A NEW METHOD TO CORRECT VOID RATIO FOR EXPANSIVE SOIL

by

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A NEW METHOD TO CORRECT THE VOID RATIO FOR EXPANSIVE SOIL

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Abstract

Expansive soil undergoes significant volume variations due to changes in soil water content. Understanding this problematic soil is key for an effective design of landfill layers, dams, pavements, and other structures on this type of soil. Not much has been discussed about the effect of soil expansion on the calculation of soil void ratio, which is the space between soil grains. Increase in moisture occupying the soil voids causes water infiltration into the space between adjacent clay layers leading to increase in the mineral d-spacing and consequently the volume of the clay particle. Therefore, the global soil void ratio, in the case of expansible soil, is composed of the effective space between grains and the space occupied by the water in the interlayer space. This research proposes a new method to calculate an effective void ratio for expansible soil which is calculated after deducting the interlayer water from the global void ratio. This new method differs from others previously put forth because it can be applied to all expansible soils. The effective void ratio concept is developed using soil-water characteristic curves, where the void ratio is a key parameter. Also, the effect of time on particle hydration was evaluated via hydrometer and laser diffraction tests. The experimental results indicated that the volume of the clay particles increased, which was observed to be due to change in the specific gravity of the expansible soil. The void ratio and water content were corrected, and the conclusion is that the interlayer water should be considered separately for expansible soil even for a soil composed of small amounts of expansible minerals.
Lay Summary

Expansive soil present significant volume variations when there is a change in its water content. Thus, expansive soils can impose significant and costly damages to the structures and therefore their behavior must be well understood. One reason for the volume change is the variation of the particle volume due to ingress of water into the individual particles. Traditional design calculations assume that the particle volume remains constant, which raises some problems because the water that has entered the particles no longer behaves like water around the particles. In addition, the particle volume increase also affects the void space between the particles, which consequently modifies the soil permeability. This research proposes a new simple calculation method which requires less testing. The outcome and benefit of the proposed method is that this calculation can be used for any type of expansible soil and grants a better understanding of expansive soils.
Preface

This research was performed under the supervision of Dr. Sumi Siddiqua. The laboratory experiments were conducted in the Geo-Material Research Laboratory at the School of Engineering, University of British Columbia (UBC). Ethics approval from the UBC Research Ethics Board was not required for this research study.

The list of publications related to this thesis is presented below. While earning my M.Sc, I was able to publish one conference papers associated with this research at peer-reviewed geotechnical engineering conference.

- Camapum de Carvalho, J., Guimaraes, R., Siddiqua, S., Bigdeli, A., Barreto, P.
  “Relationship between soil plasticity and the soil air entry value in the soil-water characteristic curve” (in Portuguese) GEOCENTRO 2017, Goiânia, Brazil.
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<table>
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<th>Symbol</th>
<th>Description</th>
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<tbody>
<tr>
<td>AEV</td>
<td>Air-entry value</td>
</tr>
<tr>
<td>CEC</td>
<td>Cation exchange capacity</td>
</tr>
<tr>
<td>(d_{001})</td>
<td>d-spacing or basal spacing, distance between two clay layers</td>
</tr>
<tr>
<td>(d_p)</td>
<td>Clay particle diameter</td>
</tr>
<tr>
<td>(e)</td>
<td>Void ratio</td>
</tr>
<tr>
<td>(e_{corrected})</td>
<td>Corrected void ratio</td>
</tr>
<tr>
<td>EDS</td>
<td>Energy-dispersive spectroscopy</td>
</tr>
<tr>
<td>(g)</td>
<td>Acceleration due to gravity</td>
</tr>
<tr>
<td>GS</td>
<td>Specific gravity</td>
</tr>
<tr>
<td>(I_p)</td>
<td>Plasticity index</td>
</tr>
<tr>
<td>LL</td>
<td>Liquid limit</td>
</tr>
<tr>
<td>(m_s)</td>
<td>Mass of solids</td>
</tr>
<tr>
<td>(m_{iw})</td>
<td>Mass of interlayer water</td>
</tr>
<tr>
<td>(m_w)</td>
<td>Mass of water</td>
</tr>
<tr>
<td>(n)</td>
<td>Number of stack layers per particle</td>
</tr>
<tr>
<td>(N_p)</td>
<td>Number of expansible particles</td>
</tr>
<tr>
<td>PI</td>
<td>Plasticity index</td>
</tr>
<tr>
<td>PL</td>
<td>Plastic limit</td>
</tr>
<tr>
<td>PSD</td>
<td>Particle-size distribution</td>
</tr>
<tr>
<td>RH</td>
<td>Relative humidity</td>
</tr>
<tr>
<td>RO</td>
<td>Reverse osmosis</td>
</tr>
<tr>
<td>SEM</td>
<td>Scanning electron microscopy</td>
</tr>
<tr>
<td>Symbol</td>
<td>Definition</td>
</tr>
<tr>
<td>--------</td>
<td>------------</td>
</tr>
<tr>
<td>$S_r$</td>
<td>Degree of saturation</td>
</tr>
<tr>
<td>$S_{r,\text{corrected}}$</td>
<td>Corrected degree of saturation</td>
</tr>
<tr>
<td>$SWCC$</td>
<td>Soil-water characteristic curve</td>
</tr>
<tr>
<td>$USCS$</td>
<td>Unified soil classification system</td>
</tr>
<tr>
<td>$V$</td>
<td>Soil volume</td>
</tr>
<tr>
<td>$V_{air}$</td>
<td>Volume of air</td>
</tr>
<tr>
<td>$V_{iw}$</td>
<td>Volume of interlayer water</td>
</tr>
<tr>
<td>$V_p$</td>
<td>Volume of one particle</td>
</tr>
<tr>
<td>$V_{p,\text{dry}}$</td>
<td>Volume of one particle when dry (not hydrated)</td>
</tr>
<tr>
<td>$V_s$</td>
<td>Volume of solids</td>
</tr>
<tr>
<td>$V_{s,'}$</td>
<td>Corrected volume of solids</td>
</tr>
<tr>
<td>$V_{s,\text{swell}}$</td>
<td>Volume of swelling solids</td>
</tr>
<tr>
<td>$V_{v,p}$</td>
<td>Inter-particle void volume</td>
</tr>
<tr>
<td>$V_v$</td>
<td>Volume of voids</td>
</tr>
<tr>
<td>$V_w$</td>
<td>Volume of water</td>
</tr>
<tr>
<td>$V_{w,p}$</td>
<td>Volume of water in the inter-particle space</td>
</tr>
<tr>
<td>$w$</td>
<td>Soil water content</td>
</tr>
<tr>
<td>$w_{c}$</td>
<td>Corrected water content</td>
</tr>
<tr>
<td>$w_{opt}$</td>
<td>Optimum water content</td>
</tr>
<tr>
<td>$w_L$</td>
<td>Moisture content at liquid limit</td>
</tr>
<tr>
<td>$w_P$</td>
<td>Moisture content at saturation</td>
</tr>
<tr>
<td>$\rho_w$</td>
<td>Density of water</td>
</tr>
<tr>
<td>$\rho_{w,i}$</td>
<td>Density of interlayer water</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>------------------------------------</td>
</tr>
<tr>
<td>$v_{0,w}$</td>
<td>Specific water volume</td>
</tr>
<tr>
<td>$\psi$</td>
<td>Soil suction</td>
</tr>
<tr>
<td>$\omega_v$</td>
<td>Molecular mass of water vapor</td>
</tr>
<tr>
<td>$\gamma_d$</td>
<td>Dry unit weight</td>
</tr>
<tr>
<td>$\gamma_w$</td>
<td>Unit weight of water</td>
</tr>
<tr>
<td>$\gamma_{0d}$</td>
<td>Initial dry density</td>
</tr>
</tbody>
</table>
Acknowledgments

I would like to express my sincere gratitude to my supervisor, Dr. Sumi Siddiqua, for giving me the opportunity of being her student. Her patience, support, and motivation were extremely important. Thank you is not enough for all the help and support given by Dr. Camapum. He is a brilliant engineer and I am very grateful for having the opportunity to discuss this research with him. Also, I would like to thank Dr. Dwayne Tannant and my committee members Dr. Salomon Tesfamariam and Dr. Mahmudur Fatmi for their insightful comments.

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I can not forget the support provided by my professors and colleagues from my past university, Federal University of Bahia. Even from far away they helped me in finding literature and performing experiments.

This journey would be not possible without the support of my beloved family who have been an important and indispensable source of emotional support. Thank you for believing in me and being by my side for every choice I make.

Being far from home was difficult, but it was possible thankfully to the great friends I have. I can never forget the support given by them.
Dedication

It is dedicated to my inspiring and supportive parents, Marcelo and Adriana, and

brother, Eduardo.
Chapter 1: Introduction

1.1 Overview

Expansive soils are susceptible to significant volume change when subjected to variation of water content. The soil expands when the water content increases and shrinks when it dries. The root cause for moisture induced volume changes in soil is the combination of seasonal climate conditions (rainfall, drought periods, temperature, wet and dry cycles) and engineering activities that impact the ground condition. Increase in moisture in soil voids causes water infiltration in the space between clay layers increasing the mineral d-spacing and consequently the volume of the clay particle. Therefore, the global soil void ratio, in the case of expansible soil, is composed of the effective space between grains and the space occupied by the water in the interlayer space. This research presents an effective void ratio calculation for expansive soil that is determined after deducting the interlayer water from the global void ratio calculation. The effective void ratio concept is developed using soil-water characteristic curves, where the void ratio is a key parameter. The experimental results indicate that the interlayer water should be considered separately for expansible soils, even for soils which contain a small amount of expansible minerals.

This chapter provides brief background information of expansible soil, the motivations for selecting the research project, and the novelty of the research. The research objectives and required tasks to achieve the objectives are presented in this chapter. Finally, the chapter ends with the thesis structure.

1.2 Motivations

Expansive soil is defined as soil that can undergo large volume and strength variations due to changes in moisture content regime, which in turn can cause extensive damage to infrastructure
founded on expansive soils (Canadian Geotechnical Society, 1992; Houston et al., 2011). The cost associated with such damages is significant. Nelson et al. (2015) stated that the repair for damages caused by expansive soil can exceed original cost of constructions. In Britain, during the 1990’s, the cumulative damage was estimated to be £3 billion (Bell & Culshaw, 2001). Currently, the annual cost of repairs for related damages in Sudan is $6 million per year (Zumrawi et al., 2017). In the United States, the damage is estimated to cost $15 billion per year (Jones & Jefferson, 2012), and 25% of the homes in the United State are directly affected (Dasog & Mermut, 2013). Water content increase of a potentially expansible soil causes water absorption, leading to the expansion of soil particles, consequently resulting in increasing soil volume and reducing the soil strength. With time, the soil may lose water due to evapotranspiration or variation in the state of stress. The loss of strength during expansion can cause drying cracks when the soil loses water and shrinks. Light loaded structures, such as pavements and residences, are the ones that incur the greatest losses because of expansible soils (Bell & Culshaw, 2001). Engineered structures such as shallow foundations, embankments, dams, pavements, parking and runways which are founded in these soils are subjected to damaging deformations and stresses. The expansion pressure against pipes, foundations, or other structures may cause pipe breaks, cracks, and differential settlement, putting the overall safety of the structure in danger (Peck et al., 1974). Due to the issues associated with expansive soils, there is a need for extensive analysis of expansive soil behavior to minimize damages and make improved design predictions.

Expansive soils can be found in arid and semiarid areas in more than 40 countries over 6 continents (except Antarctica), including Australia, China, India, Israel, South Africa, Brazil, Canada, and the United States (Nelson et al., 2015; Puppala et al., 2006). It is estimated that expansive soils occupy about 350 million hectares, or 2.35% of the Earth’s land (Dasog & Mermut, 2013). They
are found in all areas of North America, and most of the serious problems, such as differential settlement, occur in the West and Southwest regions of Canada and the United States due to low precipitation and the presence of overconsolidated claystone and clayshale in these areas (Nelson et al., 2015).

The swell-shrink potential, or the volume change potential, depends on the initial water content of the soil, applied vertical stress, void ratio, and the types and amounts of clay minerals in the soil structure (Bell & Culshaw, 2001). These parameters affect the soil density, which is a very important parameter for every design in geotechnical engineering. Soil density is directly related to the soil void ratio. Soil void ratio is defined as the space between soil grains. The voids can be occupied by air, partially occupied by water, or totally occupied by water (saturated condition). When water is added to soil, crystalline swelling occurs which increases the basal spacing, or interlayer spacing, of the mineral. The clay mineral volume increase causes the expansion of the clay particle. This expansion happens in the space between particles, resulting in the reduction of the actual inter-particle voids. However, the void ratio calculation does not differentiate between inter-particle voids and interlayer voids occupied by water, and void ratio is an important parameter for both mechanical and hydraulic behavior of the soil. For instance, void ratio affects permeability, an extremely important parameter for bentonite engineered barriers, and is related to leakage control. In addition, the water occupying the interlayer space is restrained between clay layers and cannot contribute to the mechanical and hydraulic behavior of the soil.

Campos et al. (2008) and later Camapum de Carvalho et al. (2017) discussed the void ratio correction for two Brazilian bentonites based on hypothetical interlayer distances found in literature. The corrected void ratio corresponds to the true volume of voids between particles and disregards the voids in the interlayer spaces. Their results indicated that the void ratio correction
causes significant difference on the soil-water characteristic curve (SWCC) analysis, but the actual interlayer distance should be measured for more conclusive findings. De la Morena et al. (2018) and Sedighi & Thomas (2014) developed a method to calculate the corrected void ratio with the interlayer distance measured for the MX-80 and FEBEX bentonites by Villar (2007). Sedighi and Thomas (2014) calculations were based on the number of moles of the clay mineral. Their method is great for a soil composed mostly of one clay mineral but makes it complex to calculate when there is a variety of mineral present in the soil. De la Morena et al. (2018) proposed a method for natural bentonites based on the content of the exchangeable cations, viz. sodium and calcium. Therefore, no method was proposed for an expansible soil other than bentonites.

This research presents a new method to calculate the interlayer water volume and correct the void ratio based on volumetric relationships. This method allows the correction of a wider range of expansible soil with a diverse amount of different expansible minerals. To test this proposed model the interlayer distance of a local expansible soil (partially composed of expansible clay) was analyzed. The same analysis was performed on the MX-80 and FEBEX bentonites which had their SWCC and interlayer distance measured by Villar (2007).

1.3 Objectives

The aim of this work is to analyse the effect of differentiating the interlayer and inter-particle water in the calculation of water content, degree of saturation, and void ratio for expansible soil. In order to achieve this goal, the following objectives must be met:

1) Develop a method to calculate the interlayer water for corrections of water content, degree of saturation, and void ratio based on volumetric relationships. The corrections are made after measuring the variation of d-spacing with change in soil water content.
2) Validate the proposed method by applying it on two bentonites with data (mineralogy analysis and SWCC) presented in Villar (2007).

3) Characterize local expansive soil (physical and mineralogical analysis) and develop the SWCC of this material to perform the corrections on the soil index parameter considering the intra-crystalline swell.

These objectives are accomplished by implementing the following primary tasks:

1) Development of a step by step calculation that characterizes the volume of the swollen particle for further correction of the void ratio, degree of saturation, and water content.

2) Characterization of the physical properties of the soil (particle size distribution, specific gravity, Atterberg limits, shrinkage curve, compaction) and determine the SWCC (filter paper test).

3) Studying soil microstructure by performing X-ray diffraction (XRD) tests under different sample conditions: smeared, glycolated with ethylene glycol, and hydrated. Later, quantitative and qualitative analyses were completed to determine the soil mineral composition and clay interlayer (d-spacing) distance. Scanning electron microscopy (SEM) images were also analyzed to evaluate structure of natural soil.

4) Application of the procedure to correct the soil parameters after particle swell on the Glenmore clay, FEBEX, and MX-80. The correction of the void ratio, degree of saturation, and water content were analyzed using the SWCC as a tool to evaluate the correction.

1.4 Novelty of research

The novelty of this research is the development of a new method to calculate the corrected void ratio and water content of a local soil (which has less expansible minerals than bentonites) developed with volumetric relationships. De la Morena et al. (2018) and Sedighi & Thomas (2014)
developed models to calculate the corrected void ratio for bentonites which are based on exchangeable cations and number of moles of the bentonites. However, these are complex models and are specific to bentonites. The proposed calculation can be used for any expansible soil and needs information from the particle size distribution and mineralogical analysis (X-Ray diffraction test).

The calculation was based on the observation of the consequences that the intra-crystalline swell has on the soil void ratio calculation. The water that occupies the clay interlayer space and causes the intra-crystalline swell is not considered separately from the overall water in the soil. This leads to an overestimation of the water that is available in the soil inter-particle voids and the soil void ratio. This research proposes that both water content and void ratio should be corrected considering the increase of the soil particle volume caused by the water that moved from the inter-particle void to the space between the clay layers. This correction affects both mechanical and hydraulic behavior of the soil.

