EFFECT OF PAPER MILL ASH ON PROPERTIES OF EXPANSIVE SOILS

By

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DEDICATION

To my family

DECLARATION

By submitting this thesis electronically, I declare that the entirety of the work contained therein is my own, original work, that I am the authorship owner thereof (unless to the extent explicitly otherwise stated) and that I have not previously in its entirety or in part submitted it for obtaining any qualification.

Date: April 2014

ABSTRACT

Expansive soils, one of the problematic soils, are encountered on all continents with exception of polar continents. Problems caused by their heaving and shrinking behaviour, particularly to light structures, have been reported from different countries to place large financial burden on developers. For this reason, many techniques have been developed and applied to prevent and/or remediate the damage caused by these soils. Soil stabilization with traditional chemical additives has been applied successfully since ancient times. In addition to traditional additives such as lime, cement, fly ash, etc., some non-traditional additives, such as polymer based products, salts, etc. have been used effectively for soil treatment.

On the other hand, industries are increasingly challenged by waste management in an acceptable and environmentally friendly manner. In this regard, a number of researches have been done on using industrial waste for soil improvement purposes. The study and understanding of basic reactions involved in lime-soil stabilization persuaded many researchers to study the applicability of lime-rich products for soil treatment. Studies conducted by Khalid et al. (2012); Muchizuki et al. (2004) and Thacker (2012) showed that lime-rich products such as pulp fly and bottom ashes and CaO by-products, can be applied for soil stabilization. This research was thus performed to investigate the effect of lime-rich paper mill waste ash on expansive soil properties.

Two commonly listed soil engineering properties namely volume change and strength were investigated. Soil strength was examined in terms of unconfined compressive strength (UCS), due to its correlations with a number of other soil properties, and the volume change in terms of free swell and swelling pressure. In addition to these two engineering properties, dry density and moisture content were also studied due to their involvement in structural design, as well as gradation, Atterberg limits and California Bearing Ratio (CBR). The choice of these properties was also influenced by the availability of a standard (ASTM D4609-08) specifically developed to assess the effectiveness of admixtures for soil stabilization.

Two main types of materials were used namely three clay materials and paper mill ash. According to the index properties, commonly used for expansive soil classification, three clays were classified into low, medium and high degrees of potential expansiveness. The ash results from the combustion of paper mill sludge, sawdust, bark, coal ash and bituminous coal in a multi-fuel boiler for the purpose of electricity and steam production. The tests mentioned

above were conducted on both untreated and treated clays and the results were compared. Since the study was carried out on this material based on the fact that it contains lime, the procedure applied for lime-soil stabilization was considered.

In general, it was observed that ash-soil treatment has a number of effects similar to lime-treatment and almost all studied properties were enhanced for all clays. It can thus be concluded that the paper mill ash from a multi-fuel boiler can be efficiently used for expansive soil treatment. For optimum use of this material for expansive soil treatment, more tests and further researches have been recommended.

OPSOMMING

Uitsettende gronde, een van die probleemtipe-gronde, kom op alle kontinente voor, behalwe die twee poolkontinente. Probleme veroorsaak deur uitswellende en inkrimpende gedrag van hierdie gronde, veral finansiële onkostes van ligte strukture is al in baie lande aangemeld. Vir hierdie rede is baie tegnieke ontwikkel en toegepas om skade wat deur hierdie tipe gronde veroorsaak is, te voorkom en/of herstel. Hierdie tegnieke sluit grondstabilisasie met chemiese bymengsels in, veral tradisionele bymengsels, wat met groot sukses in die verre verlede toegepas en na moderne tye oorgedra is. Bykomend tot tradisionele bymengsels soos kalk, sement, vlieg-as ensovoorts is ʼn aantal nie-tradisionele bymiddels soos polimeergebaseerde produkte, soute en ander produkte ontwikkel vir grondstabilisasie.

Aan die ander kant raak industrieë toenemend daarmee gemoeid om afvalstowwe op ʼn aanvaarbare en omgewingsvriendelike wyse te bestuur. Op hierdie gebied is ʼn aantal navorsingsprojekte al uitgevoer om industriële afval vir grondverbetering te gebruik en sodoende die las op nywerhede te verlig. Navorsing is onderneem om die basiese reaksies wat onstaan tydens stabilisasie van grond met tradisionele en moderne middels te bepaal en om die geskiktheid van kalkryke produkte vir grondstabilisasie te ondersoek. Baie navorsing is uitgevoer wat aangetoon het dat kalkhoudende produkte soos pulp vlieg- en oondresidu-as, asook CaO neweprodukte gebruik kan word vir stabilisasie. Gebaseer hierop is hierdie projek onderneem om die effek van papiermeulas, verkry deur die verbranding in ʼn veelvuldige brandstof-stoomketel, op die gedrag van uitsettende grond te ondersoek.

Tydens hierdie studie is twee algemene ingenieurseienskappe van grond, naamlik sterkte en volumeverandering ondersoek. Grondsterkte is geëvalueer in terme van eenassige druksterkte (EDS) as gevolg van ? deur middel van die korrelasie met ʼn aantal ander grondeienskappe, en die volumeverandering in terme van vry-swel en sweldruk. Addisioneel tot hierdie twee grondeienskappe is droë digtheid en waterinhoud ook bestudeer aangesien beide in struktuurontwerp betrokke is. Verdere eienskappe wat ondersoek is, is gradering, Atterberggrense en Kaliforniese drakragverhouding (KDV). Die keuse van hierdie eienskappe is beïnvloed deur die beskikbaarheid van ʼn toetsstandaard (ASTM D4609-08) wat spesifiek ontwikkel is om die effektiwiteit van bymengsels vir grondstabilisasie te evalueer. Hierdie standaard is deurgaans as verwysing tydens die projek gebruik.

Daar is waargeneem dat as-behandeling van grond ’n aantal effekte het soortgelyk aan kalkbehandeling, met die uitsondering van die droë digtheid en optimum waterinhoud van een van die gronde wat getoets is. Byna al die eienskappe wat ondersoek is, soos EDS, KDV, ensovoorts, is verbeter behalwe in die geval van die eerste klei waarvan die plastisiteitsindeks verhoog het en die grond meer plasties geraak het. Daar kan dus afgelei word dat papiermeule-as vanaf ’n stoomketel wat veelvuldige tipes brandstof gebruik geskik is vir die behandeling van uitsettende grond. Om die optimumgebruik van hierdie materiaal vir die stabilisasie van swellende klei te bepaal, is meer toetse en projekte nodig.

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If I have left anyone off this list, please realize that you are only missing from this page, not from my heart.

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CHAPTER 1 INTRODUCTION

1.1. BACKGROUND

Natural earth materials such as soil and rock play an important role in the design and construction of geotechnical systems where they are used to support and/or to make them. Those systems include among others retaining walls, embankments, road and airfield pavements, box culverts, and bridge abutments. Soil is thus one of the most important engineering materials for various civil engineering projects. Specifically, soil material can be used as:

- Construction material, for example in earthen structures such as dams, roadways, railways, embankments
- Supporting material in foundations; as well as
- Surrounding material in underground pipelines and tunnels.

However, as a natural material, soil exhibits high variability in its properties which vary largely from place to place and often, point to point. In addition to this variability, some geotechnical difficulties including inadequate bearing capacity, the potential for unacceptable settlements and slope instability can be observed due to some problematic soils, the most noteworthy being expansive soils, collapsible soils, soft clays and dispersive soils.
According to Diop, Stapelberg, Tegegn, Ngubelanga and Heath (2011), damage to structures in South Africa is commonly related to soil characteristics, with expansive and collapsing soils causing the most problems.

After identification of all constraints associated with soil variability and problematic soils, the designer has the choice of accepting the limitations imposed by the properties of the in-situ soils, or improving the properties as a means of fulfilling the design criteria (Lee, White and Ingles, 1983). When the choice of improving soil properties is considered, soil stabilization, soil reinforcement and other ground improvement techniques are central issues in many projects wherein land is scarce, good quality materials are in short supply, and at developed sites where the existing soil conditions must be improved.

Expansive soil, one of the most challenging soils, is found on all continents and creates a worldwide problem. The problems caused by expansive soils are considered as natural hazards that pose challenges to civil engineers, construction firms, and owners (Al-Rawas and Goosen, 2006). In the United States of America, expansive soils are reported to cause

more damage to structures, particularly light buildings and pavements than any other hazard, including earthquakes and floods (Jones and Holtz, 1973).

The problems associated with expansive soils were not recognized by soil engineers until the latter part of 1930 and the associated damages were attributed to shoddy construction and settlement of foundations (Chen, 1975). Even after recognition of such problematic soils in different countries, related damages were still observed in underdeveloped countries because buildings were constructed without any knowledge of the occurrence of expansive soils, mainly due to a lack of historical evidence (Al-Rawas and Goosen, 2006). With the rapid development in infrastructure, many research projects have been carried out and the problems associated with expansive soils have become more evident. As a result, appropriate measures were taken and treatment techniques were developed to minimise the damages caused by expansive soils.

One of the treatment procedures used for expansive soil stabilization is the use of chemical additives including among others lime, cement and fly ash. According to Nelson and Miller (1992), this technique has been used successfully on many projects to minimize swelling, reduce the soil plasticity and improve soil workability as well as soil strength. The stabilization of expansive soil with lime consists mainly of three phenomena namely water absorption and chemical binding during hydration of unslaked lime (CaO) into slaked lime ($Ca(OH)_2$) with an increase of temperature, pozzolanic reactions between lime and clay minerals which enhance the mechanical strength of soils and cation exchange between additive and clay particles (Segui, Aubert, Husson and Measson, 2011).

In additional to traditional chemical additives used to stabilize soils, other alternatives have been an important subject of research. Within this framework, Khalid, Mukri, Kamarudin and Arshad (2012) conducted a study to investigate the potential use of paper mill waste to stabilize clay soil. During their study, a waste paper sludge ash (abbreviated by WPSA or WSA) from combustion of waste paper recycling factories in Malaysia was used. The chemical composition analysis of the ash, using X-rays Fluorescence (XRF), has shown that lime content was about 62.39% by weight. This lime content of WPSA was also confirmed by the study conducted by Segui et al. (2011) where the mineralogical characterisation of WPSA showed that the residue was containing hydraulic minerals such as lime (CaO), mayenite ($Ca_{12}Al_{14}O_{33}$) and α'-Ca_2SiO_4 in addition to gehlenite, the main mineral. Further, Zhou, Smith, and Sego (2000) found that various wastes generated from paper and pulp

making contain lime. Based on the fact that commercial lime has been successfully used to stabilize expansive soil, the lime-rich paper mill wastes can be used to enhance the properties of expansive soil.

Landfilling is one of the methods mostly used for pulp and paper mill ash residuals disposal (Elliott and Mahmood, 2006). This disposal technique has its drawbacks such as holding back the land from other beneficial uses and the possibility of leading to contamination of ground water and soil. In addition to that, engineered landfills and long haulage distances to environmentally acceptable disposal places may cause huge disposal expenses to the factories. For all those reasons, utilization of paper mill waste is anticipated to be very advantageous, both economically and environmentally (Bujulu, Sorta, Priol and Emdal, 2007).

1.2. RATIONALE

Over the years, the decisions made by the builders had been influenced by ground characteristics and in some cases the locations, heights and configurations of some structures and facilities were dictated to a certain degree by the anticipated behaviour of the ground (Munfakh, 1997). However this alternative consisting in relocating facilities or changing their configurations could not always be applied to some projects including infrastructure projects such as highways, railways, airfields, etc. requiring large areas of land. Quite often, such projects are executed over areas covered with soils which are not suitable for such purpose. In order to overcome the limitations imposed by the properties of in-situ soils, various ground improvement techniques were developed to curtail the problems of poor or difficult ground such as expansive soils, collapsible soils, sensitive clays and dispersive clays.

Due to the large extent and problems caused by expansive soils, many techniques, to deal with these problems, have been developed and applied successfully. One of such techniques is the chemical stabilization with lime.

In addition to the problems associated with expansive soils encountered worldwide, the management of industrial wastes is also another issue experienced by all countries of the planet. According to Likon and Trebše (2012),

> the management of wastes, in particular of industrial waste, in an economically and environmentally acceptable manner is one of the most critical issues facing modern industry,

mainly due to the increased difficulties in properly locating disposal works and complying with even more stringent environmental quality requirements imposed by legislation.

The success of this study should become one of the solutions for problems caused by industrial waste management.

1.3.PROBLEM STATEMENT

Expansive soils cause damage to structures, particularly light buildings and transportation facilities, in many countries. According to Nelson and Miller (1992), the estimates of damage caused by expansive soils contribute much to the burden that natural hazards place on the economy. An appropriate, cost effective technique to improve those soils can be of great benefit to both public and private developers dealing with ground with expansive soils.

While builders may experience financial challenges associated with difficult soils to meet design requirements, pulp and paper industries face tremendous problems in eliminating waste, which is in the form of sludge, wood, and reject material. For some industries, incineration is seen as the most effective solution; nonetheless, this unfortunately leaves large quantities of residues such as fly ash and bottom ash for which the disposal may become a formidable problem. In order to handle this issue, many studies have been carried out to assess the effective use of waste from paper mill for construction (Mochizuki Yoshino, Saito and Ogata, 2004; Zhou et al., 2000).

This present study aimed at assessing the effect of paper mill waste on expansive soil properties namely volume change and strength. The effective application of that waste to improve the properties of expansive soils would lighten the large financial burden placed on both constructors by dealing with expansive soils on one side and pulp and paper producers by waste handling on the other side.

1.4. RESEARCH OBJECTIVE

Many techniques have been applied successfully to stabilize expansive soils and to remediate the problems caused by those problematic soils. On the other hand, even though many tests to identify expansive soils and associated problems have been developed, more occurrences may be expected especially in developing countries where adequate investigations have not yet been conducted. The new occurrences of expansive soils together with the constant land development will probably lead to the increase of problematic soil stabilization cost.

Therefore, the development of new techniques to deal with those engineering challenging soils especially by using local materials currently considered as wastes would be more beneficial to both construction and waste producing industries. The new techniques and methods must comply with technical, practical, economical and environmental rules for successful application.

As mentioned above, lime has been applied successfully to stabilize expansive soils; however various paper mill wastes, currently considered as wastes, have been proven to contain high percentage of lime (see background) which may produce the same effects on expansive soils as the lime itself. The damage caused by expansive soil is due to swelling-shrinking behaviour of the material.

The main objective of this study was to investigate the effect of paper mill waste ash on the expansiveness of expansive soils. The soil strength improvement resulting from the stabilization of swelling soil with paper mill ash was also studied due to the role of soil strength in all civil engineering works.

To achieve this main objective, the following was done:

- Classification of the studied swelling clays
- Testing of the untreated soil for physical and some engineering properties assessment
- Investigation of the effect of paper mill ash on index properties and some engineering properties of expansive soils
- Determination of optimum percentage of paper mill ash for expansive strength enhancement based on compressive strength.
- Investigation of the influence of the optimum or selected paper mill ash content on some physical and engineering properties of expansive soils

The same tests were conducted on both non stabilized and stabilized expansive soil to assess the effect of paper mill ash on its properties.

1.5. LAYOUT OF THE STUDY

This study is structured as stated below:

Chapter 1 consists of introduction. It states the background to the study, the reasons of carrying out this research, the problem statement and the objective of the study.

Chapter 2 reviews the literature related to expansive soils and their stabilization. It focuses on the problems associated with these problem soils and alternative solutions using chemical additives technique. In addition, some application cases of paper mill ash for various engineering purposes are mentioned with focus on soil treatment.

Chapter 3 describes the methodology followed to achieve the objective of the study. The tests to be performed during this research to meet the objectives of this study are also discussed. Various tests are carried out on expansive soils before stabilization and then repeated on stabilized expansive soils.

Chapter 4 focuses on the analysis of the obtained test results as well as their discussion. The results of non-stabilized soils are compared with the results after stabilization.

Chapter 5 illustrates the beneficial use of paper mill ash in soil treatment, through two design examples using the results obtained in Chapter 5.

Chapter 6 provides overview of the effect of paper mill ash on properties of expansive soils and the conclusions. In this chapter, the recommendations resulting from the comparison of obtained results and the objectives of the study are mentioned. For beneficial use of the material currently considered as a waste, recommendations for further researches were made.

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CHAPTER 2 LITERATURE REVIEW

2.1. INTRODUCTION

Geotechnical engineers often experience problems related to construction sites for which soil properties are unable to meet the required project specifications. The majority of these problems are as a result of some problematic soils such as expansive soils, collapsible soils, dispersive soils and soft clays. Consequently, many techniques have been developed and applied to improve on site soils in order to meet the intended purpose of the site development.

This literature review focuses on the problems caused by expansive soils, various common techniques applied for its stabilization and some emerging techniques of using paper industry waste for soil treatment. This section also reviews the identification of those soils, factors influencing the swelling and shrinking potential as well as the associated problems.

As indicated by the appellation, expansive soils are soils which have a potential for swelling or shrinking due to variation of moisture content and this leads to heave and settlement problems. Many definitions have been given of expansive soils with a common factor of significant volume change due to existing conditions. Patrick and Snethen (1976) defined expansive materials as natural earth materials which, because of their inherent and environmental characteristics, undergo volume change due to changes in environmental conditions. According to Steinberg (1998), expansive soils are soils which react with significant volume change due to moisture content variations. This section reviews the identification of those soils, factors influencing the swelling and shrinking potential as well as the associated problems.

Ground improvement is a key and ancient tool which allows the designer to deal with in-situ soils which don’t fulfill the design requirements. Although various techniques have been applied to improve expansive soils, there is need to introduce new techniques and materials to reduce the burden put on builders by problematic soils.

2.2. ORIGIN OF EXPANSIVE MATERIALS

Expansive soils also termed as swelling clays or shrinking soils or expansive shales, exist in various forms and may consist of clays or shales or other minerals (Steinberg, 1998). These materials absorb water and expand due to increase in moisture content and inversely shrink when the moisture content decreases by drying out.

Expansive materials may exist in various forms. Based on their physical characteristics and in geologic sense, expansive materials may be classified into three categories viz. expansive argillaceous rocks, argillaceous sediments and argillaceous soils (Snethen, Townsend, Johnson, Patrick and Vedros, 1975). The common characteristic of all those expansive materials is that they have high clay-size particles content responsible for volume change, thus the term argillaceous (Snethen et al., 1975). "Argillaceous rocks refer to relatively hard, indurated clay shales or clay stones which have been buried, consolidated, and at least partially cemented while argillaceous sediments include those materials of Mesozoic age or younger that have not been sufficiently buried, consolidated, or cemented to be included in the category of rocks" (Snethen et al., 1975). The distinction between the first two classes is not distinct such that for some materials it is difficult to differentiate the two classes (Snethen et al., 1975). The argillaceous soils result from the weathering of the two first classes and they are also referred to as residual soils (Snethen et al., 1975).

Drawing a clear distinction between expansive rocks and sediments proves to be a daunting task and it is for this reason that Patrick and Snethen (1976) classified expansive materials into two categories namely rocks and soils which are either transported or residual. As for other transported soils, transported expansive soils are transported from the place of origin by various agencies such as wind, water, ice, gravity, etc. while residual expansive soils remain in position at the place of origin after being formed by weathering of rocks. "Transported expansive soils are, geologically, fine-grained, unlithified, argillaceous sediments having expansive properties whereas residual expansive soils are the product of in situ weathering of parent material which may or may not have any expansive properties itself" (Patrick and Snethen, 1976). The expansive behaviour of those residual soils may result from parent material or weathering process which led to their formation (Snethen et al., 1975).

This part of the literature study centres on expansive soils without distinction on their origin; the emphasis is put on the expansiveness of the soil not on the process leading to expansive soil.

Donaldson (1969 cited in Chen, 1975) affirmed that expansive soils can result from the weathering of two parent materials, viz. basic igneous and sedimentary rocks. The feldspar and pyroxene minerals from basic igneous rocks such as basalts found in India, dolerite sills and dykes in the central region of South Africa and, the gabbros and norities found in Transvaal, west of Pretoria North, have led to the formation of montmorillonite and other

secondary minerals (Donaldson, 1969 cited in Chen, 1975). Moreover, montmorillonite contained in sedimentary rocks leads to the formation of expansive soils by physical weathering (Donaldson, 1969 cited in Chen, 1975). Other rocks which are associated with expansive soils are bedrock shale found in North America, marls and limestones found in Israel and the shale of the Ecca series in South Africa.

In addition to weathering, Snethen et al. (1975) found that diagenetic alteration of pre-existing minerals and hydrothermal alteration lead individually or in combination to the formation of expansive materials. Some examples of expansive materials resulting from weathering and diagenesis considered to be more important are mentioned here below (Snethen et al., 1975):

- The weathering of volcanic ash or primary silicate minerals such as feldspars, pyroxenes, or amphiboles will lead to the formation of montmorillonite provided that the bases and silica are retained during the weathering process. The formation of expansive materials are accelerated by some factors such as absence of sufficient water to leach out present bases, loss of water by high evaporation as well as resistance to water flowing through original materials, which prevent bases from being drained away from the materials.
- By diagenesis, montmorillonite also forms from the devitrification of sediments from volcanic ash particles or shards which are chemically intermediate between rhyolite and basalt in terms of composition rich in silica and bases as well. These particles rich in bases and silica are characterized by chemical instability which, most of the time, leads to the formation of montmorillonite.

Although the two phenomena viz. diagenesis and weathering consist both of chemical and physical processes with groundwater as the main factor, their slight difference is based on the depth of alteration where the first happens in depth while the second occurs within few feet of the soil surface (Snethen et al., 1975).

Snethen et al. (1975) found that some diagenetic factors which have contributed to the formation of montmorillonite from volcanic ash may also cause its destruction with time and burial. Those factors are overburden pressure, temperature change in burial depth, pore solution chemistry and duration of high pressure exposition. For that, the montmorillonite structure may, with time and burial conditions, change to illlite like structure.

2.3.EXPANSIVE SOIL MINERALOGY

2.3.1. Overview of Mineral composition

The characterization of soils requires the study of different soil parameters including particle sizes and consistency limits. However, those parameters don't suffice to describe the behaviour of natural soils which exhibit a great variability in terms of engineering properties and they have to be complemented by other properties. Reddi and Inyang (2000) found that the mineralogy plays a very important role in soil characterization, particularly clays. The mineral composition of soil highly depends on the composition of parent rock from which soils are formed. Soils are the weathering products of rocks where the first stage of weathering leads to the formation of sands and silts which then undergo weathering to form clays. For that reason, sand and silts are probably comprised of some original or primary minerals from parent rocks, more or less unchanged while clays will contain secondary minerals resulted from weathering of less resistant primary minerals (Reddy and Inyang, 2000). Some primary and secondary minerals are given in Table 2.1 and their relative proportions in soils are illustrated with Figure 2.1.

Table 2.1. Examples of primary and secondary minerals (Reddy and Inyang, 2000)

Primary minerals	Secondary minerals
Quartz (SiO_2)	Kaolinite $[Si_4]Al_4O_{10}(OH)_8$
Feldspar $(Na,K)AlO_2[SiO_2]_3$	Illite $(K,H_2O)_2(Si)_8(Al,Mg,Fe)_{4,6}O_{20}(OH)_4$
Mica $K_2Al_2O_5[Si_2O_5]_3Al_4(OH)_4$	Chlorite $(OH)_4(SiAl)_8(Mg,Fe)_6O_{20}$
Pyroxene $(Ca,Mg,Fe,Ti,Al)(Si,Al)O_3$	Montmorillonite $Si_8Al_4O_{20}(OH)_4.nH_2O$
Olivine $(Mg,Fe)_2SiO_4$	Gypsum $CaSO_4.2H_2O$

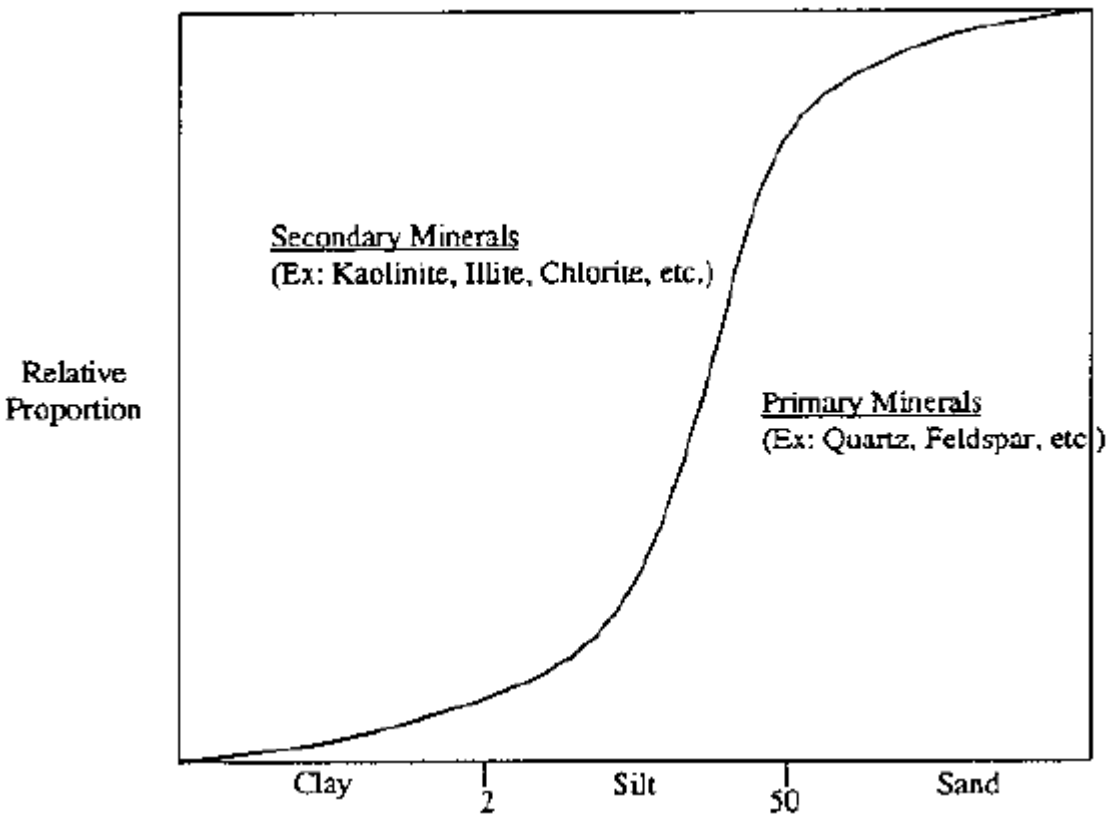


Figure 2.1: Relative proportions of primary and secondary minerals in soils (Reddy and Inyang, 2000)

Naturally, minerals form in compliance with the principle of minimum energy known as law of economics regularly observed in creation and three conditions which govern their formation with the minimum energy possible are (Reddy and Inyang, 2000):

1. The formation proceeds incorporating all the available atoms; obviously, the atoms that are most available in nature will occur frequently in the mineral structure.
2. The atoms forming the mineral should follow a geometric pattern which repeats itself in space.
3. The formation always strives for a neutrality of electronic charge, thus resulting in the alternation of anions and cations.

The fulfilment of the first condition is illustrated in Tables 2.1 and 2.2. The Table 2.1 shows that oxygen, silicon and aluminium are abundant in minerals and these elements are also the most abundant in the earth's crust. The second and the third conditions are simultaneously illustrated with the fact that the elements are arranged to achieve the charge neutrality not in terms of filling holes created by the bigger elements. For all those reasons, the oxygen which is the most abundant anion in the earth's crust in terms of volume and weight is also abundant

in element arrangement to neutralize the cations also abundant in earth's crust (Reddy and Inyang, 2000). The success of cations to participate in mineral structure will depend on the individual radii of ions as well as their capacity to fulfil the third condition of charge neutrality.

