,76	26,74	27,36	27,31	27,29	27,25

Table 4.10



Figure 4.16

The higher the bentonite content of the treated soil, the lower the solid unit weight of the material. This effect is equivalent for T12 and T3 since the two lines are parallel.

## **4.5 PROCTOR TEST**

The normal Proctor test was performed on the untreated and treated materials in order to find the optimum water content  $w_{OPT}$  and the maximum dry specific weight  $\gamma_{d OPT}$ . The standard procedure, described in the French norm NF P 94-093 (1999), was used. The test consists in humidifying a soil in order to have different water contents and to compact it with a predetermined number of blows from a standard weighted hammer at a specified height, for each water content.

Hence, for each water content, it is possible to find the dry unit weight of the material and to trace the Proctor curve of the soil, which describes the variations of the dry unit weight

with the water content.

The test can be performed with either the Proctor or the CBR mould. The first is used when  $d_{max}$ <5mm the second when  $d_{max}$ >5mm. Since the test was performed on samples obtained from the 0/5mm fraction, the Proctor mould was used.

The mould diameter  $d_{mould}$ , the mould height  $h_{mould}$ , the mould volume  $V_{mould}$ , the hammer weight  $W_{hammer}$ , the fall height of the hammer H, the number of compacted layers in the mould  $n_l$ , the number of blows for each layer  $n_{b/l}$  and the compaction energy E are presented in table 4.11.

The compaction energy E is calculated with the following equation:

d <sub>mould</sub> [m]	0,1015
h <sub>mould</sub> [m]	0,1165
V <sub>mould</sub> [m <sup>3</sup> ]	0,000942644
W <sub>hammer</sub> [N]	24,90
H [m]	0,305
nı	3
n <sub>b/l</sub>	25
E [N/m²]	604244

$$E = \frac{m_{hammer} \cdot H \cdot n_l \cdot n_{b/l}}{V_{mould}}$$
(4.9)

Table 4.11

From the Proctor curve, it is possible to acquire the maximum value of the dry unit weight ( $\gamma_{d OPT}$ ), which is obtained at a particular water content value called optimum water content ( $W_{OPT}$ ).

In order to obtain a proper Proctor curve, a minimum of 5 points corresponding to 5 different water contents is required. In particular, in this work a minimum of 6 water contents was used for each soil, in order to have 3 water contents minor than the optimum and 3 superior to the optimum.

In figure 4.17 and figure 4.18, the Proctor curves of T12 based soils and T3 based ones, respectively, are presented. In table 4.12 the values of  $w_{OPT}$  and  $\gamma_{d OPT}$  are presented for the analysed soils.



Figure 4.17



Figure 4.18

	T12				ТЗ			
	0%	5%	7%	12%	0%	5%	7%	12%
WOPT	12,3%	14,9%	16,4%	17,6%	11,1%	14,2%	15,0%	15,2%
γ <sub>d</sub> [kN/m³]	19,1	18,0	17,9	17,2	19,7	18,9	18,4	17,8

|--|

The saturation curves equations represented in figure 4.17 and 4.18 are referred to the untreated materials, respectively T12 and T3, and are obtained from the following equation after fixing the degree of saturation S to 80% or 100% (Colombo, 2004):

$$\gamma_d = \frac{S \cdot \gamma_s}{S \cdot \gamma_w + w \cdot \gamma_s} \tag{4.10}$$

Where  $\gamma_s$  is the solid unit weight (kN/m<sup>3</sup>) of T12 0% or T3 0% (table 4.9 paragraph 4.4).

It is possible to find out that the maximum dry unit weight of the two type soils T12 and T3 gets lower as the bentonite percentage of the treatment increases.

This result is in contrast with the studies by Couradin et al. (2008) and Labiod-Aloui et al. (2014) (Chapter 3 paragraph 3.2.3).

However, this could be due to the high fine content of the untreated type materials T12 and T3. In fact, as already outlined in chapter 3 paragraph 3.2.3, the addition of bentonite to a fine soil can contribute to loosen the soil structure, leading to a decrease of the dry unit weight.

# 4.6 SUMMARY OF THE SAMPLES CHARACTERISATION

In table 4.13, the parameters characterising the 0/5mm fraction of the soils under analysis are displayed. In the following chapters, these results will be used in order to study the variation of the hydraulic conductivity in function of the bentonite treatment.

	T12 (0/5mm fraction)			T3 (0/5mm fraction)				
	0%	5%	7%	12%	0%	5%	7%	12%
NF P 11-300 Classification	A1	A3	A4	A4	B5	A3	A3	A4
VB	0,96	2,22	2,88	4,41	0,65	2,04	2,37	3,45
WL	31,25%	53,74%	76,00%	110,35%	32,50%	52,87%	63,24%	109,23%
WP	21,87%	22,30%	27,13%	35,10%	22,81%	23,59%	27,70%	33,33%
PI	9,38%	31,44%	48,87%	75,25%	9,69%	29,27%	35,54%	75,90%
γ <sub>s</sub> [kN/m³]	26,79	26,77	26,76	26,74	27,36	27,31	27,29	27,25
W <sub>OPT</sub>	12,3%	14,9%	16,4%	17,6%	11,1%	14,2%	15,0%	15,2%
$\gamma_{d} [kN/m^{3}]$	19,1	18,0	17,9	17,2	19,7	18,9	18,4	17,8

#### Table 4.13

It must be outlined that, even if all the characterisation tests were performed on the 0/5 mm fraction of the soils, the *in situ* coarser fraction (complete granulometry) is not negligible. Hence, the coarse fraction could have a certain influence on mechanical and hydraulic properties of the analysed soils.

However, since the bentonite generally interacts with the 0/5 mm fraction of a soil, the analysis in the following chapters are based on the 0/5 mm fractions properties. The difference between the 0/5 mm fraction and the complete granulometry will be considered for the internal erosion criteria applied to T12 0% and T3 0% in chapter 7.

# CHAPTER 5 Influence of the treatments on hydraulic conductivity in saturated conditions

In this chapter the saturated hydraulic conductivity of T12, T3 and their bentonite treatments will be calculated using the oedo-permeameter test and some correlations with the soil properties.

After having described and applied each method, the results will be commented referring to the reliability of each method.

In particular, the main interest is to assess the efficiency of bentonite treatments on the impermeability of T12 and T3.

# **5.1 OEDO-PERMEAMETER TEST**

## 5.1.1 Samples preparation

The samples used in the oedo-permeameter tests have been prepared mixing the 0/5 mm fraction of the material with water to obtain the chosen water content. After the mixing, the material undergoes a maturation for 72 hours minimum. This phase is necessary for materials with a high fine content in order to have a water content perfectly homogeneous. After compacting the material with normal Proctor procedure (Chapter 4 paragraph 4.5), the samples have been obtained coring the soil directly with the cutting ring of the oedometric cell. The advantage of this procedure is that it is possible to have a perfect contact between the sample and the cell (Verstaevel, 2013). Regarding the choice of the water content for the compaction, the indications of the *Bureau de Recherche Géologiques et Minières (BRGM)* have been followed (Chapter 1,

paragraph 1.4.3). Hence, a gravimetric water content of w<sub>OPT</sub>+2% was chosen.

The calculation of  $w_{OPT}$ +2% and  $\gamma_{d OPT+2\%}$  will be discussed more in detail in reference to the preparation of the hanging column test samples (chapter 6, paragraph 6.2.1).

In table 5.1, the calculated values of  $w_{\text{OPT}}\text{+}2\%$  and  $\gamma_{d \text{ OPT}\text{+}2\%}$  values are displayed.

	WOPT+2% REAL	Ŷd OPT+2% [kN/m³]
T12 0%	14,29%	18,77
T12 5%	17,91%	17,51
T12 7%	19,73%	16,92
T12 12%	21,36%	16,47
T3 0%	12,63%	19,46
T3 5%	16,77%	18,13
T3 7%	17,40%	17,95
T3 12%	18,38%	17,40

Table 5.1

#### **5.1.2 Description of the method**

The hydraulic conductivity in laboratory is measured in saturated conditions and steady state. Moreover, since temperature influences the viscosity of water and hence the measure of hydraulic conductivity, the laboratory test have to be performed at constant temperature, in general 20°C.

The test is based on the validity of Darcy's law (chapter 2, paragraph 2.2.1):

$$v = K \cdot i \tag{5.1}$$

Where v is the velocity of the permeant through the soil matrix (m/s), K is the hydraulic conductivity (m/s) and i is the hydraulic gradient, calculated as following:

$$i = \frac{\Delta h}{H} \tag{5.2}$$

Where  $\Delta h$  is the difference of hydraulic charge between the upper and the lower part of the sample (m) and H is the drainage distance of the sample in the flow direction (m). Using the Darcy's law it is possible to write the continuity equation with the following expression:

$$Q = v \cdot A = K \cdot i \cdot A \tag{5.3}$$

Where Q is the flow rate through the sample (m<sup>3</sup>/s). Knowing the water volume V (m<sup>3</sup>) which passes through the sample in a time interval  $\Delta t$  (s)

it is possible to calculate the corresponding flow rate Q:

$$Q = \frac{V}{\Delta t} \tag{5.4}$$

Substituting the equation 5.4 in the equation 5.3, the hydraulic conductivity can be found:

$$K = \frac{V}{A \cdot i \cdot \Delta t} \tag{5.5}$$

The measure of hydraulic permeability through the permeameter test can be done with two methods:

- Constant head test;
- Falling head test.

In the <u>constant head test</u> the sample is subjected to a vertical flow imposed by a water charge maintained constant during the test. This method assumes laminar flow conditions and independency between the hydraulic conductivity and the hydraulic gradient. Generally, the constant charge method is used for coarse-grained soils. In the <u>falling head test</u> the sample is subjected to a vertical flow imposed by a variable water charge. This method is used for both coarse-grained and fine-graded soils.

In this work rigid wall permeameters with the falling head procedure have been adopted. The tests were performed referring to the French norm NF X 30-441 (1998). In these tests, the samples are placed between two porous plates (one over and one under the sample) in an oedometric cell, which blocks horizontal and vertical dilatations.

The test is organised in three phases according to the French norm NF X 30-442 (1998):

- 1<sup>st</sup> Phase.
   The confinement charge is applied;
- 2<sup>nd</sup> Phase.

The sample is saturated applying a hydraulic charge until the steady state is reached.

• 3<sup>rd</sup> Phase.

The interval of time  $\Delta t$  necessary to the loss of hydraulic charge  $\Delta h$  is measured. Finally, it is possible to obtain the hydraulic conductivity K for each interval with the following expression:

$$K = \frac{H \cdot s}{A \cdot (t_1 - t_0)} ln\left(\frac{h_0}{h_1}\right)$$
(5.6)

Where H is the drainage length of the sample in the flow direction and, hence, the height of the sample (m), s is the section area of the tube providing water supply ( $m^2$ ),  $h_0$  and  $h_1$  (m) are the water heights in the supply tube respectively at time  $t_0$  and  $t_1$  (s).

In this work, the tests have been performed using a confinement of 26 kPa, corresponding to a layer of soil characterised by a thickness slightly superior to 1 m, and a hydraulic charge of 10 cm.

#### 5.1.3 Results

The values of the hydraulic conductivities in saturated conditions are displayed in figure 5.1 and table 5.2.



Figure 5.1

		K <sub>SAT</sub> [m/s]	R
	0%	6,70E-10	-
T12	5%	5,60E-10	16%
112	7%	1,50E-10	78%
	12%	1,30E-10	81%
	0%	9,60E-10	-
тз	5%	5,40E-10	44%
	7%	2,60E-10	73%
	12%	2,40E-10	75%
		Table 5 2	

Table 5.2

The R value in table 5.2 can be used as a parameter to evaluate the efficiency of the bentonite treatment. It indicates the reduction of hydraulic conductivity referred to the raw material and is calculated with the following expression:

$$R = \frac{K_{SAT (untr)} - K_{SAT (tr)}}{K_{SAT (untr)}} \%$$
(5.7)

The bentonite treatment contributes to decrease significantly the hydraulic conductivity of the raw materials.

Referring to R values, it is possible to notice that the bentonite treatment tends to reach the maximum of its effectiveness at 7%, since increasing the bentonite rate to 12% produces an additional decrease of hydraulic conductivity, which is not sufficient to justify the bentonite content increase.

In figure 5.2, the logarithmic value of  $K_{\text{SAT}}$  is represented in function of the bentonite content.

From figure 5.1 and figure 5.2, it is possible to notice that there is not an evident linear correlation between the hydraulic conductivity measured with the oedo-permeameter and the bentonite percentage of the treatment.



Figure 5.2

# **5.2 CORRELATIONS BETWEEN KSAT AND SOIL PROPERTIES**

Different authors have tried to find a relation between hydraulic conductivity, and the parameters that describe the nature of the soil (Magnan, 2000).

In the following paragraphs, some of these approaches will be applied on the studied soils.

In paragraph 5.2.1, three equations thought for sandy materials will be applied to the untreated soils, T12 and T3.

In paragraph 5.2.2, the Sivakumar correlation, thought for cohesive materials, will be applied to analyse the effect of the bentonite treatment.

The analysis will be referred to the 0/5 mm fraction of the materials. In particular, the reference diameter  $d_{10}$  and  $d_{15}$  will be used;  $d_{10}$  and  $d_{15}$  are the diameters in m

corresponding to, respectively, the 10% and 15% of cumulative passant in the granulometric curve.

All the applied equations consider the compaction conditions of the materials using the porosity value (n) or the void ratio value (e). The analysis will be referred to the results of the standard Proctor tests performed on each soil with a water content equal to  $w_{OPT}+2\%$  as suggested by *BRGM* (Chapter 1, paragraph 1.4.3).

Therefore, the porosity was calculated with the following equation:

$$n = 1 - \frac{\gamma_{d \ OPT+2\%}}{\gamma_s} \tag{5.8}$$

Where  $\gamma_{d \text{ OPT+2\%}}$  is the dry unit weight calculated with the Proctor test and  $\gamma_s$  is the solid unit weight (chapter 4, paragraph 4.4).

The void ratio can be directly calculated with the following equation:

$$e = \frac{n}{1 - n} \tag{5.9}$$

		d <sub>10</sub> [mm]	d <sub>15</sub> [mm]	γ₅ [kN/m³]	<sup>γ</sup> d OPT+2% [kN/m <sup>3</sup> ]	n	е
	0%	0,00093	0,00156	26,79	18,77	0,3049	0,4386
T12	5%	-	-	26,77	17,51	0,3515	0,5421
112	7%	-	-	26,76	16,92	0,3732	0,5954
	12%	-	-	26,74	16,47	0,3901	0,6395
	0%	0,00129	0,00391	27,36	19,46	0,2793	0,3875
Т3	5%	-	-	27,31	18,13	0,3284	0,4889
	7%	-	-	27,29	17,95	0,3353	0,5045
	12%	-	-	27,25	17,40	0,3554	0,5514

In table 5.3, the values of  $d_{10}$ ,  $d_{15}$ ,  $\gamma_s$ ,  $\gamma_{d \text{ OPT+}2\%}$ , n and e are displayed.

#### 5.2.1 Untreated soils

In this paragraph, the hydraulic conductivity of the untreated soils, T12 and T3, will be calculated using three correlations which are generally used to obtain the hydraulic conductivity of cohesionless soils.

<u>Kozeny</u> (1927) proposed an expression to evaluate the hydraulic conductivity of granular materials. This expression was refined by Carman in 1937 and 1956.

The authors represent the porous structure of the soil as a system of pipes characterised by tortuosity and a little diameter (capillary model). The expression for the hydraulic conductivity K (m/s) is the following:

$$K = \frac{1}{k_0 T^2 S_0^2} \cdot \frac{\gamma_w}{\mu} \cdot \frac{n^3}{(1-n)^2}$$
(5.10)

Where  $k_0$  is the pore shape factor, T is the tortuosity of the flow path,  $S_0$  is the specific surface area of soil particle,  $\mu$  is the dynamic viscosity of water (kg/(ms)),  $\gamma_w$  is the unit weight of water (N/m<sup>3</sup>), n is the porosity of the soil.

Equation 5.10 is difficult to apply because it is necessary to evaluate two coefficients:  $k_0$  and T.

In order to apply the formula in a simpler way, different ways of calculating  $k_0$ , T and  $S_0$  have been proposed. In this work the following form will be applied:

$$K = \frac{1}{80} \cdot d_{10}^2 \cdot \frac{\gamma_w}{\mu} \cdot \frac{n^3}{(1-n)^2}$$
(5.11)

Where  $d_{10}$  is expressed in m.

Chapuis (2004) elaborated the following correlation for loose sands:

$$K = 0,0246 \cdot \left[\frac{d_{10}e^3}{1+e}\right]^{0,7825} = 0,0246 \cdot \left[\frac{d_{10}n^3}{(1-n)^2}\right]^{0,7825}$$
(5.12)

Where  $d_{10}$  is expressed in m.

<u>Bezuijen</u> (2010) proposed the following correlation, based on Kozeny and Carman's studies:

$$K = \frac{1}{160} \cdot d_{15}^2 \cdot \frac{\gamma_w}{\mu} \cdot \frac{n^3}{(1-n)^2}$$
(5.13)

Where  $d_{15}$  is expressed in m.

In table 5.4 and in the semi-logarithmic graphic of figure 5.3, the results of the analysis are displayed. The values obtained with the oedo-permeameter are shown too.

	K <sub>SAT</sub> [m/s]			
	T12 0% T3 0%			
Kozeny Carman	5,81E-09	9,69E-09		
Chapuis	4,84E-08	5,60E-08		
Bezuijen	8,13E-09	4,43E-08		
Oedo-Permeameter	6,70E-10	9,60E-10		

	<b>4</b> ,84E-08	5,60E-08 4,43E-08	
1,00E-08	<ul> <li>8,13E-09</li> <li>5,81E-09</li> </ul>	<b>9</b> ,69E-09	<ul> <li>Kozeny Carman</li> <li>Chapuis</li> <li>Bezuijen</li> </ul>
1,00E-09	<b>6</b> ,70E-10	<b>9</b> ,60E-10	• Oedo-Permeamete

Table 5.4



The values obtained from the oedo-permeameter are used as a reference to evaluate the reliability of the correlations. In fact, these values can be considered the most realistic ones since they are obtained from a direct experimentation on the soils.

The oedo-permeameter gives the lowest values of hydraulic conductivity.

All the three correlations tend to overestimate the hydraulic conductivity of the soils. This is mainly because these correlations were obtained from studies on cohensionless soils and are not suitable for clayey soils (Mitchell, 1993).

However, Kozeny-Carman equation 5.11 gives results which are nearer to the oedopermeameter ones, with a difference of about an order of magnitude.

#### 5.2.2 Treated soils

In this paragraph, the effect of bentonite treatments on the hydraulic conductivity of T12 and T3 will be analysed using the Sivakumar correlation.

<u>Sivakumar</u> proposed the following equation to calculate the hydraulic conductivity of normally consolidated clays:

$$2,50 + 0,24 \cdot \log K = \frac{e}{e_L} \tag{5.14}$$

Where e is the void ratio corresponding to the Proctor standard compaction at  $w_{OPT}+2\%$  (table 5.2),  $e_L$  is the void ratio of the soil with a water content equal to the liquid limit  $w_L$ . From equation 5.14, it is possible to obtain K:

$$K = 10^{\wedge} \left( \frac{\frac{e}{e_L} - 2.5}{0.24} \right)$$
(5.15)

In order to calculate the  $e_{L}$ , it is possible to suppose that at a water content equal to  $w_{L}$  the soil is saturated. When the soil is in saturation conditions the void volume is equal to the water volume inside the soil, hence it is possible to calculate  $e_{L}$  with the following equation:

$$w_L = e_L \frac{\gamma_w}{\gamma_s} \tag{5.16}$$

Where  $\gamma_w$  is the water unit weight (kN/m<sup>3</sup>) and  $\gamma_s$  is the solid unit weight (kN/m<sup>3</sup>).