1.5 Thesis organization

This dissertation consists of five chapters, as follows:

**Chapter One** presents the importance of a in depth understanding of expansive soils in engineering applications. Further, it highlights the importance of suction on soil swelling and the gap of knowledge in this area. This chapter emphasizes the objectives of this study that can contribute on advancing the knowledge of expansive soils. Finally, the steps to achieve the objectives are listed.

**Chapter Two** reviews background information on expansive soils to characterize and understand the soil swell and shrinkage. The causes of soil volume change and methods to estimate the soil
swell potential are also presented in this chapter. Later, the SWCC is explained. Finally, chapter ends with the discussions on corrected void ratio discussion proposed in literature.

Chapter Three presented the proposed model for the interlayer water volume calculation. The corrected values of water content, void ratio and degree of saturation were also presented.

Chapter Four refers to the experimental methods adopted in this study. Preliminary tests were performed to characterize the studied soil. SEM images and XRD analysis were performed to understand the soil’s structure. In addition, SWCC of the soil was measured.

Chapter Five presents the results of the performed tests. The effect of hydration on particle size was analyzed after hydrometer, and laser diffraction tests. XRD data was analyzed and soil mineral composition presented. The d-spacing (interlayer distance) of soil mineral was evaluated after soil hydration. The SWCC was determined and the transformed model was applied. The d-spacing data was used in the interlayer void ratio and the corrected void ratio was determined.

Chapter Six contains a summary of this dissertation and presents the outcomes of each objective. Moreover, this chapter presents the limitations of this study and the recommendations for future studies in this area of research.
Chapter 2: Literature Review

2.1 Overview

Both microscale and macroscale factors contribute to soil expansion. Microscale factors are related to the particles, mineralogy and structure of soil as well as pore fluid chemistry. Macroscale factors are directly related to the initial water content, suction, applied stress, and void ratio (Bell & Culshaw, 2001). This present study focuses on the consequences of soil expansion caused by microscale factors on soil macroscale factors of void ratio, water content, and degree of saturation. This chapter reviews background information of the soil expansion process, where indirect and empirical methods to identify potentially expansible soils are discussed. The increase in interlayer space after soil hydration is reviewed. Since suction tests were performed to evaluate the void ratio correction, the concept of soil suction is reviewed and the most common methods of suction measurement are presented.

2.2 Soil expansibility

The clay fraction of the soil is responsible for the swelling of the soil. Clay expands when in contact with water and shrinks with water loss. The expansion process is considered reversible (Madsen & Müller-Vonmoss, 1989). There is some variation of the vertical deformation and volumetric deformation of the soil for each swell and shrinkage cycle, until the equilibrium state is reached and the deformations due to wetting and drying are almost constant for further cycles (Tripathy & Subba Rao, 2009). Since clays are responsible for the swelling behavior of the soil, it is important to understand their mineralogy and structure for comprehension of the expansion process.
2.2.1 Clay mineralogy and soil structure

Clays are formed after chemical weathering of rocks (Suguio, 1973). Climate, topography, vegetation, parent material, and time are the main factors that form different clay minerals. For instance, it is likely to find expansible soils in places where the annual precipitation is less than 380mm (Grim, 1959). Clay particles behave differently than sand and silt particles due to their small sizes (diameter smaller than 2 µm), charged surfaces, and the influence of ions in the water. Three components form a clay micelle unit: solid clay particles, cations, and bound water. Nelson et al. (2015) describes the clay particle as a “negatively charged inner mineral core surrounded by positively charged cations that neutralize the mineral charge”.

Clays are composed of clay minerals and have a sheet or layered crystalline structure formed by silicates of aluminum and/or iron and magnesium (Grim, 1959). Clay minerals are formed after the combination of fundamental units: tetrahedral and octahedral. Grim (1959) explains the structure of each of these units. The tetrahedral unit is formed by a silicon atom (Si$^+$) equidistant from four oxygen atoms (O$^-$). The tetrahedra are linked by oxygen atoms on the same plane to form the sheet structure, meaning that two tetrahedra share the same oxygen on the base, satisfying the two necessary bonds of oxygen. However, the oxygen on the top of the tetrahedra has an unsatisfied bond, which means that a chemical bond with a positively charged ion is possible (Grim, 1959; Nelson et al., 2015). The octahedral unit has one aluminum (Al$^{+3}$) atom with six hydroxyls (OH$^-$) and forms an octahedral shape with all chemical bonds satisfied. Octahedral units are able to form a sheet structure by sharing hydroxyls (Grim, 1959; Nelson et al., 2015). The cation in the center of the tetrahedral or octahedra can be substituted by other cations without changing the original crystal form. This process is called isomorphous substitution and results in a net negative charge on the clay particle (Lambe, 1958; Nelson et al., 2015). Different
arrangements of these fundamental units form different clay minerals and the most common are kaolinites, montmorillonites, illites, and chlorites; these minerals are described below.

The kaolinite structural unit is formed by an aluminum octahedral layer on top of a layer of tetrahedral units. The oxygen from the tetrahedra strongly bonds with the hydroxyl in the octahedra sheet through hydrogen bonding (Camapum de Carvalho et al., 2015; Nelson et al., 2015), forming the kaolinite unit. The attractive force between these units due to the hydrogen bonds is strong, making it difficult for water to penetrate. It is the most stable form of clay minerals. Each kaolinite structural unit is about 7 Å thick (1 Å = 10⁻¹⁰ m) and the stacking of these units form the kaolinite mineral (Grim, 1959). Since there is no cation substitution for this mineral, its fixed formula is Si₄Al₄O₁₀(OH)₈. The particles have a lamellar shape which favors high plasticity; however, it is still less plastic than montmorillonites and illites. Additionally, they have low cation exchange capacity, CEC (Sobral, 1956).

The montmorillonite structural unit is formed by an aluminum octahedral layer between two layers of tetrahedral units (2:1 structure). The attraction force between the structural units is of van der Walls type and water can penetrate it easily, leading to cation hydration and soil expansion (Camapum de Carvalho et al., 2015). This very weak attraction force does not allow the particle to sustain a number of structural units so the particles can be as small as 9.5 Å, especially in water (Grim, 1959; Nelson et al., 2015). Montmorillonite particles are very thin with its shape characterized as flaky sheets (Lambe, 1958). They have intense surface activity which causes high CEC and high water absorptive capacities. Depending on the clay-mineral formation process, the exchangeable base may be calcium or magnesium, which increases the material's ability to expand. The theoretical formula of the montmorillonites is Al₄Si₈O₂₀(OH)₄nH₂₀, there is an n variable because the number of water molecules varies, which causes variable particle sizes (Camapum de
Carvalho et al., 2015). The difference in particle size is due to the distances between the top of two successive layers, or inter-planar distance.

Illites is composed of 2:1 structure units like montmorillonites, the difference is in the interchangeable ions that create a more stable relationship between the structural units, meaning they are less expansive (Grim, 1959). The exchangeable ion of illites is potassium, which prevents the absorption of water located between the molecular planes (Sobral, 1956). The mineral crystal shape are irregular flaky sheets (Lambe, 1958).

Chlorites are a group of clay minerals that may also present expansible behavior if their magnesium cations are not exchanged by other cations (Camapum de Carvalho et al., 2015). This mineral has the shape of sheets and its structure is the 2:1 type composed of a central octahedral sheet between two tetrahedral sheets. The mineral general formula is (Si, Al)₈(Mg, Fe)₆O₂₀(OH)₄ with a layer of brucite (Mg,Al)₆(OH)₁₂ between the 2:1 structures. Brucite is a layer that has an octahedral arrangement with a magnesium atom in the center surrounded by six hydroxyls (Azevedo & Torrado, 2009; Camapum de Carvalho et al., 2015; Grim, 1959). Due to this brucite layer between the 2:1 structural units, some authors also consider chlorite as having 2:1:1 structure.

**Figure 2-1: Clay minerals arrangement**
One important characteristic of the clays is the specific surface area. The specific surface area is the total external area of every particle in one gram of soil. Larger specific surface areas are related to the particle size (one gram of soil will contain more particles with smaller sizes) and to the available area for reaction. Montmorillonites have a specific surface value between 620 and 770 m\(^2\)/g, which is high compared to 70 to 100 m\(^2\)/g for illite and chlorite and 10 to 30 m\(^2\)/g for kaolinite (Azevedo & Torrado, 2009). The high specific surface area of montmorillonite provides a greater contact area between the grains and water, stimulating soil expansion.

As previously described, other than the particles, cations are also part of the clay micelle. Cations play an important role because they neutralize the charge, reducing the repulsive forces between soil particles. Lambe (1958) explains that on the surfaces of dry clay particles, cations neutralize the negative charges. When in water the cations move around the clay particle, attracted by the negative forces on the clay surface forming the electrical double layer. More cations are concentrated closer to the clay particle, reducing the electrical concentration with distance, which causes a gradient of cation concentration. Therefore, the clay particle and the electric double layer form the clay micelle (Lambe, 1958). Cations are also responsible for the connection of the clay units, as previously described, and for clay expansion. These ions and their effect on the soil swell will be discussed in section 2.2.2.

Cation exchange capacity (CEC) is the maximum capacity of the soil to absorb cations (Camapum de Carvalho et al., 2015). Exchangeable cations can be used to transport minerals for plant growth and are important for soil chemical stabilization (Bell, 1996; Grim, 1953). The three causes of cation exchange are broken bonds around atoms sheets, isomorphous substitution, and replacement of some hydroxyls for other cations (Grim, 1953). According to Grim (1953), exchangeable
cations affect the physical properties of the clay such as plasticity, compaction, and shrinkage properties. For example, clays with Na\(^+\) as exchangeable cations are more plastic than ones with Ca\(^{2+}\).

Table 2-1: Range of cation exchange capacity for each mineral group (Grim, 1953)

<table>
<thead>
<tr>
<th>Clay mineral</th>
<th>CEC (meq/100g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kaolinite</td>
<td>3-15</td>
</tr>
<tr>
<td>Montmorillonite</td>
<td>80-150</td>
</tr>
<tr>
<td>Illite</td>
<td>10-40</td>
</tr>
<tr>
<td>Chlorite</td>
<td>10-40</td>
</tr>
</tbody>
</table>

2.2.2 The process of expansion

Soil expansion is related to the energy variation within the soil and its mineral composition. Expansion due to energy variation can occur in all clay minerals and is related to decrease in applied stress (external energy) and increase in soil water content (internal energy) (Camapum de Carvalho, 2007). The expansion caused by water content increase is described as inter-particle swelling, where the expansion happens when water occupies the space between particles, caused by the decrease in capillary forces (Grim, 1962).

The clay minerals that have expansible behavior are from the smectite, illite, vermiculite, and chlorite groups (Camapum de Carvalho et al., 2015). Greater amounts of these minerals lead to higher swell-shrink potentials in soils. The clay mineralogy is responsible for two types of swell: the intra-crystalline swell, caused by hydration of the exchangeable cations, and the osmotic swell,
caused by the difference in ion concentration between the clay surface and the pore water (Madsen & Müller-Vonmoss, 1989).

Osmotic expansion occurs due to a difference in cation concentration between adsorbed water and pore water in the soil. These positively charged ions repel each other causing electric repulsion. Therefore, osmotic swell is a result of the repulsion between electric double layers (Madsen & Müller-Vonmoss, 1989). As described by Lambe (1958), dry clays do not have a diffuse double layer; so when water is added to the soil, a diffuse double layer will develop for each particle. These layers will overlap and cause the electric repulsion, creating distance which is dependent on the double layer size and concentration of electrolytes in the pore water (Madsen & Müller-Vonmoss, 1989). Osmotic expansion stops when equilibrium is reached and varies when soil internal energy changes (e.g. after an excavation).

Intra-crystalline swell is caused by hydration of the exchangeable cations which leads to an increase in clay interlayer space (up to 1 nm) and occurs when clay is initially dry (Grim, 1962; Laespiga, 2001; Madsen & Müller-Vonmoss, 1989). The water molecule is electrically neutral, but its charges are unevenly distributed. Cations in between clay layers attract the polar water molecule and hydrate the cation, increasing the clay particle d-spacing (Moore & Reynolds, 1989). The cation hydration process can be explained using a simplified model of a hydrated ion (Frank & Wen, 1957; Nelson et al., 2015). This model has three regions, the first is the region of immobilization, where water is strongly adhered to the ion and is therefore immobile. In the second region, water molecules are adhered to the ion but can be partially removed by heat. Finally, in the third region, the water is held to the ion by a weak ionic field and is absorbed to the clay particle where it can be removed by air drying (Nelson et al., 2015). When the cation is hydrated the diameter of the ion increases considerably. Lambe and Whitman (1969) indicated that the radius
of the sodium ion varies from 0.98 Å when dehydrated to 7.8 Å after hydration. Since the cations are located between clay particles, when there is an increase in the water content of the soil the cations will be hydrated, increase their volume, and consequently cause soil expansion (Nelson et al., 2015), as shown in Figure 2-2. According to Madsen & Müller-Vonmoss (1989), intra-crystalline swell in montmorillonites can cause a swell pressure of about 100 MPa.

![Diagram](image)

(a) d=10Å  (b) d=20Å

(c) d-spacing interlayer space

Figure 2-2: Intra-crystalline swell of expansible clay minerals: (a) dry particle; (b) expanded particle; (c) difference between d-spacing and interlayer space

### 2.2.2.1 Interlayer water and d-spacing

As explained above, one of the reasons for soil swelling is intra-crystalline expansion, which is caused by the hydration of the interlayer cations, resulting in an increase of the interlayer space (Grim, 1962). This interlayer space contains only water, differently than the inter-particle space that may contain liquid and vapor water, and air (Sedighi & Thomas, 2014). The amount of absorbed water depends on the particle specific surface area and chemical interactions between the clay surface and porewater (Romero et al., 2011; Tuller & Or, 2005). The d-spacing increase is a
function of the size, location, and charge of the interlayer cation, temperature, applied stress, and water salinity (Moore & Reynolds, 1989; Villar et al., 2012).

Moore and Hower (1986) measured the d-spacing of a sodium bentonite from Wyoming (USA), in which the value varied from 9.6 Å at 0% relative humidity to 12.4 Å at 60-65% relative humidity. According to the authors, the distance can go up to 18 Å when the relative humidity is 100%, concluding that the interlayer distance increases continuously, but not linearly. In contrast, Villar et al. (2012) observed a linear relationship of the interlayer space increase with an increase in the water content of the soil for two distinct bentonites. These authors tested a calcium bentonite from Spain (FEBEX) and a sodium bentonite (MX-80) from Wyoming - both materials are used for the construction of deep high-level nuclear waste (HLW) repositories. Figure 2-3 exhibits a reconstruction of the data obtained by Villar et al. (2012) showing the linear relationship between the variation in basal spacing, or d-spacing, with water content. The relationship for the sodium bentonite seems steeper than that for the calcium bentonite, which can be related to the observations made by Lambe & Whitman (1969), where it was described that the sodium ion was able to considerably increase its radius from 0.98 Å to 7.8 Å after hydration.
Figure 2-3: Relationship between basal spacing and water content of two bentonites using data points collected from Villar et al. (2012)

Another significant finding of Villar et al. (2012) is that the water in the clay soil can be redistributed with time. Samples with the same water content were subjected to XRD tests for the measurement of basal spacing at different times. Even though the soil water content did not change with longer periods of time, the basal space increased, which means that the water was redistributed in the soil and moved from the inter-particle voids to the interlayer spaces in the clay. Therefore, the d-spacing was found to be time dependent.

2.2.3 Factors affecting expansibility

The factors that affect the swell potential of the soil are the soil mineralogy, pore size distribution, particle orientation, soil water content, soil dry density, confining pressure applied to the soil, and the duration that the soil is in contact with water.

Soil mineralogy is the main factor that contributes to soil swell potential. Each clay mineral expands differently, depending on structural arrangement, particle shape, specific surface area, and CEC. Expansible clay minerals are able to expand and retract their interlayer space without
changing their crystal structure. Moore & Reynolds (1989) attributes the swelling property of the smectites to the small charge between mineral layers that are not strong enough to hold the layers together, causing the interlayer cations to be more attracted to water. When water is available in the soil, the exchangeable cations attracts water into the interlayer space, expanding the clay mineral. Therefore, the expansibility depends on not only the clay mineral but the exchangeable cation charge and type (Moore & Reynolds, 1989). For example, montmorillonites (clay minerals from the smectite group) present different swell potential depending on the exchangeable cations present between clay unit layers. When calcium is the exchangeable cation, the soil can swell from 50-100%; however, when sodium is the exchangeable cation the soil can swell up to 2,000% (Bell & Culshaw, 2001). Nelson et al (2015) explain this difference in swell potential of different montmorillonites by the fact that when multivalent cations are present in the micelle, less cations are needed to neutralize the negative charges, meaning the micelle volume is smaller than if monovalent cations were adsorbed.