Table 2.2: Percent Weight and Volume in the Earth's Crust, Valence, Ionic Radii, and Radius Ratios (with Respect to Oxygen) of the Most Abundant Elements in the Earth's Crust (Reddy and Inyang, 2000).

Element	Weight (%)	Volume (%)	Valence	Ionic radius (Å)	Radius ratio
O	46.6	93.8	−2	1.32	—
Si	27.7	0.9	+4	0.31–0.39	0.23–0.30
Al	8.1	0.5	+3	0.45–0.79	0.34–0.60
Fe	5.0	0.4	+2	0.67–0.82	0.51–0.62
Mg	2.1	0.3	+2	0.78–0.89	0.59–0.68
Na	2.8	1.3	+1	0.98	0.74
Ca	3.6	1.0	+2	1.06–0.17	0.80–0.89
K	2.6	1.8	+1	1.33	1.0

The relative arrangement of different elements is determined by the radius ratio defined as the ratio of the radius of cation to the radius of anion and the number of anions around the cation is directly proportional to this radius ratio. According to Reddy and Inyang (2000), two kinds of arrangements result from the radius ratios mentioned in Table 2.2 namely tetrahedral type and octahedral type. Tetrahedral structure which requires at least a radius ratio of 0.22 consists of four anions around a cation whereas octahedral structure requires a minimum radius ratio of 0.41 with six anions around a cation. Therefore, silicon has got a great chance to form a tetrahedral structure while aluminum, iron and others will make octahedral structure. Most minerals are formed based on these arrangements and the schematic illustrations are given in Figure 2.2.

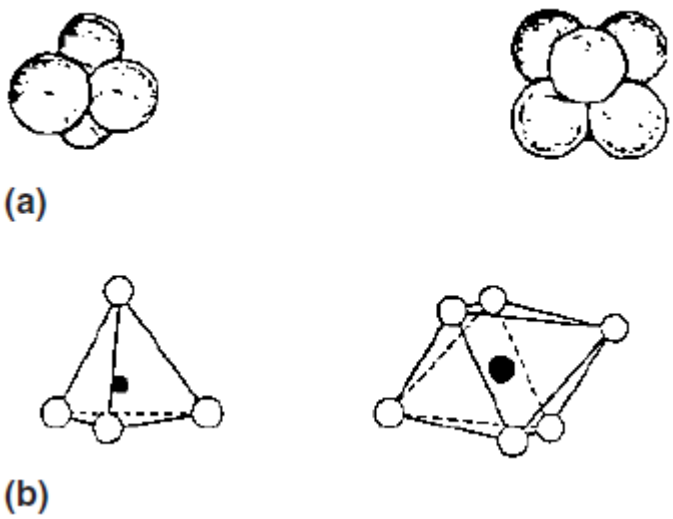


Figure 2.2: Tetrahedral and octahedral packing arrangements: (a) actual coordination, and (b) idealization (Reddy and Inyang, 2000).

It's observable that tetrahedral structure and octahedral structure will leave net negative charge of -4 and -10 respectively which requires the arrangement of similar basic units to fulfil the principle of charge neutrality. This arrangement of basic units, which may be in both vertical and horizontal directions, leads to the formation of layers with a net charge tending to zero by a sharing of the oxygens with the surrounding tetrahedra or octahedra. The two types of layers are schematically illustrated in figure 2.3 with symbolic notations.

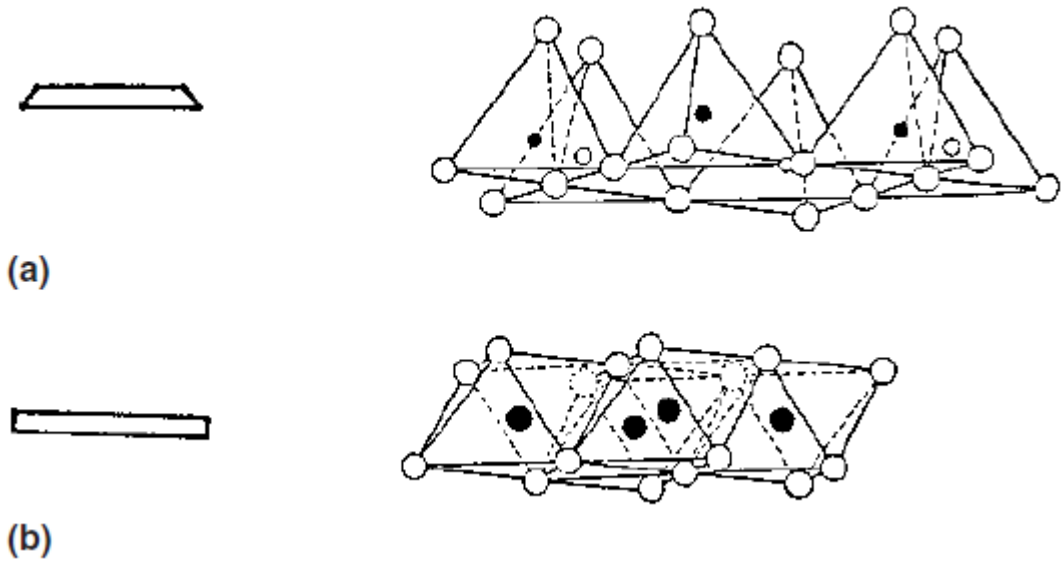


Figure 2.3: (a) Tetrahedral and (b) octahedral layers with symbolic notations (Reddy and Inyang, 2000)

The association of layers requires chemical bonding between them and the ionic bonds consisting in sharing ions mostly develop between tetrahedral and octahedral layers. This association of layers consisting of sharing anions at the interface leads to the formation of two main units. The first one is represented as 1:1 and is formed by a stacking together between one octahedral layer and one tetrahedral layer and the other one is represented as 2:1. It consists of two tetrahedral layers with a sngle octahedral layer sandwiched between them (Reddy and Inyang, 2000). The majority of the minerals forming clays are of these kinds of layer associations.

2.3.2. Clay minerals

Many classification systems described clay minerals as particles which have an effective diameter of two microns or less (Chen, 1975). However, it has been found that all particles whose diameter is less than two microns are not necessarily clay minerals (Sarsby, 2000). For this, the grain size doesn't suffice to describe clay mineral; hence the necessity to study the mineralogical composition which is probably the essential property of fine-grained soils (Chen 1975). Das (2010) defined clay minerals as aluminosilicate compounds resulting from the combination of two basic units namely silica tetrahedron and aluminium octahedron.

Each tetrahedral unit is formed by a silicon atom surrounded by four oxygen atoms and the combination of identical units results in the formation of a silica sheet in which neighbouring tetrahedra share three oxygen atoms at the base of each tetrahedron (Das, 2010). On the other hand, an octahedron consists of an aluminum atom surrounded by six hydroxyls. The combination of octahedral units leads to the formation of octahedral sheet also called gibbsite sheet. When the aluminium atom is replaced by the magnesium, the octahedral sheet is named a brucite sheet (Das, 2010). Those basic units and their combination are illustrated in Figure 2.4.

According to Chen (1975), Sarsby (2000) and Das (2010), there exist a number of clay minerals among which the important ones are kaolinite, illite and montmorillonite. All of them consist of a combination of silica-gibbsite sheets and are characterized by high specific surface i.e. the surface area per unit mass. According to Snethen et al. (1975), "the small grain size and resulting large surface are due to the clay mineral's origin by weathering or diagenetic alteration of pre-existing minerals".

Das (2010) describes the three important clay minerals as follows:

- Kaolinite is composed of a combination of repeating layers of elemental silica-gibbsite sheets held together by hydrogen bonding in 1:1 lattice. The estimated specific surface of kaolinite is $15m^2/g$.
- Illite is composed of a gibbsite sheet sandwiched between two silica sheets bonded together by potassium ions which are balanced by the negative charge left by the substitution of aluminum for some silicon in the tetrahedral sheets. This phenomenon consisting in substitution of an element by another one without alteration of crystalline form is called isomorphous substitution. The specific surface of illite particles is estimated to $80m^2/g$.
- Montmorillonite has the same structure as illite with attracted water molecules between layers instead of potassium ions. The specific surface is estimated to $800m^2/g$.

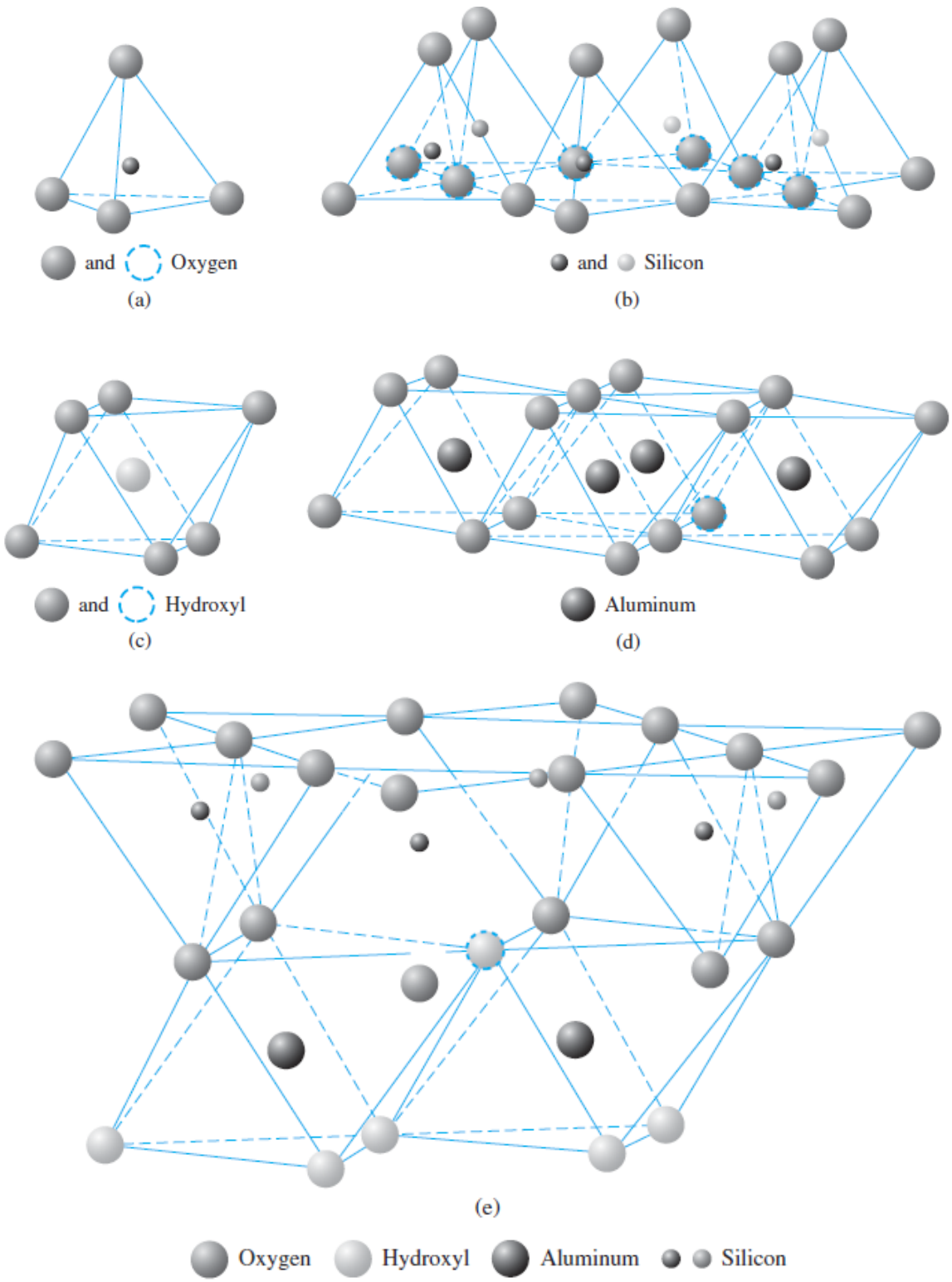


Figure 2.4: (a) Silica tetrahedron; (b) Silica sheet; (c) alumina octahedron; (d) octahedral (gibbsite) sheet; (e) elemental silica-gibbsite sheet (Das, 2010).

In addition to those important clay minerals, other common clay minerals are chlorite, halloysite, vermiculite and attapulgite (Das, 2010). The diagram in Figure 2.5 illustrates the structures of the three important clay minerals mentioned above.

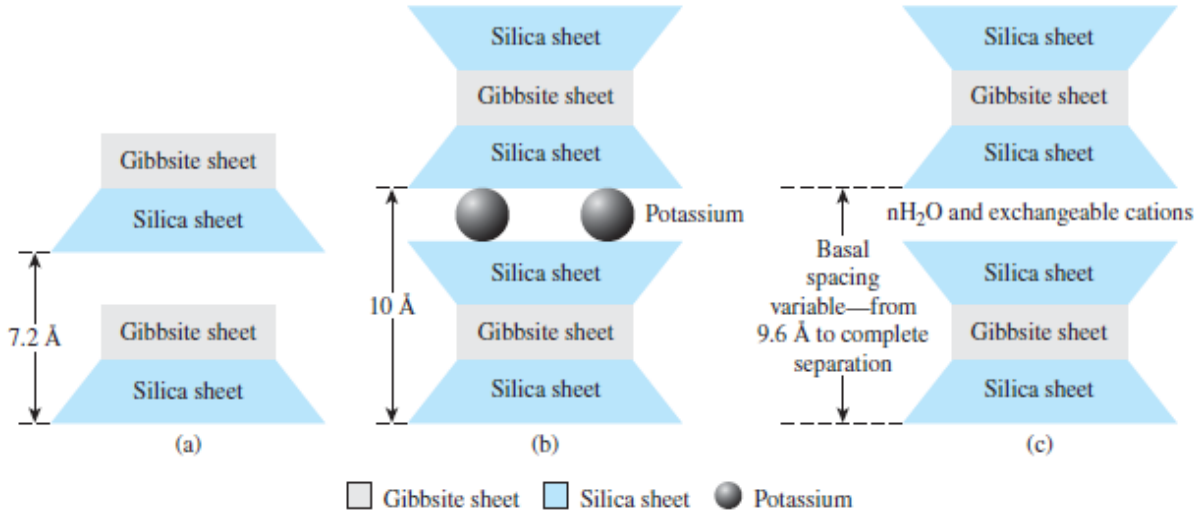


Figure 2.5: Diagram of the structures of (a) kaolinite; (b) illite; (c) montmorillonite (Das, 2010)

For Hunt (2007), "clays are hydrous aluminium silicates that are classified into a number of groups based on their crystal structure and chemistry". Based on their stability, Hunt (2007) divided clays into two main categories namely common groups comprised of kaolinite, halloysite, illite, and montmorillonite, and less common groups including vermiculite and chlorite which change quickly to other types. The structures of common clay minerals are illustrated below in Figure 2.6.

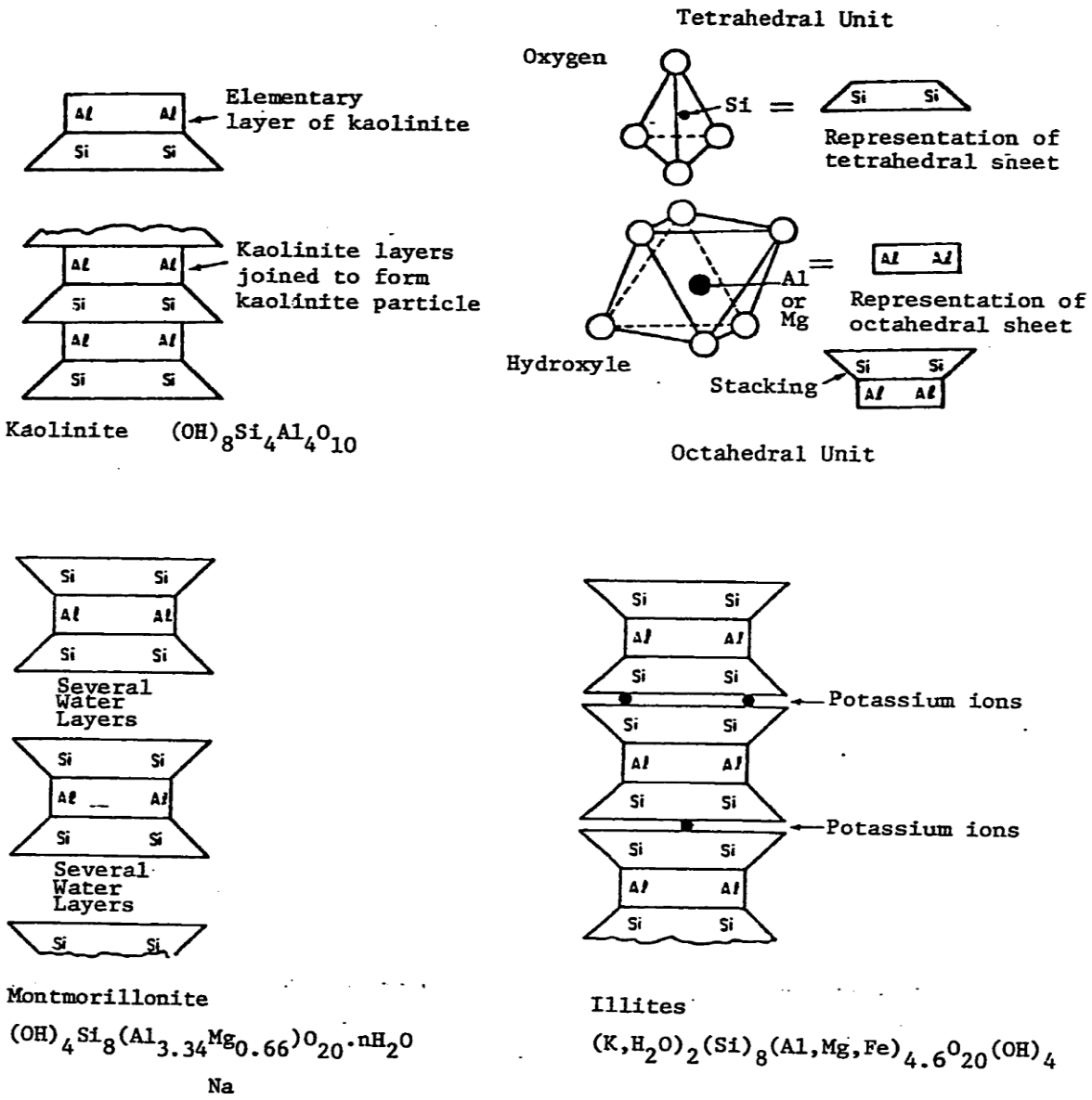


Figure 2.6: Structures of important clay minerals (Young, 1975, redrawn in Ahmad, 1988).

2.3.3. Some typical clay characteristics

Some typical characteristics of clay particles are their properties including cohesion, adhesion, plasticity, consistency and activity. Hunt (2007) describes those properties as follows:

- Cohesion of clay is defined as its ability to stick to itself. It results from a bond developing at the contact surfaces of clay particles, caused by electrochemical attraction forces. Two elements are responsible for the cohesion of clay particles namely the high specific surface of the particles (surface area per unit weight), and the

electrical charge on the basic silicate structure resulting from ionic substitutions in the crystal structure.

- Adhesion of clay is its ability to stick to another material to which it comes into contact.
- Plasticity is the ability of a material to undergo a change in shape without undergoing a change in volume, with its moisture content held constant.
- Consistency is defined as the relative ease with which a soil can be deformed. It involves cohesion and adhesion as well as its ability to resist deformation and rupture. In other words, soil consistency refers to the description of the resistance of a soil at various moisture contents to mechanical stresses or manipulations. The consistency of soils is generally described at three soil moisture levels: wet, moist and dry. With decreasing moisture content, clays pass from the fluid state (very soft) through a plastic state (firm), to a semisolid state, and finally to a hard brick-like state. The moisture contents at the transitions between these various states are defined by the Atterberg limits, which vary with the clay type and its purity.
- Activity is defined as the ratio of the plasticity index to the percent by weight finer than 2 μm.

Hunt (2007) and Mitchel (1976) respectivley summarized other characteristics of some clay minerals in tables 2.3 and 2.4 given here below.

Table 2.3. Characteristics of common clay minerals (Hunt, 2007).

Mineral	**Origin**	**Activity**	**Particles**
Kaolinite	Chemical weathering of feldspars Final decomposition of micas and pyroxenes in humid climates or well drained conditions Main constituents of clay soils in humid-temperate and humid-tropical regions	Low. Relatively stable material in the presence of waters	Platy but lumpy
Halloysite	Similar to kaolinite, but from feldspars and mica (primarily sialic rocks)	Low, except properties are radically altered by intense drying. Process not reversible	Elongated rod-like units, or hollow cylinders
Illite	Main constituent of many clay shales, often with montmorillonite	Intermediate between kaolinite and montmorillonite	This plates
Montmorillonite (smectite)	• Chemical decomposition of olivine (mafic rocks) • Partial decomposition of micas and pyroxene in low rainfall or poor drainage environment • Constituent of marine and clay shales • Alteration of rock during shearing by faulting • Volcanic dust	• Highly expansive and the most troublesome of the clay minerals in slopes and beneath foundations • Used as an impermeabilizing agent	Under electron microscope, appears as a mass of finely chopped lettuce leaves

Table 2.4. Characteristics of some clay minerals (Mitchell, 1976, summarized in Nelson and Miller, 1972).

Mineral group	Basal Spacing (Å)	Particle features	Interlayer bonding	Specific Surface (m^2/g)	Atterberg Limits			Activity (PI/% clay)
					LL (%)	PL (%)	SI (%)	
Kaolinites	14.4	Thick, stiff 6-sided flakes 0.1 to 4 x 0.005 to 2µm	Strong hydrogen bonds	10-20	30-100	25-40	25-29	0.38
Illites	10	Thin, stacked plates 0.003 to 0.1 x 1.0 to 10 µm	Strong potassium bonds	65-100	60-120	35-60	15-17	0.9
Montmorillonites	9.6	Thin, filmy, flakes > 10 Å x 1.0 to 10 µm	Very week van der Waals bonds	700-840	100-900	50-100	8.5-15	7.2

2.3.4. Expansive clay minerals

Some clay minerals such as montmorillonite, vermiculite, chlorite and mixed-layer combinations of these minerals between them or with other clay minerals, which exhibit significant volume change due to variations in moisture content, are termed expansive clay minerals (Snethen et al, 1975). On the other hand, kaolinite and illite, which don't exhibit the same amount of expansion as montmorillonite, vermiculite or chlorite, are classified as non-swelling clay minerals. Although kaolinite is considered as a non-swelling clay mineral, "halloysite, the tubular, hydrous member of kaolinite group may exhibit expansive properties" (Snethen et al, 1975).

According to Snethen et al (1975), the high volume change exhibited by clay minerals is due to three factors viz. the electrical charge characteristics, degree of crystallinity, and size. The charge deficiency of clay minerals which mainly results from the lattice substitution plays a great role in their volume change. The cations located in the proximity of clays minerals which are easily hydrated are attracted to the surfaces of clay particles to maintain electrical balance. The crystallinity has an influence on clay minerals swelling because the water adsorption may depend, to some extent, on the degree of crystallinity of minerals. The clay minerals size also has effect on their swelling because their small particles have considerable specific surface areas (the total surface area of the particles per unit of mass) which will be surrounded by water and this will have effect on the thickness of double-layer water. The increase in double layer thickness, which sometimes results in overlapping of diffuse double layers between particles, will lead to the development of repulsive forces between clay particles which result in swelling. In other words, the swelling potential increases with the augmentation of the diffuse double layer in thickness (Nelson and Miller, 1992).

Snethen et al. (1975) described the expansive clay minerals on the basis of those factors as follows:

- Montmorillonite: is normally a dioctahedral because two Al^{3+} ions are required to retain the neutrality of the sheet which has a 6-fold octahedral site. However, montmorillonite usually contains magnesium substituted for aluminum in the octahedral layer and becomes a trioctahedral which leads to the electrical charge deficiency. This electrical deficiency is then neutralized by cations present in the lattice such as Na^{+}, Ca^{++}, or Mg^{++} which occupy the space on and above the tetrahedral layers. Due to the neutrality of the octahedral layer, the ions are not

strongly held and can be easily exchanged and this will have a great effect on the swelling of montmorillonite. These attracted cations may be hydrated thus allowing water to enter the mineral layers causing expansion.

- Vermiculite: vermiculite as well as chlorite minerals are fine grained minerals which result from the weathering and diagenesis of preexisting mica, illite, chlorite and vermiculite. The swelling and cation exchange behaviour of vermiculite and chlorite is similar to montmorillonite which complicates the differentiation between their minerals and montmorillonite minerals. In addition to this, these minerals are usually mixed with montmorillonite or other expansive minerals hence the swelling behavior. The same as montimorillonite, vermiculite may present charge deficiencies which are balanced by cations such as Mg^{++}, Na^{+} or K^{+} ions and the hydration of these interlayer cations has influence on the expansiveness of these minerals.
- Chlorite: this mineral consists of mixed-layer interstratification of a di- or trioctahedral three layer clay. This mineral often exhibits the expansiveness in association with other clay minerals.
- Mixed-layer types: this category of clay minerals which consists of a mixed-layer of swelling minerals mentioned above and other clay minerals may exhibit an expansive behavior. The degree of expansion will depend on the amount of montmorillonite or other swelling clay minerals present in that association.

The insignificant expansiveness exhibited by kaolinte and illite is mainly due to the tight bond between their layers (Snethen et al, 1975). In kaolinite, the strong bond between octa- and tetrahedral layers is established by opposing electrical charges on the adjacent layers. In addition, the ionic substitution between layers is virtually nil which leads to the neutrality of kaolinite minerals and the absence of the compensation cations. The slight swelling of kaolinite minerals is generally caused by the adsorption of water on the individual grains (Snethen et al, 1975). For illite, the potassium ion present between layers within the hexagonal openings establishes the neutrality of the tetrahedral layer and is thus strongly bonded such that it's difficult to be exchanged (Snethen et al, 1975). As a result, significant water is prevented to enter the mineral layers.

The water absorption exhibited by illite, thus its tendency to swell, is greater than kaolinite because the interlayer bonding of potassium (K^{+}) ions in illite is less stable than the hydrogen bonding in kaolinite (Ahmad, 1988).

2.4. DISTRIBUTION OF EXPANSIVE SOILS

The studies have shown that, with the exception of polar continents, expansive soils are encountered on all other continents (Steinberg, 1998; Gourley, Newill and Schrein, 1993). In the following, a review of distribution of expansive soils worldwide as well as in South Africa, is conducted.

2.4.1. Distribution of expansive soils worldwide

The reported existences of expansive soils worldwide were summarized by Donaldson (1969, cited in Chen, 1975) in figure 2.7 here below. The cases of expansive soils have been reported from many countries and different continents (Donaldson, 1969 cited in Chen, 1975) as shown in the following list.

Argentina	Cuba	Israel	Zimbabwe	U.S.A
Australia	Ethiopia	Iran	South Africa	Venezuela
Burma	Ghana	Mexico	Spain	
Canada	India	Morocco	Turkey	

According to Chen (1975), the extent of expansive soils was not accurate because in underdeveloped countries the problems associated with expansive soils may not have been recognized due to lack of related research. For this, Chen predicted that more regions with expansive soils will be discovered as the land development expands.