In table 5.5, the values of  $w_L$  and  $e_L$  are displayed.

		WL	e∟				
	0%	31,25%	0,8534				
T12	5%	53,74%	1,4665				
	7%	76,00%	2,0730				
	12%	110,35%	3,0079				
	0%	32,50%	0,9064				
тз	5%	52,87%	1,4717				
	7%	63,24%	1,7592				
	12%	109,23%	3,0340				
II							

Table 5.
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In figure 5.4 and table 5.6, the results of equation 5.15 are displayed. In figure 5.4, the hydraulic conductivity values obtained with the oedo-permeameter test (paragraph 5.1.3) are also represented.



Figure 5.4

	K <sub>SAT</sub> (Sivakumar) [m/s]		
	T12	Т3	
0%	4,68E-09	2,82E-09	
5%	1,22E-09	1,04E-09	
7%	5,64E-10	6,55E-10	
12%	2,80E-10	2,29E-10	

	Tal	ble	5.	6
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The difference between the results obtained with the oedo-permeameter and Sivakumar, decreases significantly with the bentonite content increase.

If the hydraulic conductivity is plotted using logarithmic values (figure 5.5), it is possible to notice that the results obtained with Sivakumar's equation 5.15 are linearly correlated to the bentonite percentage of the treatment.



Figure 5.5

# **5.3 SUMMARY OF RESULTS**

In table 5.7, all the values of hydraulic condu	ctivity obtained in the previous paragraphs
are displayed.	

		Oedo- Permeameter	Sivakumar	Kozeny Carman	Chapuis	Bezuijen
T12	0%	6,70E-10	4,68E-09	5,80841E-09	4,8414E-08	8,1321E-09
	5%	5,60E-10	1,22E-09	-	-	-
	7%	1,50E-10	5,64E-10	-	-	-
	12%	1,30E-10	2,80E-10	-	-	-
тз	0%	9,60E-10	2,82E-09	9,68712E-09	5,6012E-08	4,4344E-08
	5%	5,40E-10	1,04E-09	-	-	-
	7%	2,60E-10	6,55E-10	-	-	-
	12%	2,40E-10	2,29E-10	-	-	-

Та	ble	5.	7
10	210	Ο.	

The reference values for the following chapters will be the oedo-permeameter ones. In fact, they are the most affordable since they were obtained analysing directly the water movement in the soils of interest and following the standard procedure described in the French norm (NF X 30-441 and NF X 30-442).
# CHAPTER 6

# Influence of Bentonite Treatments on Soil in Unsaturated Conditions

In this chapter, the behaviour in unsaturated conditions of the untreated and treated soils described in chapter 4 is analysed.

The hanging column test was performed on the samples and, from the obtained results, it was possible to build the first part of the Soil Water Retention Curve (SWRC) corresponding to low suction values.

Since the exposition of all the calculations would be too burdensome for this report, the complete results can be consulted in the attachment.

# **6.1 DESCRIPTION OF THE HANGING COLUMN TEST**

The hanging column test consists in applying a predetermined value of suction to soil samples put in a box (sandbox) in order to find the corresponding moisture content. The instrumentation used for the test is shown in figure 6.1 and schematised in figure 6.2 (Eijkelkamp, 2007).



Figure 6.1



Figure 6.2

The sandbox (1) contains a fine synthetic sand separated from the samples by a nylon filter cloth and characterised by a flexible surface to guarantee contact between the sand and the samples. In fact, the sand has the function to convey the amount of suction from the drainage system to the samples.

The box lid (3) is used to prevent evaporation during the test.

Tap A can be turned to "supply", "discharge" and "closed" position. When it is turned on "supply" position water can flow from the supply bottle (6) to the sandbox. When it is turned on "discharge" position, water can flow from the sandbox to the discharge pipe (9), Mariotte's bottle (12) and finally the outflow pipe (13). When it is turned on "closed" position, no flow of water from or to the sandbox can take place.

Mariotte's bottle (12) is a device used to maintain constant the flow rate when tap A is not closed.

The amount of suction is determined by the negative pressure head, which is the difference in height between the suction regulator (11), and the middle of the soil samples (14). Since the suction can be increased lowering the suction regulator, there is a physical limit to the suction applicable to the samples. In particular, this device is likely to apply a pressure head from 0,01 m to 1 m, corresponding to a range of suctions from 0,10 Pa to 10 Pa.

Due to this limit, the hanging column test can be used to find the moisture content of the samples corresponding to low values of suction.

As already outlined, the moisture content of the samples is measured referring to predetermined levels of applied suction which correspond to a specific value of pF marked on the sliding measuring stand (10).

The pF expresses the force with which soils hold water in the capillarity between soil particles. Therefore, it is a measure of suction with which is in relation with the following equation:

$$pF = log_{10}h[cm]$$

$$pF = log_{10}h[m] + 2$$
(6.1)

Where h is the pressure head (m) which calculated with the following expression:

$$h = \frac{s}{\gamma_w} \tag{6.2}$$

Where s is the suction (kPa) and  $\gamma_w$  is the water unit weight (kN/m<sup>3</sup>).

Knowing the value of the applied pF, it is possible to find the corresponding pressure head and suction respectively from the equation 6.1 and the equation 6.2.

In table 6.1 the suction levels to be considered during the test are shown.

Level pF		h [m]	s [Pa]
1	<b>1</b> 0,00		0,10
2	<b>2</b> 0,40		0,25
3	1,00	0,1000	1,00
4	1,50	0,3162	3,16
5	<b>5</b> 1,80		6,31
6	<b>6</b> 2,00		10,00

To	610	6	1
ıa	pie	6.	1

It is possible to distinguish two phases of the test: the saturation phase and the suction phase.

During the saturation phase, the samples are put into the sandbox in which the water level is at 1 cm under the soil samples top. The time necessary for the soil sample to reach saturation depends on the soil composition and granulometry.

During the suction phase, when all the samples in the sandbox reach the equilibrium at a specific applied suction, than, it is considered that these weights are the nominal value for this suction value. After that, the suction is increased lowering the suction regulator till the next pF value. The procedure is repeated for each suction level.

After the test end, the soil samples are dried in the oven and weighed in order to deduce from the dry mass measure the water content corresponding to each suction level.

The test procedure will be described in more detail in paragraph 6.4.

# **6.2 PREPARATION OF THE SAMPLES**

A typical sample used for the hanging column test is shown in figure 6.3 and figure 6.4. It is composed by the following parts:

- The soil sample compacted with the Proctor procedure.
- A metal ring used to retrieve the sample from the Proctor mould.
- A filter used to prevent the loss of material from the sample during the suction experiment and the direct contact between the soil and the sandbox.
- A string used to keep in tension the filter and to prevent it from moving.



Figure 6.3



Figure 6.4

The hanging column test was performed at the same time on the following 15 samples:

- 2 samples of T12;
- 2 samples of T12 5%;
- 2 samples of T12 7%;
- 2 samples of T12 12%;
- 2 samples of T3;
- 1 samples of T3 5%;
- 2 samples of T3 7%;
- 2 samples of T3 12%;

In figure 6.5 all the samples just before being put in the sandbox are shown.



Figure 6.5

Only one sample was made for the T3 5% due to the lack of a metal ring necessary for the sample preparation. It has been chosen to sacrifice the T3 5% because the results of this material are less important for the project of the CSM cap cover system. In fact, as described in chapter 1, the CSM area is mainly formed by the T12 material while the T3 interests a smaller part of the site. Regarding the bentonite treatments, since in the final cap cover system the use of the 7% one is previewed (chapter 4 paragraph 4.2.2), the main concern is to show that T12 7% and T3 7% are characterised by an adequately low hydraulic conductivity and that the 12% treatment does not decrease significantly this value.

For the preparation of the samples the following procedure was followed:

- 1. Compaction of the untreated and treated soil with the Standard Proctor procedure.
- 2. Extraction of the sample from the Proctor mould with the normal compression device.
- 3. Application of the filter and measure of the sample mass, height and volume.

The scale used for all the mass measures, made during the samples preparation, was characterised by an accuracy of  $\pm 0.01$  g.

### 6.2.1 Compaction with Proctor procedure

The first step in the preparation of the samples consists in compacting the soil and water mixture with the Standard Proctor Procedure, already described in chapter 4. Water was added to the mixture of soil and eventually bentonite in a quantitative equal to  $w_{OPT}$ +2%, where  $w_{OPT}$  is the optimum water content found for the considered material with the Standard Proctor test (chapter 4 paragraph 4.5).

After drying the material, it is possible to measure the effective water content of the material compacted in the mould and the corresponding dry unit weight  $\gamma_{d \text{ OPT+2\%}}$ . All the mentioned parameters are displayed for each material in table 6.2.

	WOPT	WOPT+2% Calculated value	WOPT+2% Measured value	<b>ү</b> d орт+2% [kN/m³]
T12 0%	12,30%	14,30%	14,29%	18,77
T12 5%	14,90%	16,90%	17,91%	17,51
T12 7%	16,40%	18,40%	19,73%	16,92
T12 12%	17,60%	19,60%	21,36%	16,47
T3 0%	11,10%	13,10%	12,63%	19,46
T3 5%	14,20%	16,20%	16,77%	18,13
T3 7%	15,00%	17,00%	17,40%	17,95
T3 12%	15,20%	17,20%	18,38%	17,40

Table 6.2

It is possible to notice that, for almost all the materials, the actual water content is different from the one calculated for the mixing. This happens due to inevitable imprecision committed during the mixing (water not absorbed in a perfectly homogeneous way by the soil, evaporation of water) and due to the initial water content of T12 and T3. In fact, the actual water content values generally result slightly higher than expected. However, for the purpose of the study, this does not represent a problem since, as mentioned in chapter 1 paragraph 1.4.3, soils used in cap cover systems are generally installed with a water content equal to the optimum +2% or +3% and, in any case, included in the range  $[W_{OPT+2\%}; W_{OPT+6\%}]$  (ADEME, BRGM, 2001).

From now on, the  $w_{OPT+2\%}$  expression will be used to refer to the measured value and not to the calculated one to prepare the mixture.

# 6.2.2 Retrieving of the sample with normal compression

After the compaction, a metal ring was weighed and put on the surface of the compacted soil in the Proctor mould. In order to make the metal ring penetrate into the soil, without disturbing the sample, a normal compression device was used (figure 6.6). Moreover, a lubrification by a thin layer of grease was applied on the metal ring surfaces before using the normal compressor to make the punching easier.



Figure 6.6

The process is repeated another time using a second metal ring put on the opposite surface of the soil in the mould.

The samples were obtained removing all the extra soil of the mould and eventually levelling the surfaces of the sample to make them more homogeneous.

Therefore, from each mould prepared for a specific material, it was possible to obtain two samples, which have been called E1 and E2. The only exception is the T3 5% for which only the E1 has been prepared.

# 6.2.3 Measure of samples mass, height and volume

After putting the filters and the strings, the final samples have been obtained.

It is important to register the sample masses during their preparation in order to be able to know the soil sample masses at every step. The following masses have been registered:

- Metal ring mass (m<sub>ring</sub>).
- Soil and metal ring mass (m<sub>soil+ring</sub>).
- Soil, metal ring, filter and string mass (m<sub>soil+ring</sub>)
- Filter and string mass obtained from difference between m<sub>soil+ring+filter+string</sub> and m<sub>soil+ring</sub> (m<sub>filter+string</sub>).

These values, obtained at the end of each sample preparation, are displayed in table 6.3 and table 6.4 respectively for T12 and T3. The values are referred to a water content equal to  $w_{OPT+2\%}$ .

	T12 0%	T12 0%	T12 5%	T12 5%	T12 7%	T12 7%	T12 12%	T12 12%
	E1	E2	E1	E2	E1	E2	E1	E2
m <sub>ring</sub> [g]	41,03	41,40	43,26	42,82	55,41	42,80	43,24	43,03
m <sub>soil+ring</sub> [g]	162,95	162,80	155,46	157,35	163,07	154,12	153,74	153,79
m <sub>soil+ring+filter+string</sub> [g]	164,78	163,95	157,30	159,09	164,90	155,99	155,55	155,66
m <sub>filter+string</sub> [g]	1,83	1,15	1,84	1,74	1,83	1,87	1,81	1,87

Table 6.3

	T3 0%	T3 0%	T3 5%	T3 7%	T3 7%	T3 12%	T3 12%
	E1	E2	E1	E1	E2	E1	E2
m <sub>ring</sub> [g]	42,39	41,92	43,55	41,35	42,77	43,25	41,80
m <sub>soil+ring</sub> [g]	162,03	166,84	159,23	158,20	156,57	157,87	158,13
m <sub>soil+ring+filter+string</sub> [g]	163,89	168,78	161,20	160,09	158,47	159,58	159,79
m <sub>filter+string</sub> [g]	1,86	1,94	1,97	1,89	1,90	1,71	1,66

Regarding the geometry of the samples, the height H was measured with the Vernier calliper, the diameter D was constant for all the samples and the volume V was calculated with the cylinder formula.

The values of D, H and V are indicated in table 6.5 and table 6.6 respectively for T12 and T3.

	T12 0%	T12 0%	T12 5%	T12 5%	T12 7%	T12 7%	T12 12%	T12 12%
	E1	E2	E1	E2	E1	E2	E1	E2
D [cm]	5,00	5,00	5,00	5,00	5,00	5,00	5,00	5,00
H [cm]	2,97	2,97	3,00	3,03	2,98	3,00	3,02	3,00
V [cm³]	58,32	58,32	58,90	59,49	58,51	58,90	59,30	58,90

	T3 0%	T3 0%	T3 5%	T3 7%	T3 7%	T3 12%	T3 12%
	E1	E2	E1	E1	E2	E1	E2
D [cm]	5,00	5,00	5,00	5,00	5,00	5,00	5,00
H[cm]	2,96	3,01	3,03	2,99	2,98	3,00	3,03
V [cm³]	58,12	59,10	59 <i>,</i> 49	58,71	58,51	58,90	59,49

### Table 6.6

# **6.3 PRELIMINARIES TO THE TEST**

Before describing the test process (paragraph 6.4) it is necessary to introduce the parameters and the equations used during the test to control its progress.

Moreover, since before concluding the test, samples dry masses are not accesible directly, these values are in first approximation calculated from the Proctor test as explained in paragraph 6.3.1. At the end of the test, after the drying in the oven, the true dry mass values will be used for the parameters calculation (paragraph 6.5).

In paragraph 6.3.2 the equations used to calculate the water retention parameters are shown.

Regarding the gravitational acceleration constant g and the water unit weight  $\gamma_w$ , the following values were used:

- Gravitational acceleration constant: g=10 m/s<sup>2</sup>.
- Water unit weight:  $\gamma_w$ = 10 kN/m<sup>3</sup>.

#### 6.3.1 Calculation of samples dry masses and dry unit weights

The dry masses of the samples are calculated with the following equation, which derives directly from the definition of gravimetric water content (chapter 2 paragraph 2.1.1):

$$m_d = \frac{m_{soil+ring} - m_{ring}}{1+w} \tag{6.3}$$

Where  $m_d$  is the dry mass (g) of the considered sample,  $m_{soil+ring}$  and  $m_{ring}$  are the masses (g) already defined in paragraph 6.2 and w is the gravimetric water content expressed in decimal form.

The parameters of the equation 6.3 are assessed in reference to the Proctor procedure. Therefore  $m_{soil+ring}$  corresponds to the mass measured for the soil sample with the Proctor mould and w corresponds to the  $w_{OPT+2\%}$ .

The values of  $m_{soil+ring}$  and w have already been presented respectively in table 6.4 and table 6.2 (paragraph 6.2).

The unit weights of the samples are calculated with the following equation:

$$\gamma_d = \frac{m_d}{V}g \tag{6.4}$$

Where  $\gamma_d$  is the unit weight of the considered sample (kN/m<sup>3</sup>), m<sub>d</sub> is the dry mass of the sample (kg), g is the gravitational acceleration (m/s<sup>2</sup>) and V is the volume of the sample (m<sup>3</sup>) as calculated in paragraph 6.2.3 (table 6.5).

#### 6.3.2 Calculation of water content parameters

In this paragraph, the equations used to calculate the gravimetric water content w, the volumetric water content  $\theta$  and the degree of saturation S are presented.

Gravimetric water content 
$$w = \frac{m_w}{m_d} = \frac{m_h - m_d}{m_d}$$
 (6.5)

Where  $m_w$  (g) is the water mass in the sample calculated as the difference between the soil sample mass  $m_h$  (g) and the dry sample mass  $m_d$  (g).

For measuring m attention must be paid in considering only the soil mass and not the other elements forming the sample (metal ring, filter and string).

Volumetric water content 
$$\theta = w \cdot \frac{\gamma_d}{\gamma_w}$$
 (6.6)

Where  $\gamma_d$  is the dry soil unit weight (kN/m<sup>3</sup>) calculated with equation 6.4 and  $\gamma_w$  is the water unit weight (kN/m<sup>3</sup>).

Degree of saturation 
$$S = w \cdot \frac{\gamma_s \gamma_d}{\gamma_w (\gamma_s - \gamma_d)}$$
(6.7)

Where  $\gamma_s$  is the unit weight of the solid fraction of the sample (kN/m<sup>3</sup>). The equation 6.7 derives directly from the definition of degree of saturation (chapter 2 paragraph 2.2.1) in which the residual volumetric content was considered equal to zero. The value of S must be between 0% (perfectly dry conditions) and 100% (saturation

,

conditions).

# 6.4 THE TEST PROCESS

In this paragraph, the running of the hanging column test will be described.

The test started the 22<sup>nd</sup> April 2014 and ended the 1<sup>st</sup> July 2014, for a total duration of 71 days.

In the following paragraphs, these aspects of the test process will be covered:

- The test phases (preparation of the sandbox, saturation phase, suction phase).
- Survey of temperature and humidity oscillations in the sandbox.
- Problems encountered during the test.

The scale used for all the mass measures, made during the test, were characterised by an accuracy of  $\pm 0,01$  g.

# 6.4.1 Preparation of the sandbox

Before putting the samples in the sandbox, the instrumentation must be prepared for the test. The detailed assembling of the sandbox is exposed in the *Operating Instructions* by Eijkelkamp (2007), but in this work only the last steps of the preparation will be described since the laboratory instrumentation had already been set for the utilization.

At first, the sand surface in the sandbox was smoothed and left to settle.

Then, the tap A (figure 6.2) was turned to the "supply" position and the tap B was opened (figure 6.2) so that the water could flow from the supply bottle to the sandbox through the sand drain removing the residual air bubbles.

Once about 1 cm of water level was reached above the sand, all the taps were closed. Then, in order to remove possible bubbles entrapped between the sand and the protection cloth, the surface was gently smoothed from the centre of the box to its sides.

After that, the tap A was turned to the "discharge" position to lower the water level to 0,5 cm above the sand surface. This 0,5 cm layer of water must be ensured in order to prevent air to enter into the sand layer.

At the end of this process, all the taps were closed and all the prepared soil samples were put in the sandbox with the bottom part, covered by the filter, in contact with the protection cloth above the sand surface.

Before proceeding to the saturation phase and starting the test, it is fundamental to prepare a map of the box so that the samples can be replaced to the exact position each time they are taken out of the sandbox for measurements. Rotations of the samples in relation to the original positions must be avoided too.

This procedure is important in order to be able to re-establish the same contact conditions between the soil samples and the sandbox and, hence, to guarantee the same leakage conditions during the test.