Even though soil mineralogy is the most important factor, the pore’s distribution in the soil may cause significant effect on the soil swell potential. Pedarla et al. (2013) showed that a soil composed of 60% expansible minerals but with a well distributed pores’ gradation presented more swell than another soil composed of 90% expansible minerals and poor pore size distribution. A well pore size distribution allows better access of the water to the soil (Pedarla et al., 2013).

Another factor that affects the soil swell potential is how the particles are oriented in the soil mass. Nelson et al. (2015) explain that the particle orientation depends on their process of deposition and that the orientation can range between flocculated and dispersed (Figure 2-4). In the flocculated form, particles are randomly oriented, increasing voids between particles, so mineral expansion takes place in each particle but does not cause significant increase of the soil mass volume. This
condition is shown in Figure 2-4 (a). In the dispersed structure, the particles are more consistently oriented, so less space exists between particles as shown in Figure 2-4 (b) (Nelson et al., 2015). It is important to note that compaction leads to a more oriented particle structure, and that a sample compacted with a water content at dry of optimum has a flocculated structure and wet of optimum has a dispersed structure (Lambe, 1958).

![Particle orientation: (a) flocculated; (b) dispersed](image)

Besides the variation in the water content during soil compaction (dry or wet of optimum), the dry density obtained after compaction also affects the soil swell potential. Nelson et al. (2015) discuss the results obtained by Chen (1973) which analyses the effect of the initial water content and dry density on the volume change obtained after oedometer tests. Greater dry densities led to an increased volume change (Chen, 1973) because there is less space between particles. As illustrated in Figure 2-1, a more oriented structure implies fewer void spaces between particles, so the crystalline and osmotic swell are more effective, leading to greater swell potential. The swelling pressure was found to increase exponentially with dry density increase (Villar & Lloret, 2008). On the other hand, the volume change potential decreases with an increase of initial water content before inundating the sample. Generally more swelling is expected for a soil with a higher dry density and lower initial water content (Lambe, 1960; Nelson et al., 2015). Villar & Lloret (2008) show that samples saturated under the same vertical pressure and compacted at low dry densities are less significantly affected by the water content. Between confining pressure, initial water
content and dry density, the confining pressure is the variable that affects the soil swell potential the most (Villar & Lloret, 2008). Greater confining pressure leads to less swell; soils with no or very low confining pressure exhibit a significant decrease in the soil swell based on a slight increase in the confining pressure (Lambe, 1960).

Another important parameter is the time given for the soil to swell. Sufficient time should be allowed to elapse for water to reach all particles in the soil for swelling to occur. Due to clay’s low permeability, the time required to hydrate all particles is longer than that for coarser soils, for instance.

In conclusion, the mineral composition, and consequently the chemical characteristics of the soil, affect expansibility since high specific surface area allows more contact between the clay particles and water, which causes change in the double layer thickness with consequent volume increase (Lambe, 1960). Second, the swell potential depends on the initial water content and degree of saturation. In addition, soil volume change is influenced by soil density, confining pressure, and time allowed for swell. Therefore, if a soil is composed of expansible minerals and has low initial water content, high density, and low confining pressure when subjected to a high water content variation over time, high swell is expected.

2.2.3 Identification of expansible soils and swell potential estimation

Expansible soils can be identified by performing tests to determine soil physical properties, such as plasticity, free swell, potential volume change, and expansion index. Expansible soils can also be determined based on clay content, minerals, and chemical characteristics (CEC and specific surface area) (Nelson et al., 2015). The soil swell potential can be estimated from the collected samples by performing oedometer, shrinkage, or swell tests.
The Atterberg limits are used to predict the soil swell potential. Plastic limit ($w_p$) is the water content of the soil that separates the semi-solid and plastic state. Liquid limit ($w_l$) is the water content at which the soil changes from plastic to liquid state. If the water content of the soil is greater than its liquid limit, the soil does not present any shear strength (Nelson et al., 2015). Both liquid and plastic limit values are related to soil texture, mineral composition, and chemical characteristics (Camapum de Carvalho et al., 2017). The difference between the liquid limit and plastic limit is called plasticity index (PI).

Peck et al. (1974) present a relationship between swell potential and plasticity index of the soil (Table 2-2). Skempton (1953) suggests that expansible soils can be identified by relating plasticity index with soil texture. The author defined the term activity ($A_c$) as the ratio of the plasticity index and the percentage of soil by weight that is finer than 2 μm (Equation 2-1). If the value found in Equation 2-1 is less than 0.75 the clay is considered to be inactive. If the value of $A_c$ is greater than 1.25 the soil is active and has potential to expand. Activity values between 0.75 and 1.25 suggest that the soil has normal activity. Holtz et al. (2011) relates the values of plasticity index with the soil mineral composition. The authors identified zones in the plasticity chart where montmorillonite, illite, and kaolinite rich soils typically fall. Therefore, through this modified plasticity chart it is possible to identify if the soil has potential to swell based on mineral composition.
Free swell tests measure the volume change of soil subjected to swell at a low or no confining pressure. The volume change is the ratio of the change in volume and the initial soil volume. High swell soils like sodium bentonite present a swell above 1200% (Nelson et al., 2015). However, other authors reported that even soils with swell less than 100% have caused considerable damage under light loaded structures or at extreme climatic conditions (Dawson, 1953; Holtz & Gibbs, 1956).

The swell pressure of soil is tested by maintaining the soil at a constant volume with a load cell on top of the sample, usually in an oedometer cell (Lambe, 1960). Water is then added to the sample which pushes the load cell as the soil tries to swell but cannot since the volume is kept constant. Therefore, swell pressure is the stress required to prevent the soil to expand when water penetrates the soil. This is important information for soil beneath lightly loaded structures, such as pavement (Soundara & Robinson, 2009), because the structure can be designed to apply the swell pressure on the soil and prevent soil expansion.
The method used to determine the soil swell potential must be carefully chosen to account for the in-situ condition and obtain a proper design for the structures constructed on expansible soils. The consolidation, free swell, and swell pressure tests do not reproduce the in-situ stress condition (Lambe, 1960), but provides valuable information for the design of the structures. Plasticity tests are simple, accessible, and their results can be considered for initial investigation of soil expansibility. Historical damages give valuable information for preliminary studies. Chemical and mineralogical tests are the best indicators of whether or not the soil is expansible, but these tests are expensive and not always available.

2.3 Soil suction

Soil swell is related to the variation of the water content, which also affects soil suction. Many authors explained the behavior of expansive soil by studying the performance and mechanism under unsaturated condition by determining the soil water content – suction relationship (Al-Yaqoub et al., 2017; Campos et al., 2008; Marinho, 2005; Tripathy et al., 2014; Villar, 2007; Villar et al., 2012). A review of these important finding is presented below.

2.3.1 Suction theory

Traditional soil mechanics developed by Terzaghi in 1943 deals with soil in its saturated state. The soil is saturated when the degree of saturation (Sr) is 100% (Equation 2-2). Hence, a saturated soil (Sr = 100%) means that all voids in the soil are filled with water. However, soils are not always saturated, especially at the surface where they interact with vegetation and are exposed to environmental conditions such as seasonal climate change. As a result, degree of saturation of such soil is less than 100% and soil become unsaturated. Unsaturated soils have different engineering behavior than saturated one due to the air-water interface in the soil voids (Fredlund & Rahardjo,
1993), which causes negative pore water pressure in the soil (Lee, 2013). This negative pore water pressure is named soil suction.

\[ S_r = \frac{V_w}{V_v} \]  

were \( V_w \) is the volume of water and \( V_v \) is the volume of voids in the soil.

Marinho (1994) describes that soil suction is composed of three components: total suction (\( \psi \)), matric suction (\( u_{a} - u_{w} \)), and osmotic suction. Total suction is the sum of osmotic and matric suctions. Total suction is the free energy of the water in the soil and can be determined by measuring the relative humidity of the air around the soil. Matric suction is the difference between the air pressure (\( u_{a} \)) and water pressure (\( u_{w} \)) and it is usually the largest component of the total suction. Matric suction is a result of capillary rise caused by surface tension at the soil-water-air interface, and adsorption forces, particularly for clays since these forces are caused by electrical and van der Waals electric fields (Marinho, 1994; Nam et al., 2009). The osmotic suction occurs due to salt concentration in water which changes the water vapor pressure. Osmotic suction is related to the osmotic concept where the pure solvent flows to the solution to equilibrate the unbalance of ions. Higher ion concentration can be found in the top layers of soil since when the water table rises it carries ions. When the top soil layer dries the ions remain, creating an ion concentration gradient (Marinho, 1994). The total suction is calculated based on the partial pressure of the pore water vapor as expressed in Equation 2-3 (Fredlund & Rahardjo, 1993).

\[ \psi = -\frac{RT}{v_0\omega_v}\ln RH \]  

where \( R \) is the universal gas constant (J/(mol.k)), \( T \) is the absolute temperature (K), \( v_0\omega_v \) is the specific water volume (m\(^3\)/kg), \( \omega_v \) is the molecular mass of water vapor (g/mol), and \( RH \) is the relative humidity.
Interest in soil suction began in the field of soil science when scientists were studying the soil-water-plant systems (Fredlund & Rahardjo, 1993). Later, the unsaturated soil concept was used in geotechnical engineering. The hydraulic and mechanical properties of unsaturated soil are determined with the SWCC (Marinho, 2005). The SWCC relates the suction in the soil to the amount of water present in it. Since the suction in the soil depends on the water content of the soil and one of the causes of the volume change of expansible soil is moisture variation, the swell-shrink behavior of the expansive soil depends on the change in suction (Houston et al., 2011).

### 2.3.2 Soil-water characteristic curve

The soil-water characteristic curve (SWCC) represents the relationship between the amount of water in the soil and the negative pressure (suction) needed to remove this water from the soil (Camapum de Carvalho & Leroueil, 2004). The suction can be presented in terms of gravimetric water content, volumetric water content, or degree of saturation. Suction can be represented in terms of Pascals or pF, where pF is the logarithm of suction in the height of a column of water in centimeters. In other words, pF is the pressure, in kPa, divided by the unit weight of water (kN/m³) times 1000.

SWCC can be used to obtain key parameters such as air entry value (AEV) and residual degree of saturation (S\textsubscript{residual}) (Figure 2-5). These terms are best inferred from the curve relating to suction and degree of saturation. The AEV represents the pressure at which the pores in soil are no longer saturated, and the soil transition to unsaturated state. It is found by the prolongation of two lines where there is a change of the inclination from saturated to unsaturated condition (Figure 2-5). The residual degree of saturation is the minimum amount of water that soil can hold where an increase in suction does not cause a significant change in the degree of saturation. The SWCC can be divided in three zones which are separated by the AEV and residual water content (Nam et al.,
From zero suction to the air entry value is the capillary zone. The region of the curve below the residual water content is the zone of residual saturation. The region in between is the desaturation zone.

![Soil-water characteristic curve](image)

**Figure 2-5: Soil-water characteristic curve**

### 2.3.3 Factors affecting soil-water characteristic curve

The shape of the SWCC is affected by the soil pore distribution, initial water content, sample density, stress history, and mineralogy (Camapum de Carvalho & Leroueil, 2004; Marinho, 2005; Romero et al., 2011). According to Tripathy et al. (2014), compaction effort does not affect the suction curve. However, results obtained by Nam et al. (2009) indicated distinct suction values for disturbed and undisturbed samples, suggesting that the structure of the soil sample affects the SWCC. Nam et al. (2009) measured the matric suction of disturbed and undisturbed silt and clay samples. Disturbed soil samples presented a SWCC with linearly increasing suction with water content decrease in logarithmic scale. Undisturbed samples, on the other hand, presented bilinear or steeper curve with lower air entry values, which indicated the existence of two defined pore sizes. Bigger pores in the disturbed samples led to higher water content at lower suction values.
and a low AEV (Nam et al., 2009). In other words, the disturbed samples had their original pore
distribution disturbed, affecting the soil AEV and water content.

The pore size distribution determines the shape of the curve and its air entry value (Camapum de
Carvalho & Leroueil, 2004; Marinho, 2005). Well-distributed pores have a range of sizes, from
bigger to smaller sizes in a smooth transaction. Poorly distributed pores have a gap in the pore
sizes or not a smooth transition between them. In a uniform pore distribution, the pores have the
same diameter. Pores can also be distinguished between macropores and micropores. Macropores
are the voids between soil aggregates, while micropores are the pores in the interior structures of
the aggregates, created by clay bonds and cementing agents.

Marinho (2005) described the effect of pore distribution on the shape of the curve. Theoretically,
for a soil with all pores at the same size, the soil should lose all water in the pores at the AEV, as
shown in curve (1) of Figure 2-6. However, due to the water surface phenomena, a small amount
of water would still be inside the pores and be removed with suction increase, as represented in
curve 2. According to Camapum de Carvalho & Leroueil (2004), highly weathered soils (usually
some tropical soils) have two defined pore sizes, both macropores and micropores, which causes
poor pore distribution and the existence of two AEV, forming a SWCC with bimodal shape, shown
in curve 3. The first AEV is related to the air entering macropores and the second to the micropores.
The slope and length of the curve after each AEV depend on the sizes of the micropores and
macropores, the pore distribution, soil shrinkage, and soil mineral composition (Camapum de
Carvalho & Leroueil, 2004). If the soil has well-distributed pore sizes, the water content reduces
gradually, as shown in curve 4 (Camapum de Carvalho & Leroueil, 2004; Marinho, 2005).
For fine grained soils, the initial water content affects the soil structure and aggregation, and consequently the SWCC (Vanapalli et al., 1999). Camapum de Carvalho & Leroueil (2004) discuss that adding water to fine soils forms lumps with diameters of 3 to 4 mm. The lumps have micropores and the macropores are formed between lumps. When the water content is below the optimum water content ($w_{opt}$) the suction is relatively high which causes some stability maintaining the macropores. Therefore, when the soil is compacted with a moisture content on the dry side of compaction curve, this causes a poor pore distribution, which leads to a SWCC with a bimodal shape. In contrast, soil compacted at water contents above the $w_{opt}$ tends to have pore sizes that are well distributed and have only one AEV. Higher water content decreases soil suction, leading to more deformable soil lumps (Camapum de Carvalho & Leroueil, 2004). Vanapalli et al. (1999) also discuss the differences in SWCC when soil is compacted at dry and wet side of optimum. According to the authors, soils compacted at dry side of optimum have a steeper SWCC than when compacted at optimum or higher. Overall, soils compacted dry of optimum behave like coarse
grained soils and have their macrostructure (i.e. applied stress and stress history) dominating soil behavior (Vanapalli et al., 1999).

Even though the compaction effort does not modify the SWCC (Tripathy et al., 2014), the dry density affects the SWCC of fine soil. Marcial (2003), Villar (2002), Villar & Lloret (2004), and Villar (2007) observed that bentonites compacted at different dry densities presented similar SWCCs for suctions above 10 MPa. Villar (2007) explained that for higher suctions the water is mostly in the microstructure, which is not affected by density change, since the compaction affects soil macrostructure. However, when the suction is less than 10 MPa, at same suction level, less dense soil presented higher water content than denser samples because less dense soil present bigger pore volumes, so the water occupies this space (Villar, 2007).

2.3.4 Methods to determine the soil-water characteristic curve

There are several methods to determine soil suction and the SWCC. The election of a method depends on whether total or matric suction is desired and on the range of suction values. The most commonly used methods with their principle, suction range, advantages and limitations are summarized in Table 2-3.

Tripathy, Elgabu, et al. (2014) compared suction measurements using null-type axis translation, filter paper test, and chilled mirror dewpoint test methods on clay and sandy soil samples. Pressure plate (axis translation) and vapor equilibrium (salt solution) tests were performed to measure matric and total suction of the soil samples, respectively, and these results were used as a basis to compare the other three methods. Both non-contact filter paper and chilled mirror dewpoint tests obtained similar results for total suction measurements which were in accordance with the vapor equilibrium tests for suctions above 400 kPa. For suctions lower than 400 kPa, the results obtained by filter paper and chilled mirror dewpoint test were higher than the control SWCC. The authors
also measured the matric suction with filter paper and null-type axis translation methods. The results obtained with the filter paper method were similar to the control SWCC. In conclusion, the filter paper test worked well for clay and sandy soils to measure both matric and total suction. Better results were obtained when suction was above 400 kPa (Tripathy, Elgabu, et al., 2014).

Mabirizi & Bulut (2009) compared the total suction of high plastic clays obtained using filter paper, thermocouple psychrometers, and chilled mirror psychrometers test methods. Total suctions were measured at higher ranges and the results obtained from the filter paper test were similar to other methods. The author concluded that the filter paper test is cost effective and covers a wide range of suction (Mabirizi & Bulut, 2009).

Lee (2013) measured the total suction of plastic clays with filter paper test and capacitive polymer-based sensors. The sensors reached equilibrium in 2 to 3 hours and provided good results for soils with low water content, but the results were not reliable when the relative humidity was greater than 98%. In addition, the filter paper method was suggested for soils with high water content because of its reliability (Lee, 2013).