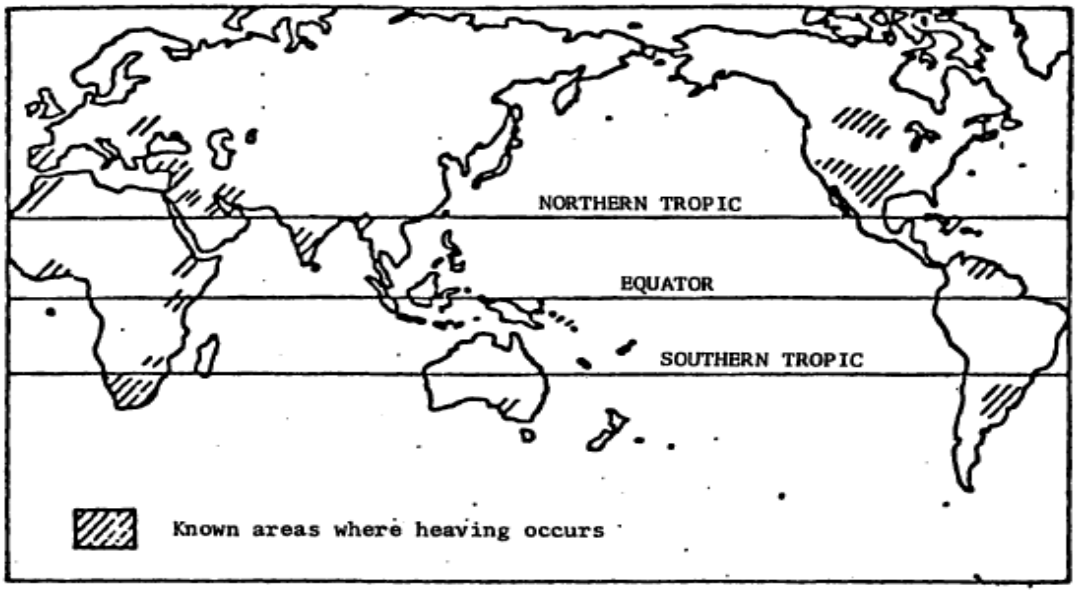


Figure 2.7: Distribution of reported instances of heaving (Donaldson, 1969, cited in Chen, 1975).

According to figure 2.7, the potentially expansive soils are confined to semi-arid regions of the tropical and temperate climate zones. They are abundant in areas where precipitations are relatively low compared to annual evapotranspiration. "This follows the theory that in semi-arid zones, the lack of leaching has aided to formation of montmorillonite" (Chen, 1975).

2.4.2. Distribution of expansive soils in South Africa

While the problems associated with expansive soils were recognized in the United States of America in the latter part of 1930, in South Africa, those problems were taken into consideration for the first time in 1950 and the first symposium on expansive soils was published in 1957 (Chen, 1975). Serious problems associated with heaving were observed in various areas of South Africa such as Leeuhof, Vereeniging, and Pretoria in Transvaal on fluvio-lacustrine deposits considered as source of swelling soils (Chen, 1975). Other foundation movement problems were recorded at Odendaalsrus in the Orange Free States Goldfields due to the presence of the Ecca shale largely found in South Africa (Chen, 1975). According to Williams, Pidgeon and Day (1985), expansive clays are widely found in South Africa. Diop et al. (2011) found that expansive soils in South Africa generally result from either basic igneous rocks or argillaceous sedimentary rocks. Basic igneous rocks which lead to the formation of expansive soils are among others norite from the Busveld igneous complex, dolerite of the Karoo supergroup, diabase and andesite from the Pretoria group and andesite from the Ventersdorp supergroup. On the other hand, the argillaceous rocks from which expansive soils are formed include Karoo group, the shales and mudrocks of the Dwyka, Ecca and Beaufort group (Diop et al., 2011). The Karoo group is considered as the main source of expansive soils in the Southern Africa and some heaving clays resulted from the weathering of shales and mudrocks are encountered in some areas of South Africa such as KwaZulu-Natal Province.

The occurrence of expansive soils in South Africa has been summarized by Diop et al. (2011) as follows:

- Soils originated from lava cover different parts of South Africa such many areas in Limpopo Province; area between Makhado, Alldays and Musina; a band all along the eastern borders of the Limpopo and Mpumalanga Provinces; the Mookhophong area and the Bela Bela, the northern part of the Eastern Cape Province;
- Residual norite soils known as black "turf" are encountered in the Onderstepoort to Rustenburg area and northwards towards Thabazimbi;
- Andesite and diabase soils are found in the Pretoria and Lydenburg areas;

- Soils originated from lava are also found in the south-eastern parts of the North West Province and in some areas south of Johannesburg;
- Expansive soils, formed from mudstone and/or shales known as Karoo mudrock and tillite, are found in almost all provinces. They are found in the western parts of the Northern Cape, northern parts of the Free State, eastern parts of Mpumalanga, western parts of Kwa-Zulu Natal, northern parts of the Eastern Cape and north-eastern parts of the Western cape;
- Soils derived from mudstone and/or shales are also encountered in other parts of the Eastern Cape Province namely the Port Elisabeth and Uitenhage area;
- Swelling soils formed from clayey sandstones and shale are found in Western Cape in the Malmesbury area;
- Soils resulting from the weathering of dolerite rock are found in forms of sills and dykes countrywide.

In addition to the formation of residual expansive soils from the in situ chemical weathering of basic igneous rocks and argillaceous rocks, Jennings and Brink (1978, cited in Williams et al., 1985) found that expansive soils may also result from transported soils. The transported expansive soils found in South Africa are described by Williams et al. (1985) as follows:

- Alluvial deposits composed of different materials which are transported by rivers, may exhibit swelling characteristics. One of these deposits is the Vereeniging type clay adjacent to the Vaal River;
- Expansive soils may also be found in lake, pan or subterranean pool in cavernous rock where they are transported by stream and are known as lacustrine deposit;
- Expansive soils like the black clays found in the Pretoria Moot result from the transported soil from local catchment and they may contain various types of active soils. They are known as gulleywash and their composition depends on catchment soil;
- Expansive hillwash also known as fine colluvium, results from the transportation of active material from various rocks such as basic igneous or sedimentary argillaceous rocks. Those soils composed of sheetwash, although found in some areas, are not common.

The Figure 2.8 illustrates the distribution of expansive clays and collapsible sand in South Africa (Williams et al., 1985).

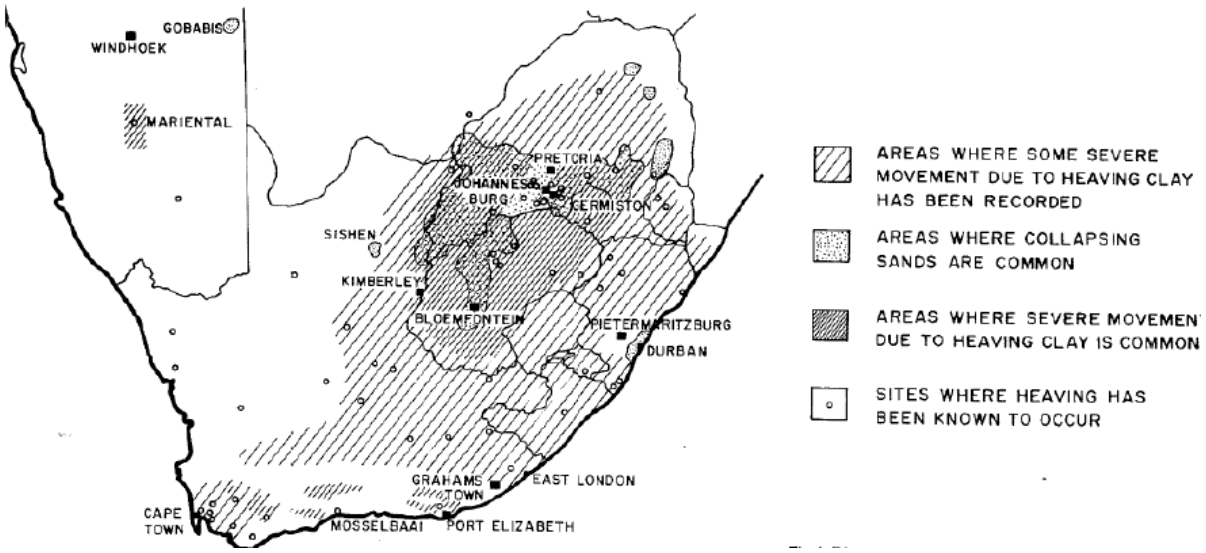


Figure 2.8: Distribution of expansive clays and collapsing sands (Williams et al., 1985).

2.5. IDENTIFICATION AND CLASSIFICATION OF SWELLING SOILS

2.5.1. Introduction

Like other problem soils, the identification of expansive soils is an important phase for every construction project. “Early identification of expansive soils, during the reconnaissance and preliminary stages of a project, is essential to allow for appropriate sampling, testing, and design in later stages” (Nelson and Miller, 1992). For that reason, prior to any other construction project phase, the site investigation should be conducted for identification of the site soil as a problem soil or not for appropriate measures. This identification is followed by soil sampling and preliminary tests which provide the key information for adequate and economic decision making and design.

2.5.2. Identification and categorization of expansive soils

Many tests have been developed to identify the expansive soils. Some of them are based on standard classification tests; whilst others are based on the soil mineralogy and chemistry (Nelson and Miller, 1992). Given the importance of identification phase of problem soils, a review of the identification tests was carried out and different tests used to identify the swelling potential of expansive soils are summarized in Table 2.5. In addition, a brief overview of them was also mentioned to make them a bit clear.

Table 2.5. Laboratory tests used in identification of expansive soils (Nelson and Miller, 1992)

Test	Reference	Properties Investigated	Parameters Determined
Atterberg limits	ASTM Standards 1991	Plasticity, consistency	
Liquid limit (LL)	ASTM D-4308	Upper limit water content of plasticity	PI = LL − PL = plasticity index
Plastic limit (PL)	ASTM D-4318	Lower limit water content of plasticity	$LI = \frac{w - LL}{LL - PL}$ = liquidity index
Shrinkage limit (SL)	ASTM D-427	Lower limit water content of soil shrinkage	R = shrinkage ratio L_s = linear shrinkage
Clay content	ASTM D-422	Distribution of fine-grained particle sizes	Percent finer than 2 μm
Mineralogical tests	Whittig (1964)	Mineralogy of clay particles	
X-ray diffraction	ASTM STP 479 (1970)	Characteristic crystal dimensions	Basal spacings
Differential thermal analysis	Barshad (1965)	Characteristic reactions to heat treatments	Area and amplitude of reaction peaks on thermograms
Electron microscopy	McCrone and Delly (1973)	Size and shape of clay particles	Visual record of particles
Cation-exchange capacity	Chapman (1965)	Charge deficiency and surface activity of clay particles	CEC (meq/100 g)
Free swell test	Holtz and Gibbs (1956)	Swell upon wetting of unconsolidated unconfined sample of air dried soil	Free swell = $(V_{wet} - V_{dry})/V_{dry} \times 100\%$
Potential volume change meter	Lambe (1960b)	One-dimensional swell and pressure of compacted, remolded sample under semi-strain controlled conditions	SI (swell index) (lb/ft^2) PVC (potential volume change)
Expansion index text	Uniform Building Code	One-dimensional swell under 1 psi surcharge of sample compacted to 50% saturation initially	Expansion index (EI)
California bearing ratio test	Yoder and Witczak (1975); Kassiff et al. (1969)	One-dimensional swell under surcharge pressure of compacted, remolded samples on partial wetting	Percent swell CBR (%)
Coefficient of linear extensibility (COLE) test	Brasher et al. (1966)	Linear strain of a natural soil clod when dried from 5 psi (33 kPa) to oven dry suction	COLE and LE (%)

2.5.2.1. Classification tests

According to Nelson and Miller (1992), classification tests such as grain size distribution, clay content and plasticity are used widely to identify and categorize expansive soils. The study carried out by Seed, Woodward and Lundgren (1962, cited in Chen, 1975), has demonstrated that the plasticity index is an important parameter to identify the swelling behaviour of most clays. Their study led to the establishment of the relationship between the plasticity index and the swell potential.

$$S = 60K(PI)^{2.44}$$

Where,

S = swell potential

K= $3.6x10^{-5}$, a constant and

PI = Plasticity Index (expressed as a percentage).

"The potential swell is defined as the percentage swell of a laterally confined sample which has soaked under a surcharge of one pound per square inch after being compacted to maximum density at optimum moisture content according to AASHO compaction test" (Chen, 1975). This relation between the Plasticity Index and the swelling behaviour of soils which applies to soils whose clay fraction is comprised between 8 and 65 percent can clearly be explained by the fact that both are influenced by the presence of water (Chen, 1975). The relation shows that the swell potential increases with the plasticity index.

In addition to this relation between the plasticity index and the swell potential, Skempton (1953, cited in Nelson and Miller, 1992) established another relation, used for swelling soils identification, the activity which falls between Atterberg limits and clay content.

$$Activity\ A_c = \frac{Plasticity\ Index}{\%\ by\ weight\ finer\ than\ 2\mu m}$$

The high activity shows the high potential of swelling.

Based on index properties, expansive soils have been classified into different categories by different researchers. The following section gives the overview of some studies results.

Based on classification tests carried out on undisturbed samples, Holtz and Gibbs (1954), classified soil expansion as shown in Table 2.6.

Table 2.6: Expansive soil classification based on colloid content, plasticity index and shrinkage limit (Holtz and Gibbs, 1954)

Data from Index Tests[a]			Probable Expansion (% Total Volume Change)	Degree of Expansion
Colloid Content (% minus 0.0001 mm)	Plasticity Index	Shrinkage Limit		
>28	>35	<11	>30	Very high
20–31	25–41	7–12	20–30	High
13–23	15–28	10–16	10–20	Medium
<15	<18	>15	<10	Low

A similar classification was established by Altmeyer (1955) but the parameter of clay fraction was suppressed because the hydrometer test used to determine that fraction was not considered as a routine laboratory test. The classification is shown in Table 2.7.

Table 2.7: Expansive soil classification based on shrinkage limit or linear shrinkage (Altmeyer, 1955).

Linear Shrinkage	SL (%)	Probable Swell (%)	Degree of Expansion
<5	>12	<0.5	Noncritical
5–8	10–12	0.5–1.5	Marginal
>8	<10	<1.5	Critical

Another simple method to identify and classify the expansive soils based on their plasticity index was established by Chen (1988, cited in Nelson and Miller, 1992). The results of his study are shown in Table 2.8.

Table 2.8: Expansive soil classification based on plasticity index (Chen, 1988).

Swelling potential	Plasticity Index
Low	0-15
Medium	10-35
High	20-55
Very high	35 and above

Apart from tables showing the relationship between the degree of expansion and the engineering index properties, some classification charts were also established. The chart in Figure 2.9, developed by Seed, Woodward and Lundgren (1962, cited in Nelson and Miller) shows the relationship between the activity of soils and the percent of clay fraction.

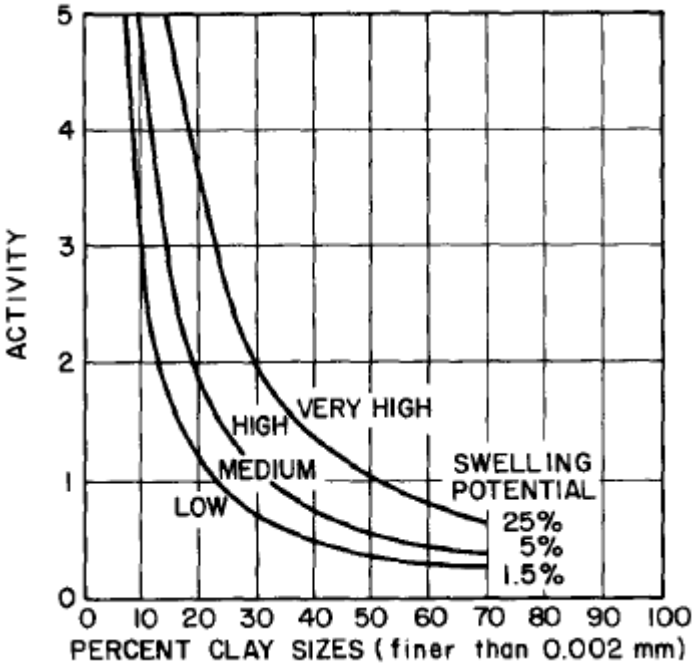


Figure 2.9: Classification chart for compacted clays based on activity and percent clay (Seed et al., 1962b, redrawn in Nelson and Miller, 1992 and in Murthy, 2007).

Another chart combining several soil properties has been developed by Holtz and Gibbs (1954) to show the relationship between the colloid content, plasticity index and shrinkage limit. Based on their experiments carried out on thirty eight samples, they found that this combination of these three parameters provide adequate indicators of the swelling characteristics of clays. The results of their research are shown in Figure 2.10.

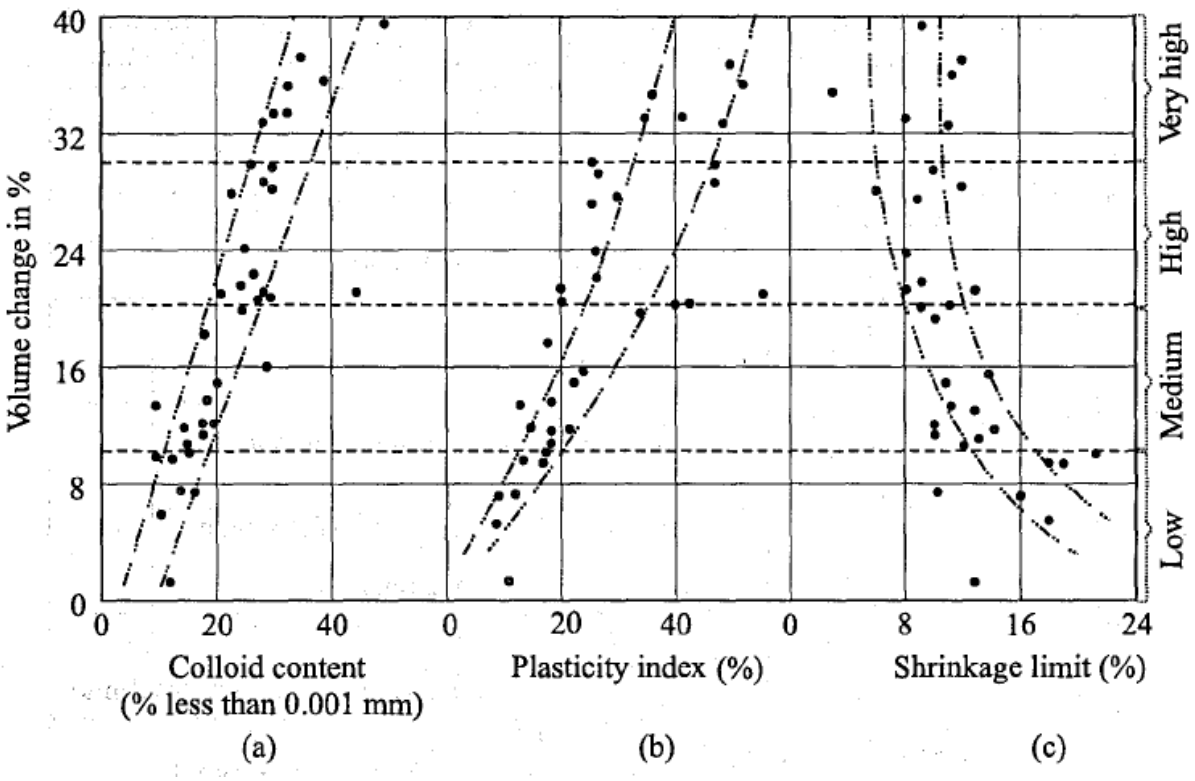


Figure 2.10: Relation of volume change to (a)colloid content, (b) plasticity index, and (c) shrinkage limit (air-dry to saturated condition under a load of 1lb per sq in) (Holtz and Gibbs, 1954).

Another method widely used in South Africa to determine the amount of heave caused by expansive soil is the double oedometer carried out on undisturbed samples (Van der Merwe, 1964). He nonetheless found that the method was mainly applied to big projects due to the expenses associated with it. In this regard he developed a simple chart based on the plasticity index (PI) and the soil fraction passing the 2μm sieve to determine the potential expansiveness of soils (Figure 2.11). According to him, the chart has been used successfully to assess the potential expansiveness, one of the two main factors which influence the amount of heave, namely potential expansiveness and moisture content under the structure.

According to the chart (Figure 2.11), soil is classified into very high, high, medium and low degrees of potential expansiveness according to its PI and clay fraction (% < 2μm). The plasticity index (P.I.) of the whole sample, also called gross PI for the total soil (Savage, 2007), is given by the product of the plasticity index and the fraction passing 425μm sieve. Nevertheless, the degree of expansion could not be separated from the soil moisture content because soil with low natural moisture content will exhibit higher volume change due to

moisture content variation whereas soil with high moisture content or where water-table is high will exhibit lower volume change.

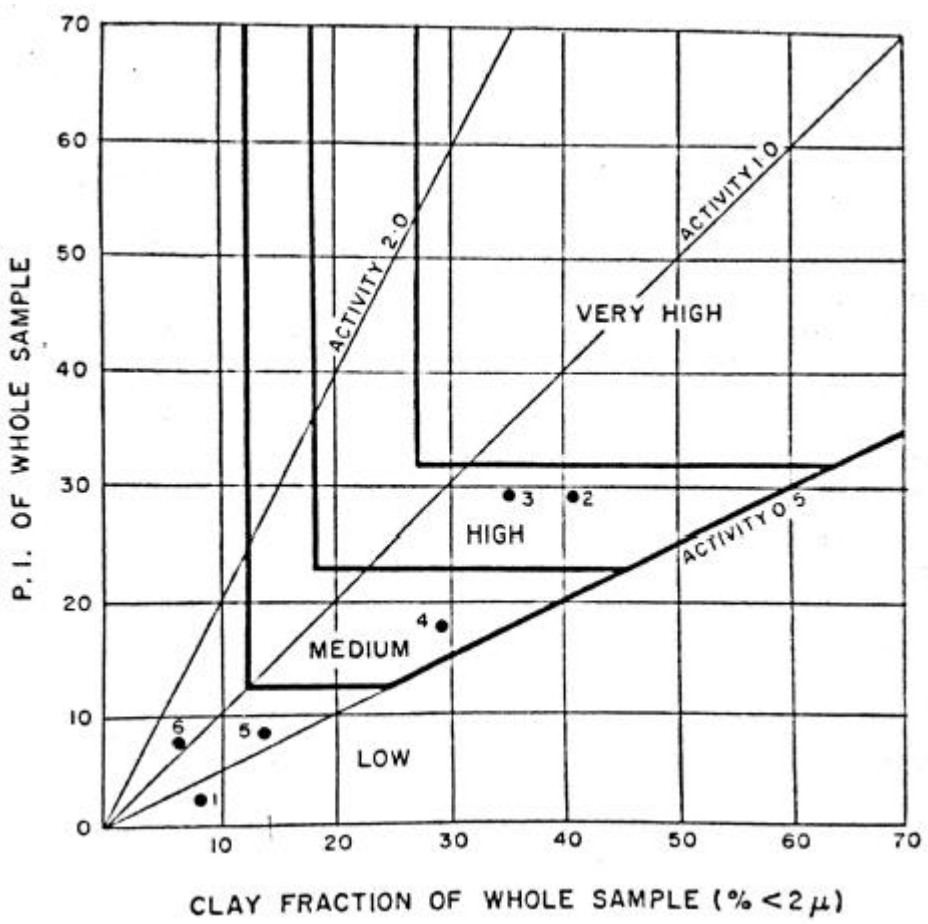


Figure 2.11. Determination of potential expansiveness of soils (Van der Merwe, 1964).

2.5.2.2.Mineralogical tests

The mineralogical composition has a great influence on the swelling behaviour of clay due to clay minerals' affinity with water. Nelson and Miller (1992) defined some methods used to study the mineralogy of clay. Those methods are:

- X-ray diffraction,
- Differential thermal analysis,
- Dye adsorption,
- Chemical analysis, and
- Electron microscopy resolution.

According to Chen (1975), contrary to the classification methods, these methods require specialised equipment as well as an expert for results interpretation. Among those methods, the X-Ray diffraction is widely used and the most appropriate to identify the clay minerals because the wavelengths of the X-rays and the atomic spacing planes of clay minerals crystals are comprised in the same range (Nelson and Miller, 1992).

2.5.2.3.Cation exchange capacity (CEC)

One of the typical characteristics of clay minerals is the presence of the negative charge on their surfaces. The quantity of the exchangeable cations required to balance that negative charge is called cation exchange capacity. CEC is expressed in milliequivalents per hundred grams of dry clay. The higher the CEC, the higher is the activity of clay particles, and then high swelling potential (Nelson and Miller, 1992).

2.5.2.4.Free swell

The free swell, defined as the ratio between the volume change and the initial volume, following the absorption of water, gives an indication of the expansiveness of clay soils (Nelson and Miller, 1992). According to Nelson and Miller (1992), the free swell is determined by putting a known volume of dry soil passing the No. 40 sieve (425µm) into a graduated cylinder, and filling it with water. After the soil has completely settled, the volume change is recorded and the free swell, expressed as a percentage, is determined. This test has been identified by Holtz and Gibbs (1954) as the simplest test which can provide satisfactory data for preliminary design. For them, an amount of dry soil of 10cm^3 passing the No. 40 sieve is poured into a 100 cm^3 cylinder slowly and the volume change is recorded after settlement. The free swell is given by the ratio between the change in volume and the initial volume expressed in percentage. Based on their research, Holtz and Gibbs (1954) found that the high swelling commercial bentonite has a free swell comprised between 1,200% and 2000%. They also found that soil with a free swell in the range of 100% may exhibit significant volume change when wetted under negligible loads whereas soils with a free swell below 50% were found to exhibit low volume change. However, based on the study carried out on the Texas clays, Dawson (1954) found that those soils, although their free swell is in the range of 50%, have caused significant trouble due to their expansive potential combined with climatic conditions. For that, Dawson recommended the consideration of climatic conditions of a given area when classifying the expansive soils.

Although this method has been used for a long time to determine free swell, Chen (2000) found that this test consisting of using a graduated cylinder to measure the swelling potential has been given up since it is not a standard method and recommended not to use it anymore.

The free swell can also be determined using the consolidometer. In this case, compacted soil is placed into a consolidometer ring under a seating pressure of 1kPa and then inundated. Following the absorption of water, the specimen will increase in height. The free swell, expressed as a percentage is given by the ratio between the change in height of the specimen and its initial height (ASTM D4546-08, 2008).

2.5.2.5.Potential volume change (PVC)

By means of a PVC meter, the swell pressure of a compacted soil can be determined both in the field and laboratory (Nelson and Miller, 1992). This method consists of placing in an oedometer ring the sample, compacted with a modified Proctor compactive effort and then inundated. This inundation results in swelling of the material against the proving ring of the PVC meter. From this method the swell index, reported as the pressure exerted on the proving ring, is determined and its correlation with PVC allows to qualitatively determine the ranges of PVC. The chart showing the correlation between the swell index and the PVC is shown in Figure 2.13. The main advantages of this method are its simplicity and standardization but it cannot be applied for design parameters for in situ soils since remolded samples are used (Nelson and Miller, 1992).

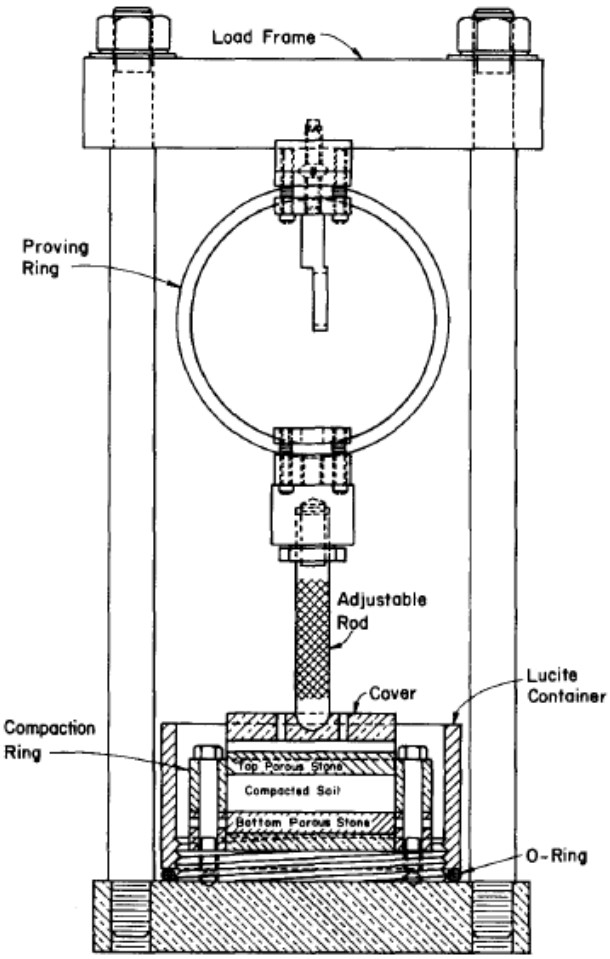


Figure 2.12 Potential volume change (PVC) apparatus (Lambe, 1996, redrawn from Nelson and Miller, 1992).