Figure 6.7 shows the samples in the sandbox and points out their positions.

Figure 6.7

# 6.4.2 Saturation phase

After setting the samples in the sandbox, they were left for adapting for 1 hour. Then, the tap A was turned to "supply" position to raise the water level until it reached 1 cm below the samples top (figure 6.8) and then turned to the "closed" position. The water level must raise slowly to prevent air from entrapping into the samples and damaging the soil structure (Eijkelkamp, 2007).



Figure 6.8

After having closed the box with the lid to prevent evaporation, the samples were left into the sandbox to allow saturation.

The duration of the saturation phase depends on the soil type: for sandy soils, it lasts few days while for clayey soils it can last more than a week.

The analysed soil samples are characterised by a high content of fine particles (chapter 4, paragraph 4.3.1). In particular, the higher the bentonite content, the longer the time needed to reach saturation. Therefore, the saturation phase lasted for four weeks, more precisely, from 23<sup>rd</sup> April until 19<sup>th</sup> May, for a total of 26 days.

In order to verify if the sample had reached the saturation the sample masses were measured after a week and again after 24 hours to check mass variations between the two measures. In fact, assuming ring, filter, string and soil particles masses as constants, the only variable that can change the sample mass is water. Hence, when the sample mass does not show variations, it means that no more water can fill the soil sample porosities and that the soil is saturated.

The registered weights are used to calculate water content at pF=0 (h=0,01 m) which, hence, corresponds to the saturation conditions.

Due to the fluctuations in the samples masses measures, it was difficult to understand if the samples had reached or not the saturation. Nevertheless, in order to avoid changing the soil samples structure, saturation phase was concluded when a certain difference between two consecutive mass measures of all the samples was reached. The factors at the origin of these fluctuations in the measures will be described and discussed in paragraph 6.5.

#### 6.4.3 Suction phase

When all the samples contained in the sandbox were saturated, the suction phase was started. It lasted from 19<sup>th</sup> May to 1<sup>st</sup> July for a total of 43 days.

During this phase tap A was maintained in "discharge" position and the suction regulator was lowered to the following suction level each time that equilibrium was reached for all the samples.

In order to define the equilibrium value of a sample, the sample was weighed two times with an interval of 24 hours between the two measures. The water volumetric content  $\theta$  corresponding to the two measures was calculated with the equation 6.6. If the difference between the two volumetric water contents was less than or equal to 0,20% the sample was assumed in equilibrium. If this condition was not verified, the measures were repeated at an interval of a week until the criterion was satisfied.

In some cases a difference in volumetric content slightly superior to 0,20% was accepted because of the difficulty in reaching this limit value in reasonable times, especially for the samples treated with 12% of bentonite. This choice was taken in order to avoid changes in the soil samples structures.

Since the first level (pF=0,4) corresponds to a suction value near to the zero, sometimes taking precise measures could be difficult. In our case the outflow water from the sandbox was too small and the level lowering in the sandbox too slow. Therefore, since the pF=0,4 point corresponds to the situation in which the water level is equal to the sand level, the suction regulator was temporarily lowered so that this situation could be reached. After this first critical point, the standard procedure previously described was adopted.

### 6.4.4 Temperature and humidity survey

Since temperature variations affect water viscosity and hence water retention values, the laboratory temperature should be constant between the measurements.

For verifying this condition, temperature and relative humidity in the sandbox were monitored during the test with a temperature and humidity USB data logger. Thanks to this system, temperature and humidity values can be monitored for long periods. Data have been registered in the logger from 13<sup>th</sup> May to the 24<sup>th</sup> June each hour of the day. After collecting the data in the computer, the graphic in figure 6.9 was obtained.





The average values are indicated with the dotted lines.

During the period of measure, the relative humidity was almost constant and near the medium value of 105%. The temperature exhibited more variations, with peaks of  $\pm 5^{\circ}$ C respect to the medium value of 23,70°C. These variations were due to meteorological changes and are still acceptable without alteration in the context of this test.

# 6.4.5 Dry samples masses and filters masses

When all the mass measurements of the samples were registered for each prefixed suction level, the samples were removed from the sandbox, dried in the oven at 105°C and weighted to obtain the true dry soil mass value  $m_d$ .

In order to correct the results the string mass  $m_{string}$ , the dry filter mass  $m_{d \ (filter)}$  and the saturated filter mass  $m_{sat \ (filter)}$  were measured.

The dry filter masses were measured right after the drying in the oven.

The saturated filter masses were measured after immersing them in water.

In table 6.7 and table 6.8 the values of  $m_d$ ,  $m_d$  (filter),  $m_{sat}$  (filter) and  $m_{string}$  are presented respectively for T12 and T3.

	T12 0%	T12 0%	T12 5%	T12 5%	T12 7%	T12 7%	T12 12%	T12 12%
	E1	E2	E1	E2	E1	E2	E1	E2
m <sub>d</sub> [g]	105,77	104,54	93 <i>,</i> 45	96,33	88,20	91,81	89,46	89,79
m <sub>d (filter)</sub> [g]	0,46	0,35	0,43	0,37	0,44	0,46	0,42	0,48
m <sub>sat (filter)</sub> [g]	0,99	0,65	0,92	0,76	1,04	1,14	0,80	0,99
m <sub>(string)</sub> [g]	1,55	1,56	1,54	1,54	1,52	1,52	1,57	1,54

#### Table 6.7

	T3 0%	T3 0%	T3 5%	T3 7%	T3 7%	T3 12%	T3 12%
	E1	E2	E1	E1	E2	E1	E2
m <sub>d</sub> [g]	104,70	109,18	98,07	97,65	95,56	94,85	96,99
m <sub>d (filter)</sub> [g]	0,48	0,54	0,53	0,52	0,49	0,37	0,36
m <sub>sat (filter)</sub> [g]	0,93	1,04	1,12	1,07	1,10	0,68	0,79
m (string) [g]	1,57	1,58	1,55	1,54	1,55	1,56	1,54

Table 6.8

# **6.5 PROBLEMS ENCOUNTERED AND CORRECTIONS**

Interpretation and correction of results of the hanging column test is an important and tricky phase. Actually, this test is subjected to a series of problems, which can affect measures accuracy. Moreover, since the test is performed on small samples, even the slightest change of the sample condition could affect dramatically the mass measurements and, hence, the final results regarding the water retention.

Therefore, it was necessary to record all the possible reasons of discrepancy occurring

during the saturation and suction phase in order to correct the results.

In this paragraph, the following problems will be discussed:

- Reference level for the zero suction,
- Overestimation of the water content due to the water dripping from the filters,
- Assessment of the filter mass during the experiment,
- Swelling of the samples treated with bentonite,
- Fine material loss from the samples,
- Presence of mushrooms on the sample upper surface.

# 6.5.1 Reference level for the zero suction

As already mentioned, the reference level for zero suction is the middle of the soil sample, but the free water level is 1 cm below the top of the sample. Therefore, the moisture tension ranges from +1 cm at the bottom of the sample, to H-1 cm at the top of the sample, where H is the sample height (Eijkelkamp, 2007).

This inaccuracy is intrinsic of the test saturation procedure and affects the precision of the water content calculation at saturation (pF=0). Nevertheless, this imprecision becomes less and less relevant with the suction increasing (Eijkelkamp, 2007).

# 6.5.2 Overestimation of water content

When the samples are taken out from the sandbox for the measurements, the change in water content is inevitable (Eijkelkamp, 2007).

It was noticed that extra water dripping from the sample could influence the sample mass measure leading to an overestimated water content.

In fact, calculating the degree of saturation S during the first phases of the test with the equation 6.7, values superior to 100% were found. Nevertheless, as already stated in paragraph 6.3.2, the maximum value of S is 100% at which the sample is saturated. Hence, when values of S superior to 100% were found, this meant that the soil sample water content was being overestimated.

In order to mitigate this effect to the minimum, the following procedure was adopted to obtain the mass measures both during saturation and suction phase:

1. The sample was removed from the sandbox, letting the extra water drip directly into the sandbox.

- 2. The sample was put on a plate to let further dripping of the extra water
- 3. The sample was put on the scales and the mass was recorded.

Different steps were carried out quickly in order to remove only extra water from the sample and to avoid changing the soil sample water content.

This problem was more relevant for the untreated materials than for the treated ones. This is probably due to the fact that the bentonite inside the treated samples helps the soil to retain more water in the porosity.

Moreover this problem was mainly related to the saturation phase since the samples were immersed in water. In fact, with the increasing suction and the subsequent lowering of the water level into the sandbox, the presence of extra water became less and less actual. In fact, on the plate used in the second step of the procedure we could find less and less water proceeding with the test.

# 6.5.3 Assessment of filter mass during the experiment

In order to obtain the gravimetric water content through the equation 6.5, it was necessary to calculate the humid soil mass removing from the recorded sample, the contribution of the string, the metal ring and the filters.

While the string mass and the metal ring mass do not depend on the presence of water and remain constant during all the experimentation, the filter masses depend on the kept of the water. The masses of the dry and saturated filters were recorded as explained in paragraph 6.4.5 but it is not possible to know the variation of the filter masses during the test.

In this study, it was decided to use always the saturated filter mass value for calculating the gravimetric water content with the equation 6.5. In fact, it was assumed that the filters are characterised by the ability to retain water even during the suction phase (porous medium with a high air entry pressure).

Therefore, the water mass inside each sample,  $m_{w_i}$  was calculated with the following equation:

$$m_w = m_h - m_d - m_{ring} - m_{string} - m_{sat \ (filter)} \tag{6.8}$$

Where  $m_h$  is the humid mass of the sample measured during the saturation or suction phase (g),  $m_d$  is the dry mass of the soil sample (g),  $m_{ring}$  is the metal ring mass (g),  $m_{string}$ is the plastic string mass (g) and  $m_{sat (filter)}$  is the mass of the saturated filter (g).  $m_{ring}$  values have been presented in paragraph 6.2.3.  $m_d$ ,  $m_{string}$  and  $m_{sat (filter)}$  values have been presented in paragraph 6.4.5.

# 6.5.4 Swelling of the samples

During the saturation phase, all the treated samples exhibited swelling.

Because of the high content in bentonite, the first samples to swell under a given suction were the T12 12% and T3 12%, followed by T12 7% and T3 7% and finally by T12 5% and T3 5%.

When swelling occurs, the soil sample volumes as calculated in paragraph 6.2.3, are not any more constant during the test. In order to determine the correct gravimetric water content w (equation 6.5) and consequently the volumetric water content  $\theta$  (equation 6.6) and the degree of saturation S (equation 6.7), it was necessary to recalculate the volume each time the sample mass had to be registered.

Because of the metal ring confining the sample laterally, the diameter remained constant even with the swelling. Hence, the parameter changing with the swelling was the sample height.

Each time the sample masses had to be recorded, the following procedure was followed for the samples exhibiting swelling:

- 1. The height was measured with a Vernier calliper.
- 2. The measure was repeated three times for each sample.
- 3. The final sample height was calculated as the arithmetic mean of the three measured values.
- 4. The new volume was calculated with the cylinder formula.
- The new dry unit weight of the swelled samples was calculated with the equation
   6.4.
- 6. The gravimetric water content w, the volumetric water content  $\theta$  and the degree of saturation S were calculated respectively with the equation 6.5, equation 6.6 and equation 6.7.

During the operations with the Vernier calliper, attention was paid to avoid disturbing the sample and removing soil from its surface in order not to change the mass.

Looking at the heights measurements (attachment), it is possible to notice that heights increased due to the swelling but began decreasing after a certain point. It means that the

soil samples were subjected to shrinkage in the last part of the test.

The first samples to shrink were the T12 12% and T3 12%, followed by the samples containing less bentonite, hence, the T12 7% and T3 7% and finally the T12 5% and T3 5%.

The swelling of T12 12% and T3 12% was monitored during all the experiment since these samples exhibited swelling at the beginning of the saturation phase. The swelling of T12 5%, T12 7%, T3 5% and T3 7% started to be observed during the suction phase because the swelling occurring during the saturation phase was not well visible. Nevertheless, the first measurements of swelling of T12 5%, T12 7%, T3 5% and T3 7%

were extended to the beginning of the suction phase in order to homogenise the results and obtain more realistic SWRCs.

# 6.5.5 Fine material loss from the samples

During the suction phase, the samples exhibited a loss of the finer material that could affect the results. However, the soil particles loss was particularly relevant for the untreated samples, T12 and T3 as shown in figure 6.10.



Figure 6.10

This issue will be discussed more in detail in chapter 7 in order to analyse the erosion susceptibility of the samples and the influence of the bentonite treatments on this matter.

# 6.5.6 Presence of mushrooms on the sample upper surface

During the saturation phase, some mushrooms appeared on the upper surface of the samples due to biological activity facilitated by the humid conditions in the sandbox. In figure 6.11 the surface of a sample attacked by the mushrooms is displayed. These mushrooms could change the soil properties if free growing was allowed. In order to overcome this problem a disinfectant solution with a 2,5% concentration of glutarldehyde was used. The solution was distilled directly on the surface of the infected samples. This operation had to be done always after the mass measurements in order not to add extra weight on the samples.

This treatment was repeated three times during the experiment and was sufficient to overcome this problem.



Figure 6.11

# 6.5.7 Calculation of the degree of saturation S

During the saturation phase and the beginning of suction phase (pF=0,4) the calculated value of S was superior to 100%, hence, since this is physically not possible, it was affected by a series of systematic errors.

Assuming that the measures of the metal ring diameters D and of the solid unit weights  $\gamma_s$  are sufficiently accurate, errors were mainly due to the following imprecisions:

 All the taken mass measures the soil sample dry mass m<sub>d</sub> an error of measure of ±∆m is committed. The real masses are equal to m±∆m.  Measuring the height H of samples exhibiting swelling an error of measure of ±∆H is committed. The real height is equal to H±∆H.

Therefore, these inaccuracies affect the determination of the soil sample volume V, the dry unit weight  $\gamma_d$  (equation 6.4), the gravimetric water content w (equation 6.5), the volumetric water content  $\theta$  (equation 6.6) and the degree of saturation S (equation 6.7). In particular, since S is calculated using w and  $\gamma_d$ , equation 6.7 tends to increase the error amount of the calculation.

H measure  $\rightarrow \pm \Delta H$  error m measure  $\rightarrow \pm \Delta m$  error  $V = \pi \cdot \frac{D^2}{4} \cdot H$   $\gamma_d = \frac{m_d}{V}g$   $w = \frac{m_h - m_d}{m_d}$  $S = w \cdot \frac{\gamma_s \gamma_d}{\gamma_w (\gamma_s - \gamma_d)}$ 

Figure 6.12 helps to clarify the impact of these errors on S.



In light of these considerations, in order to calculate S its definition, already presented in chapter 2 paragraph 2.1.1, was used instead of equation 6.7. The S definition is reported underneath:

$$S = \frac{\theta - \theta_r}{\theta_{sat} - \theta_r} \tag{6.10}$$

As already stated in paragraph 6.3.2 the residual volumetric water content was set equal to zero for all the studied samples. Equation 6.10 has the advantage of depending directly on the volumetric content  $\theta$ .

 $\theta$  corresponds to the porosity fraction of the sample completely filled by water. When the soil sample is completely saturated, water fills all its porosities. Hence, the volumetric water content corresponding to the saturation conditions  $\theta_{sat}$  can be set equal to the porosity n of the sample.

Nevertheless, considering the treated samples, the porosity n was not constant during the experiment because of the swelling and the shrinkage. Hence, at the end of the test, for each sample, the maximum volumetric water content  $\theta_{max}$ , calculated with the equation 6.6, was imposed to correspond to saturation conditions and, hence, to be equal to  $\theta_{sat}$ . In light of these considerations, the equation 6.10 can be written as following:

$$S = \frac{\theta}{\theta_{max}} = \frac{\theta}{n} \tag{6.11}$$

In table 6.9 and table 6.10 the maximum volumetric water content calculated during the experiment are presented and it is assumed that the  $\theta_{max}$ , corresponds to the porosity n, are respectively for T12 and T3.

	T12 0%	T12 0%	T12 5%	T12 5%	T12 7%	T12 7%	T12 12%	T12 12%
	E1	E2	E1	E2	E1	E2	E1	E2
$\theta_{max} = n$	0,3702	0,3555	0,4203	0,4189	0,4444	0,4312	0,5181	0,4906

	T3 0%	T3 0%	T3 5%	T3 7%	T3 7%	T3 12%	T3 12%			
	E1	E2	E1	E1	E2	E1	E2			
$\theta_{max} = n$	0,3821	0,3606	0,4116	0,4558	0,4360	0,5386	0,5224			

Table 6.	10	)
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It is possible to notice that the maximum values generally do not happen to be at pF=0 but towards the end of the saturation phase for the untreated samples, or at pF=0,4 for the treated ones. This means that the corresponding S values calculated with the equation 6.11 are not equal to 100% at pF=0.

In order to be able to apply the Van Genuchten model (paragraph 6.6), the S values at

pF=0 were fixed to 100%.

The final values of S obtained from the test are shown in the attachment.

In table 6.11 the equations already described in the previous paragraphs and to calculate S from the experimental data are displayed. The meaning of all the parameters appearing in the equations have already been explained in the previous paragraphs.

Water mass	$m_w = m_h - m_d - m_{ring} - m_{string} - m_{sat \ (filter)}$	[g]
Dry unit weight	$\gamma_d = \frac{m_d}{V}g$	[kN/m <sup>3</sup> ]
Gravimetric water	$w = \frac{m_w}{m_w}$	%
content	$m_d$	70
Volumetric water	$\theta = w \cdot \frac{\gamma_d}{\gamma_d}$	%
content	$\gamma_w$	,0
Porosity	$n = \theta_{max} = \theta_{sat}$	adimensional
Degree of	$S = \frac{\theta}{\theta}$	%
saturation	$\theta_{max}$	70

Table 6.11

# 6.6 APPLICATION OF THE VAN GENUCHTEN MODEL

In this paragraph, the data obtained for the degree of saturation S will be used to build the Van Genuchten model.

From this point forward, the subscripts EXP and VG will be used to indicate, respectively, a curve referred to the experimental values and a curve referred to the Van Genuchten model.

It is necessary to found the parameters of the Van Genuchten model in order to make some considerations on how the hydraulic conductivity of the studied samples changes when the soil is in partially saturated conditions.

In this study, since the data were obtained using only the hanging column test data the curves describe the soil retention properties referring only to the structural porosity of the samples. In fact, the suction values applicable with the hanging column test are not low enough to remove the water filling the microporosities of the samples.

# 6.6.1 The experimental Soil Water Retention Curve

In figure 6.13 and figure 6.14 the  $h(S)_{EXP}$  respectively for T12 and T3 are shown. The data used to plot these curves are shown in the attachment.



Figure 6.13



Figure 6.14

These curves show the suction h expressed in water column, in function of the degree of saturation S, calculated as described in paragraph 6.5.7.

Considering the two graphics, it is not easy to discuss the effect of bentonite treatments since there is not a clear tendency in the disposition of the curves. This is probably due to the problems encountered during the experiment and already discussed in paragraph 6.5.7.

Therefore, the curves  $h(S)_{\text{EXP}}$  will be used only to find the parameters of the Van Genuchten model.

In order to comment the effect of the bentonite treatments on the raw soils, the curves  $h(\theta)$  were used instead of the h(S). These curves will be discussed in paragraph 6.7.2.

# 6.6.2 The Van Genuchten model parameters

The Van Genuchten model was already described in chapter 2, paragraph 2.3.3. The equations are reported underneath:

$$S = \left(\frac{1}{1 + (\alpha \cdot h)^n}\right)^m \tag{6.12}$$

$$m = 1 - \frac{1}{n} \quad (0 < m < 1) \tag{6.13}$$

Where the meaning of the parameters is the same already explained in chapter 2 paragraph 2.3.3.