Overall, the filter paper method was compared to other SWCC measurement methods for both matric and total suction of clay and sandy soil. Results indicate a good agreement between these experimental methods. This is a widely accepted method that has low cost associated with the test and it is the only method that can be used to measure both matric and total suction at the same time on a same sample.
Table 2-3: Methods to determine soil-water characteristic curve (ASTM, 2016; Fredlund & Rahardjo, 1993; Marinho, 1994; Marinho & Oliveira, 2006; Nam et al., 2009).

<table>
<thead>
<tr>
<th>Method/Suction</th>
<th>Equipment</th>
<th>Principle</th>
<th>Suction range (kPa)</th>
<th>Pros</th>
<th>Cons</th>
</tr>
</thead>
<tbody>
<tr>
<td>Psychrometer (total or osmotic suction)</td>
<td>Psychrometer</td>
<td>Measures the temperature difference between the wet and dry bulb and relate them to relative humidity.</td>
<td>Can measure two suctions, total or osmotic, depending on what the psychrometer is exposed to.</td>
<td>High cost; Long equilibration time; Sensitive to temperature changes; Challenging reading at low suction level</td>
<td></td>
</tr>
<tr>
<td>Filter paper (total and/or matric, and consequently osmotic)</td>
<td>Ash free filter paper</td>
<td>Filter paper absorbs water or vapor until reaching equilibrium at a specific soil suction level. The water content of the filter paper is later related to the suction level using calibrated curve.</td>
<td>0 - 29,000</td>
<td>Able to measure all suction components; Inexpensive; Used in both lab and site; Simple testing set up.</td>
<td>Demands accurate weight measurements; Equilibrium reached after at least 7 days</td>
</tr>
<tr>
<td>Pressure plate (matric)</td>
<td>High air entry ceramic discs and pressure chamber</td>
<td>Based on axis translation technique, the pore water pressure is kept equals to atmospheric pressure meanwhile increasing the air pressure</td>
<td>10-1500</td>
<td>More than one soil sample can be tested at the same time; Can be incorporated to other soil testing, such as triaxial and oedometer.</td>
<td>Suction range is limited by the maximum pressure of ceramic disc; Other methods may be used to determine the full SWCC; May overestimate suction when Sr&gt;80%</td>
</tr>
<tr>
<td>Chilled mirror dewpoint potentiometer (total)</td>
<td>Camber, mirror, condensation detector, and thermometer</td>
<td>Relative humidity and temperature are measured inside the chamber with the soil, so suction can be calculated</td>
<td>&gt; 1000</td>
<td>Accurate measurement of temperature and dewpoint</td>
<td>Do not present good results for suction bellow 1000kPa</td>
</tr>
<tr>
<td>Vapor equilibrium (total)</td>
<td>Closed environment and salt solution</td>
<td>The soil sample and the air reach equilibrium due to osmotic potential caused by the salt solution.</td>
<td>0-10,000</td>
<td>Wide range of suction; Precise reading</td>
<td>Depends on salt type and purity of water and chemical; Temperature control; Long equilibration time</td>
</tr>
</tbody>
</table>
2.3.5 Filter paper test

The filter paper test uses filter papers to determine both matric and total suction. Matric suction is measured when the filter paper is in contact with the soil. Total suction is measured having the filter paper close proximity but not touching the soil. Nam et al. (2009) discuss that for high suction values there is not enough contact between the filter paper and soil, so it is not possible to measure the matric suction properly, and therefore total suction is measure instead.

The principle of the test is that the water content in the filter paper, transferred from the soil by water vapor equilibrium (total suction measurement) or capillary flow (matric suction measurement), is used to estimate the soil suction.

It is recommended to use of ash free filter paper for the test. Whatman No. 42 and Schleicher & Schuell No. 589 papers are often used and have their calibration curves provided in ASTM D5248 (ASTM, 2016). The filter paper calibration curve is the tool used to correlate the filter paper water content with the suction potential of the environment. The filter paper calibration curve is unique and independent of the type of suction measured (Marinho & Oliveira, 2006). There are some widely accepted calibration curves, such as the one presented in the ASTM (ASTM, 2016) and Chandler et al. (1992). However, some batches may present different results (Marinho & Oliveira, 2006). Marinho & Oliveira (2006) suggested evaluating whether the filter paper calibration agrees with the Chandler et al. (1992) calibration curve by determining one point on the curve using sodium chloride solution in desiccator (process similar to the vapor equilibrium method).

The filter paper test consists of preparing the soil at different water contents and allowing it to equilibrate with the filter paper for a certain amount of time. The equilibration time depends on the type of suction measured, soil type, and water content. In matric suction measurement the filter paper is in direct contact with the soil sample, so the water transfer occurs faster than through
vapor equilibrium. It is suggested to allow an equilibration time of at least 7 days. It takes longer to equilibrate the filter paper and the soil moisture content in fine soils, ranging between 2 to 3 weeks, due to their low permeability (Lee, 2013).

There are several advantages of the filter paper method. It is the only method that measures both matric and total suction in one test. In addition, it is an inexpensive test, widely available, and has a procedure outlined in the ASTM D5248 (ASTM, 2016). On the other hand, the test requires a well-defined methodology since it is prone to operator error, especially during weighting process. Another disadvantage is the number of samples needed to provide an acceptable curve. At least 10 points (samples) are needed to provide enough information for a reliable SWCC. If the test is conducted properly, the results are highly reliable (Lee, 2013).

2.3.6 Soil-water characteristic curve of clays and shrinkage effect

The pores size distribution of clays usually has variable sizes of pores, so clays would have a SWCC like the curve 4 in Figure 2-6. The shrinkage behavior of some clays, particularly the ones with a liquid limit above 25%, can also affect the shape of the SWCC (Marinho, 2005). Suction increase leads to loss of water and soil shrinkage due to pore size reduction. Water removal and pore shrinkage occur at the same time. Consequently, the pores stay saturated until reaching higher suctions, which is the reason for the higher air entry value (AEV) of clays (Marinho, 2005). The AEV of clays may not be presented as well defined in the SWCC as the one for non-cohesive soils; in this case, the AEV should be determined in conjunction with the shrinkage curve (Fredlund & Rahardjo, 1993; Tripathy, Tadza, et al., 2014).

Camapum de Carvalho et al. (2017) show that for plastic soils the AEV is directly related to soil plasticity. Higher AEV are expected for soils with higher plasticity. They also show that at same AEV there is a reduction of soil void ratio when soil plasticity increases.
Marinho (2005) discusses the relationship between liquid limit, suction, and stress history in plastic soils. The author also suggests that soils with higher liquid limits give steeper curves in the semi-log scale, and the SWCCs of fine grained soils are different depending on the soil’s position on the plasticity chart. In addition, higher suctions can be found below the plastic limit (Lee, 2013). Camapum de Carvalho et al. (2017) also discuss the relationship between liquid limit and the air entry value considering the normalization of the AEV with void ratio.

2.3.7 Transformed soil-water characteristic curve and void ratio

Camapum de Carvalho & Leroueil (2004) proposed a model to present the soil-water characteristic curve relating void ratio to suction. In this transformed model, the curve is presented in terms of degree of saturation (Sr) versus multiplication of void ratio (e) and logarithm of soil suction per centimeter of water column (pF), in the form of e.pF. This transformed curve is unique for soils with the same pore distribution and it is independent of the initial void ratio. It is important to note that a same soil has different pore distributions at different dry density. Different compaction conditions affect the pore distribution, which means that the same soil can have different SWCCs (Watable et al., 2000). Therefore, if the pore distribution is kept constant, there is no need to find the SWCC for each void ratio, as demonstrated by the authors, reducing the number of tests required to fully understand the soil behavior. The authors applied this model to different types of soils even after chemical stabilization.

As mentioned earlier, plastic soils are subject to variations in void ratio due to soil shrinkage. Therefore, since the transformed SWCC proposed by Camapum de Carvalho & Leroueil (2004) can be applied to different void ratios as long as the pore distribution is the same, the transformed model allows the variation in void ratio that occurs for high plastic and expansible soils with moisture content variation. Campos et al. (2008) studied two Brazilian montmorillonite-rich
bentonites and used the transformed SWCC to represent their results. However, the results obtained were opposite of expected: the degree of saturation was increasing with increase in suction (in terms of e.pF). The reason is that the model proposed by Camapum de Carvalho & Leroueil (2004) considers the void ratio as the voids between soil particles (micropores and macropores), and not the voids created between clay layers with soil expansion. In other words, the particle volume is considered to be constant. However, when only the voids between particles were used to calculate the SWCC, the curve looked as expected (Campos et al., 2008). Therefore, a method to determine the correct void was determined.

2.4 Methods to correct the soil void ratio of expansible soil

The effect of the intracrystalline swell on the void ratio of expansible soil was studied by authors who were studying bentonites (Camapum de Carvalho et al., 2017; Campos et al., 2008; De la Morena et al., 2018; Sedighi & Thomas, 2014).

Sedighi & Thomas (2014) developed a method with a chemical based approach using thermodynamic formulations. Their calculation method is based on the equilibrium equation hydrated smectite $\leftrightarrow$ dry smectite + nH$_2$O, where the equilibrium constant is calculated. De la Morena et al. (2018) method calculates the corrected soil void ratio based on the exchangeable cations in the bentonites. These methods are suitable for a soil with uniform mineralogy but complex for a soil composed of different minerals.

Campos et al. (2008) suggested the concept of global void ratio ($e_{\text{global}}$), which is composed of the voids between particles ($e_{\text{corrected}}$) and the voids between clay layers. This concept was also discussed in Camapum de Carvalho et al. (2017), where the authors suggest that the Atterberg limits for expansible soils should take into account only the water that is in the voids between particles ($e_{\text{corrected}}$). The authors explain that the water that is in the interlayer void space does not
contribute to the mechanical behavior of the soil, so this water should not be considered in the liquid limit (related to the soil strength) or plastic limit (related to soil tensile strength).

Hence, no study has been conducted on other expansible soil beside bentonites. Overall, the available methods are complex and require extensive testing. The methods proposed by Sedighi & Tomaz (2014) and De la Morena et al. (2018) do not use the d-spacing concept for calculating the corrected soil index properties. The method proposed by Campos et al. (2008) used hypothetical d-spacing values to perform the calculations. Therefore, a method to correct the soil index properties due to intra-crystalline swell that can be applied to any expansible soil and uses the d-spacing measurements is needed.

2.5 Summary of literature review

Expansible soils are challenging for the design of structures due to significant volume variation with change in moisture condition. Understand the expansion process is a key parameter for cost-effective designs. Many authors have discussed how the water is distributed, stored, and retained inside the particles of expansive soils. The water, rather than simply being stored in the voids between grains, is also stored in the space between clay units of each particle. However, current soil index properties calculations for expansive soils account for water in the soil as if it is only stored in the inter-particle voids, as for non-expansive soil. The water that moves into the clay interlayer space is different from the remainder of the water that is left in the void spaces. This interlayer water does not contribute to the mechanical and hydraulic behavior of the soil, which suggests that the effects of this phenomena may have a significant impact on the soil void ratio, water content, and degree of saturation. This research proposes to make alterations to the existing calculations that accounts for this. This analysis is done by deriving and creating a new set of calculations and modifications to existing calculations of particle volume, water content, void
ratio, and degree of saturation. The new parameters shall be evaluated based on the unsaturated soil mechanics theory since in most designs the soil is not saturated.
Chapter 3: Methodology

3.1 Overview

This chapter refers to the materials and methods used in the experiments performed on the Glenmore clay, material collected in the Okanagan region which has a medium swell potential. The experimental approach to analyze the local expansible soil is described. In addition, the methods used to evaluate the hydration effect of particle size are explained as well. The sample preparation for the microstructure and image analysis is described. Finally, the method to measure soil suction is detailed. For all tests, the soil sample was air dried for more than five days, grinded, and had its hygroscopic water content measured. All sample passed through sieve #10 (2 mm); therefore, the Glenmore clay is a fine grained soil.

3.2 Determination of the soil particle size distribution

The soil particle size distribution (PSD) was determined performing the sieve analysis followed by hydrometer test and laser diffraction test.

3.2.1 Sieve analysis and hydrometer test

The PSD was defined based on ASTM D7928-17 (ASTM, 2017). About 50 g of the material passing sieve #10 was soaked in a sodium hexametaphosphate (40 g/L) solution for 16 hours. Then the slurry was transferred to the dispersion cup and blended in the stirring apparatus for 1 minute. The slurry was then transferred to the sedimentation cylinder, water was added to the cylinder until the 1000 mL mark, and the cylinder was repeatedly turned upside down and back for one minute. Hydrometer readings were taken after 0.5, 1, 2, 4, 8, 15, 30, 60, 120, 240, 480, and 1440 minutes. After the last reading the slurry was dumped on sieve #200 (0.074 mm) to retain the material larger
than 0.074 mm (fine sand), the retained material was then oven dried for 24 hours and sieved. The combination of the sieving and sedimentation results was used to determine the soil PSD.

The effect of hydration time was analysed following a similar procedure to the one described above. Two other samples were prepared from a separate soil sample that was sieved on sieve #200 (0.074mm), since only the clay fraction is affected by hydration. One sample was prepared with reverse osmosis (RO) water and the other with sodium hexametaphosphate solution. The sample with RO water was used to compare the effect of the dispersive salt as a dispersive agent. Both samples were stirred prior to being placed in glass cylinders. Readings were taken at 0.5, 1, 2, 4, 8, 15, and 30 minutes. After the last reading the material was mixed again in the cylinder and a new set of readings was taken. In the first reading the soil hydrated for 0 hours and the test ran for 4 hours. In this way, the hydration time effect could be evaluated.

3.2.2 Laser diffraction test

The laser diffraction test measures the particles size based on the interaction between the light and soil particles. There is an inverse relationship between the angle of the diffracted light and the particle size, where smaller particles cause higher angles (Loizeau et al., 1994). The light diffraction theory was developed by Lorentz-Mie and is further explained in Loizeau et al. (1994). The laser diffraction test was performed using Malvern Mastersizer 3000 to evaluate the hydration time, the use of sodium hexametaphosphate solution (40g/l), and the ultrasound effect. The hydration time studies were performed after 0 and 16 hours of hydration. The hydration time of 16 hours was chosen since ASTM D7928-17 (ASTM, 2017) suggests that for the hydrometer test a minimum soaking time of 16 hours in sodium hexametaphosphate solution (40g/l) should be used prior to testing. In total 4 samples were prepared: 0 hours immersion in water, 0 hours immersion in sodium hexametaphosphate solution, 16 hours immersion in water, and 16 hours immersion in
sodium hexametaphosphate solution. All samples were tested with and without ultrasound with the Mastersizer 3000 machine. A dense solution of soil and solvent (water or sodium hexametaphosphate solution) was made by mixing 1.5 g of oven dried soil passing sieve #200 and 15 ml of solvent. For every combination tested, 10 readings were taken, with the results presented at the end being the average of 10 readings.

3.3 Specific gravity

The soil specific gravity was determined according to ASTM D854 (ASTM, 2014). The soil was air dried and sieved on sieve #10. The water content of the soil was tested apart. De-aired RO water to fill the pycnometer was left inside a cooler for more than 12 hours to guarantee temperature equilibrium. About 50 g soil was placed inside a 500 mL pycnometer and de-aired RO water was added until half of the depth of the main part of the pycnometer was filled. Then, vacuum was applied until there were no more air bubbles, which took about 2 hours. After vacuuming, more de-aired RO water was added until the 500 mL mark was almost reached, and then the sample sat overnight inside a cooler to have clean water at the top of the mixture. Finally, the pycnometer was filled with water until the 500 mL mark. The mass and temperature of the pycnometer with soil and water was measured right after.

3.4 Atterberg limits

3.4.1 Liquid limit

The soil liquid limit was determined based on the multipoint method presented in ASTM D4318 (ASTM, 2017a). The sample was prepared by air drying the soil, grinding it, and sieving on the sieve #40 (0.475 mm). The material passing through sieve #40 was used as the sample. The sample was then mixed with water and allowed to hydrate for at least 16 hours prior to testing. The test
was performed using the Casagrande liquid limit apparatus. At least five points with different water contents were tested (multipoint method). The test was repeated 3 times.

3.4.2 Plastic limit

The sample was prepared in a similar way as the liquid limit test. The hand rolling procedure was used to determine the soil plastic limit. The moist soil was rolled on the ground-glass plate by hand until a thread of 3.2 mm was obtained. When this diameter was reached the sample was squeezed again and re-rolled. This procedure was done until the sample could not be re-rolled or began crumbling for a thread diameter larger than 3.2 mm. At least 5 repetitions were performed to determine the soil plastic limit.

3.4.3 Shrinkage limit

The soil shrinkage curve is determined from slurries, compacted, or undisturbed samples, but a curve determined from slurry sample provides more points which gives a more complete curve (Fredlund et al., 2002). The soil shrinkage limit was determined by preparing samples at water content slightly above the soil liquid limit, forming a soil slurry, which should be allowed to hydrate for at least 16 hours. After 16 hours, the soil water content was determined, and the test set up. For this set up, the slurry was placed on a ring \((d= 40 \text{ mm}, h= 12 \text{ mm})\) with the help of a spatula; in this process there should be no air trapped in the molded sample. After that, the samples inside the rings were air dried for at least 7 days. The mass and volume of the samples were often measured until there was no difference between three consecutive measurements. Finally, the void ratio was calculated and the water content–void ratio relationship determined.