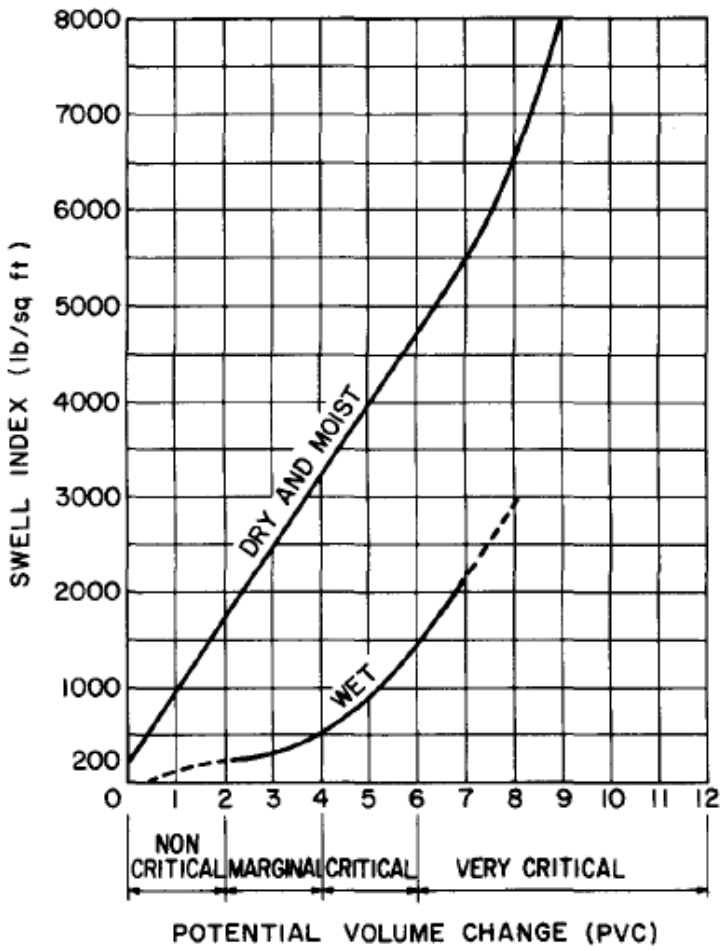


Figure 2.13. Swell index versus PVC (Lambe, 1960, redrawn from Nelson and Miller, 1992).

From the Figure 2.13, four categories of PVC have been established as shown in Table 2.9.

Table 2.9. Categories of potential volume change

PVC Rating	Category
Less than 2	Noncritical
2 to 4	Marginal
4 to 6	Critical
Greater than 6	Very Critical

2.5.2.6.Expansion Index test

This test applies on soil passing No 4 sieve (4.75mm) and is similar to PVC tests with the exception that it is conducted under constant surcharge (Nelson and Miller, 1992). In addition to this, before wetting the sample, it is brought to the degree of saturation

approximately equal to 50%. The expansion index, expressed to the near whole number, is given by the following equation (Nelson and Miller, 1992).

$EI=100x\Delta hxF$

Where Δh = percent swell and

F = fraction passing No. 4 sieve.

Different categories of expansion potential, established based on the expansion index, are shown in Table 2.10 (Nelson and Miller, 1992).

Table 2.10. Categories of expansion potential

Expansion index (EI)	Expansion potential
0-20	Very low
21-50	Low
51-90	Medium
91-130	High
>130	Very high

2.5.2.7.California Bearing Ratio (CBR)

This test, used widely in highway design, was developed to determine the penetration resistance of compacted soil to a piston of a loading machine. Most of the time, prior to conducting the CBR test, the specimen loaded with a surcharge approximately equal to the overburden pressure in the field, is soaked for 4 days and the change in volume of the specimen is recorded. Although the CBR test was not developed to determine the potential expansive behaviour, Kassif and others (1969, cited Nelson and Miller, 1992) established the relation between percent swell during CBR test and the plasticity index (PI) as well as the percent clay for various moisture contents. The disadvantage of this method is that the correlation between those parameters is specific to a particular soil.

2.5.2.8.Coefficient of linear extensibility (COLE)

"The COLE test determines the linear strain of an undisturbed, unconfined sample on drying from 5 psi (33 kPa) suction to oven dry suction (150,000 psi = 1000 MPa)" (Nelson and Miller, 1992). In other words, the coefficient of linear extensibility is the ratio of the difference between the moist length and dry length of a clod to its dry length. The COLE can also be estimated from the bulk densities as shown by the relation here below.

$COLE = \Delta L/\Delta L_D = (\gamma_{dD}/\gamma_{dM})^{0.33}-1$

Where $\Delta L/\Delta L_D$=linear strain relative to dry dimensions

γ_{dD} = dry density of oven dry sample

γ_{dM} = dry density of sample at 33 kPa

As discussed in the section related to clay minerals, the clay mineralogy plays a great role in the swelling behaviour of soil. The study carried out by the National Soil Survey Laboratory (1981, cited in Nelson and Miller, 1992) has shown that there exists a relationship between the linear extensibility and clay minerals. The relation between the clay mineralogy and the linear extensibility is shown in Table 2.11.

Table 2.11. Relationship between clay mineralogy and linear extensibility

Linear extensibility/Percent clay	Mineralogy
>0.15	Smectites (montmorillonite)
0.05-0.15	Illites
< 0.05	Kaolinites

2.6. FACTORS INFLUENCING SWELLING AND SHRINKING OF SOILS

As defined above, expansive soils are soils which swell or shrink subsequently to change in water content or suction. The resulting swell is a consequence of complex mechanism which involves a lot of intrinsic as well as environmental factors (Nelson an Miller, 1992 and Snethen et al., 1975). The intrinsic factors are related to soil properties whereas extrinsic factors are the result of the environmental influence on soils. According to Patrick and Snethen (1976), the intrinsic properties which affect the expansiveness of soils include type and amount of clay minerals, fabric, grain size distribution, cementation, and the extrinsic properties are climate, particularly amount and distribution of rainfall, topography, depth, soil moisture, fluctuation of ground water level, and the effects of man including compacted density, type of structure and the drainage features related to the structure. The intrinsic factors can, in their turn, be divided into microscale and macroscale factors (Nelson and Miller, 1992). The microscale factors include the mineralogical and chemical properties of soil while the macroscale factors are among others the engineering and physical properties of soils. In this regard, the swelling-shrinking behaviour of expansive material depends on both type of minerals and the chemistry of soil water on one hand; and, on the other hand, the physical and engineering properties, such as plasticity and density, will have great influence on soil volume change.

The soil properties and environmental factors involved in volume change of expansive soils as studied by Nelson and Miller are summarized in Tables 2.12 and 2.13.

Table 2.12: Soil properties that influence shrink-swell potential (Nelson and Miller, 1992)

Factor	Description	References
Clay mineralogy	Clay minerals which typically cause soil volume changes are *montmorillonites*, *vermiculites*, and some *mixed layer minerals*. Illites and Kaolinites are infrequently expansive, but can cause volume changes when particle sizes are extremely fine (less than a few tenths of a micron)	Grim (1968); Mitchell (1973, 1976); Snethen et al. (1977)
Soil water chemistry	Swelling is repressed by increased cation concentration and increased cation valence.For example, Mg^{2+} cations in the soil water would result in less swelling than Na^{+} cations	Mitchell (1976)
Soil suction	Soil suction is an independent effective stress variable, represented by the negative pore pressure in unsaturated soils. Soil suction is relaed to saturation, gravity, pore size and shape, surface tension, and electrical and chemical characteristics of the soil particles and water (see Chapter 4)	Snethen (1980); Fredlund and Morgenstern (1977); Johnson (1973); Olsen and Langfelder (1965); Aitchison et al. (1965)
Plasticity	In general, soils that exhibit plastic behavior over wide ranges of moisture content and that have high liquid limits have greater potential for swelling and shrinking. Plasticity is an *indicator* of swell potential	See Section 3.1
Soil structure and fabric	Flocculated clays tend to be more expansive than dispersed clays. Cemented particles reduce swell. Fabric and structure are altered by compaction at higher water content or remolding. Kneading compaction has been shown to create dispersed structures with lower swell potential than soils statically compacted at lower water contents	Johnson and Snethen (1978); Seed et al. (1962a).
Dry density	Higher densities usually indicate closer particle spacings, which may mean greater repulsive forces between particles and larger swelling potential	Chen (1973); Komornik and David (1969); Uppal (1965)

Table 2.13: Environmental conditions that influence shrink-swell potential (Nelson and Miller, 1992)

Factor	Description	References
1. Initial moisture condition	A desiccated expansive soil will have a higher affinity for water, or higher suction, than the same soil at higher water content, lower suction. Conversely, a wet soil profile will lose water more readily on exposure to drying influences, and shrink more than a relatively dry initial profile. The initial soil suction must be considered in conjunction with the expected range of final suction conditions	
2. Moisture variations	Changes in moisture in the active zone near the upper part of the profile primarily define heave. It is in those layers that the widest variation in moisture and volume change will occur.	Johnson (1969)
2.1 Climate	Amount and variation of precipitation and evapotranspiration greatly influence the moisture availability and depth of seasonal moisture fluctuation. Greatest seasonal heave occurs in semiarid climates that have pronounced, short wet periods	Holland and Lawrence (1980)
2.2 Groundwater	Shallow water tables provide a source of moisture and fluctuating water tables contribute to moisture	
2.3 Drainage and manmade water sources	Surface drainage features, such as ponding around a poorly graded house foundation, provide sources of water at the surface; leaky plumbing can give the soil access to water at greater depth	Krazynski (1980); Donaldson (1965)
2.4 Vegetation	Trees, shrubs, and grasses deplete moisture from the soil through transpiration, and cause the soil to be differentially wetted in areas of varying vegetation	Buckley (1974)

Table 2.13 Continued

2.5 Permeability	Soils with higher permeabilities, particularly due to fissures and cracks in the field soil mass, allow faster migration of water and promote faster rates of swell	Wise and Hudson (1971); De Bruijn (1965)
2.6 Temperature	Increasing temperatures cause moisture to diffuse to cooler areas beneath pavements and buildings	Johnson and Stroman (1976); Hamilton (1969)
3. Stress conditions		
3.1 Stress history	An overconsolidated soil is more expansive than the same soil at the same void ratio, but normally consolidated. Swell pressures can increase on aging of compacted clays, but amount of swell under light loading has been shown to be unaffected by aging. Repeated wetting and drying tend to reduce swell in laboratory samples, but after a certain number of wetting–drying cycles, swell is unaffected	Mitchell (1976); Kassiff and Baker (1971)
3.2 In situ conditions	The initial stress state in a soil must be estimated in order to evaluate the probable consequences of loading the soil mass and/or altering the moisture environment therein. The initial effective stresses can be roughly determined through sampling and testing in a laboratory, or by making in situ measurements and observations	
3.3 Loading	Magnitude of surcharge load determines the amount of volume change that will occur for a given moisture content and density. An externally applied load acts to balance interparticle repulsive forces and reduces swell	Holtz (1959)
3.4 Soil profile	The thickness and location of potentially expansive layers in the profile considerably influence potential movement. Gratest movement will occur in profiles that have expansive clays extending from the surface to depths below the active zone. Less movement will occur if expansive soil is overlain by nonexpansive material or overlies bedrock at a shallow depth	Holland and Lawrence (1980)

2.7. PROBLEMS ASSOCIATED WITH EXPANSIVE SOILS

As mentioned above in the section of expansive soils distribution, problems caused by expansive soils are encountered worldwide. According to Steinberg (1998), great efforts were made in the commencement of twentieth century to understand expansive soils, with the introduction of civil engineering discipline known as Soil Mechanics by Dr. Karl Terzaghi, professor of engineering practice at Harvard University. The first studies focussed on the destructive behaviour of expansive materials and their impact on highways as well as the efforts to be made to control that impact. As a result of those studies, it was found that some damages which were attributed to structural design shortcomings or faulty construction were caused by expansive soils (Steinberg, 1998).

Expansive soil causes damage because of its volume change due to water content fluctuation. When the swelling soil absorbs water, it swells and results in creation of swelling pressure defined as the maximum force per unit area that needs to be placed over a swelling soil to prevent its volume increase (Sabat, 2012). The swelling pressure is the most useful indicator of the damaging potential of a swelling soil (Sabat, 2012). If the swelling pressure of a soil is very high the damaging potential of the soil will also be very elevated and vice versa.

The most affected constructions are the light civil engineering structures namely low-rise buildings, retaining structures, channel and reservoirs linings, utility lines, railways and roads (Popescu, 1979; Kayabali and Demir, 2011; Bozbey and Garaisayev, 2010). The vulnerability of the light constructions to damage is due to the fact that they are not heavy and strong enough to counterbalance the swelling pressure (Bell, 2002). The first indicator of heaving problem, are the cracks in building walls and pavements (Williams, Pidgeon and Day, 1985). Consequently, in buildings, the doors start to jam, the distortions appear on floors and the ceilings are pulled away from walls. However, not all cracks in wall buildings result from swelling soils. Chen (1975) has found that, the vertical and horizontal cracks in basement walls are mainly due to earth pressure on walls and sometimes caused by earth moving equipment such as backhoe, if not designed and monitored carefully. He also found that, the cracks caused by swelling movement are most of the time diagonally located below windows and above doors.

According to Williams, Pidgeon and Day (1985), “the difficulties arise, not because of a lack of adequate engineering solutions but largely owing to a failure to recognize, during the early

stages of the project, the potential problem or the magnitude of movement that can be expected". The most serious challenge is that, it is possible to determine how much soil will swell through laboratory testing but not how much the soil volume will change beneath a particular structure (Holtz and Gibbs, 1954). This situation becomes more complicated when the climatic conditions have to be taken into consideration for swelling and shrinking prediction.

The damages caused by the heaving of expansive soils have been reported from different countries. Jones and Holtz (1973) estimated the damage caused by expansive soil in USA to 2, 255 millions of dollars per year. The affected structures and corresponding average annual loss are mentioned in table 2.14 here below.

Table 2.14: Estimated damage attributed to expansive soils (Jones and Holtz, 1973).

Construction category	Estimated average annual loss (millions of dollars)
Single-family homes	300
Commercial buildings	360
Multi-story buildings	80
Walks, drives, parking areas	110
Highway and streets	1,140
Underground utilities and service	100
Airports	40
Urban landslides	25
Others	100
TOTAL ($)	**2,255**

The study carried out by Bell (2002) showed that the cost of problems caused by expansive soils in the United States is evaluated to more than $2 billion each year. He also mentioned that this cost associated to expansive soil damage in most of the time is twice the cost of flood damage or of damage caused by landslides and, more than twenty times the cost of earthquake. Williams et al. (1985) found that the damage caused by expansive soil to buildings and light structures, in the USA and the United Kingdom, is estimated to $1,000 million and $100 million, respectively. In South Africa, a study carried out by Kitcher (1980 cited in Williams et al., 1985) estimated the cost of repair of houses built or to be built on

swelling clay, over a period of 20 years 1980 to 2000, to R1, 000 million. In China, the damage to railroad has been estimated at 12million U.S. dollars per year (Steinberg, 1998).

2.8. TREATMENT OF EXPANSIVE SOILS

2.8.1. Introduction

Most of the time, the in-situ soils don't meet the project requirements. Ample solutions to this constraint are available and the choice depends on various parameters such as the type of facility, cost implication, etc. Some of those solutions are among others:

1. relocation of the facility,
2. replacement of the challenging in-situ soil,
3. special design to overcome the suspected problems and
4. soil treatment, etc.

The first solution can only apply on some projects requiring a relatively small space and its choice depends on the availability of another site. For instance this alternative may be judged impractical for many linear facilities such as highways, pipelines, tunnels; and large projects such as airports, harbour which require large spaces. For this reason, due to the land demand which increases exponentially day after day, the solution cannot be considered adequate. The second solution may be found impractical or uneconomical depending of the extent of the problem and the project. The third and fourth solutions are mostly considered in practice. In this section, these solutions were briefly discussed with emphasis on soil treatment.

Due to global distribution of expansive soils and their increasing cost implications, many technical methods have been developed for their stabilization before and after construction of buildings on expansive soils (Nelson and Miller, 1992; Chen, 1975). In addition to preliminary investigations and evaluation of soil properties which must be undertaken to identify the extent of the problem and the probable factors which may influence the shrink-swell of the soil, other parameters are required to choose the appropriate soil treatment method. Those parameters are among others; the project nature and size to evaluate admissible limits and working conditions; cost comparison of different alternative methods, etc. (Nelson and Miller, 1992). The treatment methods applied for expansive soil stabilization to support pavements may differ from those applied for house foundations.

Many methods have been applied successfully to stabilize expansive soils before as well as after construction of various facilities. Those methods include among others soil replacement, surcharge loading, pre-wetting, moisture fluctuation control and chemical treatment (Nelson

and Miller, 1992). Another common method to minimise the swelling potential is the compaction control (Chen, 1975). Those methods are discussed below and this study is extended to the use of paper mill waste for expansive soil treatment.

2.8.2. Removal and replacement by suitable soil

The replacement of expansive soils by non-swelling soils is one of the techniques applied to control the problems caused by swelling and shrinking behaviour of expansive soils. Depending on the thickness of the expansive soils layer and the extent of the project, the expansive strata may be entirely or partially removed. However, in some cases, the expansive soil layer is too deep to economically allow complete removal and replacement (Nelson and Miller, 1992).

Before applying this technique, appropriate swell tests have to be carried out to estimate the expected potential heave. The depth to which nonexpansive backfill should be placed will be dictated by the weight necessary to control the expected uplift movement due to heaving and the ability of the backfill to mitigate differential displacements (Nelson and Miller, 1992). Although no definitive guidelines for the required depth to minimise or cancel heaving have been specified, some authors have suggested some values. Chen (1975) recommended that the thickness of the fill should be at least 3 feet, although the thickness of 5 feet was found to be more suitable. Jones and Gibbs (1973) have also found that the ground movement caused by expansive soil is higher near the ground surface and decreases to almost nothing between 5 and 30ft underground. These observations should be taken into account when considering this technique.

In addition to the depth, the designer has also to take into consideration the type of fill to be used for replacement. Chen (1975) has found that the material ranging from well graded-gravel (GW) to clayey sand (SC), according to the Unified Soil Classification system (USCS) may fulfil the first requirement of non-expansiveness. However, some materials within this range such as the clean granular material and the fill material with a high portion of plastic clay may cause the problem of water ingress in underlain layer and swelling respectively (Chen, 1975). In fact, when the swelling clay is partially replaced, the use of permeable layer may cause ingress of water into underlain swelling layer and cause surface heave or shrinkage. Based on experience, Chen pointed out that the criteria given in Table 2.15 have been used successfully.

Table 2.15: Criteria for expansive soil replacement fill (Chen, 1975)

Liquid limit (of replacement fill), percent	Percent minus No 200 sieve (for replacement fill)
Greater than 50	15-30
30-50	10-40
Less than 30	5-50

Another parameter that the designer has to consider carefully is the determination of swelling characteristics on undisturbed samples because the removal of soil will affect the swelling behaviour due to change in stress history (Nelson and Miller, 1992).

2.8.3. Loading

By this method, a surcharge is laid over swelling clay to restrain its heaving. The limitation of this method is that it can only be used for low swelling clay, up to a swelling pressure of 25 kPa (Nelson and Miller, 1992).

2.8.4. Prewetting

As mentioned above in the section of the factors influencing the volume change of expansive soils, the moisture content is one of the factors influencing the behavior of an expansive soil. By this method, water is added to expansive soil and allowed to swell prior to facility construction until no further significant swelling is observed (Nelson and Miller). The study conducted by Dawson (1953) has shown that when a swelling soil is allowed to expand a bit, the swelling pressure will be significantly reduced. He also found that the swelling reduces as the compacting moisture content increases (figure 2.14). For that, for a structure or pavement, the swelling pressure can be reduced by allowing the soil with moisture content greater than the optimum to expand by water absorption before placing the structure or pavement. The method becomes effective when the high moisture is kept more or less constant preventing absorption of additional high amount of water thus preventing significant volume change. To achieve the goal of this method, two techniques can be used. The first method consists of constructing dykes or small earth berms surrounding the construction before flooding whereas the other method consists of placing the foundation used as a dike to flood the construction area over prewetted foundation trenches (Chen, 1975). According to Poor (1979, cited in Nelson and Miller, 1992), the expansive soils prewetted to moisture contents 2 to 3% above the plastic limit lead to satisfactory results.

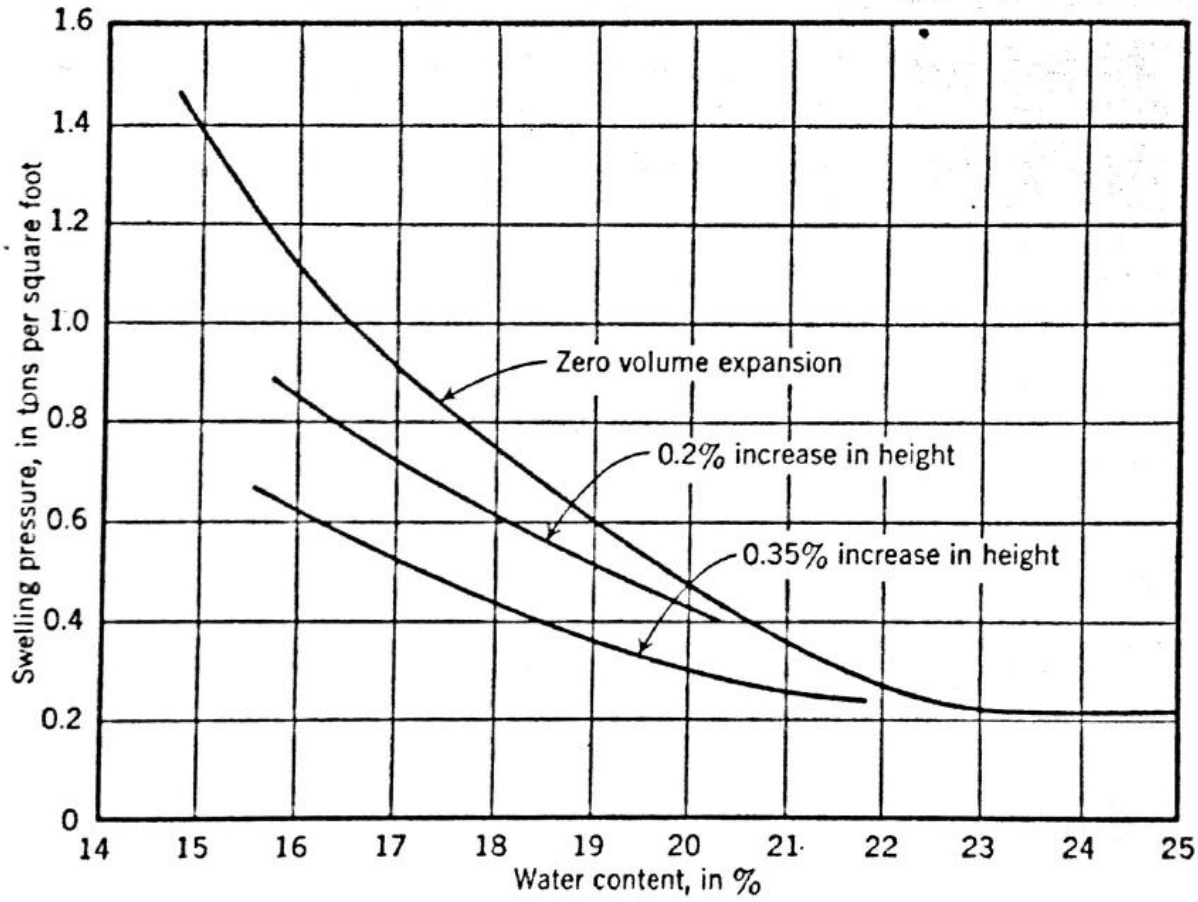


Figure 2.14: Relation of swelling pressure to moisture content (Dawson, 1953).

Even if this method has been used successfully in various projects, some drawbacks, which may complicate its successful use, have been observed. These drawbacks include, but are not limited to the following explained by Chen, 1995and Nelson and Miller, 1992:

- Long period for complete wetting, usually several years due to low hydraulic permeability characteristic to expansive soils,
- Great loss of soil strength, particularly stiff clay, in saturated conditions, which may lead to structure failure
- Difficulty in making uniform distribution of moisture content under the structure particularly in stiff clay,
- Unfavorable working conditions due to wet soils

According to Chen (1975), this method is more suitable to construction of slabs than construction of building foundations. Furthermore, this technique is applicable to soils with low to moderate swelling pressure (Nelson and Miller, 1992).

2.8.5. Moisture control

Soil expansion problems are primarily caused by the moisture content change within swelling soils (Nelson and Miller, 1992; Snethen et al., 1975). For that, it is obvious that any technique which will allow keeping the moisture content more or less constant can minimise the volume change. In this regards, moisture barriers have been used to control the moisture content in the subsoil by limiting water access and minimising moisture content change (Nelson and Miller, 1992; Snethen et al., 1975). The moisture barriers may either be vertical or horizontal and may consist of sheets or asphalt impermeable membranes and concrete (Nelson and Miller, 1992). Some layouts of moisture barriers are illustrated in Figures 2.15, 2.16 and 2.17, here below.

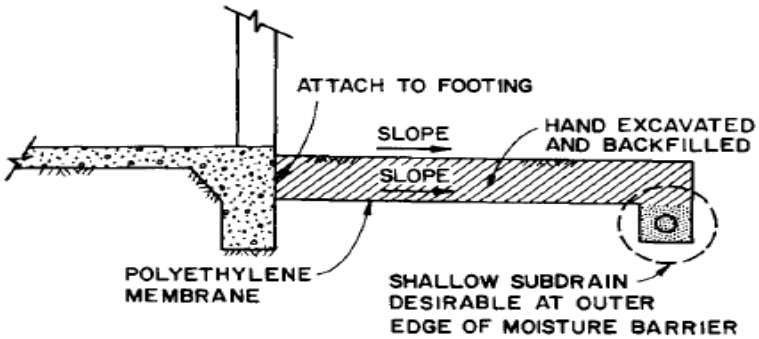
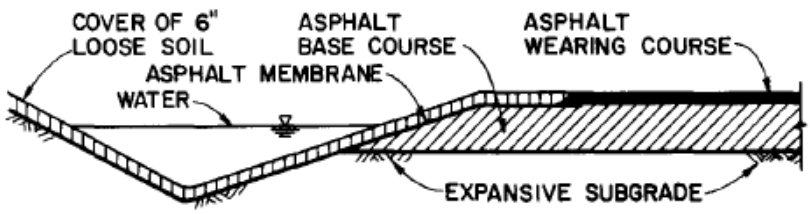
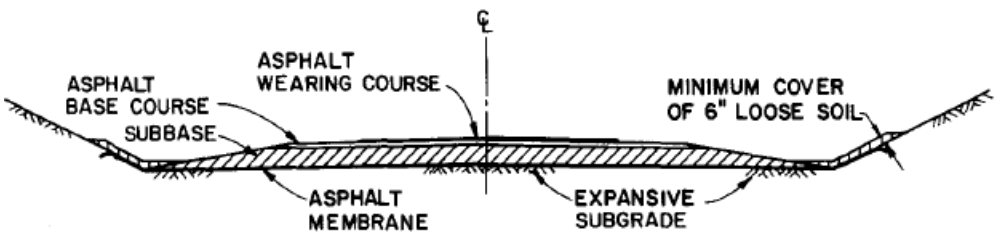


Figure 2.15: Typical detail of horizontal membrane (Nelson and Miller, 1992).