In order to build the model, it is necessary to estimate the values of  $\alpha$ , m and n. Nevertheless, since m and n are correlated with equation 6.13, equation 6.12 can be written with the following expression:

$$S = \left(\frac{1}{1 + (\alpha \cdot h)^{\frac{1}{1-m}}}\right)^{m}$$
(6.14)

Therefore, it is sufficient to estimate the value of the two parameters:  $\alpha$  and m. For doing so, the Scilab program reported in the attachment was implemented for each soil sample used in the hanging column test.

Chosen the sample to analyse, the program aim is to find a couple  $(\alpha,m)$  for which the variance between the experimental values of the saturation degree and the corresponding model values is minimal. In order to realize this objective, the following function was defined:

$$RMS = \frac{\sqrt{\sum_{k=1}^{N} (S_{k EXP} - S_{k VG})^2}}{N}$$
(6.15)

Where k is the index corresponding to an equilibrium point in the curve and N is the total number of equilibrium points of the experimental curve, hence, N=5 in this study. Summarizing, the program uses the following procedure for each soil sample:

- 1. The parameters used in the program are defined.
- The input data, that are the values of suction h and of saturation degree S<sub>EXP</sub>, are defined.
- 3. The number of test couples  $(\alpha,m)$  is chosen.
- 4. The values of α<sub>i</sub> and m<sub>i</sub> for the i-th test couple are randomly chosen by the program in the interval [0;1].

- 5. For each k-th suction value h, the program calculates the  $S_{i VG}$  with the equation 6.14.
- 6. For each i-th test couple, the program calculates the RMS with the equation 6.15.
- 7. Steps from 4 to 6 are repeated for all the test couples (cfr. Step 3).
- The program extrapolates the (α,m) values corresponding to the minimum RMS and calculates the n value with the equation 6.13.

Since the couple  $(\alpha,m)$  is chosen randomly by the program, the model reliability strictly depends on the number of test couples chosen at point 3: the higher this number, the more reliable the model. Thanks to the Scilab program, it was possible to test 1000000 of couples for each sample. After, in order to check the model reliability, it was checked that already with 100000 couples, the minimum values of RMS were negligibly higher than the ones obtained with 1000000 couples.

In table 6.12 and table 6.13 the gained values of  $\alpha$ , m and n are displayed respectively for T12 and T3 samples.

	T12 0%	T12 0%	T12 5%	T12 5%	T12 7%	T12 7%	T12 12%	T12 12%
	E1	E2	E1	E2	E1	E2	E1	E2
α [cm <sup>-1</sup> ]	0,64408	0,68817	0,56654	0,57217	0,63934	0,63569	0,55224	0,52495
n	1,03304	1,02839	1,03562	1,03385	1,04475	1,03657	1,02622	1,02481
m	0,03199	0,02761	0,03439	0,03275	0,04284	0,03528	0,02555	0,02421
RMS	0,01671	0,01410	0,02154	0,02078	0,02979	0,02425	0,01562	0,01428

Table 6.12

	T3 0%	T3 0%	T3 5%	T3 7%	T3 7%	T3 12%	T3 12%
	E1	E2	E1	E1	E2	E1	E2
α [cm <sup>-1</sup> ]	0,54837	0,47148	0,52909	0,49910	0,62907	0,54670	0,46845
n	1,06387	1,04517	1,04202	1,05179	1,04171	1,03839	1,03793
m	0,06004	0,04322	0,04033	0,04924	0,04004	0,03697	0,03654
RMS	0,03332	0,02264	0,02365	0,02743	0,02737	0,02271	0,01936

Table 6.13

# 6.7 RESULTS

In this paragraph, the data collected thanks to the hanging column test will be elaborated and discussed.

As already disclosed in paragraph 6.6.1, since the calculation of the degree of saturation was problematic (paragraph 6.5.7), the curves h(S) were not used to comment the effect of the bentonite treatments. Therefore, in order to draw some conclusions concerning the bentonite treatments of T12 and T3,  $h(\theta)$  and  $\gamma_d(\theta)$  curves were used.

The volumetric water content was chosen as a reference since it is less influenced than the degree of saturation by the variations of the sample volumes as already explained in paragraph 6.5.7. The intent of this approach was to study separately the variations of the samples water contents, and the effects of swelling and shrinkage on the treated samples. In fact, from the comparison between the 6.6 ( $\theta$ ) and the 6.10 (S), it is possible to notice that the influence of swelling and shrinkage lies only in the  $\gamma_d$  value for the calculation of  $\theta$ , and both in  $\theta$  and in the  $\theta_{max}$  value for the calculation of S.

In paragraph 6.7.1, the solid unit weight  $\gamma_s$  will be estimated and compared to the ones resulting from the water pycnometer test (Chapter 4 paragraph 4.4).

In paragraph 6.7.2 the h( $\theta$ ) will be presented to discuss the relation between these curves and the retention properties of the samples.

In paragraph 6.7.3 the  $\gamma_d(\theta)$  will be presented to study the effect of swelling and shrinkage. Finally in paragraph 6.7.4, the hydraulic conductivity of the samples will be extrapolated thanks to the Van Genuchten model.

#### 6.7.1 Estimation of the solid unit weight

The solid unit weight  $\gamma_s$  was determined with the pycnometer method as explained in chapter 4 paragraph 4.4. However, in order to calculate the degree of saturation S these values were not used since the equation 6.10 was used.

In this paragraph, the solid unit weight of the hanging column test will be compared to the values obtained from the pycnometer method in order to make some considerations on the reliability of the test results.

The solid unit weight calculated from the test will be called  $\gamma_{s (test)}$  while the one obtained from the pycnometer method will be called  $\gamma_{s (pyc)}$ .

 $\gamma_{s (test)}$  was calculated from the definition of solid unit weight:

$$\gamma_s = \frac{m_s}{V_s} \cdot g \tag{6.16}$$

Where  $\gamma_s$  is the solid unit weight (kN/m<sup>3</sup>),  $m_s$  is the solid mass (kg),  $V_s$  is the volume occupied by the solid mass (m<sup>3</sup>) and g is the gravitational acceleration (m/s<sup>2</sup>). The solid mass  $m_s$  is equal to the samples dry mass  $m_d$  (paragraph 6.4.5) while  $V_s$  is calculated with the following expression:

$$V_{\rm s} = V - n \cdot V \tag{6.17}$$

Where V is the sample volume (m<sup>3</sup>) and n is the sample porosity (adimensional). Therefore,  $\gamma_{s \text{ (test)}}$  was calculated with the following equation obtained from the equation 6.16 and the equation 6.17:

$$\gamma_{s\,(test)} = \frac{m_d}{V \cdot (1-n)} \cdot g \tag{6.18}$$

Due to swelling and shrinkage, V and n are not constant. For the assessment of  $\gamma_{s \text{ (test)}}$  the saturation condition was chosen as a reference and, hence, the values of V and n corresponding to S=100% were considered.

The error of  $\gamma_{s (test)}$  was calculated in percentage with the following expression:

$$E\left[\%\right] = \frac{\gamma_{s\,(test)} - \gamma_{s\,(pyc)}}{\gamma_{s\,(pyc)}} \tag{6.19}$$

In table 6.14 and table 6.15 the values of  $\gamma_{s (test)}$ ,  $\gamma_{s (pyc)}$  and E are displayed respectively for T12 and T3.

	T12 0%	T12 0%	T12 5%	T12 5%	T12 7%	T12 7%	T12 12%	T12 12%
	E1	E2	E1	E2	E1	E2	E1	E2
γs (test) [kN/m³]	28,80	28,80	28,51	28,23	28,70	28,51	28,32	28,51
Υs( pyc) [kN/m <sup>3</sup> ]	26,79	26,79	26,77	26,77	26,76	26,76	26,74	26,74
Е	7,49%	7,49%	6,50%	5,44%	7,25%	6,53%	5,90%	6,61%

Table 6.14

	T3 0%	T3 0%	T3 5%	T3 7%	T3 7%	T3 12%	T3 12%
	E1	E2	E1	E1	E2	E1	E2
γ <sub>s (test)</sub> [kN/m <sup>3</sup> ]	28,90	28,42	28,23	28,61	28,70	28,51	28,23
γ <sub>s( pyc)</sub> [kN/m <sup>3</sup> ]	27,36	27,36	27,31	27,29	27,29	27,25	27,25
E	5,61%	3,85%	3,36%	4,81%	5,17%	4,63%	3,60%

Table 6.15

It is possible to notice that the  $\gamma_{s \text{ (test)}}$  is always higher than the real solid unit weight  $\gamma_{s(pyc).}$ For how it has been calculated, E represents the error committed for the evaluation of  $\gamma_s$ . The value of E does not depend on the treatment since it is not possible to find any particular tendency in function of the bentonite percentage.

The error is higher for the T12 but, in any case, it is always under the 8% for all the samples studied with the test. Hence, it is possible to conclude that the problems encountered during the saturation phase have not invalidated the test results.

# 6.7.2 Soil Water Retention Curves

For the reasons already explained in the introduction of paragraph 6.7, the SWRC of the samples were finally traced referring to the volumetric water content. In figure 6.15 and figure 6.16, respectively, the  $h(\theta)$  curves of T12 and T3 and their treatments are displayed. The graphics show both the experimental points obtained with the hanging column test and the curves obtained with the Van Genuchten model. The model underestimates the initial  $\theta$  (pF 0 and pF 0,4 – saturation conditions) and the last two points (pF 1,8 and pF 2) and overestimate the central points (pF 1 and pF 1,5). Underestimations and overestimations are mainly due to the lack of a consistent number of points ranging from saturation conditions (zero suction) to very dry conditions (high suction values).

In fact, the Van Genuchten model was thought to describe the complete SWRC but, with the hanging column test, it is possible to study only its first part corresponding to low suctions. Nevertheless, the adaptation of the curve can be considered enough precise for the purpose of this work.



Figure 6.15



Figure 6.16

It is possible to observe that, increasing the bentonite content, the samples are able to retain more water. In fact, the volumetric water content of a sample with a higher bentonite rate is always higher than a sample containing less bentonite. This observation is valid, for both T12 and T3, at saturation and during the entire suction phase.

Therefore, the higher the bentonite content, the higher the water content that the sample is able to retain.

# 6.7.3 Observations on swelling and shrinkage

As already exposed in paragraph 6.5.4 the samples exhibited swelling during the initial phases of the test and shrinkage in the ending phase. In order to study these phenomena the  $\gamma_d(\theta)$  curves have been traced.

In this paragraph, only the  $\gamma_d(\theta)$  curves for T12 12% and T3 12% will be shown because the effects of swelling and shrinkage are more evident. Nevertheless the observations that will be made for these samples remain valid for the T12 5%, T12 7%, T3 5% and T3 7%.



Figure 6.17



In figure 6.17 and figure 6.18 the  $\gamma_d(\theta)$  curves of T12 12% and T3 12% respectively are exposed. In the graphics  $\theta$  increases from left to right in the x axis, hence, considering the hanging column test procedure, the suction increases from right to left. The labels next to the points indicate the corresponding suction level in pF value. The dotted lines indicate the dry unit weight values of the samples at the beginning of the test.

Since swelling and shrinkage change the samples volumes, they affect the calculation of the dry unit weight  $\gamma_d$  (equation 6.4).

In particular,  $\gamma_d$  changes in the following manner:

- Swelling  $\rightarrow$  V increases  $\rightarrow$   $\gamma_d$  decreases.
- Shrinkage  $\rightarrow$  V decreases  $\rightarrow$   $\gamma_d$  increases.

This behaviour is confirmed by the graphics.

It is possible to notice that there is a consistent decrease of the dry unit weight from the initial values (Proctor unit weights referred to the sample).

Moreover, it is possible to outline another aspect observing the last three points of suction (pF=1; pF=1,5; pF=2). These three points belong to the shrinkage phase and tend to distribute along a vertical line. This means that in the suction interval pF=[1 – 2],  $\gamma_d$  increases while  $\theta$  remains almost constant.

This aspect is schematised in figure 6.19, where, considering a volume unit of the sample, A represents a suction equilibrium point in the interval pF=[1-2] and B the successive equilibrium point in the same interval.



Figure 6.19

The unit volume consists of a triphasic system (gas, solid and water). The water phase can be considered constant in the passage from A to B due to the observation made previously. Since the mass contribute of the gas phase is not significant, the increase of the dry unit weight is due to an increment in the solid phase to the detriment of the gaseous phase. Therefore, when the sample passes from A to B, the soil particles are attracted into the triphasic unit volume with a consequent decrease of the sample porosity.
#### 6.7.4 Estimation of the hydraulic conductivity

In this paragraph, the evolution of the hydraulic conductivity with the suction will be presented.

The values of hydraulic conductivity can be extrapolated from the hanging column test results thanks to the Van Genuchten model which provides a relation between the relative hydraulic conductivity  $K_r$  and the saturation degree S. The relation have been already discussed in chapter 2 paragraph 2.3.3, and it is reported underneath:

$$K_r = \frac{K(S)}{K_{sat}} = S^{1/2} \left[ 1 - \left( 1 - S^{1/m} \right)^m \right]^2$$
(6.20)

Where K(S) is the hydraulic conductivity corresponding to a certain degree of saturation (m/s),  $K_{sat}$  is the saturated hydraulic conductivity (m/s) and m is the Van Genuchten parameter as calculated for each sample (paragraph 6.6.2).

In the equation 6.20, the experimental values of the degree of saturation,  $S_{exp}$ , were considered because the ones provided by the Van Genuchten model,  $S_{VG}$ , are less realistic. In fact, the use of  $S_{VG}$  would give back very low values of  $K_r$  and, in particular the first point at saturation (pF 0) would not correspond to  $K_r$ =1 because the model, as already observed in paragraph 6.7.2, does not give back S=100% at the zero suction (pF 0). After having obtained the values of  $K_r$ , they were traced in function of the measured volumetric water content  $\theta$ .

The  $K_r(\theta)$  curves are shown in figure 6.20 and figure 6.21 respectively for T12 and T3.



Figure 6.20



Figure 6.21

Thanks to these graphics it is possible to comment the effect of the bentonite treatments on the raw material in slightly unsaturated conditions.

The rate at which the relative hydraulic conductivity  $K_r$  of the materials changes with the volumetric water content  $\theta$  slightly increases with the increase of bentonite content. It is possible to observe that some of the samples can reach lower values of  $K_r$  for the same suction level. In order to comment this aspect the h(K) curves were traced. The hydraulic conductivity K was calculated for each sample at each suction level using the definition of  $K_r$  (equation 6.20), in which the value of  $K_{sat}$  is the one obtained with the oedopermeameter (chapter 5 paragraph 5.1.3). In fact, this test was repeated multiple times for each sample and, hence, its results can be considered reliable.



In figure 6.22 and figure 6.23, h(K) curves are presented respectively for T12 and T3.

Figure 6.22



Figure 6.23

In the graphics, the K value corresponding to zero suction (pF 0) is the hydraulic conductivity measured in saturated conditions ( $K_{sat}$ ).

In saturated conditions, the higher the bentonite content, the lower the measured  $K_{sat}$  but the efficiency of the treatment reaches its maximum with the 7% of bentonite, among the studied percentages. In fact, increasing the bentonite percentage from 7% to 12% does not improve significantly the impermeability of the material.

Regarding the suction phase, the same considerations are valid.

Moreover, it is possible to observe that the higher the suction level, the lower the hydraulic conductivity K. This effect is evident for the raw materials but becomes less visible increasing the bentonite content of the material. In fact, observing figure 6.22 and figure 6.23, it is possible to see that the h(K) curves of T12 7%, T12 12%, T3 7% and T3 12% tend to assume the shape of a vertical line.

This behaviour depends on the fact that the samples containing more bentonite are able to retain more water, as already seen in paragraph 6.7.2, and that the suction applied with

the hanging column test is not sufficiently high to see a significant change in the hydraulic conductivity from the saturated value obtained with the permeameter.

### 6.8 CONCLUSIONS ON THE HANGING COLUMN TEST

The conclusions drawn for the hanging column test will be presented and summarised in this paragraph.

- The treated samples exhibited swelling during the saturation phase and shrinkage in the last part of the suction phase. These phenomena did not take place during the oedometric tests in which the samples were more confined. The volume variations of the treated soil due to the water content should be considered for the project of the final cap cover system of the CSM, in consideration of the confinement *in situ* of the cap cover.
- Considering equal suction conditions, the higher the bentonite content of the analysed material, the higher the water content.
- Increasing suction, the hydraulic conductivity of all the samples decreases but the effect is more evident for the raw materials than for the treated samples.
- In the final part of the suction phase pF=[1,2] of the treated samples, the dry unit weight lowers while the water content remains constant.
  This aspect may be relevant for the CSM project. In fact, the future cap cover system could exhibit a variance of the dry unit weight when slightly under saturation S=[80%, 90%].
- The raw material (T12 and T3) tend to lose the fine particles during the test. Increasing the bentonite content this phenomenon becomes less and less relevant. This issue will be analysed in chapter 7.

The considerations presented above are valid in the field studied with the hanging column test and, hence, in the suction interval pF=[0,2] corresponding, for the analysed soils, to a degree of saturation approximatively in the interval S=[80%,100%].

Since the suction applicable with the hanging column test are limited, it was possible to remove only the water filling the structural pores of the soil samples.

For further studies on the effect of bentonite treatments in unsaturated conditions other tests, able to reach higher suction values, should be performed.

# CHAPTER 7 Soils sensitivity to erosion

In this chapter, the sensitivity to erosion of T12 and T3 is analysed.

The urge to analyse this matter and to intend to explain the erosion process is due to the loss of fine particles shown by the T3 in the permeameter test (chapter 5) and in the hanging column test (chapter 6).

After describing the problem (paragraph 7.1), the internal stability of the untreated materials will be analysed in paragraph 7.3 referring to the complete granulometric curve (T12 and T3) and the 0/5 mm fraction (T12<sub>0/5</sub> and T3<sub>0/5</sub>).

Since the results of the internal stability criteria will show that there is not much difference in considering the complete granulometric curve and the 0/5mm fraction, the external erosion criteria will be applied only to the 0/5mm fraction (paragraph 7.4).

Moreover, the external erosion criteria based on Atterberg limits will be applied to analyse the effect of bentonite treatments on the sensitivity to erosion of T12 and T3.

## 7.1 PRESENTATION OF THE PROBLEM

In chapter 5 and chapter 6 the oedo-permeameter test and the hanging column test, respectively, were described. For both the test soil particles loss from the tested samples was observed.

Regarding the permeameter test, the deposit of fine soil in the outflow pipe indicated that the water outflowing from the sample contained fine particles. This tendency was particularly obvious for T3.

T12 and T3 exhibited the same tendency in the hanging column test. In fact, the untreated samples, in particular T3 more than T12, lost the fine particles leaving in the sandbox a coloured circle of material as shown in figure 7.1.



Figure 7.1



Moreover, thanks to the hanging column test, it was possible to find out that also the treated samples suffer a loss of finer material but with a minor rate as the bentonite content increases. This behaviour is clearly presented in figure 7.2, showing the filter put at the base of every sample above the sandbox, after having removed all the samples at the end of the test.



Figure 7.3

The tests conditions are summarised underneath.

The oedo-permeameter test was performed under the following conditions:

- The tested samples have been realised from the 0/5 mm fraction of the granulometric curve;
- The laboratory temperature is kept around 20°C;
- The tested samples have been compacted with the Proctor procedure using different water contents but always equal or slightly superior to the optimum content (w<sub>OPT</sub>);
- A low confinement of 26 kPa corresponding to a layer of soil a little bit thicker than 1m;
- The applied hydraulic charge is of 10 cm.