The shrinkage curve was also determined from the specimens prepared for the filter paper test. In this case, the samples were compacted under same condition at optimum water content and maximum dry density. The specimens had their mass and volume determined right after
compaction and at the end of the filter paper test, as it will be described in section 3.5. The water content was measured and the relationship between void ratio and water content determined.

![Figure 3-1: Shrinkage limit determination samples.](image)

### 3.5 Soil-water characteristic curve

All specimens prepared for the determination of the SWCC were casted by static uniaxial compaction. The target water content and dry density were the optimal values which were defined in the compaction curve at standard Proctor energy. This measure was taken to ensure the same initial condition and pore distribution for all specimens. The soil was mixed with water and placed inside two bags at a constant temperature for 48 hours prior to compaction to homogenize the water content in the sample. The specimens were compacted in an aluminium mold divided into three parts to distribute more evenly the stress and do not create areas with high tension. The mold was 50 mm in diameter and 50 mm in height to accommodate all the soil to be compacted, as shown in Figure 3-2. The soil was then statically compacted in three layers at constant speed until the applied stress was equal to the Standard Proctor energy (600 kPa). After compaction, the measurements for the volume and mass were taken for each specimen. The specimens were
accepted if the dry density was plus or minus 5% of the optimum dry density. To make sure the specimens had the same volume of pores, the coefficient of the variation of the void ratio was measured. All specimens obtained a coefficient of variation of 1.5±0.5%.

Figure 3-2: Tools used in the preparation of the specimens for the filter paper test.

The specimens for the filter paper test had both drying and wetting paths to achieve a wide range of water contents and obtain a more complete SWCC. The wetting specimens had drops of water added to the soil surface and could swell. The drying specimens were air dried and shrunk freely. Both wetting and drying processes occurred until the desired water content was reached, which was confirmed by measuring the sample mass. Then, the sample mass and volume were again measured.

After specimens reached the desired water content, the specimens were set up for the filter paper test. The experiment occurred inside a can and was arranged as follows. At the bottom of the can
a plastic film was used to prevent the corrosion of the metal can and prevent the filter paper from sticking to the bottom. Then, three filter papers were placed: the two filter papers at bottom were used to measure the matric suction and the last one to avoid soil contamination of the two filter papers. Then the soil sample was placed on the top of the filter papers. A perforated disk was placed at the top of the sample with two filter papers above it. Two filter papers were used for repeatability. The disk was to provide enough distance between the two filter papers and the soil so that the total suction could be measured. The water absorption of perforated disk was tested, and it did not absorb water. The disk was 45 mm in diameter and 8 mm height. The cans were closed with their metal lids, then electrical tape was wrapped around the contact surface where the can and the lid meet. The can was then wrapped in plastic foil and put inside a zipploc bag. These measures were taken to make sure the sample was as isolated as it could be, and not losing water as evaporation to the environment. The temperature was controlled by placing the specimens inside a cooler which was placed in the laboratory with the temperature equal to 23±2°C.

The soil specimens and filter papers were allowed to equilibrate for a period of three weeks. After this period the filter papers were carefully removed, one by one, with the help of two tweezers, and placed inside cans with lids for the measurement of the filter papers’ water content. The entire process of removing the filter paper from the sample, placing it in the can, capping it, and weighing it should take less than 5 seconds. Therefore, this procedure is best performed with two people. After this measurement, the filter papers were put in the oven at 105°C for 24 hours. The dry weight of the filter papers was measured by removing the hot can with the dried filter papers from the oven, placing them on a metal block for 20 seconds, and then weighting the hot tare mass plus the dry filter papers. Finally, the filter papers were removed from the hot can, and the hot tare mass
without the filter paper was measured. The calculation procedure followed the ASTM D5298 (ASTM, 2016).

### 3.6 Mineralogical analysis

The soil mineral composition and d-spacing were measured performing the XRD test. The tests were performed in the Department of Earth, Ocean and Atmospheric Sciences at the University of British Columbia, Vancouver, BC. The sample sent for testing was oven dried and the fraction of the soil particles finer than 75 µm was studied. At the laboratory in Vancouver, the sample was ground down under ethanol in a vibratory McCrone Micronizing Mill for 7 minutes and reduced to the optimum grain-size range for quantitative X-ray analysis (<10 µm) by grinding. The loose powder was tested for quantitative analysis of the clay minerals present in the clay fraction of the soil. The micronized sample was also smeared on to three glass slides with ethanol to test the clay minerals in oriented structure. One specimen was treated with ethylene glycol for 3 hours at 60°C. The ethylene glycol solvation method (glycolated) was used to analyze the presence of smectite minerals. Ethylene glycol penetrates the clay interlayer space and helps to determine the minerals present. Expansible minerals are affected by this addition (Moore & Reynolds, 1989). The second glass slide was hydrated with distilled water for one week at room temperature (23±2°C). The hydrated sample was weighed before (0.05 g) and after (0.08 g) hydration, having a final water content of 60%. The third glass slide was run without any modification for comparison. Continuous-scan X-ray powder-diffraction data were collected over a range of 3-80° 2θ with Co-Kα radiation on a Bruker D8 Advance Bragg-Brentano diffractometer equipped with an Fe filter foil, 0.6 mm (0.3°) divergence slit, incident- and diffracted-beam soller slits and a Lynx Eye-XE detector. The long fine-focus Co X-ray tube was operated at 35 kV and 40 mA, using a take-off angle of 6°.
The results of the XRD test show the relationship between the intensity of the diffracted X-ray beams (counts) and their angles ($2\theta$). From these relationships the clay minerals present in the soil can be identified. More information on this can be found in Moore & Reynolds (1989). The d-spacing ($d_{001}$) of the clay mineral can also be calculated using Bragg’s Law (Equation 3-1).

$$2d_{001} \sin 2\theta = n\lambda$$

(3-1)

where $2\theta$ is the deflected angle, $n$ is equal to 1, and $\lambda$ is the wavelength of the beam.
Chapter 4: Interlayer water volume calculation and void ratio correction

4.1 Overview

Section 2.2.2 showed that introducing water to an expansible soil causes variation of basal spacing. However, there is no modification in the clay structure, therefore the diameter is not modified. The increase of basal spacing causes an increase in the particle volume. Soil is a three-phase material composed by grains, water, and air. Water and air are in the void space between the grains. The water initially goes to the void space, where the interlayered cations in the clay attracts the water (sections 2.3.2 and 2.3.6) and results in the grain volume increasing. This increase in volume is accommodated by the soil voids (Villar, 2007). This particle expansion causes the reduction of the pore size.

Results obtained by Villar et al. (2012) showed that the water in expansible clays travels from the inter-particle space to the interlayer space as time progresses, but without any variation in the water content. The movement of water between the inter-particle and interlayer spaces decreases the void sizes as explained above. Another consequence is that less water is in the inter-particle space because a part of it is “locked” in the clay particle. Thus, the water inside the particle cannot contribute to the mechanical and hydraulic behavior of the soil because it is restrained between the clay layers, which reduce soil permeability.

4.2 Assumptions

In order to develop the procedure to determine the corrected soil index parameters some assumptions were made. The first assumption is that the density of the water in the interlayer space is the same as the density of water in the inter-particle space. Jacinto et al. (2012) obtained higher densities for the water in the interlayer space than the pore water; however, Navarro et al. (2015)
did not obtain significant differences in their results when the same density for the interlayer and free water was assumed. The second assumption is that all inter-particle pores in the soil are connected, so the water is able to reach all of the expansive particles in the soil.

In addition, it was assumed that all particles have the same number of clay sheets and diameter. Even though soil is a natural material which is affected by different sources of weathering resulting in varying shapes, these assumptions were be made to develop the method. Further analysis can be performed to improve the accuracy of the proposed method. The number of clay sheets per particle was evaluated and will be shown in Table 5-4. Both, number of clay sheets and the particle diameter have influence on the volume of particle, but the analysis showed that the soil index properties (Sr, w, e) did not change. This indicates that the soil porosity remainsthe same but the pore size distribution is affected.

4.3 Procedure

This research proposes the re-calculation of the parameters that are affected by the water restrained in the clay particles: volume of solids, water content, void ratio, and degree of saturation. The step-by-step calculation explanation is presented as follows.

Step 1: Determination of the d-spacing of the clay mineral as a function of the soil moisture content. As presented in section 2.2.2.1 Error! Reference source not found., there is a linear relationship between the d-spacing and the soil water content. This relationship must be determined after the XRD test where the soil should be tested in both its dry and wet conditions.

After the XRD test, the relationship between the water content and the d-spacing can be determined.

Step 2: Volume of one particle. Characterization of the clay particle by the determination of the particle diameter (dp) and the number of stacked clay units in each particle. The particle diameter
can be defined after the laser diffraction test or hydrometer test. The number of clay units, or stack layers (n) can be estimated from literature and depends on the clay mineral composition. The volume of one particle ($V_p$) is determined by multiplying the particle area with particle height. The particle height is equal to the d-spacing ($d_{001}$) multiplied by the number of stack layers (n).

\[ V_p = \frac{\pi d_p^2}{4} \times d_{001} \times n \]  

**Step 3:** Determination of the total volume of swelling solids ($V_{s,swell}$) in the soil. First, the volume of solids ($V_s$) must be determined by the calculation presented in Equation 4-2. In this equation, $m_s$ is the mass of solids in the soil (determined after oven drying the sample or by knowing the wet mass of the casted sample and the water content), $G_s$ is the soil specific gravity, and $\rho_w$ is the water density. However, this formula calculates the total volume of particles in the soil, which considers both swelling and non-swelling particles. Therefore, this value should be multiplied by the percentage of clay sized particles and the percentage of swelling minerals in the clay fraction for the determination of the volume of swelling solids ($V_{s,swell}$). For example, if 80% of the particles in a soil sample is clay sized and the mineral composition of the clay shows that 90% of the soil is composed of swelling minerals, the total volume of solids in the soil must be multiplied by 80% and 90%, or by 0.72.

\[ V_s = \frac{m_s}{G_s\rho_w} \]  

\[ V_{s,swell} = V_s \times \% \ of \ swelling \ minerals \]  

**Step 4:** Number of expansible particles in the soil. The number of expansible particles ($N_p$) in the sample is determined by the ratio of the volume of expansible solids ($V_{s,swell}$) and the volume of a single dry particle (d-spacing of dry particle, $V_{p,dry}$) (Equation 4-4). $V_{p,dry}$ is calculated with equation 4-1 with the $d_{001}$ equals to the d-spacing found for the dry sample.
Step 5: Calculation of the corrected volume of solids. The corrected volume of soil particles \( (V_{s'}) \) after the hydration of swelling minerals can be determined with Equation 4-5. The first part of the equation represents the volume of non-swelling particles (the particles that do not have their volume affected by hydration). The second part is the volume of the swelling particles after hydration.

\[
V'_{s} = (V_{s} - V_{s,swell}) + (V_{p} \times N_{p})
\]

Step 6: Determination of the volume of interlayer water \( (V_{iw}) \). The volume of water in the basal spacing of the expansible minerals is found by subtracting the corrected volume of solids \( (V_{s'}) \) by the volume of solids when the water content is zero \( (V_{s}) \). The mass of interlayer water is found by multiplying the volume of interlayer water by the water density (Equation 4-7).

\[
V_{iw} = V'_{s} - V_{s}
\]
\[
m_{iw} = V_{iw}\rho_{w}
\]

Step 7: Inter-particle void volume calculation. The inter-particle void volume \( (V_{v,p}) \) is found by subtracting the volume of total voids \( (V_{v}) \) in the soil by the volume of interlayer water \( (V_{iw}) \). It is important to remember that the volume of voids is calculated as the difference between the total volume of the sample \( (V) \) and the volume of solids \( (V_{s}) \). However, in the global volume of voids the interlayer water is also considered a void because for non-expansive soil water and air occupy the inter-particle place. The total volume of the soil is composed of solids, air, and water. The volume of solids \( (V_{s}) \) is calculated assuming the particle is dry.

\[
V_{v,p} = (V - V_{s}) - V_{iw}
\]

Step 8: Determination of the volume of water inter-particles \( (V_{w,p}) \). This is the volume of water which is free to move in the inter-particle voids. This volume is equal to the volume of inter-
particle voids minus the volume of air in the soil (Equation 4-9). The volume of air in the soil is not affected by the soil expansion in the sense that the air stays in the inter-particle voids, since the interlayer voids is only occupied by water. The volume of air ($V_{air}$) is calculated by subtracting the total volume of water in the soil from the volume of voids in the soil (Equation 4-10).

$$V_{w,p} = V_{v,p} - V_{air} \quad (4-9)$$

$$V_{air} = V_{v} - V_{w} \quad (4-10)$$

$$V_{w} = \frac{m_{w}}{\rho_{w}} \quad (4-11)$$

**Step 9:** Calculation of the corrected void ratio ($e_{corrected}$). Void ratio is defined as the volume of voids divided by the volume of solids. These voids are the actual space between the soil grains, the space where water can flow freely. For expansible soils, this is the inter-particle volume defined in step 7 ($V_{v,p}$). When the expansible grains are hydrated, the volume of particles must be determined as explained in step 5. The final equation is described below (Equation 4-12).

$$e_{corrected} = \frac{V_{v,p}}{V_{s}} \quad (4-12)$$

**Step 10:** Calculation of the corrected degree of saturation ($S_{r_{corrected}}$). By definition, the degree of saturation is the ratio of the volume of water between the voids and the void ratio. Therefore, the corrected void ratio is the ratio of the volume of water inter-particles ($V_{w,p}$) and the inter-particle void volume ($V_{v,p}$).

$$S_{r_{corrected}} = \frac{V_{w,p}}{V_{v,p}} \quad (4-13)$$

**Step 11:** Water content correction. Gravimetric water content is calculated by dividing the total mass of water in the soil by the mass of solids. These masses are determined after oven-drying the soil for at least 24 hours to remove the water from the soil. However, when water is added to a soil
with expansible minerals, a part of this water occupies the mineral interlayer space increasing the mineral d-spacing. Therefore, a portion of the water stays in the interlayer space inside the particle, and the remainder stays in the space between soil particles, meaning that not all the water determined by the water content calculation is available for soil interactions. To consider the true water content of the soil, it is necessary to understand what circumstances the particle is under at the moment of calculation. When the soil is hydrated, the particles have water in the interlayer space, which means that the grain is now composed of solids plus the water occupying the interlayer space. So, the calculation of the corrected water content should be calculated as described in Equation 4-14. The numerator refers to the mass of water available in the inter-particle space, which is equal to the mass of water ($m_w$) minus the mass of interlayer water ($m_{iw}$). The denominator is the sum of the dry mass of soil ($m_s$) and mass of interlayer water, since the interlayer water is now part of the particle. The mass of water ($m_w$) is determined by the difference between the mass of the wet soil (which consists of the water mass in the inter-particles and interlayers) and the mass of the dry soil. Mass of solids is determined by the mass of the dry soil after oven drying it. The mass of interlayer water ($m_{iw}$) is calculated as in Equation 4-7.

$$w_c = \frac{m_w - m_{iw}}{m_s + m_{iw}} \quad (4-14)$$

Therefore, by following this 11-step procedure, it is possible to correct the void ratio, degree of saturation, and water content of the expansive soil and then perform the analysis on the soil-water characteristic curve and other soil parameters, such as strength and permeability. The comparison between the traditional and proposed equations is presented in Table 4-1.
Table 4-1: Proposed equations

<table>
<thead>
<tr>
<th>Equation</th>
<th>Void ratio (e)</th>
<th>Degree of saturation (Sr)</th>
<th>Gravimetric water content (w)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Traditional</td>
<td>$\frac{V_v}{V_s}$</td>
<td>$\frac{V_w}{V_v}$</td>
<td>$\frac{m_w}{m_s}$</td>
</tr>
<tr>
<td>Corrected</td>
<td>$\frac{V_{v,p}}{V_{s'}}$</td>
<td>$\frac{V_{w,p}}{V_{v,p}}$</td>
<td>$\frac{m_w - m_{i_w}}{m_s + m_{i_w}}$</td>
</tr>
</tbody>
</table>
Chapter 5: Result analysis

5.1 Overview

This chapter introduces and analyzes the results obtained from the experimental approach as well as the interlayer water calculation described in Chapter 3. The objective of this chapter is to evaluate the effect of water infiltration into the interlayer space which causes the particle swell and its consequences on the soil void ratio. Void ratio is a key factor in the unsaturated soil mechanical analysis, and it is one of the governing factors responsible for the shape of the soil-water characteristic curve. This study was based on mineral composition analysis of the clay fraction. Therefore, this chapter analyzes the consequences of particle swell on the soil void ratio and discusses the idea of considering a corrected void ratio on the unsaturated analysis. The analysis was performed on a local expensive soil and two others with data available in literature.