(a) Full-depth asphalt pavement with lined ditches



(b) Continuous asphalt membrane applied to subgrade and ditches

Figure 2.16: Typical construction of moisture barriers used to minimize subgrade moisture variations from surface infiltration (Snethen, 1979, cited in Nelson and Miller, 1992; Brakey, 1970, cited in Snethen et al. 1975)

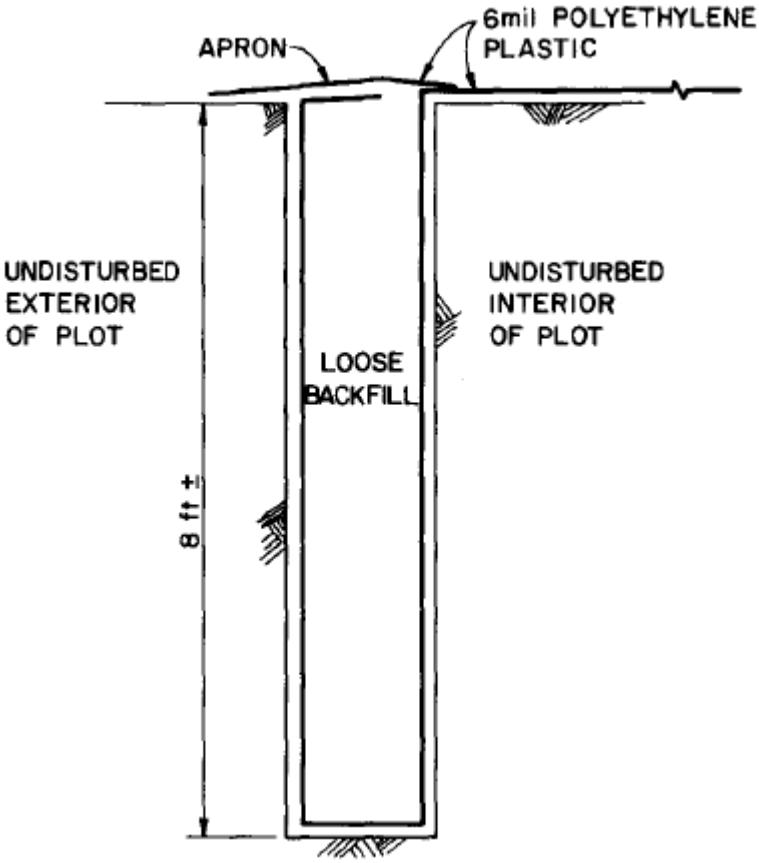


Figure 2.17: Vertical moisture barriers used in experiments by Goode (1982, cited in Nelson and Miller, 1992).

2.8.6. Compaction control

The volume change of a compacted swelling clay subseqent to additional moisture content depends on various parameters such as compacted dry density and initial moisture content (Chen, 1975). The study carried out by Holtz and Gibbs (1954) has shown that the low expansion of expansive soil can be achieved when the soil density is lower than the one obtained by standard compaction and the moisture content is close to or higher to the optimum moisture content (OMC) as shown on Figure 2.18.

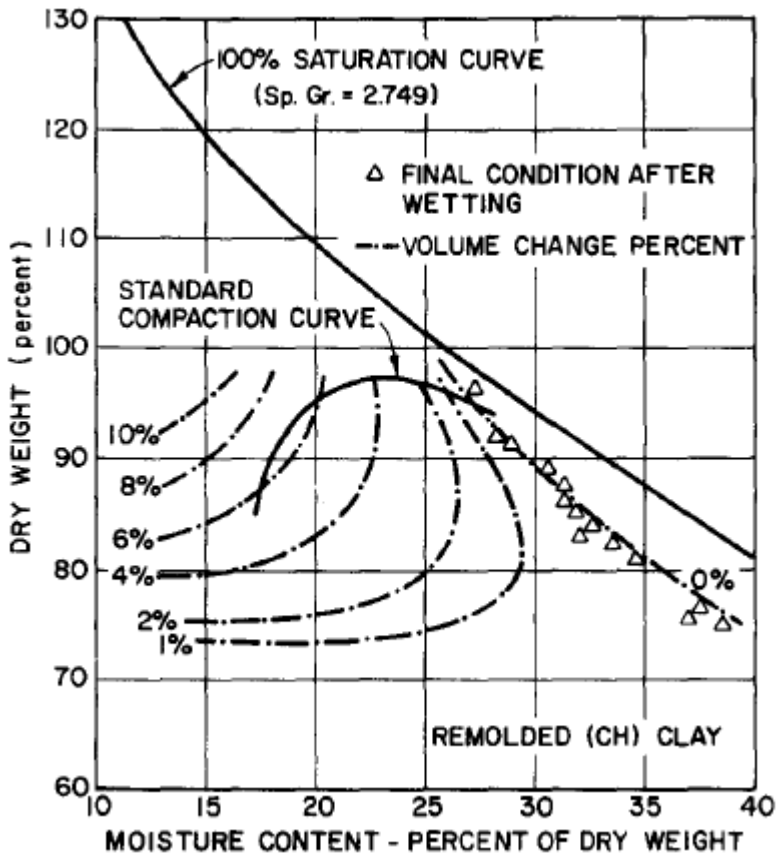


Figure 2.18. Percentage of expansion for various placement conditions for soil under a load of 1 lb per sq in. (Holtz and Gibbs, 1954).

As described in Figure 2.18, each dash line shows different points which exhibit equal expansion under different moisture content and density conditions. The figure shows that at densities less than those obtained in standard compaction, the expansion curve tends to 0% expansion curve at the moisture content greater than the OMC. The figure also shows that the expansion becomes lower and lower as the moisture content increases because the dash curve approaches the 0% expansion curve. However, this approach may not be applied because of the dificulties which may raise from wet soil. For that reason, the expansion may be mitigated by a combination of moderately high moisture content and low density control (Holtz and Gibbs, 1954). As it is not easy to compact stiff clay at water contents 4 to 5 percent above optimum, Chen (1975) and Nelson and Miller (1992) have found that the compaction of swelling clays at moisture contents slightly above their natural moisture content and at low density can provide good results in terms of expansion control. However, the designer must take into consideration problems which may be caused by low density such as low bearing capacity, etc.

2.8.7. Chemical stabilization

2.8.7.1. Introduction

The soil stabilization is defined as "chemical or mechanical treatment designed to increase or maintain the stability of a mass of soil or otherwise to improve its engineering properties" (ASTM D 653-11). Mechanical stabilization is achieved by compaction or blending soil with other materials to improve their properties without chemical reaction (Grogan, Weiss and Rollings, 1999). The addition of aggregates in fine soil to improve gradation and the mixture of soil with asphalt cement are some of the cases of mechanical stabilization. For instance, when asphalt is added to soil, the soil particles are coated then improving the soil properties (Grogan et al., 1999). On the other hand, chemical stabilization occurs when the added materials either react with soil or react on their own. These reactions lead to the formation of new cementing compounds which play a great role in soil stabilization (Grogan et al., 1999). When lime is added to clay soil, it reacts with clay whereas when cement is added to soil, it reacts on its own. Many additives such as salt, polymers, surfactants, cement, lime, fly ash, are used to stabilize expansive soils (Al-Mukhtar, Khattab and Alcover, 2012). In the following sections, some chemical admixtures used in stabilization of expansive soil are briefly discussed.

2.8.7.2. Lime stabilization

Generally, the stabilization of clay aims at controlling the volume change of the material, improving its workability and strength (Petry and Little, 2002). The stabilization of expansive soil has been achieved successfully by using lime to minimise the volume change and improve the workability as well as the plasticity (Nelson and Miller, 1992). In addition to this, the lime also improves the strength of expansive soils (U.S. Department of the Interior Bureau of Reclamation, 1998; Office of Geotechnical Engineering, 2008). While the improvement of workability follows immediately after the addition of lime to expansive soils, the enhancement of strength increases with time when the added lime reacts with silica and alumina from soil to form silicates and aluminates (U.S. Department of the Interior Bureau of Reclamation, 1998).

The U.S. Department of the Interior Bureau of Reclamation (1998) also found that the workability enhancement results from various reactions which follow the addition of lime to expansive soils namely the reduction in soil plasticity, agglomeration of colloids to form

larger particles, disintegration of clay clods and the reduction in moisture content due to hydration of lime which results in soil drying.

The effects of lime on Atterberg limits are illustrated with Figure 2.19.

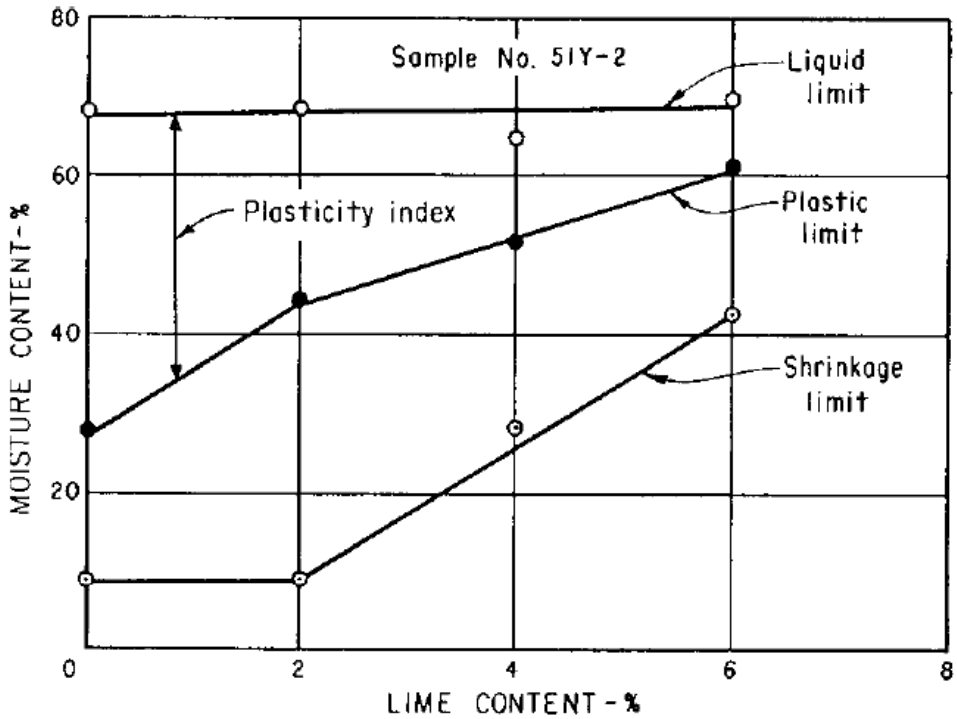


Figure 2.19: Effect of lime on shrinkage limit, plastic limit, and liquid limit (U.S. Department of the Interior Bureau of Reclamation, 1998).

From the figure it can be noticed that subsequent to the addition of lime in the studied material, the liquid limit reduces slightly or remains constant, whereas the plastic limit increases considerably which leads to the plasticity index reduction.

According to the National Lime Association (2004), the lime can be used in three main forms namely quick lime or calcium oxide (CaO), hydrated lime or calcium hydroxide [$Ca(OH)_2$] and lime slurry consisting of suspension of hydrated lime in water. Additionally, dolomitic lime is also used for soil stabilization. The types of lime used for expansive soils stabilization are given in Table 2.16.

Table 2.16: Lime materials used in construction (Nelson and Miller, 1992).

Type	Formula
Calcia (high-calcium quicklime)	CaO
Hydrated high-calcium lime	$Ca(OH)_2$
Dolomitic lime	$CaO + MgO$
Normal hydrated or monohydrated dolomitic lime	$Ca(OH)_2 + MgO$
Pressure hydrated or dehydrated dolomitic lime	$Ca(OH)_2 + Mg(OH)_2$

When lime is added to soil, different reactions such as cation exchange, flocculation, agglomeration, lime carbonation, and pozolanic reaction take place (Thompson, 1966; Nelson and Miller, 1992; Carmeuse, 2002; Little, 1995). These reactions are responsible for the soil properties improvement. According to Thompson (1964, cited in Thompson, 1966), cation exchange, flocculation and agglomeration contribute to the change of plasticity, shrinkage and workability characteristics whereas the strength enhancement is mainly a result of lime-soil pozzolanic reactions with a minor influence of lime carbonation. Water is the most important factor for soil-lime reaction but the soil-water mixture is not itself cementitious (McNally, 1998). In presence of water, lime reacts with silica and alumina from clay minerals to produce the bonding agents as in the case of Portland cement (McNally, 1998, Carmeuse, 2002). The basic pozzolanic reactions are as follows:

$Ca^{++}+OH^{-}$ + soluble clay silica → calcium silicate hydrate (CSH)

$Ca^{++}+OH^{-}$ + soluble clay alumina → calcium aluminate hydrate (CAH)

The results of flocculation/ agglomeration and pozzolanic reactions are illustrated in Figures 2.20 and 2.21, respectively (Carmeuse, 2002).

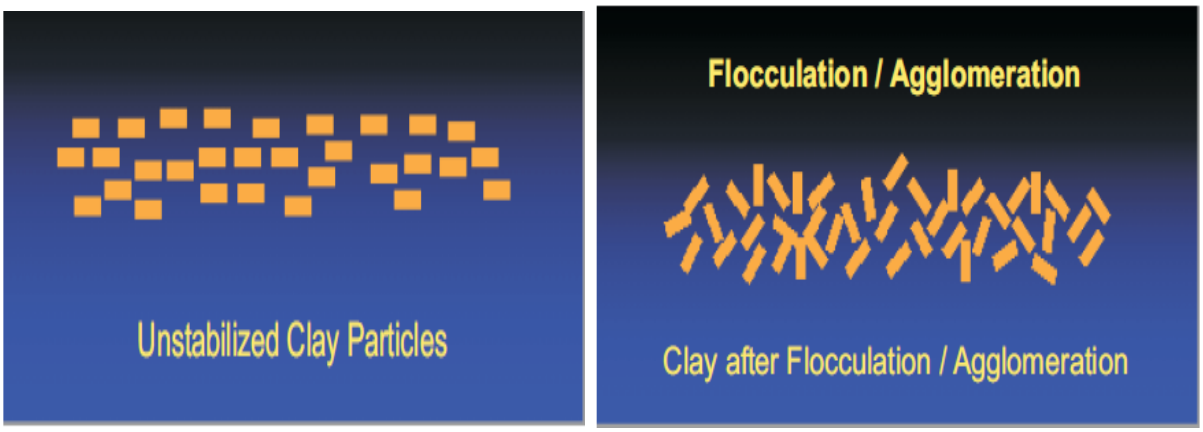


Figure 2.20. Clay particles before and after treatment (Carmeuse, 2002).

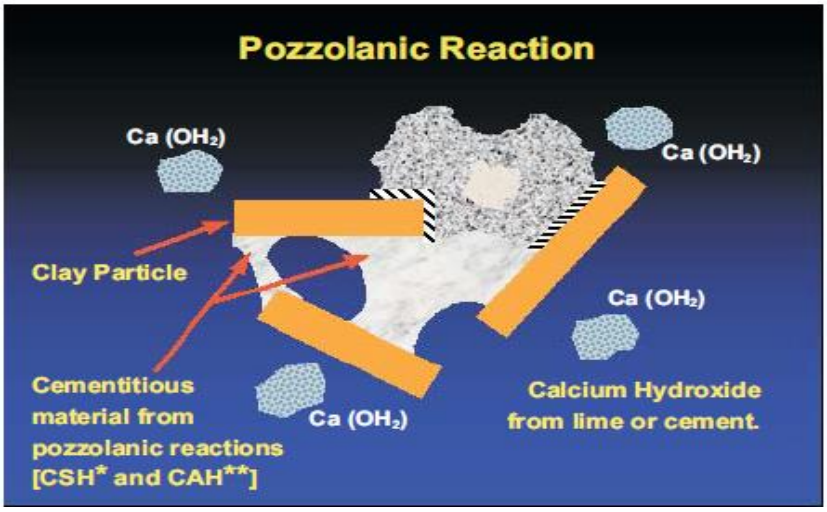


Figure 2.21. Formation of cementing agents due to pozzolanic reaction (Carmeuse, 2002).

The lime stabilized soil also exhibits higher friability than untreated material which reduces the plasticity of the soil (McNally, 1998). Although the stabilization of soil with lime is generally similar to that with cement in terms of technique and results, there are specific characteristics of the lime-stabilized process (McNally, 1998):

- Lime reacts strongly with heavy clays but poorly with granular soil; whereas the latter is more suitable for cement stabilization,
- For lime, more preparation time, i.e. mixing and compaction, is allowed lime sets slowly compared to cement,
- Lime-stabilized soil exhibits lower strength than cement-stabilized soil,
- Cement-stabilized soil is more prone to cracks than lime-treated soil.

The degree of strength improvement of lime-stabilized soil properties depends on various parameters such as type of soil, type of lime, percentage of lime and curing conditions (Nelson and Miller, 1992). Petry and Little (2002) also found that the quality of lime-soil stabilization is strongly affected by the moisture and the degree of pulverisation of the treated material. They found that the large soil particles delay the pozzolanic reaction of lime-soil mixture and the dry soil leads to the non-completion of chemical reactions.

The study carried out by Thompson (1966) revealed that some factors such as organic carbon content and soil pH, can significantly affect soil-lime reactions thus reducing the degree of strength improvement. The effect of those factors on the ability of a soil to react with lime to achieve a strength increase is illustrated in Figures 2.22 and 2.23. From Figure 2.22, it can be seen that organic matter has a great detrimental effect on soil-lime reaction. Due to its high base-exchange capacity, soil organic matter combines with the added lime thus causing the retardation of soil-lime strength gain mechanism (Thompson, 1966). For that, soil containing organic matter will require more lime to promote lime-soil pozzolanic reactions. Thompson (1966) also found that organic matter can prevent the dissolution of soil silica and/or alumina.

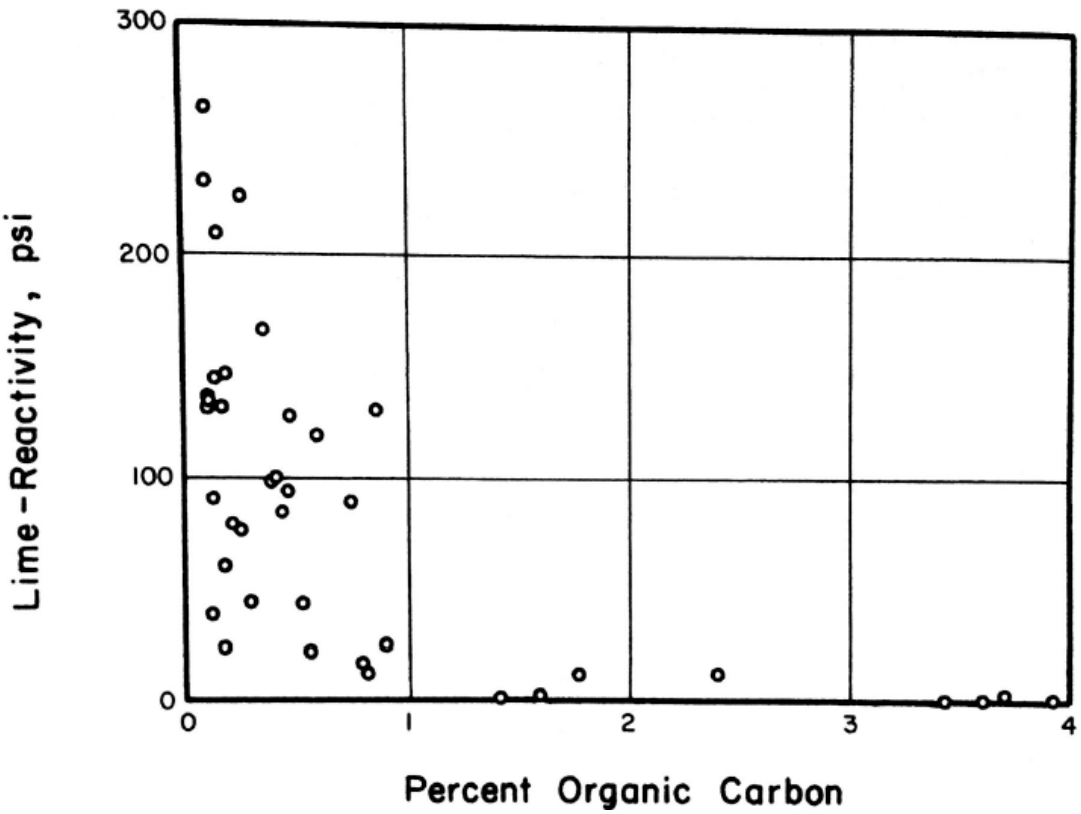


Figure 2.22. Influence of organic carbon on lime reactivity (Thompson, 1966).

Figure 2.23 shows the influence of soil pH on soil-lime reactivity. It shows that high soil pH is conducive to soil-lime strength improvement. The influence of pH on clay-silica and clay-alumina solubility was also studied by Keller (1964, cited in Little, 1995) and the results are illustrated in Figure 2.24.

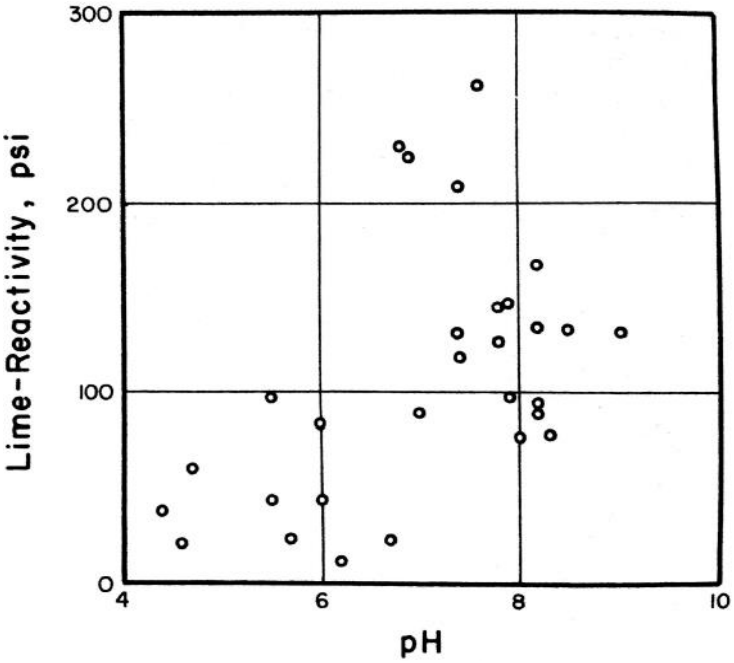


Figure 2.23. Influence of pH on lime reactivity (Thompson, 1966).

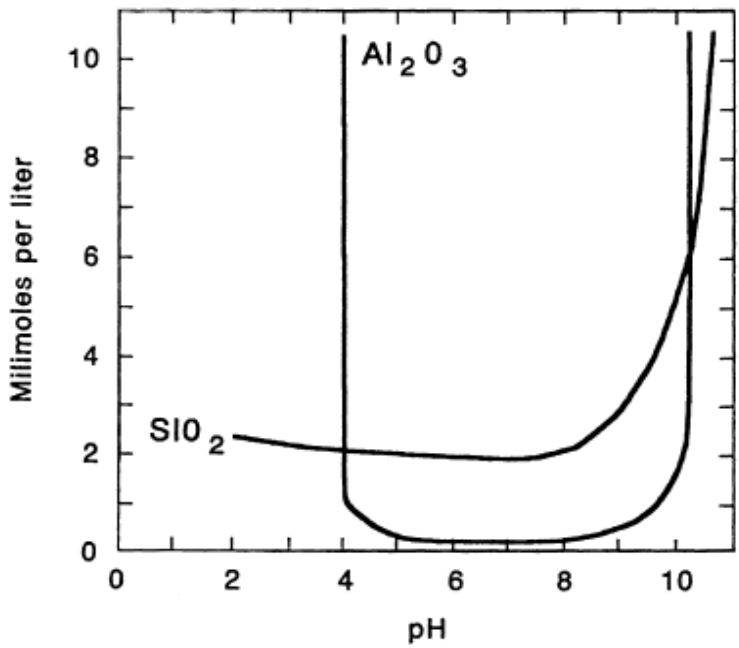


Figure 2.24. The effect of a high pH system is to release silica and alumina from the clay surface (Keller, 1964, cited in Little, 1995).

The study showed that there is no significant correlation between lime reactivity and clay fraction. However it has been found that high clay fraction will assure adequate silica and/or alumina sources for pozzolanic reactions between soil and lime.

Another effect of lime on soil is the decrease of maximum dry density (MDD) and increase in optimum moisture content (OMC). According to Rajasekaran and Rao (1998, cited in Khattab et al. 2007), reduction in MDD is attributed to the formation of cementitious products due to flocculation and agglomeration which lead to the formation of more open structure thus resulting in the MDD decrease. The author found that increase in OMC involves different phenomena such as water consumption during lime-soil ractions, lubrication of new formed materials and evaporation of water due to high temperature released during soil-lime reactions.

2.8.7.3.Cement stabilization

When cement is mixed with water, its calcium silicates and calcium aluminates hydrate to form the cementing compounds of calcium silicate hydrate and calcium aluminate hydrate (Lafarge, 2012). This reaction releases calcium hydroxide which plays a great role in cement-fine-grained soil stabilization in the same way as for lime-soil stabilization.

The stabilization of soil with cement produces the same effects as lime namely reduction in liquid limit and plasticity index, as well as in the soil expansion (Nelson and Miller, 1992).The stabilization of soil with cement also increases the shear strength and the shrinkage limit (Nelson and Miller, 1992). According to Al-Rawas and Goosen (2006), 'Cement stabilization develops from the cementitious bonds between the calcium silicate (three-calcium silicate) and aluminate hydration products of cement and the soil particles'. The choice of suitable stabilizer between lime and cement highly depends on the soil characteristics. Figure 2.25 shows the role of classification tests when choosing appropriate soil stabilizer. From Figure 2.25, it can also be noted that lime can be used alone for high plastic soils whereas the stabilization of high plastic soil with cement requires prior soil modification by lime. The prior addition of lime to high plastic soil reduces the plasticity and index and improves the workability and this facilitates the soil stabilization with cement (Puppala and Chttoori, 2010). Even if cement can be used in stabilizing various types of soils, from fine grained to coarse grained soil, Puppala and Chittoori found that it is more effective when used with granular materials.