The hanging column test was performed under the following conditions:

- The tested samples have been realised from the 0/5 mm fraction of the granulometric curve;
- The sandbox temperature is constantly measured to verify that it remains around 20°C;
- The tested samples have been compacted with the Proctor procedure using a water content equal to w<sub>OPT</sub>+2%;
- The samples have been obtained using metal rings (diameter = 5 cm, height ≈ 3 cm);
- The samples are separated from the kaolinitic sand of the sandbox by a filter directly put on the sand and by a filter put on each sample;
- The samples are dried increasing the suction value to the next one and waiting for the stabilisation until all the samples reach the equilibrium, hence, when the difference in the volumetric water content θ between two measures of two consecutive days is approximately less than 0,2%.

#### 7.1.1 Possible impact on the behaviour of the landfill soil cover

The loss of material could be a problem for the cover system since the phenomenon of erosion may affect negatively the mechanical resistance and the impermeability of the

system.

In general, it is possible to distinguish two kinds of possible causes of erosion depending on whether it has an impact on the mechanical resistance or on the impermeability of the structure (Pham, 2008).

A low mechanical resistance may be due to:

- Poor compaction;
- Absence of a proper filter;
- Use of dispersive materials ( clay, silt, fine sand).

An increase of permeability may be due to:

- Bad choice of the materials;
- Presence of cracks.

Even if the causes of low mechanical resistance and bad impermeability were presented separately, in reality the two conditions are interdependent.

In order to analyse the tendency of the two soils to lose material the internal stability, geometric criteria (paragraph 7.3) and external erosion criteria (paragraph 7.4) will be applied.

## 7.2 INTERNAL AND EXTERNAL EROSION

When a soil is subjected to water flow, its smaller particles may be removed by the flux of water causing a rearrangement of the soil structure.

According to how this phenomenon takes place, different terms have been introduced leading sometimes to confusion in literature to describe the process. In this paragraph, this issue will be clarified distinguishing the different types of erosion, which may take place when a soil is subjected to water flow.

For the purpose of this work, the classification by Pham (2008) will be used.

In general, it is possible to distinguish external and internal erosion.

The *internal erosion* consists in the rearrangement and the transport of the fine particles of a soil due to the water passage through the granular skeleton of the soil (Ziems, 1969). Kezdi (1979) and Wan and Fell (2004) underline the distinction between the primary structure (coarse particles) which does not change throughout the phenomenon and the secondary structure (fine particles) which, on the contrary, is subjected to the removal and

transport processes. Moreover, they suggest that the internal erosion takes place when the following conditions are satisfied:

- The diameter of the finer particles must be smaller than the spaces between the coarser particles of the primary structure;
- The total quantity of fine particles must be less than the quantity necessary to fill all the voids of the primary structure;
- The speed of the water flow must be capable of causing the movement of the finer particles through the primary structure.

When these conditions are encountered, the phenomenon stops only when the fine particles find constrictions that are too small to let the fine particles go through the soil primary structure.

The <u>external erosion</u> consists in the loss of the particles at the surface boundaryof a soil subjected to the action of the atmospheric agents (rain, waves, wind, cycle of freezing and thawing, etc.). Generally, the water is the main responsible of external erosion; therefore depending on the water action three kinds of external erosion may be identified: rain erosion, river erosion, coastal erosion.

## 7.3 INTERNAL STABILIY GEOMETRIC CRITERIA

Shuler (1995) states that the sensitivity to suffusion depends on several factors such as the granulometry, the shape of grains, the porosity, the roughness of grains, the cohesion of grains, the hydraulic gradient, the flow direction and the fluid properties.

Moreover, in the literature it is generally accepted that the cohesive soils are not exposed to suffusion. In fact, the process would be too slow because of the small pores and the attraction between the particles.

There are two typologies of criteria that may be used:

- Geometric (or granulometric) criteria, which are based only on the granulometric curve and the fine content of the soil;
- Hydraulic criteria, which consider also the speed of the water flow throughout the soil matrix.

In this work, the geometric criteria will be used to analyse the susceptibility of T12 and T3 to internal instability.

In general, a geometric criterion addresses the comparative sizes of finer and coarser particles in the grain size distribution curve of a soil. Therefore, if constrictions in the pore network of the coarse particles are larger than some of the finer particles, these particles can be carried by seepage flow (Li, 2008). Several approaches were proposed in the literature to describe such geometric constraints to instability, and the use of these various methods has not been critically reviewed (Li, 2008).

In the following paragraphs, after a brief description, the criteria will be applied to for the internal instability analysis of T12, T3, T12<sub>0/5</sub> and T3<sub>0/5</sub>. It is important to notice that all the criteria that will be applied have been formulated generally for granular soils while T12 and T3 have an important fine fraction, which, however, is mainly composed by silt with low cohesion.

The results of the criteria must be critically analysed also considering the experimental evidence that T3 shows a greater tendency to lose material than the T12.

#### 7.3.1 Kezdi's criterion

Kezdi's criterion consists in applying the Terzaghi's retention criterion (1939) to each point of the granulometric curve. Therefore, the curve is split into two parts, the coarser and the finer, for each reference diameter d. The following criterion has to be verified for each d:

$$d_{15}^C < 4 \cdot d_{85}^F \tag{7.1}$$

Where  $d_{15}^{C}$  is the diameter of the 15% of the passing material in the coarse fraction and  $d_{85}^{F}$  is the 85% of the passing material in the fine fraction.

Kezdi verified this method on three gap graded soils that are soils presenting an excess or a deficiency of a certain particle sizes or soils having at least one particle size missing. Fannin and Moffat (2006) tested five gap-graded soils and widely graded soils in a permeameter cell, finding that the coefficient of 4 of the Kezdi's criterion is consistent with the experimental data.

In figure 7.4, the results for T12, T3, T12 $_{0/5}$  and T3 $_{0/5}$  are presented.



Figure 7.4

The semi logarithmic graphic represents the evolution of the ratio  $4d_{85}^F/d_{15}^C$  in function of the diameter d corresponding to the point of the division between the coarser and finer fraction. When this ratio is higher than the limit  $4d_{85}^F/d_{15}^C = 1$  the criterion is verified and the soil can be considered stable.

Considering the complete granulometry, T12 is unstable for all the particles while T3 is stable only for the particles bigger than 2,5 mm.

Considering the 0/5mm fraction,  $T12_{0/5}$  is stable for particles bigger than 0,025 mm while  $T3_{0/5}$  is stable for particles bigger than 0,13 mm.

Hence, in both cases Kezdi's criterion goes against the experimental evidence that the T3 is more unstable than the T12. However, Kezdi's criterion results could not be affordable, as demonstrated by Chapuis (1992), Wan and Fell (2004) and Li and Fannin (2008).

Chapuis (1992) indicated that Kezdi's criterion should be applied with caution since there is little information on the influence of the following factors (Li, 2008):

- 1. The severity of disturbance forces due to either vibration or seepage,
- 2. The stabilizing or destabilizing effects of disturbing forces,

- 3. The void ratio,
- 4. The initial segregation of the soil.

However, it must be considered that the hydraulic gradient is high enough in the oedopermeameter test not to have disturbance effects (point 1 and point 2). The same thing is not valid for the hanging column test. Moreover, since all the tested samples were compacted with the standard Proctor procedure at a fixed water content value, the void ratio and the initial segregation (point 3 and point 4) should not pose a problem in this analysis.

Wan and Fell (2004), investigating the internal stability of clay-silt-gravel and silt-sandgravel mixtures, claimed that methods involving the splitting of the granulometric curve into two a coarse and fine fraction are too conservative since they tend to classify as unstable soils that in reality are stable.

Li and Fannin (2008) have applied different geometric criterions to 29 soils characterized by a discontinuous granulometric curve and 33 soils characterized by a continuous granulometric curve. They found out that Kezdi's criterion gives better results for the internal stability of materials with a discontinuous granulometry when the criterion is applied at the point of the granulometric discontinuity.

#### 7.3.2 Kenney and Lau's criterion

In Kenney and Lau's criterion, the passing material percentage P(d) at grain size d is compared to the percentage H(d) representing the increment of passing material occurring in the grain size interval [d – 4d]. Therefore, H(d) is calculated as the difference between the percentage passing to diameter 4d and the percentage passing to diameter d:

$$H(d) = P(4d) - P(d)$$
(7.2)

The soil is stable if in the chosen range the following relationship is verified:

$$H(d) > P(d) \tag{7.3}$$

Moreover, the following limits are considered:

- The criterion is valid for compacted powdery materials with a dimension of particles superior to 80 µm.
- For soils characterized by a coefficient of uniformity CU=d<sub>60</sub>/d<sub>10</sub>>3 only the points with P(d) < 20% are considered.</li>

 For soils characterized by a coefficient of uniformity CU=d<sub>60</sub>/d<sub>10</sub><3 only the points with P(d) < 30% are considered.</li>

This criterion was obtained studying the porosity of the materials used as granular filters. Kenney and Lau performed permeability tests on these materials with a downward vertical flow, a variable hydraulic gradient and applying vibrations to the material.

Both T12 and T3 are characterised by C.U.>3 (chapter 4, paragraph 4.3.1), hence, the zone with P(d)>20% will be excluded.



In figure 7.5, the results are shown for T12, T3, T12 $_{0/5}$  and T3 $_{0/5}$ .

Figure 7.5

If the [P(d);H(d)] points is under the limit line P(d)=H(d) the criterion is not verified and the soil is unstable. The dotted lines indicate the zone excluded by the criterion (P(d)>20%). No analysed material is stable according to this criterion and there is not much difference in considering the complete granulometric curve or only the 0/5 mm fraction.

Chapuis (1992), Wan and Fell (2004) and Li and Fannin (2008) demonstrated that Kenney and Lau's criterion could be too conservative.

Chapuis (1992) arrived to the same conclusions already presented for Kezdi's criterion

#### (paragraph 7.3.1).

Kenney and Lau (1985, 1986) themselves affirm that this criterion could be too conservative since the experimental conditions with which it was obtained (high speed of the fluid and vibration of the material) are in general more severe than the real ones. Wan and Fell (2004), in their study of the internal stability of clay-silt-gravel and silt-sand-gravel mixtures, assessed that Kenney and Lau's criterion is too conservative since it tends to classify as unstable soils that in reality are stable.

Li and Fannin (2008) found out that Kenney and Lau's criterion seems to give better results for soils with a more uniform granulometry. They assessed that for P(d)<15% the Kenney and Lau's criterion is more conservative while for P(d)>15% the Kenney and Lau's criterion is more conservative.

#### 7.3.3 Burenkova's criterion

Burenkova (1993) deduced his criterion from the results of a series of laboratory tests performed on 22 granular materials characterized by different shapes of the granulometric curve (concave, convex, linear), a maximum CU of 200 and a maximum particle size of 100 mm.

The basic assumption of this method is that the internal stability of the material is strictly dependent from the shape of the granulometric curve and that the fine fraction does not form part of the soil skeleton if it does not cause a volume increase when mixed with a coarser size fraction (Li, 2008).

In figure 7.6, the results of Burenkova's experimentation are shown. In the graphic, the black points are the unstable soils (zone I and III) while the crosses are the stable soils (zone II).



Figure 7.6

Two straight lines delimit the domain of stable soils. The equations of the two straight lines are the following ones:

$$\frac{d_{90}}{d_{60}} = 0.76 \cdot \log_{10} \frac{d_{90}}{d_{15}} + 1 \tag{7.4}$$

$$\frac{d_{90}}{d_{60}} = 1.86 \cdot \log_{10} \frac{d_{90}}{d_{15}} + 1 \tag{7.5}$$

Where  $d_{90}$ ,  $d_{60}$  and  $d_{15}$  represent the diameters of the soil particle corresponding, respectively, to 90%, 60% and 15% of passing material.

In table 7.1, the values of  $d_{90}/d_{60}$  and  $d_{90}/d_{15}$  are displayed. Each couple of values represents a point in the Burenkova's chart presented in figure 7.7.

	T12	Т3	T12 <sub>0/5</sub>	T3 0/5
d <sub>90</sub> /d <sub>15</sub>	8024,67	767,10	1259,58	922,97
d <sub>90</sub> /d <sub>60</sub>	32,05	3,89	10,58	5,27

Table 7.1



Figure 7.7

T12 and T12<sub>0/5</sub> are in the unstable zone while T3 and T3<sub>0/5</sub> are in the stable zone. This result is conflicting with the experimental evidence of T3 being more unstable than T12 (paragraph 7.1).

However, it must be considered that the analysed soils are outside the area studied by Burenkova as highlighted in figure 7.7. Moreover, Wan and Fell (2004) stated that the Burenkova's method is less conservative than Kenney and Lau's one since it sometimes classifies as stable soils that, in the reality, are unstable.

#### 7.3.4 Liu's criterion

Liu's criterion was obtained in China to evaluate the stability of pulverulent and cohesionless materials.

The basic assumption of this method is that the soil is stable if the finer particles fully occupy void spaces formed by the coarser particle (Liu, 2008). Using a combination of theoretical analysis and experimental results, the following criterion was suggested:

• P(d<sub>n</sub>) < 25 % Unstable

- $25 \% < P(d_n) < 35 \%$  Transition Condition
- P(d<sub>n</sub>) > 35 % Stable

Where  $P(d_n)$  is the passing material percentage at the diameter  $d_n$ , corresponding to the division between finer and coarser fraction. In gap-graded soils,  $d_n$  is the diameter in correspondence to the gap, in continuously graded soils  $d_n$  is calculated with the following expression:

$$d_n = \sqrt{d_{70}d_{10}} \tag{7.6}$$

Where  $d_{70}$  and  $d_{10}$  represent the diameters of the soil particle corresponding respectively to 70%, and 10% of passing material.

	T12	Т3	T12 0/5	T3 0/5	
d <sub>n</sub> [mm]	0,0582	0,3511	0,0192	0,0404	
P(d <sub>n</sub> )	35,37%	26,22%	32,78%	28,33%	
Result	P(d <sub>n</sub> ) > 35% Stable	25%< P(d <sub>n</sub> ) < 35% Transition Condition	25%< P(d <sub>n</sub> ) < 35% Transition Condition	25%< P(d <sub>n</sub> ) < 35% Transition Condition	

In table 7.2, the results of the criterion are displayed.

Considering the complete granulometric curve, T12 is stable while T3 is in transition condition.

Considering only the 0/5 mm fraction, both the soils result in transition conditions, but  $T12_{0/5}$  closer to the stable conditions than the  $T3_{0/5}$ .

Although this criterion was elaborated for pulverulent soils, it gives results more compatible than the others do with the observations made in paragraph 7.1.

#### 7.3.5 Mao's criterion

Mao's criterion, as Liu's one, was elaborated to evaluate the stability of pulverulent and cohesionless material.

Its formulation is similar to Liu's criterion but, in this case, the limit to assess the stability depends from the soil porosity n.

The criterion is following:

Table 7.2

$$P(d_n) < \frac{100}{4(1-n)}\%$$
 Unstable (7.7)

$$P(d_n) \ge \frac{100}{4(1-n)}\%$$
 Stable (7.8)

Where n is the porosity of the soil and  $P(d_n)$  is the passing material percentage at the diameter  $d_n$ , corresponding to the division between finer and coarser fraction. In gap-graded soils,  $d_n$  is the diameter in correspondence to the gap, in continuously graded soils  $d_n$  is calculated with the following expression:

$$d_n = 1.3\sqrt{d_{85}d_{15}} \tag{7.9}$$

Where  $d_{85}$  and  $d_{15}$  represent the diameters of the soil particle corresponding respectively to 85%, and 15% of passing material.

In order to apply Mao's criterion, the porosity was calculated as following:

$$n = 1 - \frac{\gamma_{d \ OPT + 2\%}}{\gamma_s} \tag{7.10}$$

Where  $\gamma_{d \text{ OPT}+2\%}$  is the dry unit weight (kN/m<sup>3</sup>) referred to the Proctor test performed with a content equal to the optimum +2% (chapter 6 paragraph 6.2.1)

 $\gamma_s$  is the solid unit weight (kN/m<sup>3</sup>) already calculated in chapter 4 paragraph 4.4.

In table 7.3, the values of  $\gamma_{d \text{ OPT}+2\%}$ ,  $\gamma_s$  and n are displayed.

	T12	Т3	T12 <sub>0/5</sub>	T3 <sub>0/5</sub>
γ <sub>d OPT+2%</sub> [kN/m <sup>3</sup> ]	18,77	19,46	18,77	19,46
γ₅[kN/m³]	26,79	27,36	26,79	27,36
n	0,2995	0,2888	0,2995	0,2888

Table 7.3

In table 7.4, the results of the criterion are presented.

	T12	Т3	T12 <sub>0/5</sub>	T3 <sub>0/5</sub>
d <sub>n</sub> [mm]	0,3251	1,3866	0,0599	0,1387
P(d <sub>n</sub> )	50,51%	37,11%	47,25%	38,00%
Limit	35,69%	35,15%	35,69%	35,15%
Result	Stable	Stable	Stable	Stable

	Та	ble	7.	4
--	----	-----	----	---

All the analysed soils are stable according to this criterion and in particular, T12 and T12 $_{0/5}$  result more stable than T3 and T3 $_{0/5}$ .

The results are similar to Liu's criterion ones. Therefore, in this case too, even if the criterion was elaborated for pulverulent soils, the results are more compatible with the observations made in paragraph 7.1. Moreover, this criterion presents the advantage of taking into account the compaction of the soil through the porosity parameter used in the limit calculation.

#### 7.3.6 Conclusions on the internal stability criteria

	T12	Т3	T3 T12 <sub>0/5</sub>	
Kezdi (1969)	Unstable	Stable for d>2,5mm	Stable for d>0,025 mm	Stable for d>0,13 mm
Kenney and Lau (1986)	Unstable	Unstable	Unstable	Unstable
Burenkova (1993)	Unstable	Stable	Unstable	Stable
Liu (2005)	Stable	Transition condition	Transition condition	Transition condition
Mao (2005)	Stable	Stable	Stable	Stable

The results obtained from the internal stability criteria are summarised in table 7.5.

The results depend heavily from the criterion used to verify the internal stability. Therefore, it is important to choose the right criterion in function of the soil under analysis.

Therefore, the results could not be very affording since the internal stability criteria have

Table 7.5

been created to study the behaviour of cohesionless soils in view of designing filters, while T12 and T3 have a high content of fine material.

Kezdi's criterion (1969) is verified only for the finer fractions but, as stated by Chapuis (1992) and Wan and Fell (2004), is quite conservative.

The same statement is valid for the Kenney and Lau's criterion (1986).

Burenkova's criterion (1993), on the contrary, can classify as stable materials that in reality are unstable (Wan and Fell, 2004). Moreover, all the results of this analysis are outside the field studied by the author (figure 7.7).

Liu's criterion (2005) and Mao's criterion (2005) have the advantage to consider the nature of soil. In particular, Mao's criterion considers the porosity of the soil to calculate the limit of the criterion. The results of these criteria fit better to the experimental evidence that the T3 is less stable than T12.

As already mentioned in the chapter introduction, it could be possible that the samples in the oedo-permeameter and in the hanging column test were subjected to superficial erosion and not to internal erosion.

In the following paragraphs, external erosion criteria will be applied in order to verify if the results are more compatible with the experimental observations than the results of the internal stability criteria.