The chapter organization is as follows. Section 5.2 presents the material used in this study and where it was collected. The mineralogical analysis using XRD is described in section 5.3 where the minerals presented in the material clay fraction and the interlayer spacing are determined. Section 5.4 discusses the physical properties of the material including natural water content, specific gravity, particle size distribution, Atterberg limits, and shrifikage test. The compaction characteristics, which include the optimal water content and maximum dry density, is presented in section 5.5. Section 5.6 investigates the microstructure of the material using SEM. The calculation proposed in Chapter 3 is applied to the expansible soil collected in the Okanagan Valley is presented in section 5.7. The relationship of suction and water content in the soil is discussed in section 5.8 where the SWCC, determined by filter paper test, is presented. In this same section numerical curve fitting models were applied to the data. In addition, the effect of interlayer water
volume was analysed and the SWCC was modified considering the corrected void ratio. Finally, the same calculation was performed on bentonites with data available in literature and the results are presented in section 5.9. Section 5.10 summarizes the findings of this research.

5.2 Material

The sample was collected from the Glenmore area (49°55′20.9″N 119°26′14.7″W) in the Okanagan Valley, where soil with swell potential was identified by local constructors. Disturbed sample was collected with an excavator at a depth of 2 meters. The soil was put in buckets and sealed with lids and transported to the lab where the temperature was 23±2°C.

According to City of Kelowna (1998), the Okanagan Valley has long and warm summers (average temperature of 21°C in July) and short and mild winters (average of 2°C in January) with average annual precipitation of 31 cm. The soil deposited in the Glenmore area consists of fine textured silty and clayey glacio-lacustrine sediments (Gough et al., 1994).

5.3 Mineralogical analysis

To comprehend the behavior of clays, the soil’s mineral composition is an important factor which must be considered and well understood. XRD analysis was performed to identify the mineral composition of the clay soil and find the interlayer distance. On completing XRD, Rietveld method was used to further refine the data obtained. Table 5-1 summarizes the quantitative results of the mineral composition of the clay fraction of the soil.
Table 5-1: Soil mineral quantitative analysis

<table>
<thead>
<tr>
<th>Mineral</th>
<th>Ideal Formula</th>
<th>Clay weight (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interlayered chlorite/smectite</td>
<td>-</td>
<td>25</td>
</tr>
<tr>
<td>Clinochlore</td>
<td>$(\text{Mg,Fe}^{2+})_3\text{Al}(\text{Si}<em>3\text{Al})\text{O}</em>{10}(\text{OH})_8$</td>
<td>15.5</td>
</tr>
<tr>
<td>Plagioclase</td>
<td>NaAlSi$_3$O$_8$ – CaAl$_2$Si$_3$O$_8$</td>
<td>14.5</td>
</tr>
<tr>
<td>Illite/Muscovite 2M1</td>
<td>$K_{0.65}Al_{2.6}Al_{0.65}Si_{3.35}O_{10}(OH)<em>{2}/KAl_2AlSi_3O</em>{10}$</td>
<td>14.1</td>
</tr>
<tr>
<td>Quartz</td>
<td>SiO$_2$</td>
<td>9.3</td>
</tr>
<tr>
<td>K-feldspar</td>
<td>KAlSi$_3$O$_8$</td>
<td>8.8</td>
</tr>
<tr>
<td>Biotite</td>
<td>$K(\text{Mg,Fe}^{2+})_3\text{AlSi}<em>3\text{O}</em>{10}(\text{OH})_2$</td>
<td>4.3</td>
</tr>
<tr>
<td>Calcite</td>
<td>CaCO$_3$</td>
<td>3.3</td>
</tr>
<tr>
<td>Gypsum</td>
<td>CaSO$_4 \cdot 2$H$_2$O</td>
<td>2.3</td>
</tr>
<tr>
<td>Others</td>
<td>-</td>
<td>2.9</td>
</tr>
</tbody>
</table>

Figure 5-1: X-Ray diffraction results with mineral indication.
The results indicate the presence of an interlayered mineral presented in a wide hump at 8° 2θ that was estimated to be 20 to 30% of the clay mineral weight; the broad range is due to the difficulty of identifying interlayered minerals. Interlayered minerals can be found in moderately acidic environments and in semi-arid regions (Rich, 1968), such as the Okanagan. They are formed in soil that have expansible clay minerals (Meunier, 2007). The interlayered mineral presented in the soil sample was composed of chlorite and smectite, the second most common mixed-layered clay mineral found in sedimentary soil (Moore & Reynolds, 1989). The layers of smectite and chlorite are connected by exchangeable cations (Meunier, 2007), where the layers’ disposition is irregular, with no discernable pattern or uniform distribution (Grim, 1953). Comparing the results of d001 spacing of dried and ethylene glycol solvated samples indicate that this is a R0 chlorite/smectite mineral, which means that there are mostly chlorite sheets in the interlayered minerals (Moore & Reynolds, 1989).

The clay fraction of the soil sample was composed of different minerals, but about 40% of it was from the chlorites group - most of which was in the chlorite/smectite mixed layered mineral. The rest (15.5%) was in the form of clinochlore. Clinochlore is a mineral from the trioctahedral subgroup, which has magnesium as a divalent ion in the 2:1 structure and in the octahedral sheet (brucite) between the 2:1 structures (Azevedo & Torrado, 2009; Moore & Reynolds, 1989). If the magnesium in the brucite layer is not exchanged by iron or aluminum ions there is a charge unbalance in the structure. This unbalance has to be neutralized by other exchangeable cations which can lead to an increase of the interlayer distance (Camapum de Carvalho et al., 2015). The interlayer distance increase causes the chlorides to swell, which causes soil expansion. The phenomena of swelling chlorides has been reported by Post & Janke (1974), Sutherland & MacEwan (1960), and Vivaldi & MacEwan (1960). In the soil profile, a small amount (4.3%) of
biotite was present, and knowing that chlorites can be formed from biotite minerals (Azevedo & Torrado, 2009), which may be one of the sources of the chlorite found in the soil.

The clay fraction is also composed of muscovite, a type of illite that has potassium between its layers – the potassium does not allow an increase in the interlayer space. The majority of chlorite, illite, and some smectite minerals can be explained by the low precipitation in the Okanagan Valley. According to Grim (1959), there is not much leaching of the soil in regions where the annual precipitation is less than 38 cm. Therefore, the water in the soil concentrates at the surface, where there is a large evaporation rate, developing illites, montmorillonites (smectite group), and chlorites (Grim, 1959).

Figure 5-2: X-Ray diffraction results for different treatments

Figure 5-2 show the results for different sample treatment from the XRD test. As seen in the figure, the first peak of the curves changes depending on the treatment. The other peaks are not affected,
which means that the treatments did not change the soil mineral composition, only the interlayer distance was affected. Figure 5-3 shows closely the effect of the sample treatments on the d-spacing of the first peak.

The changes in the d-spacing with treatment variation is most probably attributed to the clinochlore mineral and the interlayered chloride/smectite mineral because at the range between 5° 2θ and 10° 2θ, where there was a variation in the peaks of each sample, only clinochlore and the interlayered mineral were detected (Figure 5-1). The difference of d-spacing presented in Figure 5-3 shows the evolution of the interlayer space for each treatment. The smallest d-spacing was found for the soil in its dry state and was 12.39 Å. The largest distance was found in the hydrated sample, where the distance was 16.40 Å.

Figure 5-3: d₀₀₁ peak with d-spacing measurement.
In conclusion, the clay fraction of the soil sample is mainly composed of an interlayered mineral composed of minerals from the chlorite and smectite groups, most probably the chloride mineral is the clinochlore, since this is the second most abundant mineral is the clay fraction. Minerals from the smectite group, for instance the montmorillonite, are very swelling minerals. The non substitution of magnesium in the brucite layer may cause expansion as well. The expansion was visualized in Figure 5-3 in the 2θ range where the interlayered mineral and clinochlore were detected, indicating the expansion of these minerals. Therefore, the clay is composed of approximately 40% of expansible minerals, which explains the expansive behavior of the soil.

5.4 Soil physical properties

Physical properties of the soil like water content, specific gravity, PSD, Atterberg limits (plastic limit and liquid limit), and shrinkage limit are important properties for fine grained soil. The following sections discuss the obtained results of these tests.

5.4.1 Natural water content

The natural water content was determined on the same day the sample was collected. The sample was removed from the sealed bucket and oven dried for 48 hours at 105ºC. The sample water content was found to be 33±1%.

5.4.2 Specific gravity

Three samples where tested and the specific gravity (Gs) at 20ºC was 2.815. The maximum range between two results and standard deviation were 0.02 and 0.0089, respectively. Both values are acceptable for clayey soil according to ASTM D854 (ASTM, 2014).
5.4.3 Particle size distribution

The PSD was determined by using hydrometer and laser diffraction. The hydration time is extremely important for water sensitive materials and was tested in the hydrometer test. As explained in Chapter 2, when expansible clay minerals are in contact with water the hydration of exchangeable cations will lead to an increase in the interlayer space, and the cation concentration difference between the adsorbed water and pore water causes electric repulsion.

5.4.3.1 Hydrometer test

Figure 5-4 shows the PSD of the soil found by hydrometer test after sample immersion in sodium hexametaphosphate solution for 16 hours as described in ASTM D7928 (ASTM, 2017). The graph indicates that the soil texture is composed of 2% fine sand, 24% silt, and 74% clay. Therefore, this is a fine grained soil with almost half (53%) of the particles smaller than 1 μm.
The effect of the hydration of the clay particles with time of hydration on soil texture was analyzed following the procedure described in 3.3.1. Figure 5-5 indicates the results of the readings for the sample in water only. The reading name indicates how much time passed from the moment the soil was in contact with water until the first hydrometer reading. The curves presented in Figure 5-5 illustrate only the finer fraction of the soil since only this fraction is affected by hydration time. Therefore, the curve is not complete for particle diameters greater than around 0.035mm. A sudden decrease of the particle diameter was noticed at 0.011 mm, which occurred at about the 15 minutes reading.

Figure 5-5: Hydrometer test with sample in water.

Figure 5-6: Hydrometer test, sample in water only (left) and in sodium hexametaphosphate solution (right)
Figure 5-6 is the picture of the test after 15 minutes passed from the time the sample was mixed in the cylinder and allowed to settle. The soil grains in RO water only had mostly settled, meanwhile the sample in sodium hexametaphosphate solution was still in suspension. Fine particles, due to the electrical charges on their surfaces, tend to agglomerate in the low energy environment of the cylinder and sit at the bottom. The solution is used to dissipate the soil particles and the smaller particles stay in suspension, so the dimension of each particle can be measured.

For non-swelling soil the curves in Figure 5-5 should be overlapping each other. The curves are slightly different with different pre-wetting times because with more hydration time the particles are swelling. The calculation of the grain size distribution after the hydrometer measurements depend on the soil specific gravity, as explained in section 4.3.2. Increase of grain volume would decrease the grain density and, consequently, the soil specific gravity.

Specific gravity is the relationship between the density of soil grains and water density, as described in Equation 5-1. The soil grains density is the ratio of the mass of solids and its volume. It is important to note that a change in the grain volume would decrease the grain density and, consequently, the soil specific gravity.

\[ G_s = \frac{\rho_s}{\rho_w} \]  

(5-1)

The change of specific gravity was evaluated for both samples. For the sample tested in RO water, decrease of specific gravity value caused the overlap of the curves at different hydration times (Figure 5-7). The specific gravity used for 0 hours of immersion in water was the same as that found in the specific gravity test, 2.815. The other specific gravity values used to obtain the curves in Figure 5-7 are presented in Table 5-2. However, for the sample tested in RO water and sodium hexametaphosphate solution the specific gravity had to be increased in order to have the curves overlapping. A possible explanation is that sodium hexametaphosphate changes the charges on the
clay surface and decreases the size of the interlayer space as described by Castellini et al. (2013). Since the clay particles are decreasing in volume, the specific gravity increases.

![Figure 5-7: Hydrometer test with sample in water decreasing the soil specific gravity with the time increase.](image)

<table>
<thead>
<tr>
<th>Time in water (hour)</th>
<th>Gs</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>2.815</td>
</tr>
<tr>
<td>1</td>
<td>2.807</td>
</tr>
<tr>
<td>2.5</td>
<td>2.790</td>
</tr>
<tr>
<td>3.5</td>
<td>2.750</td>
</tr>
<tr>
<td>4</td>
<td>2.750</td>
</tr>
<tr>
<td>4.5</td>
<td>2.750</td>
</tr>
</tbody>
</table>

**5.4.3.2 Laser diffraction**

Laser diffraction tests were performed on the soil finer fraction (<0.074mm) to evaluate the particle size distribution and diameter under two different soaking times, liquids used, and ultrasound uses. Soaking times of 0 and 16 hours were chosen, the latter being the minimum time required for a
hydrometer test. The liquids tested were RO water and sodium hexametaphosphate solution (40 g/L) as a dispersive solution. Finally, ultrasound waves were used to prevent flocculation in the particles and allow more precise measurements of particle sizes (Buurman et al., 1997).

The particle size distribution of the soil tested in both laser diffraction and hydrometer tests are presented in Figure 5-8. The hydrometer test was not able to measure particles finer than 1µm, hence the curve plotted using this test data ends before 1 µm. The clay fraction of the soil, which corresponds to the number of particles smaller than 2 µm, was dependent on the liquid used, hydration time, and if there were ultrasound waves emitted. The sample tested in water after 0 hours of hydration presented the smallest amount of clay sized particles – approximately 15%. Samples tested with sodium hexametaphosphate after 16 hours of hydration measured the highest percentage of clay sized particles amongst all the samples tested with laser diffraction.

![Figure 5-8: Particle size distribution of sample clay fraction from laser diffraction and hydrometer tests.](image)
The flocculation of the particles is minimized with the use of deflocculant, as presented in Figure 5-10 (b) and (c) and discussed previously in 5.4.3.1. The use of sodium hexametaphosphate solution minimizes surface charges and allows better readings. The sample tested with 0 hours of hydration in deflocculant presented a PSD of a finer material than the sample tested in water. This comparison is shown in Figure 5-9 (b), which shows that the sample without deflocculant has a peak shifted to the right, indicating that flocculation occurred (Buurman et al., 1997).

Time also affects the efficiency of the deflocculant. Figure 5-9 (b) compares the results for 0 and 16 hours of hydration with deflocculant. For the 0 hour reading, the second peak is shifted to the right in comparison with 16 hours reading, indicating bigger particle size. Therefore, there was not enough time for the salt to facilitate particle dissociation.

The sample tested with 0 hours of hydration in water that was subjected to ultrasound waves presents the PSD of a finer material. This result is better visualized in Figure 5-9 (a), which shows that the ultrasound waves shifted the last peak to the left and minimized particles flocculation. However, Figure 5-8 Figure 5-9 (c) show that there is no significant difference when ultrasound waves are applied if the soil is allowed to soak in deflocculant for 16 hours.

Finally, Figure 5-9 (d) compares the results obtained by different soaking time but using both deflocculant and ultrasound waves. The longer time in solution allowed the particles to disperse more but not significant difference is noticed, indicating that ultrasound waves helped with the particles dissociation. Therefore, the results suggest that it may not be necessary to hydrate the sample for 16 hours when deflocculant and ultrasound waves are used.
Figure 5-9: Laser diffraction results for different treatments. Peaks indicate a high volume of particles with a determined diameter.

However, the clay content in the sample obtained by laser diffraction was significantly smaller than that measured with the hydrometer test (74%). Other research (Beuselinck et al., 1998;
Buurman et al., 1997; Eshel et al., 2004; Loizeau et al., 1994; Singh & Arif, 2014) shows that the clay content determined by laser diffraction tests tends to underestimate the content of clay sized particles, indicating smaller amounts than what is found with hydrometer tests. These authors attribute this difference to the fact that clay particles do not have spherical shapes, which is an assumption of the laser diffraction test. However, the hydrometer test uses the principle of Stokes’ law, which also assumes the particles are spherical. The key difference is that in the hydrometer test, an equivalent diameter is calculated based on the mass (density) of the particles, whereas the laser diffraction test uses measures based on the optical diameter (Beuselinck et al., 1998). Buurman et al. (1997) discuss that in addition to the particle spherical shape, another contributor to this difference in clay content between the two tests is the size of the sample tested: approximately 100mg for the laser diffraction test and 50g for the hydrometer test.