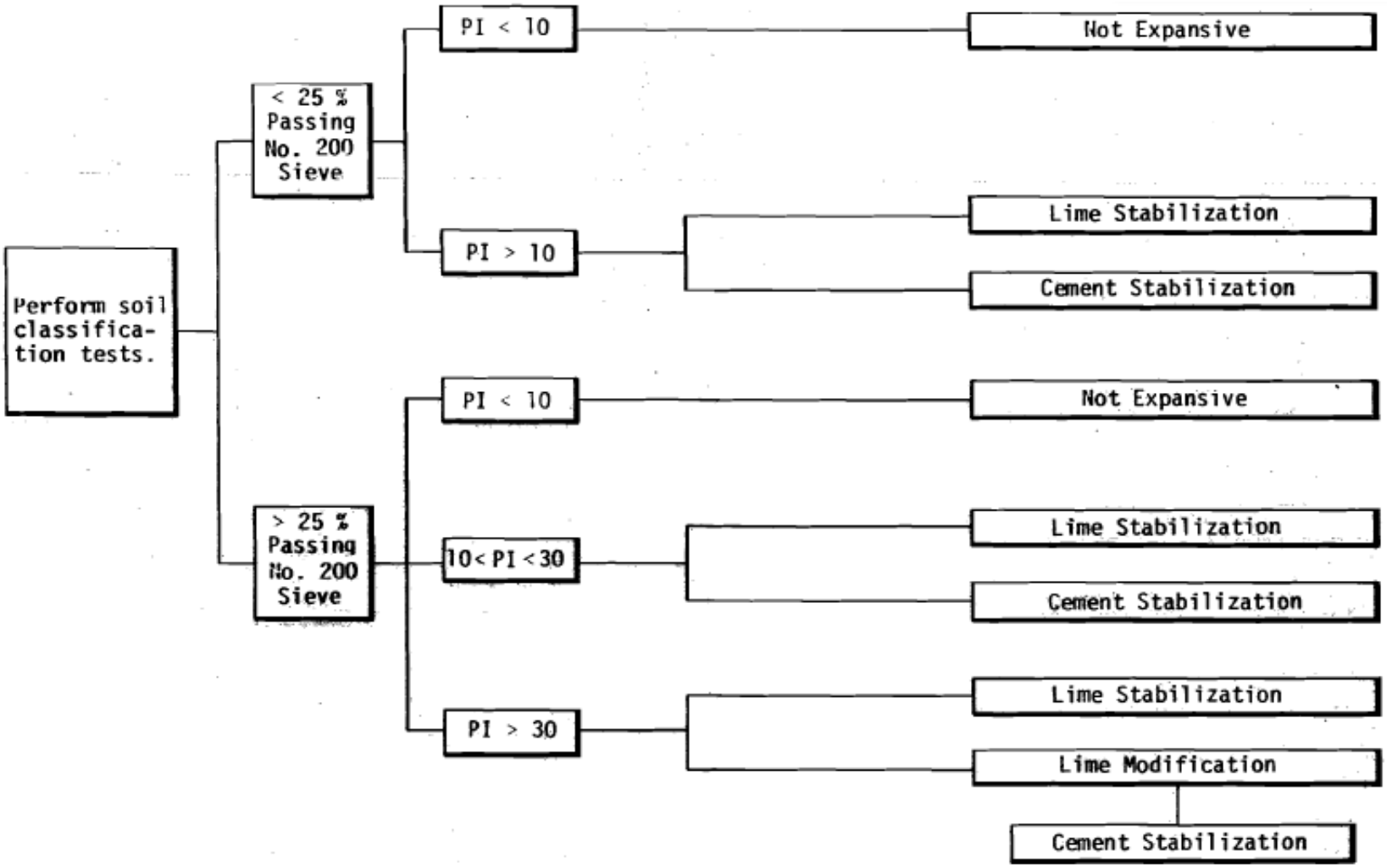


Figure 2.25: Selection of type of admixture for expansive soil stabilization (McKeen, 1976)

2.8.7.4.Salt stabilization

In addition to lime and cement, salts such as sodium chloride and calcium chloride are also used to stabilize expansive soils (Nelson and Miller, 1992). They found that the effectiveness of soil stabilization with salts greatly depends on the type of soil. Nelson and Miller also observed that, compared to lime and cement stabilization, a smaller amount of calcium chloride, 1%, is needed to stabilize most of the soils. Soil stabilization with salts has some disadvantages namely a prerequisite relative humidity of 30% and they are easily washed away from the soil which requires periodic stabilization (Nelson and Miller, 1992).

2.8.7.5.Stabilization of expansive soil with fly ash

Fly ash is increasingly used to improve the engineering properties of soils and pavement bases (Petry and Little, 2002). However, Petry and Little found that fly ash has to be combined with lime or cement for adequate stabilisation of plastic soil. When added to lime-treated soil, fly ash increases the pozzolanic activity and also plays a great role in gradation improvement of granular soil (Nelson and Miller, 1992).

According to Mackiewicz and Ferguson (2005), fly ash has a lot of applications in soil stabilization. However, the application of fly ash depends on its characteristics. The coal ash resulting from the combustion of bituminous coal exhibits no self-cementing properties and is classified as fly ash “class F”. On the other hand, the fly ash classified as “class C”, which results from the combustion of subbituminous coal, contains calcium in high percentage and thus exhibits self-cementing characteristics due to high concentration of calcium carbonate in subbituminous coal (Mackiewicz and Ferguson, 2005). While the fly ash “class C”, when mixed with water, hydrates and produces the same cementitious products in the same way as cement stabilization, fly ash “class F” needs to be mixed with lime or cement as activators to prompt pozzolanic reactions (Mackiewicz and Ferguson, 2005). The self-cementing fly ash has a lot of applications among others drying out wet soil prior to compaction, reduction of swelling potential and improvement of engineering soil properties (Mackiewicz and Ferguson, 2005).

Even though the self-cementing fly ash reduces the soil plasticity of a treated soil, the degree of reduction is not as the same as in the case of lime stabilization because only a small percentage of free lime is available in fly ash (Mackiewicz and Ferguson, 2005).

2.8.7.6.Assessment of soil admixture effectiveness

Ground or project site is not always adequate for the proposed project. To fix this problem of inadequate soil conditions, many techniques have been developed and applied successfully for different projects. Among those techniques, soil stabilization plays a great role due to a wide variety of stabilizers available for that purpose. Apart from traditional soil stabilizers such as Portland cement, lime, fly Ash, bitumen and tar and cement kiln dust (CKD), many non-traditional soil stabilizers have been found suitable and affordable to improve soil conditions in order to satisfy project requirements. Non-traditional stabilizers include among others polymers based products, copolymer based products, calcium chloride, sodium Chloride, etc.

As for other ground improvement techniques, the main objective of soil stabilization is to enhance the soil engineering properties such as strength, volume change, permeability, etc. These three properties are the key issues exhibited by expansive soil which is the subject of this study.

From the above, it's evident that the selection of a soil stabilizer is governed by a large number of factors to achieve one or more objectives. Those factors are among others soil mineralogy, soil classification, goals of treatment, desired engineering and material properties, design life, environmental conditions (drainage, water table, etc.), engineering economics (cost savings vs. benefit) (Texas Department of Transportation, 2005). It is obvious that the study of an effective stabilizer involves most of those factors which makes this step cardinal. Below, a review of evaluation of admixture effectiveness for fine-grained soils improvement, as described in ASTM D4609-08 (2008), has been made.

According to the standard, the effectiveness of a soil admixture can be assessed by comparing the unconfined compressive strength (UCS), moisture susceptibility and the compaction test parameters before and after treatment. These tests are not the only ones to be carried on a treated soil but provide enough information about adequacy of the admixture which allows making decisions regarding further tests specific to a given project. The indicators of the enhancement of fine-grained soil by a certain admixture are given here below according to D4609-08 (2008).

- **Particle-size distribution:** The coarsening or granulation of fine grained soil, due to addition of an admixture, which results in gradation curve shift, is an indicator of soil properties improvement.

- **Atterberg limits:** The reduction in liquid limit, thus in plasticity index is a good sign of soil properties enhancement.

- **Compaction parameters:** an increase in maximum dry density indicates soil strength improvement. In addition to this, the decrease in optimum moisture content will have an implication on project cost since water is most of the time purchased and also requires particular equipment for transport, storage and distribution.

- **Unconfined compressive strength (UCS) and moisture absorption:** Strength is one of the most important parameters needed for any structural design. Therefore, an increase in UCS, which has correlation with a number of soil properties, indicates the effectiveness of an admixture.

- **Volume change:** ground movement causes many distresses to structures. Volume change is mainly due to moisture change. The achievement of an adequate control of volume change by using an admixture will confirm its effectiveness in soil stabilization.

Due to a wide range of factors involved in change of those properties, it's quite difficult to achieve improvement of them with one stabilizer. According to ASTM D4609-08 (2008), a change in at least one of them but not necessary all of them, is an indicator of a certain degree of admixture effectiveness.

2.9. STABILIZATION OF CLAY SOILS WITH PAPER AND PULP MILL WASTE

2.9.1. General

As for other industries, the paper and pulp industry generates considerable wastes which require appropriate management. For instance, in Europe, the production of 99.3 million tons of paper generated 11 million tons of wastes in 2005 (Likon and Trebše, 2012). These wastes produced during pulp and paper making in the form of waste paper sludge, woodyard waste, mill trash and ash from boilers; place a large financial burden on pulp and paper industry (Scott and Smith, 1995; Likon and Trebše, 2012). To handle this issue, many techniques have been applied to manage such wastes but still more modern techniques are required to reuse or eliminate or dispose of those wastes in a sustainable way. Some techniques currently used for paper and pulp mill waste management include among others landfilling, landspreading on forest or agricultural lands, composting, energy production, production of

construction materials and landfill capping material (Likon and Trebše, 2012; Scott and Smith, 1995; Elliott and Mahmood, 2006). However, due to environmental issues, some of those techniques, such as landfilling, are progressively either restricted or sometimes banned by the legislation (Likon and Trebše, 2012). Landfilling technique may cause problems due to leachate and it also occupies land space which would be used for other important projects (Elliott and Mahmood, 2006).

In addition to the traditional paper and pulp mill waste management alternatives, other techniques have been developed and currently used to change the waste to a valuable product. Some of those techniques are mentioned below except for the soil stabilization application which will be discussed separately later.

- Use of waste paper sludge ash as component for hydraulic binders (Segui, Aubert and Husson, and Measson, 2011),
- Cement and brick manufacturing (Elliott and Mahmood, 2006),
- Glass and ceramic products manufacturing using oxides such as CaO, Al_2O_3, SiO_2, TiO_2, Fe_2O_3 and P_2O_5 present in pulp and paper as nucleating agents (Elliott and Mahmood, 2006),
- Use of ash from paper sludge incineration as paper filler (Elliott and Mahmood, 2006),
- Use of paper mill sludge as heat insulation material (Likon and Trebše, 2012),
- Use of paper mill sludge in fiber-cement sheets as replacement for virgin fibers (Likon and Trebše, 2012),
- Use of sludge as soil replacement to promote the regrowth of vegetation in the reclamation of strip mine sites (Scott and Smith, 1995),
- Use of sludge in cement tiles (Scott and Smith, 1995),
- Use of paper sludge ash for the remediation/stabilization of mine waste sites (Wajima, Shimizu and Ikegami, 2007; Mochizuki et al. 2004).

Even though many studies have been conducted to reduce the burden caused by paper and pulp waste to industries, either by recycling or reuse of the waste in various fields as mentioned above, it seems that only few studies were oriented to soil stabilization with paper and pulp mill waste. In the following, some found cases of application of paper and pulp mill waste in soil stabilization have been reviewed.

2.9.2. Stabilization of clay soil using waste paper ash

The study carried out by Khalid, Mukri, Kamarudin and Arshad (2012) has shown that the waste paper sludge ash (WPSA) resulting from the combustion waste paper in recycling factories can be used to enhance the engineering properties of soft clays. In their study, optimum WPSA was determined based on the compressive strength and the effect of the optimum WPSA on soil strength and the California Bearing Ratio (CBR) was investigated.

The chemical analysis carried on the WPSA showed that the ash possesses cementitious as well as pozzolanic properties. In addition to that, the content in lime of the ash was assessed to be more than 20% (Table 2.17). Following those characteristics, the WPSA was classified as class C fly ash. On the other hand, the clay used during their study was classified as slightly sandy clay of high plasticity.

As result, Khalid et al. (2012) found that the addition of WPSA in high plastic clay improves both compressive strength and CBR. The enhancement of the soil stabilized strength and CBR was attributed to cementation and pozzolanic reactions which led to the formation of silicate hydrates and aluminate hydrates like in the case of lime stabilization.

Table 2.17: Chemical composition of WPSA used for clay soil stabilization (Khalid et al., 2012)

Chemical Constituents	Chemical Composition (%)
Calcium Oxides (lime) CaO	62.39
Silicon Dioxide (silica) , SiO_2	23.25
Alumunium trioxide, Al_2O_3	5.26
Magnesium oxide, MgO	2.46
Iron oxide, Fe_2O_3	0.77
Sulphate, SO_3	0.58
Sodium oxide , Na_2O	0.42
Potasium oxide , K_2O	0.35
L.O.I	4.50

2.9.3. Stabilization of fibrous peat with waste paper sludge ash (WPSA)

In their study, Khalid et al. (2013) found that the same waste paper sludge ash (WPSA) used to stabilize the clay soil in the previous case, can also be used to improve the strength of a fibrous peat soil. During their study, peat soil sample from Malaysia was mixed with WPSA at different percentage to determine the optimum concentration for maximum compressive strength. While the optimum WPSA for clay soil was 10% by weight, the optimum

percentage for fibrous peat soil stabilization was found to be 7%. However the maximum compressive strength reached was higher in case of clay soil than for fibrous peat soil. Based on these two cases, various types of soils can be stabilised with waste paper sludge ash.

2.9.4. Soil improvement with paper sludge ash and re-incinerated paper sludge ash

In their study, Mochizuki et al. (2004), have found that the re-incinerated paper sludge can be used to improve soft soils or disposing of waste mud. For their study, paper sludge ash resulting from the incineration of paper sludge was re-incinerated to reduce the content in unincinerated carbon and the method of cone penetration test of compacted soils according to Japanese Industrial Standards (JIS A 1228) was used to assess the strength improvement. The waste paper sludge ash was mainly composed of silica, alumina and lime due to the addition of calcium carbonate and kaolin during the paper manufacturing (Mochizuki et al., 2004). The chemical composition of the re-incinerated paper sludge ash used in their study is mentioned in Table 2.18.

Table 2.18: Chemical components of re-incinerated paper sludge ash (Mochizuki et al., 2004).

Chemical component	Al_2O_3	SiO_2	CaO	MgO	Fe_2O_3	ZnO	P_2O_5	TiO_2	K_2O
Content (%)	22.6	33.8	18.1	5.2	1.8	0.2	0.7	2.6	0.3

The study conducted by Mochizuki et al. (2004) on various types of soils, showed that the re-incinerated waste paper sludge ash can replace cement and lime to treat mud or improve soft soils mainly due to its high water absorption capacity (115%) and no curing time is required like in the case of cement or lime. The study also found that, while more quantity of re-incinerated waste paper sludge ash was required for many construction sites improvement than lime, less quantity of re-incinerated paper sludge ash was required to achieve the designated strength for organic soil improvement than for lime or cement. An added advantage is that the effect of re-incinerated paper sludge is immediate and the cost for re-incinerated paper sludge was less than for lime.

This techniques of soil improvement has known a lot of application to the construction field in Japan namely in disposal of waste mud during earth pressure balance shield tunnelling, improvement of soft excavated soil in open caisson, improvement of river bed deposit in dredging and lake deposit after dredging (Mochizuki et al., 2004).

Furthermore, Mochizuki et al. continued their research on different ashes from different paper mills in Japan. Based on the results of the penetration tests on treated clay, they found that all paper sludge ashes have soil stabilization potential. However, contrary to the case of re-incinerated paper sludge ash where the soil improvement was likely due to its high water absorption, the other ashes exhibited chemical improvement characteristics (Mochizuki et al., 2004).

2.9.5. Use of pulp mill fly ash and lime by-products for soil strength improvement and deformation reduction in road construction

Based on the fact that commercial lime products have been used successfully to stabilise soil in roads projects and given that the pulp and paper industry generates considerable amount of lime-rich wastes, Zhou, Smith and Sego (2000) carried out a study to investigate the engineering feasibility of using those wastes as valuable material in roads construction. In addition to the assessment of the engineering potential of paper and pulp mill wastes, the study also had the objective to assess their environmental impact. For their study, four paper industry wastes namely fly ash, bottom ash, $CaCO_3$ and CaO were chemically examined to assess if they may have harmful environmental effect. On the other hand, the fly ash and $CaCO_3$ were mixed with different types of soils to assess their engineering potential for soil stabilization. In conclusion, Zhou et al. (2000) found that the fly ash and lime by-products are non-hazardous and they can be used to increase soil strength and reduce the deformation. However, it was found that soil type has a great influence on the degree of strength improvement. The strength improvement was higher for soil with more fines than the soil with lesser fines.

2.9.6. Other applications of pulp and paper byproducts in soil stabilization

In addition to the applications of paper mill wastes in soil stabilization mentioned above, Thacker (2012) also presented other cases of paper and pulp by-products applications in soil stabilization. He found that the United States pulp and paper industry produces each year a great amount of two major wastes namely waste water treatment plant (WWTP) residuals or paper mill sludge and power boiler ash. Both materials have some applications in soil stabilization as shown below.

- Stabilization of loose-sand roads in the Chequamegon National forest with WWTP residuals which started in 1977,
- Erosion control by composts made with WWTP residuals in Iowa and Virginia,

- Soil stabilization on the road side and road base with paper sludge ash (PSA) in Japan,
- Sale of PSA as a commercial product by TerraStab for road soil stabilization in Netherlands,
- Reduction of road rutting using wood ash for improvement of soil strength and stiffness,
- Increase of allowable load during winter using wood fly ash,

In conclusion, as shown in different cases mentioned above, different paper industry wastes such as fly ash, bottom ash from boilers, lime by-products have been used successfully in soil stabilization. However, given that the properties of those wastes depend on the paper production process, the types of raw materials as well the type of filler used, it is clear that each paper and pulp waste should be studied individually before its application in soil stabilization. For instance the paper ash resulting from the combustion of waste paper ash only will be different from the combustion of waste paper ash combined with sawdust, bark and some fuels.

2.10. CONCLUSION

Due to the huge damages caused by expansive soils, many researches have been carried out to understand the mechanism of swelling-shrinking behaviour of such soils as well as to develop different methods to prevent and/or remediate the associated problems. From the literature review, it is evident that the accurate identification of expansive soils is a key tool to handle those problems. For this study, the index tests were used to classify the studied materials.

From the literature, it was also found that, in addition to traditional additives which have been used since time immemorial to stabilize soils, many other products, sometimes considered as wastes, can be used for soil improvement. Particularly, for expansive soils, it is apparent that products with chemical composition which can initiate the four reactions involved in soil-lime treatment (Section 2.8.7.2) can be used for expansive soil stabilization. However, different factors affecting the soil improvement success with a given additive must be considered. The effect of paper mill ash on expansive soil properties was explicated in Chapter 4.

2.11. REFERENCES

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CHAPTER 3 RESEARCH DESIGN AND METHODOLOGY

3.1. INTRODUCTION

As discussed in the previous chapter, commercial lime has been used successfully in many projects to stabilize clay soils. Based on that, this chapter discussed the methodology adopted to study the effect of lime-rich paper mill ash on expansive soils as well as the experimental program. This chapter sets out to analyse the following issues:

- Materials used for this study,
- Tests and methods, and
- Testing program.

Figure 3.1 shows the experimental plan to evaluate the paper mill ash-stabilization process of swelling clays.

3.2.MATERIALS

Two main types of materials were used namely clay soils and a paper mill ash. Each clay soil was mixed with the ash at different proportions to evaluate the influence of the ash on some of its properties.

3.2.1. Clay materials

In order to study the effect of a paper mill ash on the properties of swelling clays, three clays were used. A brief description of each of them is given in Table 3.1 and the chemical analysis results are given in Table 3.2 as determined by Scanning Electron Microscope (SEM).

Clay 1 was collected from Stellenbosch area, Stellenbosch University property, on a construction site of New Generation Hostel and Pub next to Eendrag Hostel. As shown in Figure 3.1, the clay layer was beneath a boulder layer and starts just above the underground water level.

According to Croukamp and De Wet (2012), the boulder and sand layers overlying the clay layer consist of recent sediments of Miocene (35-5 million years ago) and the “boulders are predominantly boulders of Table Mountain Sandstone of varied shape and size with occasional fragments of in-situ weathered granite”. The clay is yellowish with some traces of reddish and white.

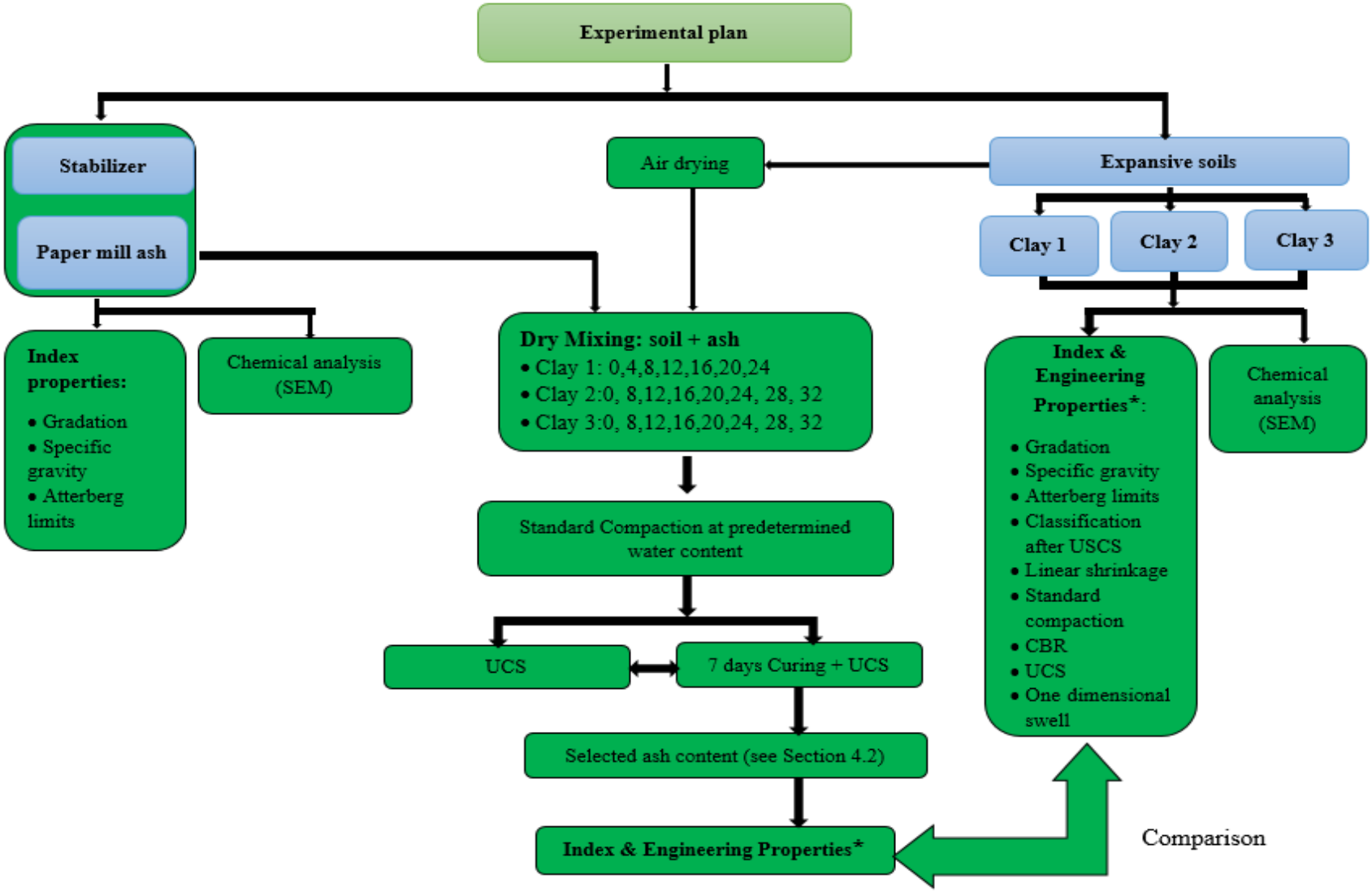
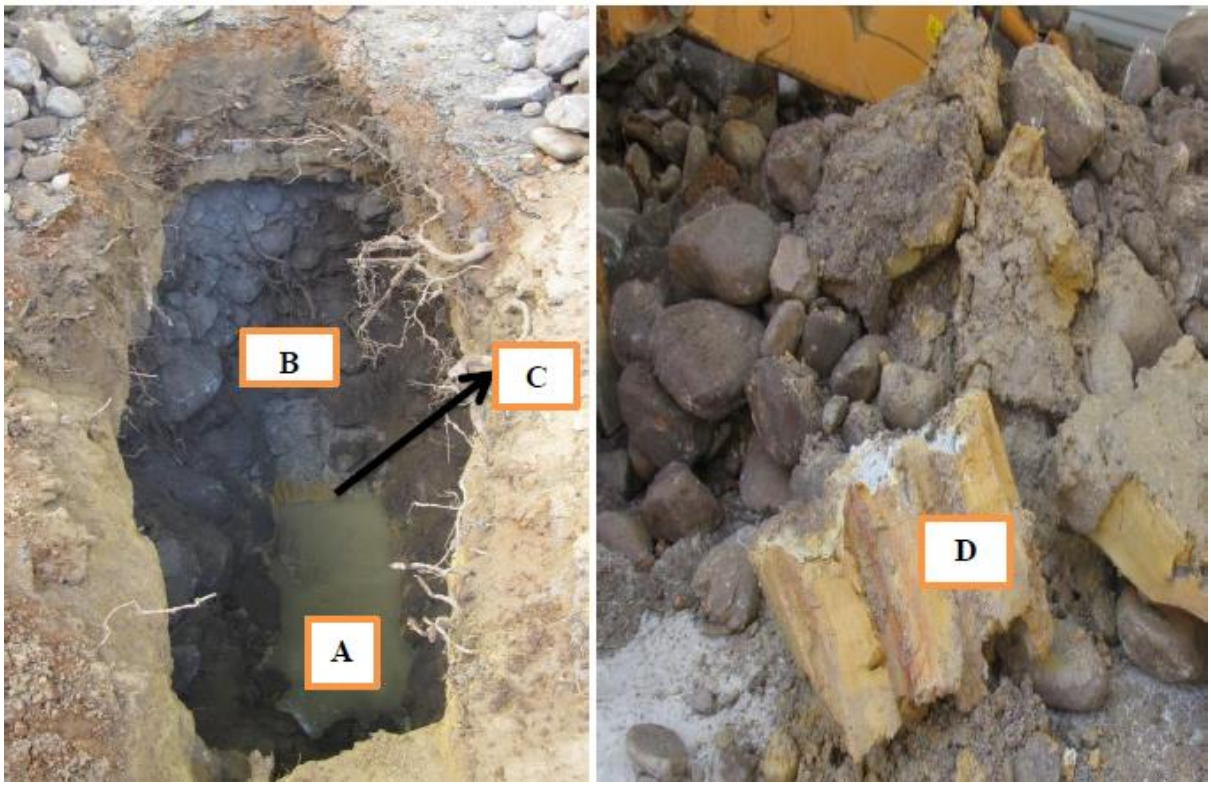


Figure 3.1. Experimental plan

During sampling, the excavated wet clay blocks were put into plastic bags and then wrapped with cling wrap plastics to prevent loss of moisture content. The clay was sampled at a depth between 2.5 and 3.0 m from the ground surface.