### 7.4 EXTERNAL EROSION CRITERIA

Different experiments were carried out to find a relationship between surface erosion and soil properties. In general, these methods provide an equation to calculate the critical shear stress, which represents the sensitivity of the soil to erosion. Therefore, when a flux of water exercises a shear stress higher than the soil characteristic critical value, the soil is subjected to erosion (Pham, 2008).

In the following paragraphs, some empirical relationships will be applied to study the susceptibility of  $T12_{0/5}$  and  $T3_{0/5}$  to external erosion and to analyse the effect of the bentonite treatments on this matter. For the analysis, only the 0/5 mm fraction granulometric curve was considered since the tests (Atterberg's limits, oedo-permeameter test, hanging column test) were performed on this fraction.

It is necessary to clarify that these relationships are still under study and do not give

always results in agree with the real conditions *in situ*. Hence, in this case too, it is important to know the limits of these relationships to analyse critically the results.

#### 7.4.1 Analysis of the untreated materials

<u>Shields (1936)</u> was one of the firsts to propose an empirical relationship to determine the critical shear stress  $\tau_c$  in function of a diameter of reference. The relationship is the following:

$$\tau_c = d_{50} \tag{7.11}$$

Where  $\tau_c$  is the critical shear stress (Pa) and d<sub>50</sub> is the diameter of the sieve through which the 50% of grains pass (mm).

Shields' theory was developed studying uniform, non-cohesive particles for which the entrainment is due only to gravity, Archimedes' force and drag forces. Even if for finer and cohesive particles electromagnetic and electrostatic forces should be considered too, Shields' theory was extended to include finer materials (Clark and Wynn, 2006).

Smerdon and Beasley (1961) proposed the following equations:

$$\tau_c = 3,54 \cdot 10^{-28,1 \cdot d_{50}} \tag{7.12}$$

$$\tau_c = 0.493 \cdot 10^{0.0182 \cdot CF} \tag{7.13}$$

$$\tau_c = 0.16 \cdot PI^{0.84} \tag{7.14}$$

Where  $\tau_c$  is expressed in Pa, d<sub>50</sub> is expressed in m, CF is the clay fraction (percentage of passing material at a reference diameter of 2 µm) and PI is the plastic index of the material.

Smerdon and Beasley obtained these equations conducting a flume study on eleven cohesive Missouri soils. The samples were not compacted but only levelled and subjected to increasing flow rates. The critical shear stress was defined as the shear stress corresponding to bed failure (Clark and Wynn, 2006).

<u>Julian and Torres</u> (2006) developed the following formula, which takes into account the silt fraction SC that is the percentage corresponding to the passing material at 0,063 mm:

$$\tau_c = 0.1 + 0.1779 \cdot SC + 0.0028 \cdot SC^2 - 2.34 \cdot 10^{-5 \cdot SC^3}$$
(7.15)

Where  $\tau_c$  is expressed in Pa and SC is expressed in percentage.

In table 7.6, the obtained results are displayed.

	τ。 [Pa]			
	T12 <sub>0/5</sub>	T3 <sub>0/5</sub>		
Shields Equation 7.11	0,08	0,34		
Smerdon Beasley Equation 7.12	3,52	3,46		
Smerdon Beasley Equation 7.13	0,97	0,81		
Smerdon Beasley Equation 7.14	1,05	1,08		
Julian Torres Equation 7.15	15,03	8,51		

Ta	hla	7	6
1a	bie	1.	0

Equation 7.11 (Shields, 1936) gives very low  $\tau_c$  values because the extension of the theory to fine-grained soils is not based on laboratory observations. The method leads to an underestimation of the critical shear stress and, hence, it is not affordable for fine-grained soils (Clark and Wynn, 2006).

Since equation 7.12, equation 7.13 and equation 7.14 (Smerdon and Beasley, 1961) were developed from laboratory tests on cohesive soils, they are more suitable to be used in this analysis.

According to equation 7.12 and equation 7.13, T12<sub>0/5</sub> results slightly more resistant to erosion than T3<sub>0/5</sub> since it is characterised by a smaller  $_{d50}$  and a higher clay fraction (T12<sub>0/5</sub>  $\rightarrow$  d<sub>50</sub>=0,08mm and CF=16,21%; T3<sub>0/5</sub>  $\rightarrow$  d<sub>50</sub>=0,34mm and CF=11,72%).

Equation 7.14 gives almost equal results for  $T12_{0/5}$  and  $T3_{0/5}$  since they are characterised by almost the same plastic index.

Equation 7.15 (Julian and Torres, 2006) gives the higher critical shear stress values. It is evident that considering the silt fraction SC instead of only the clay fraction CF, the critical shear stress increases significantly. T12<sub>0/5</sub> results more resistant than T3<sub>0/5</sub> since it is characterised by a higher silt fraction (T12<sub>0/5</sub>  $\rightarrow$  SC=47,87% ; T3<sub>0/5</sub>  $\rightarrow$  SC=31,57%).

Comparing the results of external erosion criteria (table 7.6) with the results of internal stability criteria (table 7.5), it is possible to notice that the first ones are more in

accordance with the experimental observation that  $T3_{0/5}$  is more exposed to erosion than  $T12_{0/5}$ .

#### 7.4.2 Effect of bentonite treatments

The effect of bentonite treatment will be analysed referring to two methods which refers to Atterbeg's limits: Gibbs' method (1962) and Smerdon and Beasley's equation 7.14 (1961). <u>Gibbs</u> (1962) performed laboratory erosion experiments on 45 undisturbed samples collected from different riverbanks. The results of the experiments brought to the definition of areas corresponding to different erosion rate in the Casagrande's chart (figure 7.8).



Figure 7.8

In figure 7.8, the points correspond to the analysed soils. The points corresponding to the average of T1+T2 0% and of T3 0% are represented too.

 $T12_{0/5}$  0% and  $T3_{0/5}$  0% are located in the boundary zone between the highest resistance

zone and the slight erosion zone.

 $T12_{0/5}$  5% and  $T3_{0/5}$  5% are located in the slight erosion zone.

 $T12_{0/5}$  7% and  $T3_{0/5}$  7% are already outside the plastic index range studied by Gibbs. For this reason, the points corresponding to 12% treatments were not represented.

Smerdon and Beasley's equation 7.14 (1961) gave the results displayed in figure 7.9 and table 7.7. In table 7.7 the PI values are displayed too.



Figure 7.9

	T12 <sub>0-5</sub>			T3 <sub>0-5</sub>				
	0%	5%	7%	12%	0%	5%	7%	12%
τ <sub>c</sub> [Pa]	1,049	2,897	4,199	6,031	1,078	2,729	3,212	6,074
PI	9,38%	31,44%	48,87%	75,25%	9,69%	29,27%	35,54%	75,90%

Table 7.7	7

Increasing the bentonite content, the critical shear stress value  $\tau_c$  increases. In fact, according to equation 7.14, the higher the plasticity index, the higher the resistance to erosion. For this reason,  $\tau_c$  increase has the same trend of the plastic index increase in function of the bentonite content shown in chapter 4 figure 4.13.

Equation 7.14 explains well the increase of resistance to erosion of T12 and T3 with the increase of bentonite content but it does not show significant differences between T12 0% and T3 0%.

#### 7.4.3 Conclusions on the external erosion criteria

No applied method in paragraph 7.4.1 and paragraph 7.4.2 is consolidated. Regarding equation 7.11 (Shields, 1936) and equation 7.12 (Smerdon and Beasley, 1961), the use of particle reference diameters is not considered suitable for materials containing high percentage of fine since the contribute of electromagnetic and electrostatic forces between fine particles are not considered (Briaud et al., 2001).

Regarding Gibbs' method (1962) and Smerdon and Beasley's equation 7.14 (1961), studies by Arulanandam and Perry (1983), Briaud et al. (2001), Wan and Fell (2002) have shown that there is not a direct correlation between the critical shear stress and the Atterberg's limits.

Regarding the relationship between the critical shear stress and the fine percentage (Smerdon and Beasley's equation 7.13 and Jules and Torres' equation 7.15), different authors have studied this topic but they are not always compatible with the kind of relationship between the two parameters (Pham, 2008).

Moreover, it must be underlined that the calculation of the critical shear stress of the soil has been done only in order to have an idea of the soils sensitivity to erosion. In fact, the shear stress occurring in the oedo-permeameter test and the hanging column test is not known.

However, even if the applied methods could not be affordable and the results obtained

with the different approaches are not always concordant, it is possible to draw two important qualitative conclusions:

- The critical shear stress of T12 is generally higher than the one of T3;
- The higher the bentonite rate, the higher the critical shear resistance;

These two conclusions explain the phenomenon observed in the sandbox of the hanging column test (paragraph 7.1).

## 7.5 CONCLUSIONS ON SOILS SENSITIVITY TO EROSION

In this chapter, the sensitivity to erosion of two different soils, T12 and T3, was studied focusing also on the effect of bentonite treatments on this matter.

At first the internal erosion criteria were applied to the samples which appeared more susceptible to erosion, T12 0% and T3 0%, (paragraph 7.3).

Both the complete granulometric curve and the 0/5 mm fraction curve were used. Since the internal erosion criteria are generally thought for cohesionless soils the obtained results are not always in agreement with the expectation. Only Liu's criterion and Mao's criterion (2008) show that the T3 0% is more internally unstable than the T12.

The differences between the results referred to the complete granulometric curve and the ones referred to the 0/5 mm fraction curve are not significant.

For further analysis, some external erosion criteria were applied to T12 0% and T3 0%, considering only the 0/5 mm granulometric fraction (paragraph 7.4).

The results are in general more in accordance with the expectations since they show that T3 0% is more susceptible to erosion than T12 0%.

In paragraph 7.4.3, the effect of bentonite treatment was analysed and it was found that increasing the bentonite content of the material, the soil resistance to erosion increases. Finally, it is possible to draw these conclusions:

- The problem of fine loss encountered during the hanging column test is due to superficial erosion more than internal erosion;
- The T3 is more subjected to erosion than the T12;
- The bentonite treatments increase the resistance of the material.

## CONCLUSIONS

In this work the effect of bentonite treatments on the hydraulic behaviour of two soils from a French disposal for low-radioactivity waste (*Centre de Stockage de la Manche*, CSM) was analysed.

The two untreated soils (called T12 and T3) are classified respectively as a low plasticity silt and a sand with high silt content.

T12 and T3 form the clay layer of the CSM cap cover system. In order to guarantee an impervious long-term cover system, bentonite treatments were proposed.

The influence of the treatment was analysed considering three different percentages calculated on dry mass: 5%, 7% and 12%.

The bentonite used for the treatment is a calcium activated activated bentonite. All the analysis were based on the 0/5mm fraction of the untreated and treated soils.

First, the Atteberg limits and the VB value of T12, T3 and their treatments were obtained. Increasing the bentonite content, the liquid limit, the plastic limit and the plasticity index increase. The methylene blue adsorption value (VB) increase linearly with the bentonite content.

The compaction curves of T12, T3 and their treatments were obtained with the standard Proctor procedure. It was possible to find out that the maximum dry unit weight of the soils gets lower as the bentonite percentage of the treatment increases.

The hydraulic conductivity of T12, T3 and their treatments in saturated conditions were obtained with the oedo-permeameter test and empirical relationships.

Increasing the bentonite content, the hydraulic conductivity decreases but there is not a linear correlation between the two parameters. In particular, the 7% treatment results the most efficient, since the 12% treatment determines a small additional decrease of hydraulic conductivity.

The hydraulic conductivity of T12, T3 and their treatments in unsaturated conditions were studied applying the Van Genuchten model on the experimental results of the hanging column test.

Increasing suction, the hydraulic conductivity of all the samples decreases but this effect is less evident for the treated samples.

The samples containing bentonite exhibited swelling and shrinkage during the test and, as

a consequence, variation of the dry unit weight. In particular, it was observed that in the suction interval pF=[1 - 2], the dry unit weight can change even if the volumetric water content remains constant.

Since during the hanging column test the untreated soil samples, in particular T3, were subjected to fine particles loss, the effect of bentonite on soil sensitivity to erosion was analysed referring to internal and external erosion criteria.

The results of the internal erosion criteria were not in accordance with the experimental results while the external erosion criteria described better the observations made during the hanging column test.

Hence, it was possible to verify that T3 is more sensitive to erosion than T12 and that increasing the bentonite content of the material, the soil resistance to erosion increases.

For further analysis, the study of the influence of the soil coarser fraction on the bentonite treatments is proposed. Moreover, since the unsaturated conditions were studied only with the hanging column test, it is proposed to analyse more in depth the bentonite treatment effects on unsaturated hydraulic conductivity, performing tests through which it is possible to reach higher suction levels.

## ATTACHMENT

In this section, the data from the hanging column test and the Scilab program used to find the parameters of Van Genuchten model are reported.

#### HANGING COLUMN TEST – T12 SAMPLES

		T12 0%	T12 0%	T12 5%	T12 5%	T12 7%	T12 7%	T12 12%	T12 12%
		E1	E2	E1	E2	E1	E2	E1	E2
	h [cm]	2,97	2,97	3,00	3,03	2,98	3,00	3,02	3,00
	d [cm]	5,00	5,00	5,00	5,00	5,00	5,00	5,00	5,00
	V [cm <sup>3</sup> ]	58,32	58,32	58,90	59,49	58,51	58,90	59,30	58,90
	m <sub>ring</sub> [g]	41,03	41,40	43,26	42,82	55,41	42,80	43,24	43,03
	m <sub>(string)</sub> [g]	1,55	1,56	1,54	1,54	1,52	1,52	1,57	1,54
	m <sub>(tare)</sub> [g]	6,27	6,26	6,44	6,16	5,99	6,29	6,50	3,12
Measures made	m <sub>sat (filter)</sub> [g]	0,99	0,65	0,92	0,76	1,04	1,14	0,80	0,99
after the test	m <sub>d (filter)</sub> [g]	0,46	0,35	0,43	0,37	0,44	0,46	0,42	0,48
	m <sub>h (soil+ring+filter+tare)</sub> [g]	171,50	169,93	164,53	166,84	172,08	163,70	169,46	164,94
	m <sub>d (soil+ring+filter+tare)</sub> [g]	155,08	154,11	145,12	147,22	151,56	142,88	141,19	137,96
	m <sub>d (soil)</sub> [g]	105,77	104,54	93,45	96,33	88,20	91,81	89,46	89,79
					1				r
	γ <sub>d</sub> [kN/m³]	18,77	18,77	17,51	17,51	16,92	16,92	16,47	16,47
Proctor	w	14,29%	14,29%	17,91%	17,91%	19,73%	19,73%	21,36%	21,36%
Values	m <sub>soil+ring</sub> [g]	162,95	162,80	155,46	157,35	163,07	154,12	153,74	153,79
	m <sub>soil+ring+filter+string</sub> [g]	164,78	163,95	157,30	159,09	164,90	155,99	155,55	155,66
	m <sub>filter+string</sub> [g]	1,83	1,15	1,84	1,74	1,83	1,87	1,81	1,87
							1		I
	γ <sub>d</sub> (soil sample) [kN/m <sup>3</sup> ]	18,14	17,93	15,86	16,19	15,07	15,59	15,09	15,24
00/1/110.00		164 70	1 60 07	457.00	150.00		155.00		
22/4/14 0:00	m <sub>initial</sub> [g]	164,78	163,95	157,30	159,09	164,90	155,99	155,55	155,66
	m <sub>w</sub> [g]	16,15	16,86	18,75	18,20	19,46	19,51	21,04	20,97
	γ <sub>d (E)</sub> [kN/m°]	18,14	17,93	15,86	16,19	15,07	15,59	15,09	15,24
	Ŵ	15,27%	16,13%	20,06%	18,89%	22,06%	21,25%	23,52%	23,35%
		27,69%	28,91%	31,83%	30,59%	33,26%	33,12%	35,48%	35,60%
	5	74,80%	81,33%	75,73%	73,03%	74,85%	76,81%	68,48%	12,51%
22/4/14/11/20		162.42	161.00	155.05	457.04	162.10	454.20	454.00	154.45
23/4/14 11:30	m <sub>saturation</sub> beginning [B]	102,42	101,88	155,05	157,21	103,19	154,30	154,33	154,45
	m <sub>w</sub> [g]	13,08	13,73	10,48	15,70	17,02	17,09	19,20	19,10
		18,14	12 129/	17,649/	16,19	10.20%	19,59	21 520/	15,24
		12,57%	15,15%	27.090/	26 40%	19,50%	20.01%	21,55%	21,27%
	6	22,43%	23,54%	27,98%	20,49%	29,09%	29,01%	32,48%	32,43%
	3	00,56%	00,25%	00,30%	05,24%	05,40%	07,20%	02,09%	00,10%
29/4/14 14-55	m[a]	170.02	169.06	161.02	164.60	160 57	160.20	161 70	161 52
20/4/14 14.33	m [g]	20.68	108,00	22 75	23.24	23 /0	23.02	26.63	26.17
	۳۰ <sub>w</sub> ۱۵۶ ۲۰۰۰ [kN/m <sup>3</sup> ]	18 1/	17.03	15.86	16 10	15.07	15 50	15.09	15 24
		19 55%	19.05%	24 34%	24 13%	26 53%	25.07%	29 77%	29 15%
	θ.	35.46%	3/ 1/%	29,54%	39.06%	30.00%	39.08%	1/ 01%	11 13%
	с с	05 70%	96.04%	01 88%	93.26%	90.00%	90.63%	44,91% 86.68%	90 56%
		55,7570	50,0470	51,0070	55,2070	50,0070	50,0570	00,0070	50,5070
29/4/14 11.25	m[a]	169.95	168.25	161.89	164.44	169 72	160.81	162.09	161.90
25/7/1411.25	m [σ]	20.61	20.10	22 72	22.99	23 55	23 54	27.02	26.55
	۰۰۰wL6J ۷٫۰ (۳) [kN/m <sup>3</sup> ]	18 14	17.93	15.86	16.19	15.07	15 59	15.09	15.24
		19 /19%	19.23%	24 31%	23.87%	26 70%	25.64%	30.20%	29 57%
	A	35,34%	34.47%	38,57%	38.64%	40.25%	39,96%	45.57%	45.07%
	s	95,46%	96,96%	91,76%	92,26%	90,58%	92,68%	87,95%	91,88%