In conclusion, the laser diffraction test was not able to provide a reliable particle size distribution. The test measures the optical diameter of the particle and not its volume. Thus, hydration time did not affect the measurement since this test measures the particle diameter and not the particle volume. The hydration affects the interlayer space of the clay particle and consequently the particle volume. Laser diffraction results were used to obtain the particle diameter for further calculation of interlayer volume of the Glenmore clay. The particle diameter ($d_p$) was determined based on the results obtained with ultrasound were chosen for the diameter analysis. Since the peak represents the point at which more particles presented a specific diameter, the peaks were measured. The values are presented in Figure 5-10, and the diameter of 0.737 µm was chosen for further calculations.
5.4.4 Atterberg limits

The Atterberg limits consist of the liquid limit and the plastic limit of the soil. They are related to soil texture and mineralogical composition of the soil (Camapum de Carvalho et al., 2017). Liquid limit represents the water content at which the soil changes from the plastic state to the liquid state and is related to the water retention capacity of the soil. The Casagrande method was chosen to determine its value. The plastic limit is the water content at which the soil changes from semi-solid to plastic state, and the soil behaves like a plastic material. Therefore, the plastic limit can be related to the mechanical properties of the soil and to the cracks on soil surface (Camapum de Carvalho et al., 2017). The soil liquid limit and plastic limit were 64.7% and 32.7%, respectively; which gives a plasticity index of 32%.

5.4.5 Soil classification

The Unified Soil Classification System (USCS) is used to classify soils based on the particle size distribution, liquid limit, and plasticity index. The studied sample had 100% of the material passing
through the sieve #10 (2 mm) and 74% of the particles were clay sized particles. According to the ASTM D2487 (ASTM, 2017), the studied soil was classified a highly plastic or fat clay (CH). As discussed in Chapter 2, the Atterberg limits can be used to predict the swell potential of the soil. According to Peck et al. (1974), a soil with plasticity index of 32% has medium expansion potential. However, according to Seed et al. (1962) and Daksanamurty and Raman (1973), the soil samples is considered to have high swell potential based on Atterberg limits. The International Building Code (2012) and the International Residential Code (2012) suggest that a soil can be considered expansible if some criteria are met, such as: (i) the plasticity index is equal to or greater than 15%; (ii) more than 10% of the soil passes through sieve #200 (0.075 mm); and (iii) more than 10% of the soil is smaller than 5 µm. As illustrated in Table 5-3, the soil sample met these criteria and can be considered expansible.

Table 5-3: Soil properties summary

<table>
<thead>
<tr>
<th>Soil properties</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay percentage</td>
<td>74%</td>
</tr>
<tr>
<td>Material passing sieve #200</td>
<td>98%</td>
</tr>
<tr>
<td>Soil particles smaller than 5 µm</td>
<td>85%</td>
</tr>
<tr>
<td>WL</td>
<td>64.7%</td>
</tr>
<tr>
<td>WP</td>
<td>32.7%</td>
</tr>
<tr>
<td>PI</td>
<td>32%</td>
</tr>
<tr>
<td>Gs</td>
<td>2.815</td>
</tr>
<tr>
<td>Swell pressure (at ( w_{opt} ))</td>
<td>60 kPa</td>
</tr>
</tbody>
</table>
5.4.6 Shrinkage test

Shrinkage limit test was performed to determine the water content at which there is no significant volume change in the soil. The results are presented in Figure 5-11. The shrinkage curves for nine samples followed the pattern of drying from a slurry with water content close to the soil liquid limit, as described by Fredlund et al. (2002). In the drying process the void ratio decreased along the saturation line. The saturation line was determined after the basic volume-mass relationship (Equation 5-2), where Sr is 1 and soil specific gravity (Gs) is the slope of the curve. The voids reduce linearly until the water content is close to the plastic limit, indicating that the water loss is similar to void size reduction in terms of volume (Marinho, 1994). The zero-shrinkage phase began with water contents below 19.5% and the void ratio stabilized at 0.81±0.02. The soil shrinkage limit of 29% was found graphically (Figure 5-11) as the water content in the intersection of the saturation line and the horizontal line with ordinate equals to the minimum void ratio (Mishra et al., 2019).

\[ Sr \times e = w \times Gs \]  \hspace{1cm} (5-2)

Figure 5-11: Shrinkage limit test results.
The shrinkage curve was also determined for the samples tested in the filter paper test, the results are presented in Figure 5-12. Differently than the results obtained in Figure 5-11, the points from the filter paper test did not go along the saturation line and the void ratio stabilized at a lower value (0.65±0.02). The reason for these differences is the sample preparation. First, the filter paper test samples were all compacted, what explains the lower void ratio. The objective of compaction is exactly to decrease the size of voids between grains and obtain a denser structure. The samples were all compacted at the optimum value of maximum dry density and water content. The degree of saturation at optimum is 94.3%. The samples, as explained in section 3, were allowed to air dry and to get wet to obtain a wide range of saturation for the SWCC. However, the wetting path was reached by dropping water on the sample surface and allowing it to swell. This is not the best condition to saturate a sample due to the low permeability of the soil, ideally the sample should be submerged in water or have counter-pressure applied. Since the objective was not to fully saturate the sample, but to obtain higher degrees of saturation, none of the last two procedures happened. Therefore, the points at water content higher than 20% are not fully saturated. The points are not going along the saturation line, but have similar inclination to the saturation line. Marinho (1994) showed that when the points have the same inclination as the saturation line there is no variation on the volume of air in the sample. Overall, satisfactory results for further analysis were obtained.
It is important to note that in both methods, drying from slurry and filter paper test samples, the shrinkage curves (Figure 5-11 and Figure 5-12) change inclination at a degree of saturation of around 80%. The increase in void ratio with water content is significant for the zone with degree of saturation above 80%, which means that considerable volume change occurs for degree of saturation above 80%. These results agree with the findings of Al-Yaqoub et al. (2017), who also attributed the slightly increase in void ratio with water content increase for the points with Sr <80% to suction decrease (sample recovers lost volume caused by the shrinkage process).

5.5 Compaction characteristic

The compaction characteristic is the relationship between the dry density and water content of the soil. Different compaction efforts provide different curves with their optimal condition along the curve with degree of saturation. The standard compaction energy was chosen for this study since this energy is commonly used, for example in subgrade layer of pavement. The optimal water
content and maximum dry density reached by the soil under standard compaction effort were respectively 29.3% and 1498 kg/m³. According to Lambe (1958), optimum water content is desired for clay soil because at this water content the double layer expands, increasing inter-particle repulsion, reducing flocculation, and allowing the particles to move into a more oriented distribution, which leads to denser structure.

5.6 Microstructure analysis

Scanning electron microscopy (SEM) images were taken to analyze the soil microstructure. Figure 5-13 is a micrograph of the clay sample with 5000x magnification. The image shows particles with flaky shape and an oriented structure. As shown in Chapter 2, oriented, or dispersed structured soils have higher swell potential. Based on the morphology, this is a homogeneous material.

Figure 5-13: Clay sample (5000x magnification)
5.7 Application of the proposed model in Glenmore clay

The effect of particle expansion was evaluated and the particles were corrected by following the procedure proposed in Chapter 4. This calculation was performed for further analysis on the soil suction and soil tensile strength.

The determination of the d-spacing and water content relationship was performed after the evaluation of the XRD results for the dry and hydrated samples presented in Figure 5-3. As previously explained in 2.2.2.1, the relationship between the d-spacing and water content is linear. Therefore, two points were determined with samples tested with the XRD test with their water content at 3% (hygroscopic water content) and at 60%, which is close to the soil liquid limit. The equation describing the effect of water content to the basal space is presented in Figure 5-15.

\[
y = 0.0742x + 11.948
\]

![Figure 5-14](image.png)

Figure 5-14: Relationship of d-spacing and water content for the local soil.

The volume of the particle for each water content was determined using Equation 4-1. The particle diameter (d_p) used for the calculation was 0.737 µm, a value chosen from the laser diffraction test (Figure 5-10). Only one value of diameter was chosen because there is no significant variation of the size of the clay particles, physically this difference is not considerable. The d-spacing (d001) value was calculated using the equation presented in Figure 5-14, which means that the d-spacing
was calculated for every sample since they did not present the same water content. The volume calculation also requires the estimation of the number of clay units per stack. Since clay is a natural material, the number of clay sheets per stack is variable, and an estimated value should be selected. The number of clay units per stack for chlorites varies from 3 to 9 layers per stack (Grim, 1953), meanwhile the number clay units per particle varies for other minerals. Besides that, the particle hydration leads to separation of the layers forming particles with less layers resulting in smaller particles (Saiyouri et al., 2004). An approximate value of 10 clay units per particle was used for calculations. The number of clay units per particle is responsible for the particle height and consequently its volume. The variation of the number of clay units per stack (n) was analyzed and the conclusion was that the final void ratio ($e_{\text{corrected}}$) was not affected. However, the particle volume ($V_p$) and the number of swelling particles ($N_p$) in the sample was affected, which means that the soil porosity does not change but the pore distribution does change Table 5-4.

<table>
<thead>
<tr>
<th>n</th>
<th>$V_p$</th>
<th>$V_{s,\text{swell}}$</th>
<th>$N_p$</th>
<th>$V_{s'}$</th>
<th>$V_{iw}$</th>
<th>$V_{v,p}$</th>
<th>$e_{\text{corrected}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>6.25E-15</td>
<td>6.25</td>
<td>1.20E+15</td>
<td>22.35</td>
<td>1.23</td>
<td>21.78</td>
<td>0.974726</td>
</tr>
<tr>
<td>50</td>
<td>3.12E-14</td>
<td>6.25</td>
<td>2.40E+14</td>
<td>22.35</td>
<td>1.23</td>
<td>21.78</td>
<td>0.974726</td>
</tr>
<tr>
<td>100</td>
<td>6.25E-14</td>
<td>6.25</td>
<td>1.20E+14</td>
<td>22.35</td>
<td>1.23</td>
<td>21.78</td>
<td>0.974726</td>
</tr>
</tbody>
</table>

This procedure was used for the suction test samples as described in next section.

5.8 Soil-water characteristic curve

The soil-water characteristic curve (SWCC) is the relationship between the amount of water in the soil and the negative pressure (suction) needed to remove the water (Camapum de Carvalho & Leroueil, 2004), as discussed in section 2.3.2. All samples prepared for the determination of the soil-water characteristic curve were casted by static compaction. The target water content and dry
density were the optimal values which were defined in section 5.4, to ensure the same initial condition and pore distribution for all samples. After compaction, some samples had water added and were allowed to swell and others were air dried until the desired water content was reached, which was confirmed by measuring the sample mass. After samples reached the desired water content, the SWCCs were determined using the filter paper method.

The soil samples and filter papers were allowed to equilibrate for a period of 3 weeks. Lee (2013) suggests an equilibration time of 2 to 3 weeks for plastic soil. Since the equilibrium period of 2 weeks did not obtain satisfactory results, a 3 week period was chosen instead. After equilibration time, the filter paper water content was determined and related to the filter paper calibration curve. The calibration curve used was the equation provided by the ASTMD5298 (ASTM, 2016) for Whatman 42 filter paper (Equations 5-3 and 5-4). According to Marinho and Oliveira (2006), there is no need to have two different calibration curves for matric and total suction. After the filter paper measurement, the soil samples were weighted and had their dimensions measured for further determination of the void ratio and degree of saturation. The SWCC was determined after 4 repetitions with each test having at least 12 points.

\[ h = 5.327 - 0.0779w_f, \text{for } w_f < 45.3\% \]  
(5-3)

\[ h = 2.412 - 0.0135w_f, \text{for } w_f \geq 45.3\% \]  
(5-4)

Soil air entry value (AEV) and residual suction were determined since they are primary reference points of the soil-water characteristic curves. Soil subjected to a volume change should have its AEV and residual suction obtained from the curve that relates the suction with the degree of saturation in conjunction with the shrinkage curve (Fredlund & Rahardjo, 1993; Tripathy et al., 2014), which is done to better identify the AEV (Fredlund et al., 2011). The air entry value obtained was 5,000 kPa for both metric and total suction measurements. The curve did not present
a well-defined residual stage. Similarly, Tripathy et al. (2014) did not find a residual stage for two montmorillonite-rich bentonites but did find a well-defined residual stage for a kaolinite-rich clay sample. Therefore, the tested soil presented a SWCC with similar characteristics to other expansible soils.

Since the residual condition may be required for some mathematical methods, the curve that relates suction and gravimetric water content allowed the visualization of a few points which revealed a tendency of the residual stage. The soil residual water content, which is the driest state of the soil, was about 4.5%, slightly higher than the hygroscopic soil moisture of 3.8%. This difference is because hygroscopic moisture content depends on temperature and relative humidity (Camapum De Carvalho & Leroueil, 2004). The indication of a residual stage was found for suctions greater than 70,000 kPa. The difference between the soil AEV and residual suction was one order of magnitude, as suggested by Fredlund et al. (2012). Figure 5-16 shows both matric and total suction SWCCs. It can be noticed that there is no significant difference between the matric and total suction measurement for such high suction values, which can be explained by the lack of contact within the water and filter paper (Fredlund et al., 2012; Nam et al., 2009). For further analysis, only the matric suction will be analysed.

At a low suction level, in the second section, the water content and saturation degree (Figure 5-15) is high because the pores are almost completely filled with water (Sr close to 100%). Suction increase leads to the removal of the water from the pores, reduction of the water content, and soil shrinkage. The soil shrinkage, or the reduction in pore size, occurs at the same rate as the water removal, so the soil keeps saturated until the AEV; after this value the pore size reduction is greater than the water loss, so the pores are now filled with both air and water.
Campum de Carvalho & Leroueil (2004) proposed a method to present the SWCC called the transformed model. In this model, the suction is normalized by the soil void ratio. Since most mathematical methods present the suction in the logarithm scale, as the models presented above,
the suction is presented in the pF scale. The pF represents the logarithm of the height of water column in cm, which indicated a certain suction value. The suction measured in kPa was converted into the pF scale and multiplied by the void ratio calculated at each suction value. The other axis is the degree of saturation. When the suction, in term of pF, is multiplied by the void ratio, there is a decrease of the inclination of the curve, which is usually a straight line (Camapum de Carvalho & Leroueil, 2004). This decrease in inclination is explained by understanding the relationship between void ratio and suction.

As illustrated in Figure 5-17, an increase in suction leads to a decrease in the void ratio. From the graphical points in this figure, it is evident that there is a tendency for the data to follow one of two relationships. The data diverts into these two relationships at a suction value of around 5,000 kPa, and a void ratio of 0.75. It is worth noting that in the shrinkage limit tests, the void ratio of the soil began to stabilize at a value around 0.75. The first relationship, which occurs at any suction
level below 5,000 kPa, has a high degree of saturation and water content (Figures 5-15, 5-16) because the pores in the soil are almost completely filled with water (Sr close to 100%). This is because until a suction level of 5,000 kPa, the soil shrinkage occurs at the same rate as the water removal, which keeps the soil almost completely saturated. The second relationship, occurring when the suction level is greater than 5,000 kPa, the removal of water from these pores becomes significantly higher than the soil shrinkage. This causes the soil shrinkage to quickly approach its stabilizing shrinkage limit, while the water is continually removed from the pores. This is why the saturation degree drops dramatically, from nearly 100% down to around 20%. Knowing this explains why in the first relationship, only a small alteration to the suction level was able to generate a considerable change in the void ratio; whereas in the second relationship, once the suction level surpasses 5,000 kPa, it takes a far more significant change in the suction level to see only a small difference in the void ratio. To illustrate this, a jump in the suction level from 5,000 kPa to almost 100,000 kPa is only able to bring down the void ratio value by 0.1.

As described in Chapter 2, the method proposed by Camapum de Carvalho & Leroueil (2004) works well with all types of soil except bentonites. The suction test results were applied in the e-pF versus Sr graph (Figure 5-18) and presented a behavior similar to the one observed by Campos et al. (2008), even though the amount of expansive minerals in the Glenmore clay being considerably less. From lower to higher degree of saturation values the curve is almost horizontal with only a small inclination, until the value of saturation of 80% where the suction multiplied by the void ratio increases considerably. However, this is not the expected result. The results in Figure 5-16 clearly shows that the suction decreases significantly with saturation increase. Therefore, the multiplication of the suction by the void ratio was overestimated, and since the suction was measured, with results in agreement with the theory, the problem lay in the overestimation of the
void ratio. It is important to emphasize that the void ratio (e) was calculated as the total volume of voids divided by the volume of solids, which means this was the global void ratio. Therefore, since the suction results present acceptable values, there is a problem in the way that the void ratio was calculated and, knowing about the particle volume change with consequence variation of void ratio in expansible soil, the necessity to correct these values is evident.

Figure 5-18: Soil-water characteristic curve in the e.pF vs. Sr scale.

Other mathematical methods to present the SWCC were applied to the results obtained for the Glenmore clay but no anomaly was observed. The SWCC indicating the matric suction measurements were rewritten using the SWCC equations proposed by van Genuchten (1980) and Fredlund & Xing (1994). The methods developed by van Genuchten (1980) are limited to the range between the AEV and residual conditions (Fredlund et al., 2012). This is because for most soils
there is no change in soil void ratio between soil AEV and residual suction and the methods proposed by them are simple equations.

van Genuchten’s (1980) method relates suction and volumetric water content. The function is presented in Equation 5-5, where \( \theta_r \) and \( \theta_{sat} \) are respectively the residual and saturated volumetric water content. Residual volumetric water content was calculated by multiplying the lowest gravimetric water content reached during filter paper test by the soil’s specific gravity (Gs). For the saturated condition (Sr=1), the saturated gravimetric water content was equal to the maximum soil void ratio, as can be deduced from Equation 5-2.

\[
\theta = \theta_r + \frac{\theta_{sat} - \theta_r}{1 + (\alpha \gamma)^n}^m
\]  

(5-5)

Only the range of suction between air entry value and the indication of residual condition were analyzed. It was possible to determine best fit parameters and they are presented in Figure 5-19. Experimental data could be better fitted in this method as both empirical and experimental results have a logarithmic tendency. Therefore, van Genuchten’s (1980) method presented satisfactory results.