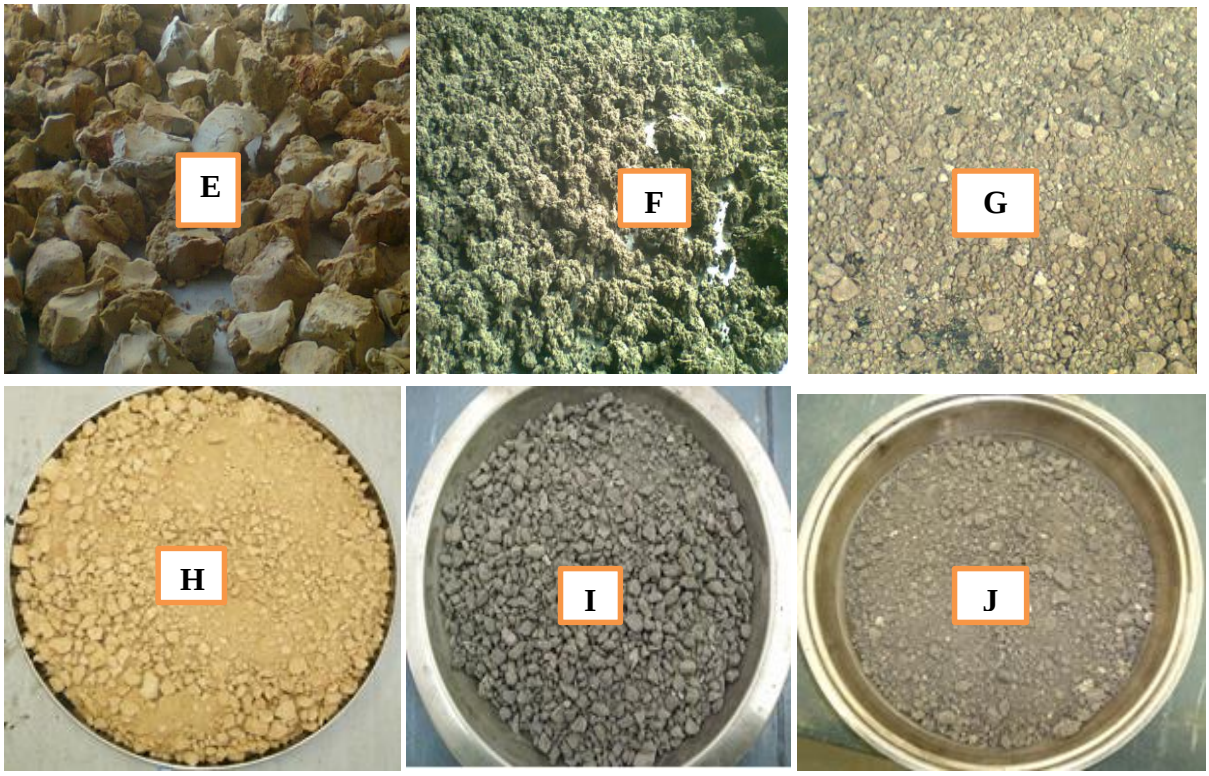
A- Underground water just beneath the upper clay limit below boulder layer, **B**-boulder layer, **C**-clay layer, **D**-excavated clay sample

Figure 3.2: Sampling pit for Clay 1 and Clay 1 material sample as excavated

Clay 2 was collected from the construction site of Mantsole weighbridge on National Road N1, in Limpopo Province. The site is located along the National Road N1 between Pretoria and Bela Bela. The clay was sampled at a depth between 0 and 0.15m from the ground surface. As shown in Figure 3.3. F, the clay is black and it was delivered wet. (See more details in Table 3.1)

Clay 3 was collected from a construction site along the National Road N1 in Ventersburg, in Free State Province. The clay was sampled at the roadbed. As for Clay 2, the Clay 3 is also black as shown in Figure 3.2. (b). (See more details in Table 3.1).

The brief description given in Table 3.1 is done on Clay 1 as sampled; but for Clay 2 and 3, the description was done on delivered samples from sampling locations.



E- wet broken clay 1 blocks; **F**-wet clay 2 as delivered; G- slightly wet clay 3 as delivered; **H**, **I** and **J** air-dried pulverized clay 1, 2 and 3 respectively.

Figure 3.3. Clay materials used in this research

Table 3.1: Soil sampling location and brief description of clay materials used for the study

Material	**Source location**	**Description of clay materials**		
		Moisture condition	**Colour**	**Consistency**
Clay 1	Stellenbosch, Western Cape	Wet	Yellowish with white and reddish traces	Very soft
Clay 2	Mantsole weighbridge (N1), Limpopo Province	Wet	black	Very soft
Clay 3	Ventersburg, Free State Province	Slightly moist	black	Soft

Table 3.2: Chemical composition of clay materials (SEM analysis)

Materials	**Oxide compounds (%)**									
	Na_2O	MgO	Al_2O_3	SiO_2	SO_2	K_2O	CaO	TiO_2	MnO_2	Fe_2O_3
Clay 1	0.00	0.49	22.26	54.68	0.00	1.49	0.11	0.65	0.00	5.32
Clay 2	0.52	2.22	14.31	46.22	0.04	1.45	0.82	0.44	0.00	3.88
Clay 3	0.56	2.75	10.31	44.00	0.00	0.42	4.30	0.42	0.00	3.18

In addition to the brief description mentioned above in Table 3.1 and the chemical composition given in Table 3.2, a summary of index properties of the used clay materials is given in Table 3.3. The index tests or physical properties tests such as gradation analysis and Atterberg limits tests are very important in soil mechanics because they are not expensive and they provide information on engineering properties of soils (USBR, 1998). The optimum moisture content and the maximum dry density, results of the moisture content and density relationships also used as index properties (USBR, 1998), are given in Table 3.3 too. Detailed procedures for the determination of those properties as well as the standard methods followed are given in Chapter 4.

Table 3.3. Index properties of clay materials.

Property	Clay 1	Clay 2	Clay 3
Specific gravity	2.67	2.80	2.68
Liquid limit (%)	43	61	46
Plastic limit (%)	28	28	23
Plasticity index (%)	15	33	23
Linear shrinkage (%)	7	10.7	7.2
Maximum dry density (kg/m^3)	1695	1404	1568
Optimum moisture content (%)	16.5	27.5	21.4
Field moisture content (%)	32.5	*-	*-

The asterisk (*) means that the property has not been determined. The natural moisture contents for Clay 2 and 3 were not determined because the materials were collected by the site engineers of the mentioned construction sites, put into plastic bags and then shipped to Stellenbosch.

In addition to the index properties, the clay materials used in this study were classified according to the expansiveness potential. To this end, the chart proposed by Van der Merwe (1964) given in Section 2.5.2.1 was used. The positions of the three clays on the chart are shown in Figure 3.3 below. From the chart, Clay 1 is classified into low degree of potential expansiveness, and Clay 2 and 3 into medium and high degree of potential expansiveness respectively.

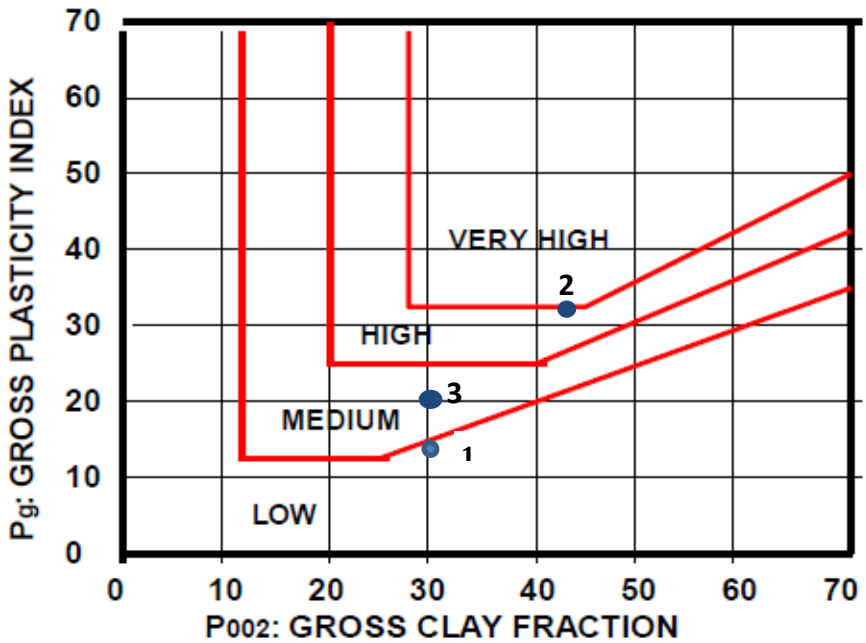


Figure 3.4. Potential expansiveness of soils (Van der Merwe, 1964, redrawn from Savage, 2007).

3.2.2. Paper mill ash

The ash used in this research was collected from Mondi, Merebank mill located on Travancore Drive, Merebank, Durban in KwaZulu Natal Province, South Africa. The ash results from the combustion of sludge, sawdust, bark, coal ash and bituminous coal (Rafiq, 2013). The pulp and paper mill sludge generated during paper and pulp manufacturing is burnt at 900oC in a Multi Fuel Boiler (MFB) together with bark, sawdust, coal boiler ash and coal to produce electricity and steam. The resulting ash from the MFB is called waste ash to

differentiate it from course ash generated in a coal boiler. The chemical composition of the ash obtained by scanning electron microscopy (SEM) analysis is given it Tables 3.4 and 3.5. Although some successful trials were done to evaluate how suitable the ash is, in block making, it is still landfilled as any other waste ((Rafiq, 2013).

Table 3.4. Elemental constituents of paper mill ash (SEM analysis)

Constituent	Mg	Al	Si	S	K	Ca	Ti	Fe	O
Content (Weight)	0.97	11.08	17.08	1.96	0.32	22.87	0.67	0.90	43.43

Table 3.5. Chemical composition of paper mill ash (SEM analysis)

Oxide compound	MgO	Al_2O_3	SiO_2	SO_2	K_2O	CaO	TiO_2	Fe_2O_3
Percentage (by weight)	1.55	22.41	35.83	4.93	0.43	32.58	1.16	1.11

As shown in table 2.5, the ash is rich in calcium oxide (CaO). For that reason, it's likely to be used as soil stabilizer as other lime-rich products.

3.3. TESTS AND METHODS

This research aimed at studying the effect of a paper mill ash produced by incineration of different raw materials as described in Section 3.2.2, on clay materials properties. As this material seemed to be new in soil stabilization application and some researches have shown that the paper mill waste contains lime (see Section 2.2.8.8), the standard procedures used for other stabilizers, particularly lime, were considered. Within this framework, the ASTM D4609-08 (2008) was considered to select some required tests. According to this standard, a series of laboratory tests including Atterberg limits, particle size distribution, compaction, unconfined compressive strength, and volume change tests are conducted on both treated and untreated soil to evaluate the potential of a stabilizer to improve the soil's engineering properties. The effectiveness is judged based on changes in one or more properties but not necessary all (ASTM D4609-08 (2008). In addition to these tests, the California Bearing Ratio (CBR) was also conducted because of its wide use in evaluating the strength of soil in big projects such as pavements, where soil stabilization and/or modification is mostly applied. This section describes the tests carried out, equipment used as well as the standard methods followed.

3.3.1. Gradation and Atterberg limits

According to Nelson and Miller (1992), "classification tests for soil index properties such as grain size distribution, clay content, and plasticity are the most widely used in practice for identifying and classifying expansive soils". In addition, the Atterberg limits tests are generally useful for engineering classification of soil (USBR, 1998; Lambe and Whitman, 1969).

As shown in Figure 3.4, the Atterberg limits define different states of a fine-grained soil as water content is increased and they correspond to the water contents between adjacent states (Lambe and Whitman, 1969).

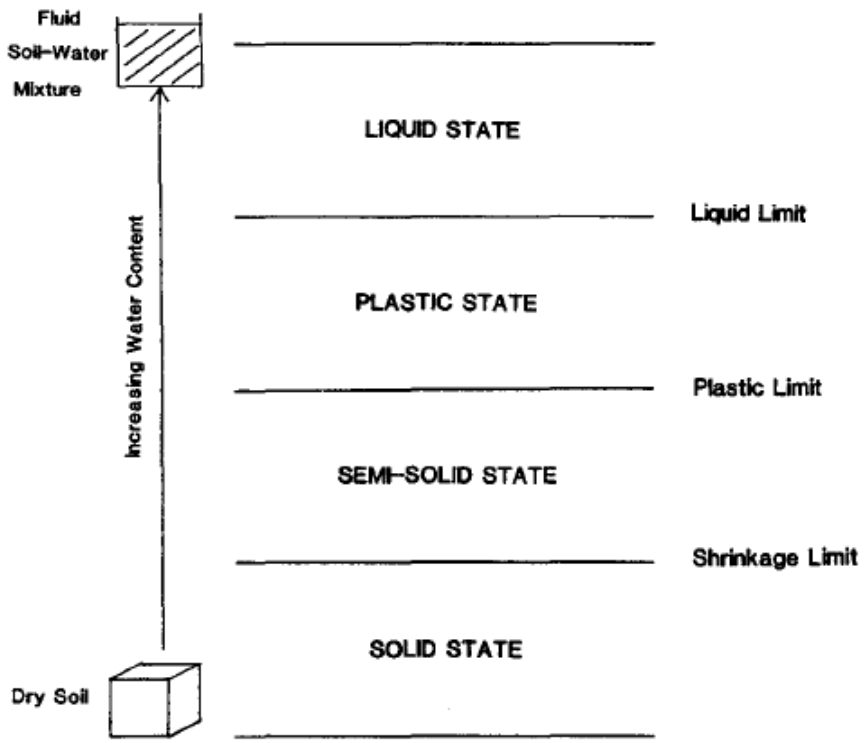


Figure 3.5. States of consistency and Atterberg limits of fine-grained soils (Lambe and Whitman, 1969, redrawn in Nelson and Miller, 1992).

Atkinson (1993) defined liquid limit as water content at which a fine-grained soil becomes very weak and starts flowing as a liquid; and plastic limit as water content at which soil becomes so brittle and starts crumbling.

During this study, the samples for particle size distribution analysis and Atterberg limits were prepared according to ASTM D421-85 (2007) regarding dry preparation of soil samples for

particle-size distribution analysis and determination of soil constants. The soil as sampled was air-dried and the lumps were broken carefully to avoid the destruction of soil grains structure and allow the soil to dry more or less thoroughly. By means of a riffle sampler, an adequate representative sample for gradation and Atterberg limits was obtained from the air-dried soil.

3.3.1.1. Gradation

The gradation which consists of distribution and size of grains was conducted on clay materials according to ASTM D422-63 (2007). The sample material prepared according to ASTM D421-85 (2007) was divided into two portions namely portion retained on No 10 (2.00mm) sieve and portion passing No 10 sieve. The portion retained on No. 10 sieve was carefully ground and sieved again to allow all fines smaller than 2mm in diameter to be separated from coarse soil grains. After the second sieving, the fraction retained on No. 10 sieve was washed then oven dried. The oven dried fraction was finally separated into different fractions using a series of 9.5mm, 4.75mm and 2.00mm sieves. From the portion passing No 10 sieve a sample of 50g was used for hydrometer test according to ASTM D422-63 (2007). In order to complete the soil gradation, the specific gravity was determined according to ASTM D854-10 (2010).

3.3.1.2. Atterberg limits

The remaining soil passing 2mm sieve after taking soil sample for hydrometer analysis was then separated into two fractions using No. 40 (425µm) sieve. The fraction passing No. 40 sieve was used to determine the Atterberg limits of clay materials to classify them in accordance with ASTM D4318-10 (2010) regarding the standard test methods for liquid limit, plastic limit and plasticity index of soils.

For the determination of the liquid limit there are two alternative tests namely the Casagrande liquid limit test and the fall cone liquid limit test (Atkinson, 1993). During this study, the first alternative was considered and carried out according to ASTM D4318-10 (2010). Among two methods provided by ASTM D4318-10, method A, called multipoint liquid limit method was used because it is assessed by this standard to be more accurate than method B referred to one point method. Figure 3.6 shows the length on which the groove made in the soil pat in the liquid limit device must close for at least three trials for which the number of blows shall be within the ranges of 25 to 35, 20 to 30 and 15 to 25 (ASTM D4318-10, 2010).

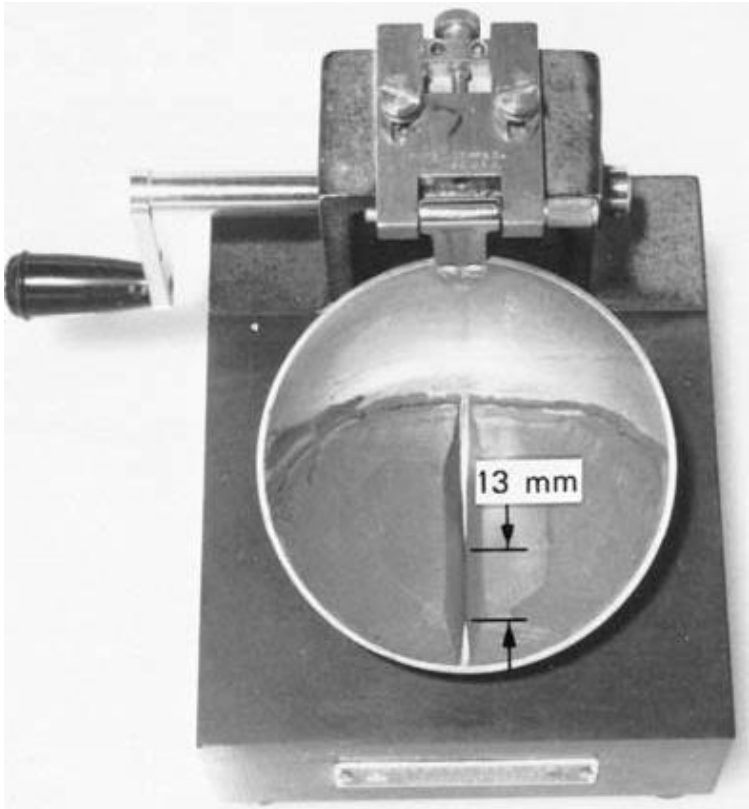


Figure 3.6. Soil pat after groove has closed (ASTM D4318-10, 2010)

The plastic limit was determined using hand method according to ASTM D4318-10 using the soil remaining after the liquid limit test. Three trials were done and the plastic limit was the average of the three water contents.

3.3.2. Compaction

In order to determine the relationship between moisture contents and densities, a series of standard Proctor tests was conducted on both untreated and treated materials according to ASTM D698-07 (2007). The optimum moisture contents determined during compaction of non-treated material and treated materials at different ash contents were used to prepare specimens for unconfined compressive strength (UCS) tests. For treated materials, the ash content in terms of percentage was added to air dried soil based on its oven dry weight, then mixed thoroughly with a mechanical mixer and then compacted using an automatic compactor for soil. After adding water, the standing soil lumps were pulverized and the batch mixed thoroughly until the batch was more or less uniform. The mixture was compacted without mellowing time.

3.3.3. Unconfined compressive strength (UCS)

The American Society for Testing and Materials (ASTM) defines the unconfined compressive strength of untreated material as either the maximum stress or the stress at 15 % axial strain whichever is secured first during the UCS test (ASTM D2166-06) and the UCS of treated material as either the maximum stress or the stress at 5 % axial strain, whichever is reached first during UCS test (ASTM D5102-09).

The UCS is one of the most used tests in pavement projects as well as in soil treatment application and is often used to assess the soil properties improvement due to treatment (Muhunthan and Sariosseiri, 2008). According to ASTM D4609-08 (2008), the UCS is one of the methods used to assess the effectiveness of admixtures for soil treatment. The standard states that the admixture is considered to be effective once an increase in UCS of at least 345 kPa is achieved. In addition to this, UCS was found to be a good indicator of soil-lime reactivity and strength gain (Little, 1995) and many other soil properties can be estimated from UCS (Little, 1995; Thompson, 1965 and 1966, cited in Thompson, 1967).

Soil properties which can be estimated from UCS are among others cohesion, one of the components of shear strength and Poisson's ratio (TRB, 1987, cited in Little, 1995); tensile strength (Little et al., 1987); compressive static modulus of elasticity and resilient modulus (Little, 1995). Cohesion is approximately estimated to 30% of UCS, tensile strength is approximately equal to 0.13 of UCS and the compressive static modulus of elasticity can be given by E (ksi) =10+0.124 UCS (in psi).

Apart from the wide use of UCS mentioned above, it is also one of the methods recommended by the U.S. Department of the Interior Bureau of Reclamation (USBR) (1998) to estimate the quantity of required lime for soil treatment. The quantity of the required lime can also be governed by other design purposes such as optimum pH, plasticity index reduction, strength gain, and prevention of harmful volumetric change (USBR, 1998).

For all those reasons, during this study, the UCS test was used to determine the ash content used for other tests carried out on different clays (see Section 4.2). A series of UCS was conducted on both untreated and treated clay materials. UCS on untreated material was done according to ASTM D2166-06 regarding standard test method for unconfined compressive strength of cohesive soil and the UCS on treated materials was conducted according to ASTM D5102-09 related to standard test methods for unconfined compressive strength of compacted soil-lime mixtures.

3.3.3.1. Preparation of UCS specimens

Specimens were prepared according to ASTM D5102-09 (2009). However, two different procedures were used for Clay 1 on one hand and Clay 2 and 3 on the other hand, as explained in the following. Procedure A consists in preparing and testing UCS specimens for which the height-to-diameter ratio is between 2.00 and 2.50 whereas procedure B consists in preparing and testing specimens using test methods D698 compaction equipment (ASTM 698-07, (2007). For method A, the standard compacted specimen was cut into four parts and each part was trimmed to obtain a cylindrical UCS specimen.

a. UCS specimen for Clay 1

As mentioned above in Section 3.3.2, compaction tests were carried out on both untreated and treated soil samples to determine the optimum moisture content and maximum dry density for each mixture (0, 4, 8, 12, 16, 20 and 24% paper mill ash content). First attempt made to prepare specimens at optimum moisture content according to procedure A (ASTM D5102-09) failed because the samples were breaking. To fix this problem, specimens in standard compaction moulds were compacted at 5% above optimum moisture content. After compaction, the specimens from standard compaction mould were cut into four parts and then carved to make specimens of 50mm diameter and 100mm height using a spatula and a metallic sharp mould (see Figure 3.7). At least two specimens without defects were prepared and one was tested immediately after preparation and another one (or more) was cured in a water bath at 25°C during 7 days. The test was conducted using triaxial machine without confinement.

Figure 3.7. Tools used to make small UCS specimen and specimen preparation of Clay 1

Figure 3.8. Specimens before and after testing, Clay 1

b. UCS specimens for Clay 2

As for Clay 1, an attempt was made to prepare small UCS specimens according to ASTM D5102-09 (2009) procedure A. Unfortunately, after adding ash, the compacted specimens were friable such that the specimens could not be made neither at optimum water content nor at 5% above optimum moisture content (OMC) like in the case of Clay 1 (see Figure 3.9). Consequently, the UCS specimens were prepared according to procedure B of that standard. The specimens prepared at different ash contents (0, 8, 12, 16, 20, 24, 28, 32% ash content) were duplicated and one set of specimens was tested without curing and another one was cured in a water bath at 25°C during 7 days. Figure 3.10 shows the specimens before and after testing.

(a) (b)

Figure 3.9. Specimen for Clay 2 treated at 12% ash, compacted at 5% above OMC : (a) before cutting into 4 portions, (b) cut friable portions.

Figure 3.10. Specimen treated at 12% paper mill ash compacted at optimum moisture content before and after testing

c. UCS specimens for clay 3

For Clay 3, UCS specimens were prepared and cured in the same way as for Clay 2. However the paper mill ash content was limited to 28 % (0, 8, 12, 16, 20, 24, 28% ash content).

3.3.4. California Bearing Ratio (CBR)

Even though CBR test is not included in tests recommended by ASTM D4609 to assess the effectiveness of soil stabilizers, it has been conducted during this study due to its great role in various big projects such as pavements. According to McNally (1998), the CBR test is the most widely used to evaluate the subgrade strength. In addition to this, the CBR is also among tests which indicate the immediate improvement of soil stabilizer (Little, 1995). During this study, CBR test was carried out on unsoaked and 4 days soaked specimens.

3.3.5. Swell tests

These tests were carried out in accordance with method A of ASTM D4546-08 to determine free swell and swell pressure using normal consolidometer apparatus. Method A consists of applying different loads on different identical specimens to build up different stresses. The range of stress values have to include the stress due to vertical overburden pressure plus any other stress imposed by the structure to be erected. The free swel is defined as the percentage of swell under stress more or less equal to 1kPa. During this study, a stress of 1.2kPa was applied on the first specimen. According to the standard, a minimum of 4 specimens are required for Method A. Depending on the value of the stress built up, the inundation of

specimen results either in swelling, collapse or both (ASTM D4546-08). For each stress, a time-swell curve or time-collapse curve was plotted and swell or collapse strains were computed as follows:

$$\varepsilon_s = \frac{100\Delta h_2}{h_1}$$

$$\varepsilon_c = \frac{-100\Delta h_2}{h_1}$$

Where:

ε_s= swell strain, %, shown as positive,

ε_c= collapse strain, %, shown as negative,

h_1=specimen height immediately prior to wetting, and

Δh_2= change in specimen height: swell or collapse after wetting

3.4. TESTING PROGRAM

Figure 3.10 shows the experimental plan followed to achieve the objective of this study. From that figure, it's clear that a set of tests have been carried out on untreated material and the results served as a reference to assess the effect of paper mill ash on different expansive soil properties.

After conducting tests on untreated soil, the later was mixed with paper mill ash at different contents and then compacted to determine the optimum moisture content. The obtained optimum moisture content was used to prepare specimens for unconfined compressive strength (UCS) test in order to determine the optimum ash content for each material. After selecting the ash content for each clay (see Section 4.2), all tests carried out on untreated materials were repeated, except UCS tests and standard compaction for which the results were available.

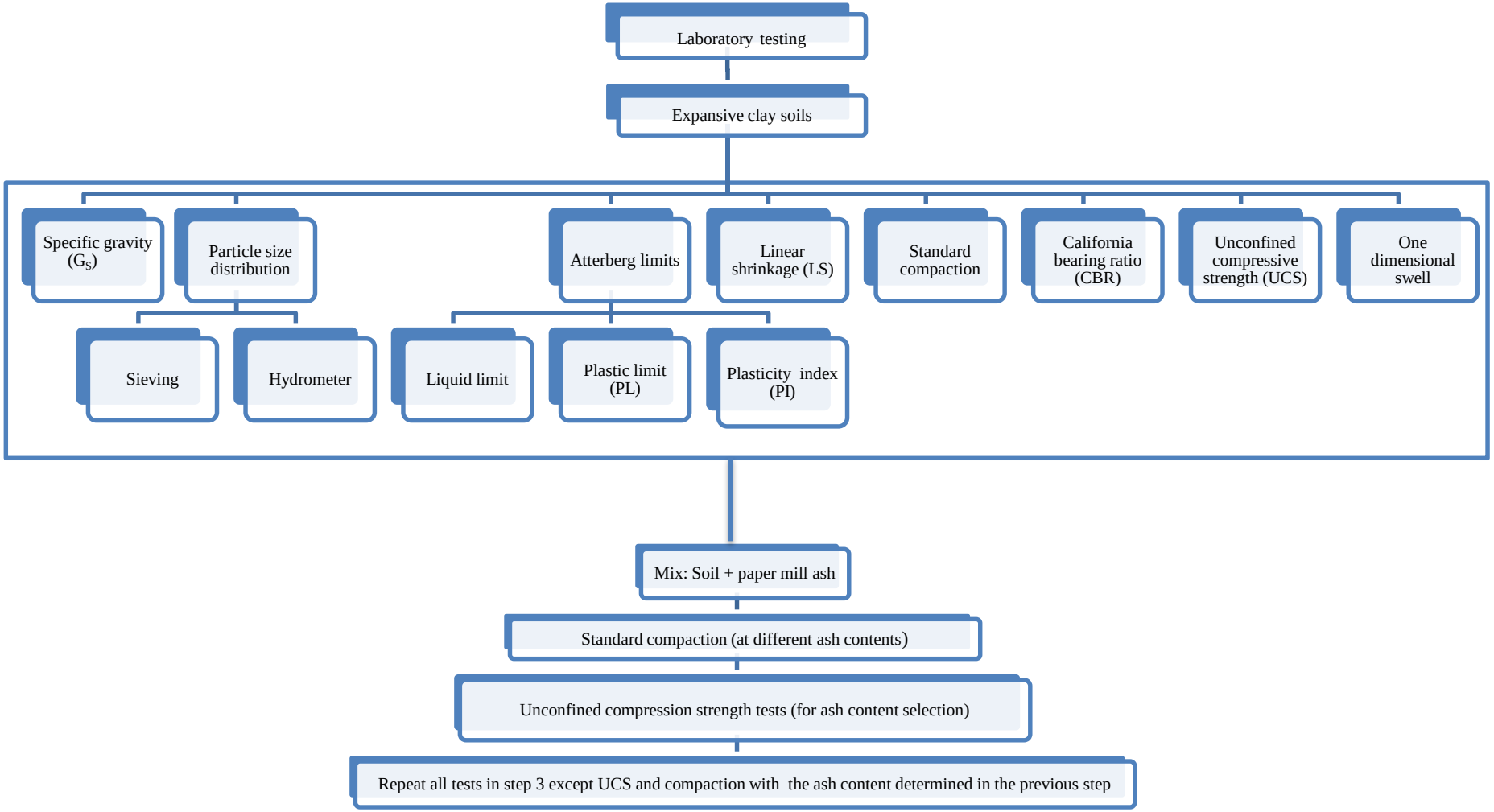


Figure 3.11. Laboratory testing flowchart

3.5. CONCLUSION

Chapter 3 has described the design of this thesis as well as the methodology adopted to achieve the objective of this study. This chapter provided the composition of different materials used in this study, test methods and the testing procedure. The chemical analysis carried out on clay materials, using Scanning Electron Microscopy (SEM), showed that three clay materials used in this study were composed mainly of silica and alumina responsible for the formation of cementitious materials once clay materials are mixed with lime or lime-rich products. Furthermore, the chemical analysis of the paper mill ash, using SEM showed that the material is rich in calcium oxide. Based on the chemical composition of both clay and paper mill ash materials, it could be expected that the clay materials will react with the available lime from the ash to initiate pozzolanic reactions responsible for strength improvement. Concerning the testing methods, all tests were conducted in accordance with ASTM specifications. To achieve the goal of this study, consisting in evaluating the effect of paper mill ash on swelling soils, a series of tests were selected to be conducted on both untreated and treated clay materials ranging from low to high swelling soils according to their index properties. Briefly, this chapter provided a comprehensive description of materials and testing procedures important for the next chapter regarding results analysis and discussion.