5/5/14 10:54	m [g]	170,29	168,69	162,49	165,74	169,99	161,18	165,15	164,66
	m <sub>w</sub> [g]	20,95	20,54	23,32	24,29	23,82	23,91	30,08	29,31
	h [cm]	-	-	-	-	-	-	3,38	3,35
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	-	-	-	-	66,37	65,78
	γ <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18,14	17,93	15,86	16,19	15,07	15,59	13,48	13,65
	w	19,81%	19,65%	24,95%	25,22%	27,01%	26,04%	33,62%	32,64%
	θ	35,93%	35,22%	39,59%	40,83%	40,71%	40,59%	45,32%	44,56%
	S	97,04%	99,08%	94,18%	97,47%	91,62%	94,13%	87,48%	90,83%
6/5/14 10:51	m [g]	170,28	168,50	162,83	164,97	169,95	161,59	165,50	164,74
	m <sub>w</sub> [g]	20,94	20,35	23,66	23,52	23,78	24,32	30,43	29,39
	h [cm]	-	-	-	-	-	-	3,38	3,35
	Vauguar [cm <sup>3</sup> ]	-	-	-	-	-	-	66.37	65.78
	Vd (E) [kN/m <sup>3</sup> ]	18.14	17.93	15.86	16.19	15.07	15.59	13.48	13.65
	W	19.80%	19.47%	25.32%	24.42%	26.96%	26.49%	34.02%	32.73%
	θ	35 91%	34 90%	40 17%	39 53%	40.64%	41 29%	45 85%	44 68%
	ŝ	96.99%	98 17%	95 56%	94 38%	91.46%	95 75%	88 50%	91 08%
	5	50,5570	50,1770	55,5070	54,5070	51,4070	55,7570	00,0070	51,0070
12/5/14 10:46	m[a]	170.03	168 78	163.06	165 62	170.60	161 51	167.02	166 51
12/ 3/ 14 10.40	m [g]	21 50	20.62	22.80	24.17	24.42	24.24	22.05	21.16
	IIIw[8]	21,59	20,05	25,69	24,17	24,45	24,24	52,65	2.42
		-	-	-	-	-	-	3,44	3,42
	V <sub>swelling</sub> [Cm <sup>2</sup> ]	-	-	-	-	-	-	67,61	67,09
	γ <sub>d (E)</sub> [κΝ/m°]	18,14	17,93	15,86	16,19	15,07	15,59	13,23	13,38
	Ŵ	20,41%	19,73%	25,56%	25,09%	27,70%	26,40%	36,72%	34,70%
	6	37,02%	35,38%	40,56%	40,63%	41,75%	41,15%	48,59%	46,45%
	S	100,00%	99,52%	96,49%	96,99%	93,96%	95,43%	93,78%	94,68%
12/5/14 11:20		170 77	100.00	102 12	105.00	170.00	101 50	107 70	167.01
13/ 5/ 14 11:20	m [g]	21.42	108,88	103,13	24.15	24.75	24.22	107,70	167,01
	m <sub>w</sub> [g]	21,43	20,73	23,96	24,15	24,75	24,23	32,03	31,00
	n [cm]	-	-	-	-	-	-	3,44	3,42
	V <sub>swelling</sub> [cm <sup>2</sup> ]	-	-	-	-	-	-	67,61	67,09
	γ <sub>d (E)</sub> [kN/m°]	18,14	17,93	15,86	16,19	15,07	15,59	13,23	13,38
Beginning of suction	Ŵ	20,26%	19,83%	25,64%	25,07%	28,06%	26,39%	36,47%	35,26%
PF=0 (-0,001 cm)	0	36,75%	35,55%	40,68%	40,59%	42,30%	41,13%	48,26%	47,19%
13/5/14 15:40	S	99,26%	100,00%	96,77%	96,91%	95,19%	95,39%	93,15%	96,20%
10/5/11/11/00		470.04	4 60 70	4.62 74	466.00	472.07	4 62 50	460 57	467.54
19/5/14 11:00	m [g]	1/0,84	168,70	163,/1	166,22	1/2,0/	162,59	169,57	167,51
PF=0 (-0,001 cm)	m <sub>w</sub> [g]	21,50	20,55	24,54	24,77	25,90	25,32	34,50	32,16
	n [cm]	-	-	-	-	-	-	3,44	3,42
	V <sub>swelling</sub> [CM ]	- 18 1/	- 17.02	- 15.86	- 16 10	- 15.07	- 15 50	12 22	13 38
Increase of suction	Yd (E) [KNV/III ] W	20 33%	19.66%	26 26%	25 71%	29 37%	27 58%	38 56%	35.82%
PF=0.4 (-2.5 cm)	θ	36.87%	35.24%	41.66%	41.63%	44.26%	42.98%	51.03%	47.94%
19/5/14 15:30	S	99,58%	99,13%	99,11%	99,40%	99,62%	99,69%	98,49%	97,72%
			, <u>,</u>	,			<u> </u>		
27/5/14 11:00	m [g]	170,67	168,63	163,93	166,37	172,17	162,67	170,10	168,26
PF=0,4 (-2,5 cm)	m <sub>w</sub> [g]	21,33	20,48	24,76	24,92	26,00	25,40	35,03	32,91
	h [cm]	-	-	-	-	-	-	3,44	3,42
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	-	-	-	-	67,61	67,09
	γ <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18,14	17,93	15,86	16,19	15,07	15,59	13,23	13,38
Increase of suction	w	20,17%	19,59%	26,50%	25,87%	29,48%	27,67%	39,16%	36,65%
PF=1 (-10 cm)	θ	36,58%	35,12%	42,03%	41,89%	44,44%	43,12%	51,81%	49,06%
27/5/14 11:30	S	98,80%	98,79%	100,00%	100,00%	100,00%	100,00%	100,00%	100,00%

2/6/14 11:00	m [g]	168,88	167,17	161,64	164,45	169,39	160,70	168,03	166,04
PF=1 (-10 cm)	m <sub>w</sub> [g]	19,54	19,02	22,47	23,00	23,22	23,43	32,96	30,69
1°measure	h [cm]	-	-	3,033	3,103	3,083	3,117	3,50	3,42
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	59,559	60,934	60,541	61,196	68,79	67,22
	γ <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18,14	17,93	15,69	15,81	14,57	15,00	13,01	13,36
	w	18,47%	18,19%	24,04%	23,88%	26,33%	25,52%	36,84%	34,18%
	θ	33,51%	32,62%	37,73%	37,75%	38,35%	38,29%	47,92%	45,66%
	S	90,50%	91,75%	89,75%	90,11%	86,31%	88,79%	92,48%	93,07%
			1		1		1		1
3/6/14 11:46	m [g]	168,79	167,03	161,52	164,37	169,29	160,59	167,91	165,92
PF=1 (-10 cm)	m <sub>w</sub> [g]	19,45	18,88	22,35	22,92	23,12	23,32	32,84	30,57
2°measure	h [cm]	-	-	3,033	3,103	3,083	3,117	3,50	3,42
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	59,559	60,934	60,541	61,196	68,79	67,22
	γ <sub>d (E)</sub> [kN/m³]	18,14	17,93	15,69	15,81	14,57	15,00	13,01	13,36
	w	18,39%	18,06%	23,92%	23,79%	26,21%	25,40%	36,71%	34,05%
	θ	33,35%	32,38%	37,53%	37,61%	38,19%	38,11%	47,74%	45,48%
	5	90,09%	91,08%	89,27%	89,80%	85,94%	88,37%	92,14%	92,71%
	<u>θ1–θ2</u>	0.15%	0.24%	0.20%	0.13%	0.17%	0.18%	0.17%	0.18%
	01 02	0,1570 Ok	No!	No!	0,1370 Ok	0,1//0 Ok	0,1070 Ok	0,1770 Ok	0,10/0 Ok
4/6/14 11:30	m[g]	168,64	166,90	161,42	164,27	169,15	160,50	167,79	165,81
PF=1 (-10 cm)	m <sub>w</sub> [g]	19,30	18,75	22,25	22,82	22,98	23,23	32,72	30,46
3°measure	h [cm]	-	-	3,033	3,103	3,083	3,117	3,50	3,42
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	59,559	60,934	60,541	61,196	68,79	67,22
	γ <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18,14	17,93	15,69	15,81	14,57	15,00	13,01	13,36
Increase of suction	w	18,25%	17,94%	23,81%	23,69%	26,05%	25,30%	36,58%	33,92%
PF=1,5 (-31,6 cm)	θ	33,10%	32,15%	37,36%	37,45%	37,96%	37,96%	47,57%	45,32%
4/6/14 14:30	S	89,39%	90,45%	88,88%	89,41%	85,42%	88,03%	91,81%	92,38%
	02.02	0.20%	0.220/	0.470/	0.10%	0.220/	0.45%	0.470/	0.10%
	02-03	0,26%	0,22%	0,17%	0,16%	0,23%	0,15%	0,17%	0,16%
		140:	140:	ÖK	ÖK	110:	ÖK	ÖK	ÖK
11/6/14 11:50	m[g]	168,20	166,62	161,06	163,87	168,97	160,27	167,30	165,43
PF=1,5 (-31,6 cm)	m <sub>w</sub> [g]	18,86	18,47	21,89	22,42	22,80	23,00	32,23	30,08
1°measure	h [cm]	-	-	3,033	3,103	3,083	3,117	3,51	3,44
	V <sub>curelling</sub> [cm <sup>3</sup> ]	-	-	59,559	60,934	60,541	61,196	68,85	67,48
	γ <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18,14	17,93	15,69	15,81	14,57	15,00	12,99	13,31
	w	17,83%	17,67%	23,42%	23,27%	25,85%	25,05%	36,03%	33,50%
	θ	32,34%	31,67%	36,75%	36,79%	37,66%	37,58%	46,81%	44,58%
	S	87,36%	89,10%	87,44%	87,84%	84,75%	87,16%	90,35%	90,87%
			1		1		1		1
12/6/14 11:05	m[g]	168,10	166,52	160,93	163,75	168,87	160,14	167,04	165,21
PF=1,5 (-31,6 cm)	m <sub>w</sub> [g]	18,76	18,37	21,76	22,30	22,70	22,87	31,97	29,86
2°measure	h [cm]	-	-	3,033	3,103	3,083	3,117	3,51	3,44
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	59,559	60,934	60,541	61,196	68,85	67,48
	γ <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18,14	17,93	15,69	15,81	14,57	15,00	12,99	13,31
	w	17,74%	17,57%	23,29%	23,15%	25,74%	24,91%	35,74%	33,26%
	θ	32,17%	31,50%	36,53%	36,60%	37,50%	37,37%	46,43%	44,25%
	S	86,89%	88,62%	86,92%	87,37%	84,38%	86,67%	89,62%	90,20%
	A1 A2	0 170/	0 170/	0.220/	0.20%	0.170/	0.210/	0.200/	0.22%
	01-02	0,17%	0,17%	0,22%	0,20%	0,17%	0,21%	0,38%	0,33%
	1	5.	5.			0.			

13/6/14 13:44	m [g]	168,03	166,40	160,80	163,58	168,64	159,96	166,81	165,03
PF=1,5 (-31,6 cm)	m <sub>w</sub> [g]	18,69	18,25	21,63	22,13	22,47	22,69	31,74	29,68
3°measure	h [cm]	-	-	3,033	3,103	3,083	3,117	3,51	3,44
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	59,559	60,934	60,541	61,196	68,85	67,48
	γ <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18,14	17,93	15,69	15,81	14,57	15,00	12,99	13,31
Increase of suction	w	17,67%	17,46%	23,15%	22,97%	25,48%	24,71%	35,48%	33,05%
PF=1,8 (-63,1 cm)	θ	32,05%	31,30%	36,32%	36,32%	37,12%	37,08%	46,10%	43,98%
13/6/14 15:40	S	86,57%	88,04%	86,40%	86,71%	83,53%	85,99%	88,97%	89,66%
	00.00	0.400/	0.040/	0.000/	0.000/	0.000/	0.000/	0.000/	0.070/
	02-03	0,12%	0,21%	0,22%	0,28%	0,38%	0,29%	0,33%	0,27%
		OK	NO:	NO:	110:	NO:	NO:	NO:	NO:
23/6/14 13:25	m[g]	167,91	166,28	160,65	163,31	168,52	159,87	166,06	164,69
PF=1,8 (-63,1 cm)	m <sub>w</sub> [g]	18,57	18,13	21,48	21,86	22,35	22,60	30,99	29,34
1°measure	h[cm]	-	-	3.033	3.103	3.043	3.043	3.42	3.38
	V [cm <sup>3</sup> ]	-	-	59,559	60,934	59,756	59,756	67.09	66.43
	V <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18.14	17.93	15.69	15.81	14.76	15.36	13.34	13.52
	W	17.56%	17.34%	22.99%	22.69%	25.34%	24.62%	34.64%	32.68%
	θ	31,84%	31,09%	36,06%	35,87%	37,40%	37,82%	46,19%	44,17%
	S	86,01%	87,46%	85,80%	85,65%	84,17%	87,71%	89,16%	90,03%
24/6/14 12:05	m [g]	167,85	166,21	160,59	163,25	168,46	159,78	165,92	164,54
PF=1,8 (-63,1 cm)	m <sub>w</sub> [g]	18,51	18,06	21,42	21,80	22,29	22,51	30,85	29,19
2°measure	h [cm]	-	-	3,033	3,103	3,043	3,043	3,42	3,38
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	59,559	60,934	59,756	59,756	67,09	66,43
	γ <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18,14	17,93	15,69	15,81	14,76	15,36	13,34	13,52
Increase of suction	w	17,50%	17,28%	22,92%	22,63%	25,27%	24,52%	34,48%	32,51%
PF=2 (-100 cm)	θ	31,74%	30,97%	35,96%	35,78%	37,30%	37,67%	45,99%	43,94%
41814,72917	S	85,73%	87,12%	85,56%	85,41%	83,95%	87,36%	88,75%	89,57%
	00.00	0.400/	0.420/	0.40%	0.40%	0.40%	0.45%	0.240/	0.220/
	02-03	0,10%	0,12%	0,10%	0,10%	0,10%	0,15%	0,21%	0,23%
		OK	UK	UK	UK	<u>UK</u>	UK	NO:	NO:
30/6/14 15:15	m[g]	167,84	166,30	160,61	163,23	168,53	159,91	165,76	164,59
PF=2 (-100 cm)	m, [g]	18,50	18,15	21,44	21,78	22,36	22,64	30,69	29,24
1°measure	h [cm]	-	-	3.020	3.077	3.043	3.043	3.39	3.34
	V [cm <sup>3</sup> ]	_	_	59 298	60.410	59 756	59 756	66.63	65 58
	Vd (E) [kN/m <sup>3</sup> ]	18.14	17.93	15.76	15.95	14.76	15.36	13.43	13.69
	W	17.49%	17.36%	22.94%	22.61%	25.35%	24.66%	34.31%	32.56%
	θ	31,72%	31,12%	36,16%	36,05%	37,42%	37,89%	46,06%	44,59%
	S	85,69%	87,55%	86,02%	86,07%	84,21%	87,86%	88,90%	90,89%
1/7/14 15:15	m [g]	167,78	166,27	160,61	163,20	168,51	159,88	165,72	164,54
PF=2 (-100 cm)	m <sub>w</sub> [g]	18,44	18,12	21,44	21,75	22,34	22,61	30,65	29,19
2°measure	h [cm]	-	-	3,020	3,077	3,043	3,043	3,39	3,34
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	59,298	60,410	59,756	59,756	66,63	65,58
	γ <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18,14	17,93	15,76	15,95	14,76	15,36	13,43	13,69
	w	17,43%	17,33%	22,94%	22,58%	25,33%	24,63%	34,26%	32,51%
	θ	31,62%	31,07%	36,16%	36,00%	37,39%	37,84%	46,00%	44,51%
	S	85,41%	87,41%	86,02%	85,96%	84,13%	87,75%	88,79%	90,73%
			1						
	θ2-θ3	0,10%	0,05%	0,00%	0,05%	0,03%	0,05%	0,06%	0,08%
	1	UK	UK	UK	UK	UK	UK	UK	UK

#### HANGING COLUMN TEST – T3 SAMPLES

		T3 0%	T3 0%	T3 5%	T3 7%	T3 7%	T3 12%	T3 12%
		E1	E2	E1	E1	E2	E1	E2
	h [cm]	2,96	3,01	3,03	2,99	2,98	3,00	3,03
	d [cm]	5,00	5,00	5,00	5,00	5,00	5,00	5,00
	V [cm <sup>3</sup> ]	58,12	59,10	59,49	58,71	58,51	58,90	59,49
								-
	m <sub>ring</sub> [g]	42,39	41,92	43,55	41,35	42,77	43,25	41,80
	m <sub>(string)</sub> [g]	1,57	1,58	1,55	1,54	1,55	1,56	1,54
	m <sub>(tare)</sub> [g]	3,17	3,32	6,55	6,42	6,46	6,38	6,50
Measures made	m <sub>sat (filter)</sub> [g]	0,93	1,04	1,12	1,07	1,10	0,68	0,79
after the test	m <sub>d (filter)</sub> [g]	0,48	0,54	0,53	0,52	0,49	0,37	0,36
	m <sub>h (soil+ring+filter+tare)</sub> [g]	167,14	172,05	168,98	168,00	167,06	175,42	175,65
	m <sub>d (soil+ring+filter+tare)</sub> [g]	152,31	156,54	150,25	147,48	146,83	146,41	147,19
	m <sub>d (soil)</sub> [g]	104,70	109,18	98,07	97,65	95,56	94,85	96,99
	-							
	γ <sub>d</sub> [kN/m³]	19,46	19,46	18,13	17,95	17,95	17,40	17,40
Proctor	w	12,63%	12,63%	16,77%	17,40%	17,40%	18,38%	18,38%
Values	m <sub>soil+ring</sub> [g]	162,03	166,84	159,23	158,20	156,57	157,87	158,13
	m <sub>soil+ring+filter+string</sub> [g]	163,89	168,78	161,20	160,09	158,47	159,58	159,79
	m <sub>filter+string</sub> [g]	1,86	1,94	1,97	1,89	1,90	1,71	1,66
						1		
	γd (soil sample) [kN/m³]	18,01	18,47	16,48	16,63	16,33	16,10	16,30
22/4/44.0.00		162.00	4.60 70	464.20	100.00	450.47	450.50	450 70
22/4/14 0:00	m <sub>initial</sub> [g]	163,89	168,78	161,20	160,09	158,47	159,58	159,79
	m <sub>w</sub> [g]	14,94	15,74	17,61	19,20	18,24	19,77	19,34
	γ <sub>d (E)</sub> [KN/m <sup>-</sup> ]	18,01	18,47	16,48	10,63	10,00%	16,10	10,30
	w	14,27%	14,42%	17,96%	19,66%	19,09%	20,84%	19,94%
	6 0	25,71%	20,03%	29,60%	32,70%	31,17%	53,50%	32,51%
	3	07,2770	75,00%	71,9170	/1,/5%	71,50%	02,51%	02,2270
23/1/11 11.20	m [a]	162.91	167.67	160.06	150.24	157.76	150.07	150.25
23/4/14 11.30	m [g]	13 22	13.95	15 77	17 73	16 78	18 73	139,23
	N. (5)	18.01	18 //7	16.48	16.63	16.33	16 10	16 30
	W	12 63%	12 78%	16.08%	18 16%	17 56%	19 75%	18 69%
	θ	22 75%	23 60%	26 51%	30,20%	28.68%	31 80%	30.47%
	s	59.52%	65.46%	64.39%	66.26%	65.78%	59.04%	58,33%
		22,02/0	,	.,	,20,0			
28/4/14 14:55	m[g]	170,00	173,88	167,41	166,03	163,87	166,87	166,72
· · ·	m <sub>w</sub> [g]	20,41	20,16	23,12	24,42	22,89	26,53	25,60
	γ <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18,01	18,47	16,48	16,63	16,33	16,10	16,30
	w	19,49%	18,46%	23,57%	25,01%	23,95%	27,97%	26,39%
	θ	35,12%	34,11%	38,86%	41,60%	39,12%	45,04%	43,03%
	S	91,90%	94,60%	94,41%	91,26%	89,73%	83,62%	82,37%
			-					
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29/4/14 11:25	m [g]	170,14	174,14	167,44	166,43	164,20	167,01	167,14
	m <sub>w</sub> [g]	20,55	20,42	23,15	24,82	23,22	26,67	26,02
	γ <sub>d (E)</sub> [kN/m³]	18,01	18,47	16,48	16,63	16,33	16,10	16,30
	w	19,63%	18,70%	23,61%	25,42%	24,30%	28,12%	26,83%
	θ	35,36%	34,55%	38,91%	42,28%	39,68%	45,28%	43,74%
	S	92,53%	95,82%	94,53%	92,75%	91,02%	84,06%	83,72%
5/5/14 10:54	m[g]	170,64	174,40	166,76	166,17	164,32	170,15	169,83
	m <sub>w</sub> [g]	21,05	20,68	22,47	24,56	23,34	29,81	28,71
	h [cm]	-	-	-	-	-	3,43	3,44
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	-	-	-	67,41	67,54
	γ <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18,01	18,47	16,48	16,63	16,33	14,07	14,36
	w	20,11%	18,94%	22,91%	25,15%	24,42%	31,43%	29,60%
	θ	36,22%	34,99%	37,77%	41,83%	39,89%	44,22%	42,51%
	S	94,78%	97,04%	91,75%	91,78%	91,49%	82,10%	81,36%
6/5/14 10:51	m [g]	171,00	174,56	166,96	166,95	164,23	170,67	170,50
	m <sub>w</sub> [g]	21,41	20,84	22,67	25,34	23,25	30,33	29,38
	h [cm]	-	-	-	-	-	3,43	3,44
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	-	-	-	67,41	67,54
	γ <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18,01	18,47	16,48	16,63	16,33	14,07	14,36
	W	20,45%	19,09%	23,12%	25,95%	24,33%	31,98%	30,29%
	θ	36,84%	35,26%	38,10%	43,16%	39,74%	44,99%	43,50%
	S	96,40%	97,79%	92,57%	94,69%	91,14%	83,53%	83,26%
12/5/14 10:46	m [g]	171,35	174,82	167,33	167,07	164,72	173,03	173,69
	m <sub>w</sub> [g]	21,76	21,10	23,04	25,46	23,74	32,69	32,57
	h [cm]	-	-	-	-	-	3,52	3,57
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	-	-	-	69,05	70,10
	γ <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18,01	18,47	16,48	16,63	16,33	13,74	13,84
	w	20,78%	19,33%	23,49%	26,07%	24,84%	34,46%	33,58%
	θ	37,44%	35,70%	38,73%	43,37%	40,57%	47,34%	46,46%
	S	97,97%	99,01%	94,08%	95,14%	93,06%	87,90%	88,94%
13/5/14 11:20	m [g]	171,56	174,79	168,19	167,17	165,17	173,23	174,36
	m <sub>w</sub> [g]	21,97	21,07	23,90	25,56	24,19	32,89	33,24
	h [cm]	-	-	-	-	-	3,52	3,57
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	-	-	-	69,05	70,10
	γ <sub>d (E)</sub> [kN/m³]	18,01	18,47	16,48	16,63	16,33	13,74	13,84
Beginning of suction	w	20,98%	19,30%	24,37%	26,18%	25,31%	34,68%	34,27%
PF=0 (-0,001 cm)	θ	37,80%	35,65%	40,17%	43,54%	41,34%	47,63%	47,42%
13/5/14 15:40	S	98,92%	98,87%	97,59%	95,52%	94,83%	88,44%	90,77%
19/5/14 11:00	m [g]	171,80	174,94	167,90	167,76	165,72	175,82	176,31
PF=0 (-0,001 cm)	m <sub>w</sub> [g]	22,21	21,22	23,61	26,15	24,74	35,48	35,19
	h [cm]	-	-	-	-	-	3,52	3,57
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	-	-	-	69,05	70,10
	γ <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18,01	18,47	16,48	16,63	16,33	13,74	13,84
Increase of suction	W	21,21%	19,44%	24,07%	26,78%	25,89%	37,41%	36,28%
PF=0,4 (-2,5 cm)	θ	38,21%	35,90%	39,68%	44,54%	42,28%	51,38%	50,20%
19/5/14 15:30	S	100,00%	99,58%	96,41%	97,72%	96,98%	95,40%	96,10%