![Figure 5-19: Van Genchten (1980) best fit equation for matric suction measurements.](image-url)
Fredlund and Xing’s (1994) method allows the determination of the entire SWCC. This method relates suction and volumetric water content as described in Equation 5-6, where $e$ is the Napierian base logarithm ($e=2.718$) and $m$, $n$, and $a$ are fitting parameters. The results are presented in Figure 5-20.

$$
\theta = \theta_{\text{sat}} \left[ 1 - \frac{\ln\left(1 + \frac{\psi}{\psi_r}\right)}{\ln\left(1 + \frac{1000000}{\psi_r}\right)} \right] \left[ \frac{1}{\ln\left(e + \frac{\psi}{a}\right)^n}\right]^m
$$

(5-6)

As shown in the figures above, no anomaly in the suction results on the Glenmore clay were observed in the van Genchten (1980) or Fredlund and Xing (1994) equations. The reason for it is that none of those methods determined the theoretical SWCC directly from each void ratio measurement. Since the method proposed by Camapum de Carvalho & Leroueil (2004) multiplies...
the void ratio correspondent to the situation of the sample at each suction measure, the effect of
the particle volume increase is evident.

The proposed method to correct soil parameters such as the volume of particles, void ratio, water
content, and degree of saturation explained in Chapter 4 and section 5.7 were applied to correct
the void ratio overestimation. The interlayer distance used to calculate the particle volume was
calculated based on the linear relationship of d-spacing \((d_{001})\) and water content \((w)\) found in the
graph in Figure 5-14 (Equation 5-7).

\[
d_{001} = 0.0742w + 11.948
\]  

(5-7)

The corrected values of the void ratio \((e_{\text{corrected}})\) and the degree of saturation \((S_{r\text{corrected}})\) were
calculated and a new curve was determined in terms of \(e_{\text{corrected}}\cdot pF\) versus \(S_{r\text{corrected}}\). When
comparing the figures with the correction (Figure 5-21) to the those without the correction (Figure
5-18) it is possible to note a difference in the inclination in the section of the curve for the degree
of saturation less than 80%, which indicates that the void ratio was corrected and the suction is
decreasing with saturation increase. However, for the points above a degree of saturation of 80%
the correction in void ratio was not enough. A possible explanation is that for degree of saturation
above 80% there is a significant increase in void ratio with water content increase, as shown in
Figure 5-12. As observed by Al-Yaqoub et al. (2017), significant volume change occurs for degree
of saturation above 80%. Therefore, the need for correction in degree of saturation and void ratio
is more evident for \(S_r > 80\).
Even after correcting the void ratio, for points with a degree of saturation greater than 80%, the void ratio correction seemed insufficient. The reason is the underestimation of the d-spacing increase with the water content increase. This underestimation was caused by the difference in the hydration time that the XRD and filter paper test samples had; the XRD samples were subjected to hydration for 5 days and the filter paper test samples hydrated for at least 30 days. Villar (2007) showed that the water phase (vapor or liquid) was not an issue because it does not affect the d-spacing; however, greater d-spacing values were obtained for samples allowed to hydrate for longer periods. Thus, XRD samples must be tested after hydration during the same period that the sample is tested. This means, that if the soil is tested after 30 days of hydration, the XRD sample should also be evaluated after the same period, so there is no underestimation of the d-spacing value. On the other hand, the calculated d-spacing for the points with degrees of saturation less than 80% seemed to represent the actual d-spacing. One explanation for this is that in the XRD,
less time was available, but more water was present, while in the suction test, more time was available, but less water was present. It was found that with these two parameters being at different ends of the spectrum, the effects they had amounted to one another, thus the obtained result was correct.

To predict the interlayer space after 30 days of hydration the d-spacing variation with the global void ratio is presented in Figure 5-22. The d-spacing increase, which was determined using Equation 5-8, presented a linear relationship with the void ratio, as expected. The reason is that a greater d-spacing causes greater pressures in swelling soils, which increases the size of the voids. There is a difference in inclination of the points after \( e = 0.75 \). The d-spacing and void ratio increase have a linear relationship where the linear inclinations change for void ratios less than and greater than 0.75. The inclination changes at the same point where the degree of saturation is 80%. Since the void ratio correction for the points with degrees of saturation less than 80% seemed sufficient, the hypothesis is that the tread line of these points was the true relationship between d-spacing and void ratio.

![Figure 5-22: Relationship of d-spacing and global void ratio. Trend line corresponds to the points with Sr<80%.

\[ y = 13.585x + 3.8098 \]
The d-spacing of all points were then re-calculated based on equation 5-8, found in Figure 5-22, which correlated the d-spacing with the global void ratio. Figure 5-23 compares the results of the d-spacing obtained after equations 5-7 and 5-8. The results obtained from equation 5-8 showed a linear relationship of the d-spacing increase and global void ratio.

\[ d_{001} = 13.585e + 3.8098 \]  

(5-8)

Figure 5-23: Relationship of d-spacing and global void ratio with d-spacing values determined after equations 5-8 and 5-9
The SWCC obtained in terms of $e_{\text{corrected}} \cdot pF$ vs. $S_{\text{corrected}}$ after the correction of the d-spacing determined with Equation 5-8 is presented in Figure 5-24. This new SWCC presented the expected behavior where the suction, multiplied by the void ratio, is decreasing with an increase in the degree of saturation. Therefore, it can be concluded that the d-spacing calculation determined using Equation 5-7 – XRD data – underestimated the interlayer distance because the test time was not sufficient to allow better hydration of the clay minerals.

Figure 5-25 compares the shrinkage curve for two void ratio: the global void ratio (non corrected) and global water content, which do not account for the intra-crystalline swell; and the corrected values of void ratio and water content, where only the water in the macrostructure and the inter-particle voids are analysed. The figure shows that there is no significant variation on the inter-particle void ratio and the slightly increase for water content above 20% is related to the decrease in suction (release of tension).
Figure 5-25: Corrected and non corrected values of void ratio in the shrinkage curve.

The SWCC with corrected values of gravimetric water content is shown is Figure 5-26. The difference between the global and the corrected water content is greater for higher water content because of the linear relationship between the d-spacing, thus the particle volume, and the water content. It is important to note that the water content correction does not affect the suction measured, the AEV is still the same, the difference is on the water that is truly available on the soil grains. For the points with higher degrees of saturation, the difference in water content from the global to the corrected value is about 6.4%, which means that 6.4% of the water content of the soil is restricted to the insides of the soil particles.
5.9 Application in other expansive soils

The same methodology suggested for the calculation of the parameters affected by the particle volume increase was applied to the data collected by Villar (2007). Two swelling bentonites, FEBEX (Ca/Mg-bentonite) and MX-80 (Na-bentonite) were tested and had their SWCC and d-spacing measured. Table 5-5 summarizes the information of these two bentonites. Since these materials have a significant amount of expansive minerals, they were used to validate the proposed method.

The data was obtained by measuring the total suction of the soil in a wetting path SWCC. The samples were compacted at their hygroscopic water content inside a cell. This cell had holes to allow the flow of air between the soil and the environment. The soil samples in the cell were then put inside desiccators with imposed suction caused by solutions of sodium chloride or sulphuric acid. The soil increased water content with relative humidity of the desiccator increase, however...
the swell was restrained, which means that the global void ratio of the soil did not change. After performing the suction test, the samples were immediately tested in the XRD for determination of the d-spacing.

Table 5-5: Bentonites properties information from Villar (2007).

<table>
<thead>
<tr>
<th>Bentonites</th>
<th>FEBEX</th>
<th>MX80</th>
</tr>
</thead>
<tbody>
<tr>
<td>Origin</td>
<td>Spain</td>
<td>USA</td>
</tr>
<tr>
<td>Clay fraction</td>
<td>67±3%</td>
<td>90%</td>
</tr>
<tr>
<td>% swelling minerals</td>
<td>92%</td>
<td>90%</td>
</tr>
<tr>
<td>Exchangeable cation</td>
<td>Ca/Mg</td>
<td>Na</td>
</tr>
</tbody>
</table>

The FEBEX bentonites had 67±3% of the soil composed of clay sized particles. The clay fraction is composed of 92% of a montmorillonite and illite mixed layered mineral. Therefore, 61.64% of the soil particles were swelling. The samples were compacted at two different dry densities and the SWCC for each dry density presented similar results for suctions greater than 10 MPa. This similarity is because at high suction level the void ratio does not affect the shape of the SWCC (Romero et al., 2011).

The correction of the soil’s parameters may cause significant change on the shape of the SWCC, especially when the soil is saturated. Figure 5-27 compares the soil’s degree of saturation before and after correction for the FEBEX bentonite. Before correction, the degree of saturation is greater
than 100%, which suggests that the volume of water in the soil is greater than the volume of soil voids (Sr=Vw/Vv). However, since the test was performed at constant volume condition, it is not possible to have more water added to the sample than the soil voids can accommodate, the sample would overflow water, and this was not reported by the authors. However, when the proposed method is applied, the degree of saturation of the last two points is corrected and equals to 100%. The correction shows what was the soil actual condition; the inter-particle voids were completely filled with water, obtaining a degree of saturation of 100%, the surplus water is in the interlayer space.

Figure 5-27: Comparison of correction on the degree of saturation on the soil-water characteristic curve of FEBEX bentonite – the global values were obtained from Villar (2007) and the corrected values were obtained after applying the model.

The corrections of the water content for the SWCC of the soils compacted at dry densities of 1.5 g/cm³ and 1.6 g/cm³ are presented in Figure 5-28. The similarity of the results of samples
compacted at different dry densities for suction values above 10MPa was maintained after the water content correction. An interesting fact is that a more significant difference between the global and the corrected water content was observed for the sample compacted at 1.6 g/cm³ - 3.9% difference for the sample compacted at 1.6 g/cm³ and 1% for the sample compacted at 1.5 g/cm³. The voids are smaller in denser soils, so most of the water is occupying the interlayer space. Therefore, the water content correction is more significant for soil with smaller soils (denser).

![Soil-water characteristic curve of FEBEX bentonite for different dry densities with water content correction](image)

Figure 5-28: Soil-water characteristic curve of FEBEX bentonite for different dry densities with water content correction – the global values were obtained from Villar (2007) and the corrected values were obtained after applying the model.

The composition of the MX-80 bentonite soil that was clay sized particles was 65-90%, the amount of 90% was assumed in the calculations. The clay fraction is composed of 65-90% of a montmorillonite mineral – assumed 90%. Therefore, 81% of the soil particles were swelling. The samples were compacted at 1.3 and 1.6 g/cm³ and their SWCC with the water content corrections
are presented in Figure 5-29. Like the FEBEX bentonite soil, the sample compacted at higher densities presented a larger difference between the global water content and the corrected water content. For a density of 1.6 g/cm³ the difference was of 8.76% and for the sample compacted at 1.3 g/cm³ the difference was 7.41%. The water content difference was greater for the MX-80, which is a sodium bentonite, and has higher swell potential than FEBEX, a calcium bentonite. Therefore, sodium bentonite has higher water holding capacities in their interlayers.

Figure 5-29: Soil-water characteristic curve of MX-80 bentonite for different dry densities with water content correction – the global values were obtained from Villar (2007) and the corrected values were obtained after applying the model.

The void ratio correction is observed in Figure 5-30, where the corrected void ratio was related to the corrected water content. The samples had their volume change restrained during the test, so there was no change in void ratio with water content increase. However, the corrected void ratio
decreased with water content increase. Water content increase caused hydration of the clay minerals and a respective increase of the particle volume. The particle volume increase caused reduction of the voids between particles ($e_{\text{corrected}}$) and this is illustrated in Figure 5-30. The void ratio of the denser soil is smaller than the one compacted at a dry density of 1.3 g/cm$^3$. It is worth noting that the difference in the corrected void ratio between the as compacted state and saturated is similar for both densities, which confirms that the d-spacing increase depends on the soil mineralogy and pore water and not on the soil density, as suggested by Villar et al. (2012)

![Figure 5-30: Relationship of the corrected void ratio and the corrected water content of the MX-80 bentonite](image)

**5.10 Summary**

The hydration of the clay minerals causes an increase of the particle volume which affects the soil void ratio, saturation degree, and water content. In order to perform the calculation to determine all of those, it is required to know the mineralogical composition of the soil and the d-spacing increase of the swelling minerals after hydration. It is important that the d-spacing measurement
should be done on a test sample which is hydrated for the same duration as the soil to be corrected. The corrections changed the SWCC of the material. In conclusion, the results indicated that the correction of the particle volume should be made for every expansible soil, even if the soil is composed of a low percentage of swelling minerals.
Chapter 6: Conclusions and Recommendations

6.1 Summary and conclusions

Expansive soils are subjected to significant volume change with a variation of the water content, as they tend to expand when there is water available and shrink with water removal. The expansibility is mainly related to the soil mineral composition. One cause of soil swell is the hydration of the cations between clay units. This cation hydration leads to an increase of the interlayer space and an increase of the particle volume.

This research proposed the analysis of the particles’ volume change with d-spacing variation. A linear relationship between d-spacing and water content was developed after XRD test results. The calculation procedure was developed and used to calculate the corrected values of void ratio, degree of saturation, and water content of a local expansible soil found in the Okanagan Valley and two commercial bentonites. The bentonites data were available in the literature and the success of the proposed method on these bentonites validated the model. In these calculations, more accurate values and understandings for the various soil parameters can be found for expansive soils, when in the past certain parameters may have been undervalued or overestimated.

The main findings were:

a. The variation of the soil’s specific gravity with hydration time was observed for hydrometer test

b. Laser diffraction test does not provide a reliable particle size distribution for fine grained soils due to the shape of the particles. Laser diffraction test provides better results for spherical particles, which is not the case for clays.
c. The sample for XRD test must be prepared in the same conditions that the sample will be tested in for further analysis or conduct the XRD test immediately after any prior tests.

d. The transformed soil-water characteristic curve model does not present satisfactory results for expansible soil and requires the correction of the void ratio and degree of saturation.

e. The particle volume correction for further correction of water content, void ratio, and degree of saturation should be made for every soil that has swelling minerals as part of its composition.

6.2 Originality and contributions

The sample analysis required for the development of the corrections is costly and not widely available. The proposed calculation does not cause significant changes to the design of structures; however, it does provide a better understanding of the behavior of expansible soil which allows a more accurate design. The analysis seems more feasible for homogeneous materials used for drastic conditions, for example in nuclear waste repositories, where expansible soils are used to prevent possible leakage of fluid from nuclear waste. The understanding of the pores’ sizes is a key factor for the soil permeability; therefore this proposed analysis provides other tools for the analysis of dams, nuclear waste repositories, and landfill barriers.

6.3 Limitations and recommendations

A few limitations and their recommendation for future studies have been listed in the Table 6-1 below:
Table 6-1: Limitations and recommendations

<table>
<thead>
<tr>
<th>No.</th>
<th>Limitation</th>
<th>Recommendation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>The studies were performed for Glenmore clay, FEBEX, and MX-80 bentonites.</td>
<td>Different mineral compositions may have different analysis. The studies should be performed on expansible soil from other locations and with other minerals composition.</td>
</tr>
<tr>
<td>2</td>
<td>Tests performed in temperature of around 20-25 degree Celsius</td>
<td>The effect of temperature should be analyzed for structures that are subjected to different temperature ranges since temperature affects the water density and the particle attraction forces. Another recommendation is to analyze the effect of freeze and thaw on the water in the interlayer space.</td>
</tr>
<tr>
<td>3</td>
<td>The density of water in the interlayer space was assumed to be the same as the water in the inter-particle space.</td>
<td>Some authors suggest that the interlayer water density is different than that for free water. Better understanding of the interlayer water density should be done analyzing the effect of hydration time, water content, and soil suction.</td>
</tr>
<tr>
<td>4</td>
<td>The XRD test was not performed in the same city that the tests were held. Therefore, the sample tested for XRD was not subjected to the same condition than the filter paper test samples, causing underestimation of the clay minerals d-spacing.</td>
<td>Perform the XRD test immediately after the suction test in both tests the samples are on the same condition of water content and hydration time. Also, more points of water content should be analyzed to build a relationship between d-spacing and soil suction.</td>
</tr>
</tbody>
</table>
Bibliography


Laespiga, M. A. M. (2001). *Estudo do fenômeno de expansão dos sedimentos da formação*
Liberdad (Quaternário): Metodologia de avaliação de mapeamento na região de Montevidéu-Uruguai. Universidade de São Paulo.