27/5/14 11:00	m [g]	171,67	175,03	168,78	168,37	166,49	177,53	177,74
PF=0,4 (-2,5 cm)	m <sub>w</sub> [g]	22,08	21,31	24,49	26,76	25,51	37,19	36,62
	h [cm]	-	-	-	-	-	3,52	3,57
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	-	-	-	69,05	70,10
	γ <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18,01	18,47	16,48	16,63	16,33	13,74	13,84
Increase of suction	w	21,09%	19,52%	24,97%	27,40%	26,70%	39,21%	37,76%
PF=1 (-10 cm)	θ	37,99%	36,06%	41,16%	45,58%	43,60%	53,86%	52,24%
27/5/14 11:30	S	99,41%	100,00%	100,00%	100,00%	100,00%	100,00%	100,00%
			1			1		r
2/6/14 11:00	m [g]	168,07	172,59	166,39	165,83	164,27	175,06	175,52
PF=1 (-10 cm)	m <sub>w</sub> [g]	18,48	18,87	22,10	24,22	23,29	34,72	34,40
1°measure	h [cm]	-	-	3,090	3,123	3,117	3,68	3,71
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	60,672	61,327	61,196	72,32	72,91
	γ <sub>d (E)</sub> [kN/m³]	18,01	18,47	16,16	15,92	15,62	13,11	13,30
	w	17,65%	17,28%	22,53%	24,80%	24,37%	36,61%	35,47%
	θ	31,80%	31,93%	36,43%	39,49%	38,06%	48,01%	47,18%
	S	83,21%	88,55%	88,49%	86,64%	87,29%	89,13%	90,31%
3/6/14 11:46	m [g]	167,70	172,42	166,23	165,40	164,18	174,88	175,24
PF=1 (-10 cm)	m <sub>w</sub> [g]	18,11	18,70	21,94	23,79	23,20	34,54	34,12
2°measure	h [cm]	-	-	3,090	3,123	3,117	3,68	3,71
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	60,672	61,327	61,196	72,32	72,91
	$\gamma_{d(E)}$ [kN/m <sup>3</sup> ]	18,01	18,47	16,16	15,92	15,62	13,11	13,30
	w	17,30%	17,13%	22,37%	24,36%	24,28%	36,42%	35,18%
	θ	31,16%	31,64%	36,16%	38,79%	37,91%	47,76%	46,80%
	S	81,54%	87,75%	87,85%	85,11%	86,96%	88,67%	89,58%
	θ1-θ2	0,64%	0,29%	0,26%	0,70%	0,15%	0,25%	0,38%
		No!	No!	No!	No!	Ok	No!	No!
								•
4/6/14 11:30	m [g]	167,44	172,13	166,06	165,30	164,08	174,75	174,96
PF=1 (-10 cm)	m <sub>w</sub> [g]	17,85	18,41	21,77	23,69	23,10	34,41	33,84
3°measure	h [cm]	-	-	3,090	3,123	3,117	3,68	3,71
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	60,672	61,327	61,196	72,32	72,91
	γ <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18,01	18,47	16,16	15,92	15,62	13,11	13,30
Increase of suction	w	17,05%	16,86%	22,20%	24,26%	24,17%	36,28%	34,89%
PF=1,5 (-31,6 cm)	θ	30,71%	31,15%	35,88%	38,63%	37,75%	47,58%	46,41%
4/6/14 14:30	S	80,37%	86,39%	87,17%	84,75%	86,58%	88,34%	88,84%
	θ2-θ3	0,45%	0,49%	0,28%	0,16%	0,16%	0,18%	0,38%
		No!	No!	No!	Ok	Ok	Ok	No!

11/6/14 11:50	m [g]	167,24	171,77	165,62	164,74	163,77	173,86	173,94
PF=1,5 (-31,6 cm)	m <sub>w</sub> [g]	17,65	18,05	21,33	23,13	22,79	33,52	32,82
1°measure	h [cm]	-	-	3,090	3,123	3,117	3,67	3,69
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	60,672	61,327	61,196	72,06	72,45
	γ <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18,01	18,47	16,16	15,92	15,62	13,16	13,39
	w	16,86%	16,53%	21,75%	23,69%	23,85%	35,34%	33,84%
	θ	30,37%	30,54%	35,16%	37,72%	37,24%	46,52%	45,30%
	S	79,47%	84,70%	85,41%	82,75%	85,42%	86,37%	86,71%
12/6/14 11:05	m [g]	167,05	171,61	165,41	164,48	163,55	173,49	173,52
PF=1,5 (-31,6 cm)	m <sub>w</sub> [g]	17,46	17,89	21,12	22,87	22,57	33,15	32,40
2°measure	h [cm]	-	-	3,090	3,123	3,117	3,67	3,69
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	60,672	61,327	61,196	72,06	72,45
	γ <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18,01	18,47	16,16	15,92	15,62	13,16	13,39
	w	16,68%	16,39%	21,54%	23,42%	23,62%	34,95%	33,41%
	θ	30,04%	30,27%	34,81%	37,29%	36,88%	46,00%	44,72%
	S	78,61%	83,95%	84,56%	81,81%	84,60%	85,41%	85,60%
	θ1-θ2	0,33%	0,27%	0,35%	0,42%	0,36%	0,51%	0,58%
		No!						
			1			1		1
13/6/14 13:44	m [g]	166,99	171,55	165,37	164,32	163,39	173,17	173,30
PF=1,5 (-31,6 cm)	m <sub>w</sub> [g]	17,40	17,83	21,08	22,71	22,41	32,83	32,18
3°measure	h [cm]	-	-	3,090	3,123	3,117	3,67	3,69
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	60,672	61,327	61,196	72,06	72,45
	γ <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18,01	18,47	16,16	15,92	15,62	13,16	13,39
Increase of suction	w	16,62%	16,33%	21,49%	23,26%	23,45%	34,61%	33,18%
PF=1,8 (-63,1 cm)	θ	29,94%	30,17%	34,74%	37,03%	36,62%	45,56%	44,42%
13/6/14 15:40	S	78,34%	83,67%	84,40%	81,24%	84,00%	84,59%	85,02%
	θ2-θ3	0,10%	0,10%	0,07%	0,26%	0,26%	0,44%	0,30%
		Ok	Ok	Ok	No!	No!	No!	No!

23/6/14 13:25	m [g]	166,68	171,39	165,07	164,22	163,10	172,03	172,20
PF=1,8 (-63,1 cm)	m <sub>w</sub> [g]	17,09	17,67	20,78	22,61	22,12	31,69	31,08
1°measure	h [cm]	-	-	3,090	3,067	3,083	3,54	3,57
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	60,672	60,214	60,541	69,57	70,10
	γ <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18,01	18,47	16,16	16,22	15,78	13,63	13,84
	w	16,32%	16,18%	21,19%	23,15%	23,15%	33,41%	32,04%
	θ	29,40%	29,90%	34,25%	37,55%	36,54%	45,55%	44,34%
	S	76,95%	82,92%	83,20%	82,38%	83,81%	84,57%	84,87%
						1		1
24/6/14 12:05	m [g]	166,56	171,29	164,96	164,09	163,01	171,84	172,02
PF=1,8 (-63,1 cm)	m <sub>w</sub> [g]	16,97	17,57	20,67	22,48	22,03	31,50	30,90
2°measure	h [cm]	-	-	3,090	3,067	3,083	3,54	3,57
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	60,672	60,214	60,541	69,57	70,10
	γ <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18,01	18,47	16,16	16,22	15,78	13,63	13,84
Increase of suction	w	16,21%	16,09%	21,08%	23,02%	23,05%	33,21%	31,86%
PF=2 (-100 cm)	θ	29,20%	29,73%	34,07%	37,33%	36,39%	45,28%	44,08%
41814,72917	S	76,41%	82,45%	82,76%	81,91%	83,46%	84,06%	84,38%
	θ2-θ3	0,21%	0,17%	0,18%	0,22%	0,15%	0,27%	0,26%
		No!	Ok	Ok	No!	Ok	No!	No!
20/6/14 15:15		166.66	171.26	164.04	164.20	162.15	171.04	171 00
BE=2(100  cm)	m [g]	17.07	171,50	20.65	22 50	22 17	21 50	20.76
PF=2 (-100 cm)	m <sub>w</sub> [g]	17,07	17,04	20,05	22,59	22,17	31,50	30,76
1°measure	h [cm]	-	-	3,073	3,050	3,043	3,55	3,53
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	60,345	59,887	59,756	69,64	69,31
	γ <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18,01	18,47	16,25	16,31	15,99	13,62	13,99
	w	16,30%	16,16%	21,06%	23,13%	23,20%	33,21%	31,71%
	θ	29,37%	29,85%	34,22%	37,72%	37,10%	45,23%	44,38%
	S	76,86%	82,78%	83,13%	82,76%	85,10%	83,98%	84,95%
1/7/14 15:15	m [g]	166,62	171,33	164,89	164,16	163,12	171,78	171,84
PF=2 (-100 cm)	m <sub>w</sub> [g]	17,03	17,61	20,60	22,55	22,14	31,44	30,72
2°measure	h [cm]	-	-	3,073	3,050	3,043	3,55	3,53
	V <sub>swelling</sub> [cm <sup>3</sup> ]	-	-	60,345	59,887	59,756	69,64	69,31
	γ <sub>d (E)</sub> [kN/m <sup>3</sup> ]	18,01	18,47	16,25	16,31	15,99	13,62	13,99
	w	16,27%	16,13%	21,01%	23,09%	23,17%	33,15%	31,67%
	θ	29,30%	29,80%	34,14%	37,65%	37,05%	45,15%	44,32%
	S	76,68%	82,64%	82,93%	82,61%	84,98%	83,82%	84,84%
	θ2-θ3	0,07%	0,05%	0,08%	0,07%	0,05%	0,09%	0,06%
		Ok						

## **SWELLING MEASURES – T12**

h [mm]	T12 5%	T12 5%	T12 7%	T12 7%	T12 12%	T12 12%
	E1	E2	E1	E2	E1	E2
Initial Value	3,00	3,03	2,98	3,00	3,02	3,00
1st Measure	-	-	-	-	3,38	3,35
2nd Measure	-	-	-	-	3,44	3,42
3rd Measure	-	-	-	-	3,50	3,42
4th Measure	-	-	3,08	3,12	3,51	3,44
5th Measure	3,03	3,10	3,04	3,04	3,42	3,38
6th Measure	3,02	3,08	3,04	3,04	3,39	3,34

## **SWELLING MEASURES – T3**

h [mm]	T3 5%	T3 7%	T3 7%	T3 12%	T3 12%
[]	E1	E1	E2	E1	E2
Initial Value	3,03	2,99	2,98	3	3,03
1st Measure	-	-	-	3,43	3,44
2nd Measure	-	-	-	3,52	3,57
3rd Measure	-	-	-	3,68	3,71
4th Measure	-	3,12	3,12	3,67	3,69
5th Measure	3,09	3,07	3,08	3,54	3,57
6th Measure	3,07	3,05	3,04	3,55	3,53

## **θ VARIATION DURING THE HANGING COLUMN TEST – T12**

h [m]	nf	T12 0%	T12 0%	T12 5%	T12 5%	T12 7%	T12 7%	T12 12%	T12 12%
n [m] þi	рг	E1	E2	E1	E2	E1	E2	E1	E2
0,01	0,00	37,02%	35,55%	42,03%	41,89%	44,44%	43,12%	51,81%	49,06%
0,03	0,40	36,58%	35,12%	42,03%	41,89%	44,44%	43,12%	51,81%	49,06%
0,10	1,00	33,51%	32,62%	37,73%	37,75%	38,35%	38,29%	47,92%	45,66%
0,32	1,50	33,35%	32,38%	37,53%	37,61%	38,19%	38,11%	47,74%	45,48%
0,63	1,80	33,10%	32,15%	37,36%	37,45%	37,96%	37,96%	47,57%	45,32%
1,00	2,00	32,34%	31,67%	36,75%	36,79%	37,66%	37,58%	46,81%	44,58%

h [m]		T3 0%	T3 0%	T3 5%	T3 7%	T3 7%	T3 12%	T3 12%
	рі	E1	E2	E1	E1	E2	E1	E2
0,01	0,00	38,21%	36,06%	41,16%	45,58%	43,60%	53,86%	52,24%
0,03	0,40	37,99%	36,06%	41,16%	45,58%	43,60%	53,86%	52,24%
0,10	1,00	31,80%	31,93%	36,43%	39,49%	38,06%	48,01%	47,18%
0,32	1,50	31,16%	31,64%	36,16%	38,79%	37,91%	47,76%	46,80%
0,63	1,80	30,71%	31,15%	35,88%	38,63%	37,75%	47,58%	46,41%
1,00	2,00	30,37%	30,54%	35,16%	37,72%	37,24%	46,52%	45,30%

## **θ VARIATION DURING THE HANGING COLUMN TEST – T3**

## S VARIATION DURING THE HANGING COLUMN TEST – T12

h []		T12 0%	T12 0%	T12 5%	T12 5%	T12 7%	T12 7%	T12 12%	T12 12%
n [m]	pr	E1	E2	E1	E2	E1	E2	E1	E2
0,0100	0,00	100,00%	100,00%	100,00%	100,00%	100,00%	100,00%	100,00%	100,00%
0,0251	0,4	98,80%	98,79%	100,00%	100,00%	100,00%	100,00%	100,00%	100,00%
0,1000	1,00	90,50%	91,75%	89,75%	90,11%	86,31%	88,79%	92,48%	93,07%
0,3162	1,50	90,09%	91,08%	89,27%	89,80%	85,94%	88,37%	92,14%	92,71%
0,6310	1,80	89,39%	90,45%	88,88%	89,41%	85,42%	88,03%	91,81%	92,38%
1,0000	2,00	87,36%	89,10%	87,44%	87,84%	84,75%	87,16%	90,35%	90,87%

#### **S VARIATION DURING THE HANGING COLUMN TEST – T3**

h [m] pf	f	T3 0%	T3 0%	T3 5%	T3 7%	T3 7%	T3 12%	T3 12%
	pr	E1	E2	E1	E1	E2	E1	E2
0,0100	0,00	100,00%	100,00%	100,00%	100,00%	100,00%	100,00%	100,00%
0,0251	0,4	99,41%	100,00%	100,00%	100,00%	100,00%	100,00%	100,00%
0,1000	1,00	83,21%	88,55%	88,49%	86,64%	87,29%	89,13%	90,31%
0,3162	1,50	81,54%	87,75%	87,85%	85,11%	86,96%	88,67%	89,58%
0,6310	1,80	80,37%	86,39%	87,17%	84,75%	86,58%	88,34%	88,84%
1,0000	2,00	79,47%	84,70%	85,41%	82,75%	85,42%	86,37%	86,71%

## PARAMETERS OF VAN GENUCHTEN MODEL – Scilab program

function [ALPHAopt, Mopt, RMSmin, Nopt, entrant]=ParametreVG()

```
stacksize('max')
```

```
format('v',20)
```

```
fichier=<u>input</u>("entrer nom du fichier ","string")
entrant=read(fichier,-1,2)
taille=size(entrant)
nbrcombinaison=<u>input(</u>"entre le nombre de couple alpha et n à tester = ")
nbrpoints=taille(1,1)
```

```
alphamin=0
alphamax=1
```

```
"RESULtab=ones(3,nbrcombinaison)"
ALPHAopt=999
Mopt=999
RMSmin=9999999
"Z=zeros(1,nbrcombinaison)"
"Y=zeros(1,nbrcombinaison)"
"X=zeros(1,nbrcombinaison)"
```

```
modele=ones(nbrpoints,1)
```

```
for i=1:nbrcombinaison

RMS=0

alpha=alphamin+rand()*(alphamax-alphamin)

m=rand()

for j=1:nbrpoints

modele(j,1)=(1+((alpha)*(entrant(j,1)))^(1/(1-m)))^(-m)

RMS=RMS+(entrant(j,2)-modele(j,1))^2

end
```

```
RMS=sqrt(RMS/nbrpoints)
"RESULtab(1,i)=RMS"
"RESULtab(2,i)=alpha"
```

```
"RESULtab(3,i)=m"
```

```
"Z(1,i)=RMS"
"Y(1,i)=alpha"
"X(1,i)=1/(1-m)"
```

```
if RMS<RMSmin then
RMSmin=RMS
ALPHAopt=alpha
Mopt=m
Nopt=1/(1-m)
end
end
```

"param3d(X,Y,Z)"

endfunction

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