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CHAPTER 4 RESULTS ANALYSIS AND DISCUSSION

4.1. INTRODUCTION

This chapter covers presentation of laboratory testing results, their analysis and discussion. As mentioned in chapter three, the tests conducted include grain size analysis, Atterberg limits, compaction characteristics, unconfined compressive strength, swell tests and California Bearing Ratio (CBR). All these tests were first carried out on untreated clays and then on soil treated with paper mill ash as determined based on the unconfined compressive strength test.

4.2. DETERMINATION OF ASH CONTENT USED FOR VARIOUS TESTS

As mentioned in Section 1.4 of the Chapter 1, this study aims to assess the effect of the paper mill ash, rich in lime, on the properties on expansive soils. For that, the approach used for lime stabilization was considered during this study. According to the literature, different methods have been developed to determine the optimum lime content for soil modification and stabilization. The most widely used method is the pH test developed by Eades and Grim (Departments of the Army and Air Force, 1994) which consists of determining the lowest percent lime to produce a pH of 12.4. This lime content is termed the “lime modification optimum” or LMO.

Although this method is widely used, Currin, Allen and Little (1976, cited in Nelson and Miller, 1992) found that this amount of lime doesn’t ensure the soil-lime reactivity. For that reason, Nelson and Miller (1992) recommended other factors such as the Plasticity Index (PI) and the soil strength which have to be considered to assess the effectiveness of lime use for expansion reduction.

Another method to determine the optimum stabilizer, known as Illinois procedure, is based on the maximum unconfined compressive strength (UCS) (Khattab, Al-Juari and Al-Kiki, 2008). By this method, the optimum lime content is given by the lime content corresponding to the peak of lime content-strength curve for specific soil and curing conditions (Thompson, 1967). However the study carried out by Jambor (1963, cited in Thompson, 1967) showed that soil stabilized with excessive lime may exhibit high porosity and results in strength reduction of lime-pozzolan reaction products.

It is clear from the mentioned studies that the lime content strongly depends on the purpose of the treatment. For instance, the lime required for soil modification will be less than the amount required for soil stabilization and the amount required for volume change control will differ from the amount required for maximum strength.

During this study, the paper mill ash content, used to assess its effect on expansive soil properties, was determined based on the strength improvement. The procedure followed for different clay materials used in this study is mentioned in Sections 4.2.1 to 4.2.3, here below.

4.2.1. Paper mill ash used for Clay 1

The ash content used to study its effect on Clay 1 properties was determined as the percentage required for maximum unconfined compressive strength after 7 days curing. To this end, soil samples were prepared and mixed with ash at different percentages based on the mass of oven-dry material then compacted to determine the optimum moisture contents (OMC) as well as the maximum dry density (MDD) for each soil-ash mixture. The quantity of ash required for each compaction specimen was determined as follows:

$$C=\frac{YW}{100}$$

Where

C= mass of ash required,
Y= percentage of ash required,
W= mass of oven-dry clay material.

After determining the OMC, the specimens for UCS were compacted. As mentioned in Chapter 3, Section 3.3.3.1, UCS specimens for this Clay 1 were prepared according to the procedure A of ASTM D5102-09 (2009) at 5% above OMC. The compaction test results to determine OMC and MDD are given in Table 4.4 and the moisture-density relationships are illustrated with Figure 4.6. After UCS testing of 7-days cured specimens prepared at different ash contents, it was found that the maximum UCS occurred at 20% ash. This percentage was used to assess the effect of the paper mill ash on different properties (as mentioned in Section 4.1) of Clay 1.

4.2.2. Paper mill ash used for clay 2 and 3

For these clays, the same procedure as for Clay 1 was applied with some changes dictated by state of the specimens. The common characteristic of the UCS specimens for Clay 2 and Clay 3 is that the attempt made to prepare the specimens according to the procedure A of ASTM D5102-09 (2009) was not successful for both clays neither at OMC nor at 5% above OMC as determined from compaction tests at different ash contents. The specimens from compaction were friable such that they were breaking easily. For that reason, UCS specimens for Clay 2 and Clay 3 were prepared according to the procedure B of ASTM D5102-09 (2009) as described in Section 3.3.3.1. Despite this change, it was also difficult to extrude the specimens for Clay 3 compacted at OMC since they were breaking easily. This results in preparing UCS specimens for Clay 3 at 3 % above OMC.

The unconfined compressive strength tests carried out on Clay 2 and Clay 3 showed that, after 7-days curing, the ash content-UCS was still going up and not reaching the peak at higher ash content than Clay 1. Therefore, for both clays, a new approach to determine the ash content to be used to assess the effect of the ash on clay properties was established.

According to ASTM D4609-08 (2008), one of the indications of the effectiveness of a soil stabilizer is the increase in UCS of 345 kPa. Based on this, the ash percentage used to assess the effect of paper mill ash on properties of Clay 2 and Clay 3 was chosen to be the minimum ash content required to produce an increase in UCS of 345 kPa after 7 days curing at 25° Celsius. This percentage was found to be 24% and 8% for Clay 2 and Clay 3 respectively. The compaction and UCS tests results are given in Sections 4.4 and 4.5 respectively.

4.3. GRAIN SIZE ANALYSIS AND ATTERBERG LIMITS

As stated in Section 4.1 of this chapter, each test, except compaction and UCS tests, was conducted twice, on both untreated and treated material. The untreated material served as reference to evaluate the effect of the predetermined ash content on studied clays. This section shows the results of grading and Atteberg limits. From the results of these tests, different clays studied were classified according to ASTM D2487-11 (2011). To complete the classification, liquid limits on oven dried samples were determined for the three clays. For all of them it was found that the ratio between the liquid limit on oven dried soil and air-dried soil was greater than 0.75, which means that all soils were inorganic according to ASTM D2487-11.

For both tests, the soils samples were prepared according to ASTM D421-85 (2007). The grain size distribution results of different clays are given here below.

For all hydrometer analyses, an ASTM hydrometer 152 was used and for dispersion, a solution of sodium hexametaphosphate was used. The dispersing apparatus with dispersing paddle was used during the test.

4.3.1. Clay 1

The grain size distribution of the Clay 1 before and after treatment at 20% paper mill ash and paper mill ash is shown in Figure 4.1. From the figure, it is seen that the percentages of all fractions of soil particles finer than 0.3mm (300µm) have increased. From Figure 4.1, it can also be seen that all three materials are fine-grained soils since the fraction of soil passing 75µm sieve is more than 50% for all of them. The fines or portion of soil finer than 75µm have increased from 62.8% for untreated Clay 1 to 76.1% after treatment with 20% paper mill ash. The reason of this is that, more than 90% of the paper mill ash particles are smaller than 75µm which will contribute to the increase in fines. Another reason is that, the grading was carried out immediately after mixing and at dry state such that there was neither time nor water to activate reactions between ash-clay materials.

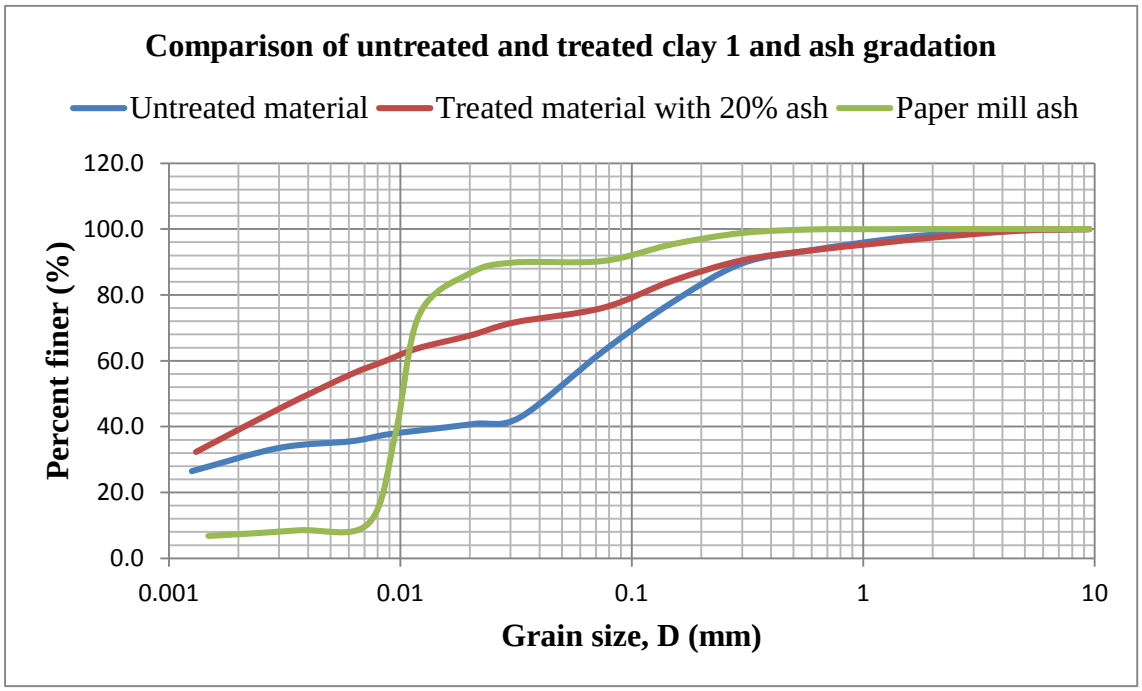

Figure 4.1. Grain size distribution of clay 1 and paper mill ash.

Figure 4.1 shows that after mixing, the grading of Clay 1 was strongly affected by the paper mill ash. The percentage of all particles finer than 0.3 mm in diameter was increased with paper mill ash stabilization.

The Atterberg limits test was conducted immediately after mixing. The test results for Clay 1 before and after treatment are given in Table 4.1. The results showed that liquid limit (LL) and plastic limit (PL) as well as the plasticity index were increased with the addition of the ash. The increase in Atterberg limits is due to the increase in percentage of clay fraction. This is in accordance with the findings of different researches which showed that there is a correlation between clay fraction and Atterberg limits (Dumbleton and West, 1966; Clare [no date]; Odell, Thornburn, and McKenzie, 1960; Davidson and Sheeler, 1952; Brown and Mengel, 1983; Novais-Ferreira 1967, cited in Cerato, 2001).

In order to be sure that the stabilization results in plasticity reduction or not, Gautrans (2004) recommends the use of soil materials from UCS cured specimens after testing, for Atterberg limits.

Table 4.1. Atterberg limits, PI and linear shrinkage (LS), Clay 1

Ash content (%)		LL	PL	PI	LS
Before treatment (C1BT))	0%	44	29	15	7.1
After treatment (C1AT))	20%	63	42	21	7.0

Note that C1BT stands for Clay 1 before stabilization and C1AT is referred to as Clay 1 after treatment. These Atteberg limits value are used in Figure 4.2 for clay classification according to the Unified Soil Classification (USCS).

The classification of Clay 1 before and after treatment, according to ASTM D2487-11 (2011) is shown in Figure 4.2. To complete the Unified Soil Classification (USCS), the liquid limit on oven dried (untreated) sample was determined and it was found to be 36, which corresponds to a ratio of 0.8 between liquid on oven-dried and air-dried soil. This ratio allows to distinguish inorganic from organic soil.

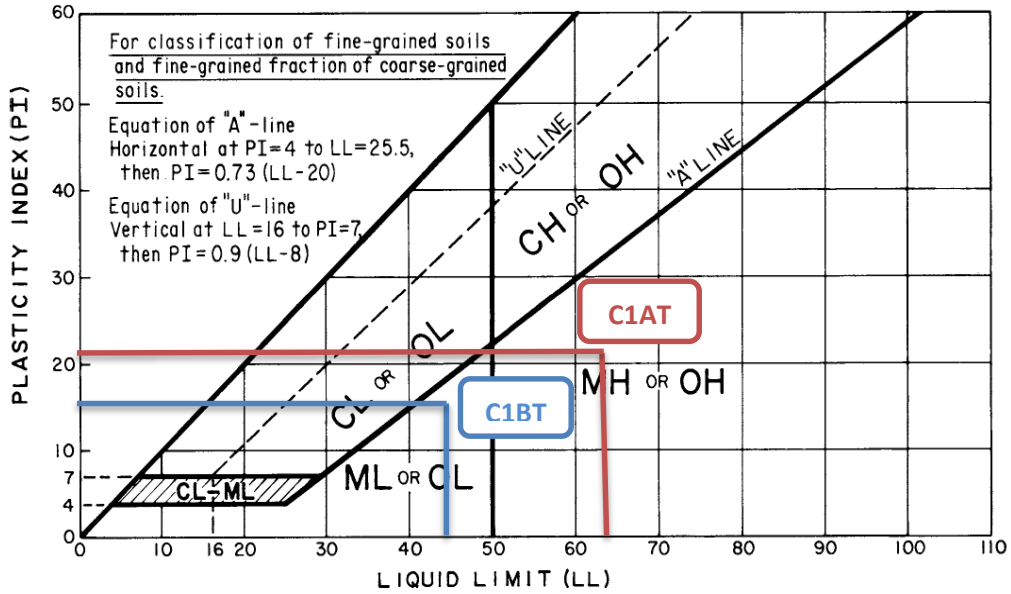

Figure 4.2. Plasticity chart for Clay 1 classification (Modified from ASTM D2487 – 11, 2011).

According to the plasticity chart (Figure 4.2) used for classification of fine-grained soils and fine-grained fraction of coarse-grained soils, before treatment the material was silt (ML) and it shifted to elastic silt (MH) after treatment.

4.3.2. Clay 2

The grain size distribution of the Clay 2 before and after treatment at 24% paper mill ash and paper mill ash is shown in Figure 4.3. The Atterberg limits and linear shrinkage values are given in Table 4.2 and the classification is shown in Figure 4.4. Contrary to Clay 1, the treatment of Clay 2 at 24% results in Atterberg limits reduction.

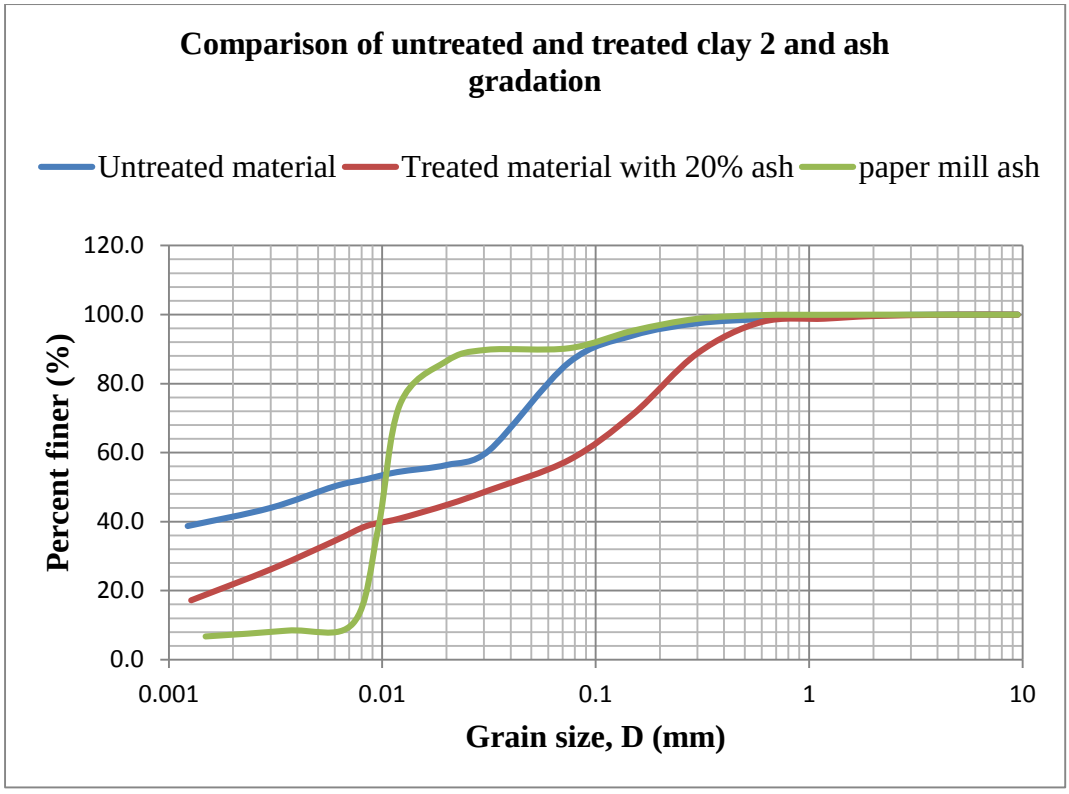

Figure 4.3. Grain size distribution of Clay 2 and paper mill ash.

Table 4.2. Atterberg limits, PI and linear shrinkage (LS), Clay 2

Soil constant		LL	PL	PI	LS
Before treatment (C2BT)	0%	61	30	31	10.7
After treatment (C2AT)	24%	51	39	12	5.2

Figure 4.4 shows that the material changed from fat clay (CH) to elastic silt (MH). This is the cause of high reduction in Atterberg limits. The liquid limit of the oven-dried clay was found to be 48. The ratio between liquid on oven-dried and air-dried (untreated) soil is therefore equals to 0.79 greater than 0.75. According to USCS, Clay 2 is inorganic material.

As for Clay 1, C2BT is referred to as Clay 2 before mixing with ash and C2AT is Clay 2 after treatment. The change of the material from fat clay to elastic clay would make the workability of ash-treated clay materials.

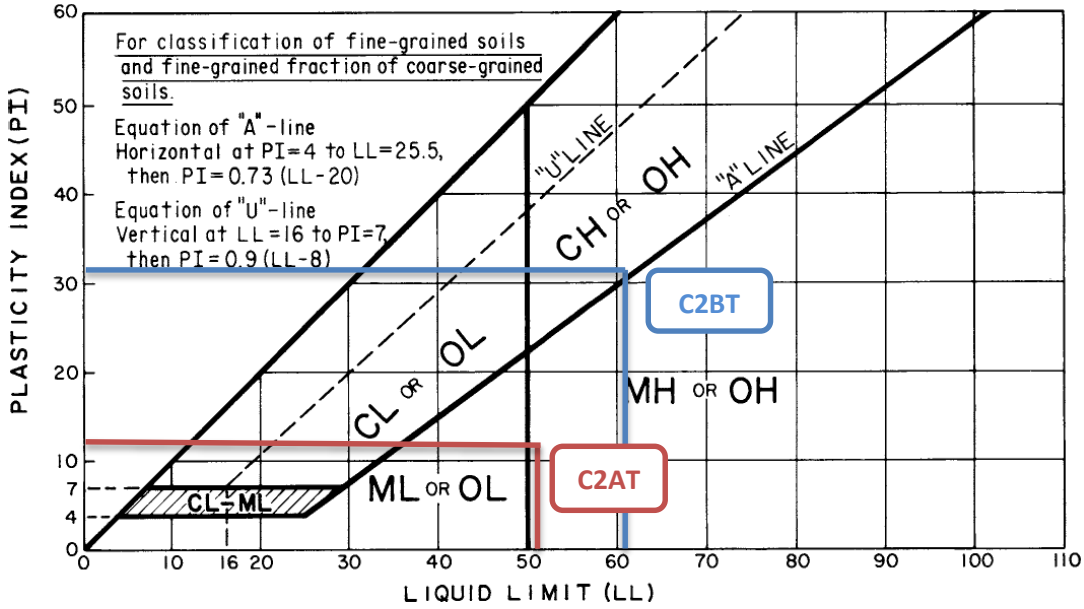

Figure 4.4. Plasticity chart for clay 2 classification (Modified from ASTM D2487 – 11, 2011).

4.3.3. Clay 3

As for Clay 1 and Clay 2, the gradation, the Atterberg limits and linear shrinkage, as well as the classification of Clay 3 before and after treatment are given in the following.

Figure 4.6 shows that the material 3 changed from lean clay (CL) to silt (ML). As for Clay 2, this also resulted in Atteberg limits reduction.

Table 4.3. Atterberg limits, PI and linear shrinkage (LS), Clay 3

Soil constant		LL	PL	PI	LS
Before treatment (C3BT)	0%	46	23	23	7.2
After treatment(C3AT)	8%	43	33	10	6.1

C3BT and C3AT were referred to as Clay 3 before and after treatment, respectively.

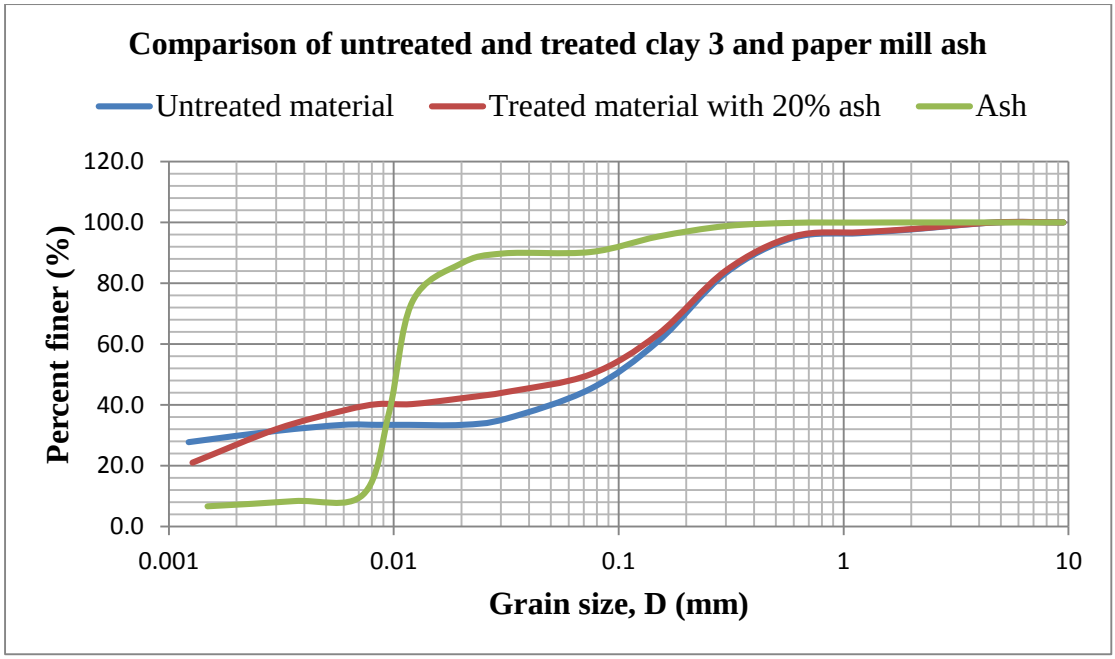

Figure 4.5. Grain size distribution of Clay 3 and paper mill ash

The liquid limit of the oven-dried Clay 3 was found to be 41. The ratio between liquid on oven-dried and air-dried (untreated) soil is therefore equals to 0.89 greater than 0.75. According to USCS, Clay 3 is inorganic material.

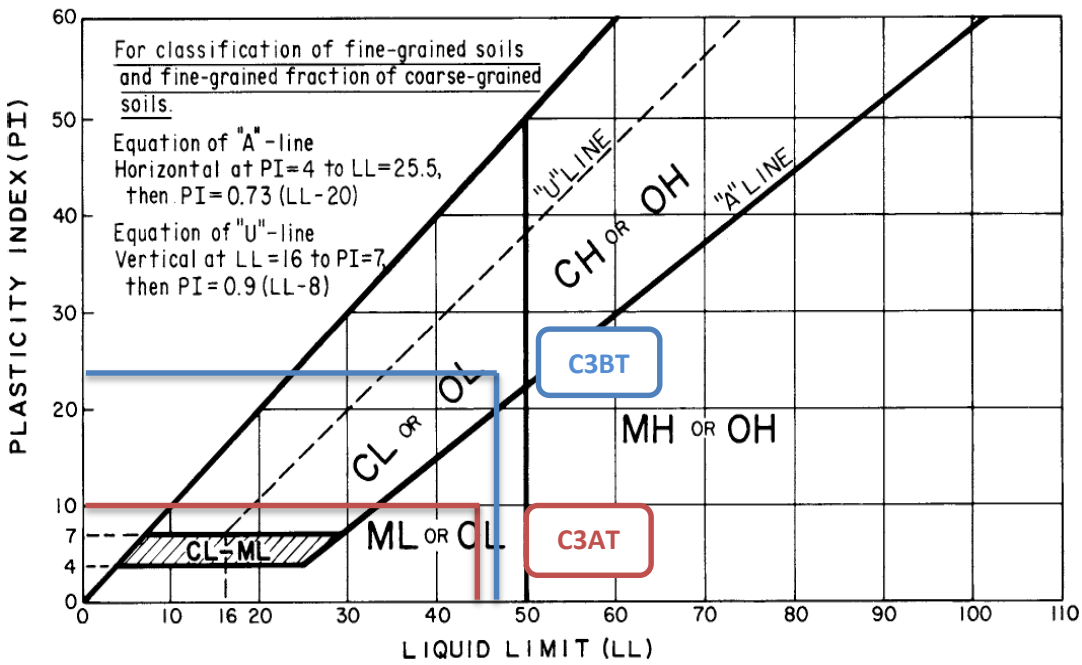

Figure 4.6. Plasticity chart for Clay 3 classification (Modified from ASTM D2487 – 11, 2011).

4.4. COMPACTION CHARACTERISTICS

As mentioned in Section 4.2, different compaction tests were conducted at different paper mill ash contents to determine optimum moisture content used for preparation of unconfined compressive strength specimens. The compaction tests, for all three clays, were conducted in accordance with ASTM D698-07 (2007), Method C with a mechanical rammer. The moisture-density relationship for different ash contents are shown in the following for the different clays.

4.4.1. Clay 1

The moisture-density relationships for different paper mill ash contents are shown in Figure 4.7. From Figure 4.8, it can be noticed that as the ash content increases, the maximum dry density decreases while the optimum moisture content increases. The variation of maximum dry density and optimum water content with ash content is illustrated in Figure 4.8.

Table 4.4. Optimum moisture content (OMC) and maximum dry density (MDD) of Clay 1 at different ash contents

Ash content (%)	0	4	8	12	16	20	24
OMC (%)	16.5	17.3	18	21	21	21.5	24
MDD (kg/m^3)	1695	1639	1570	1555	1523	1518	1479

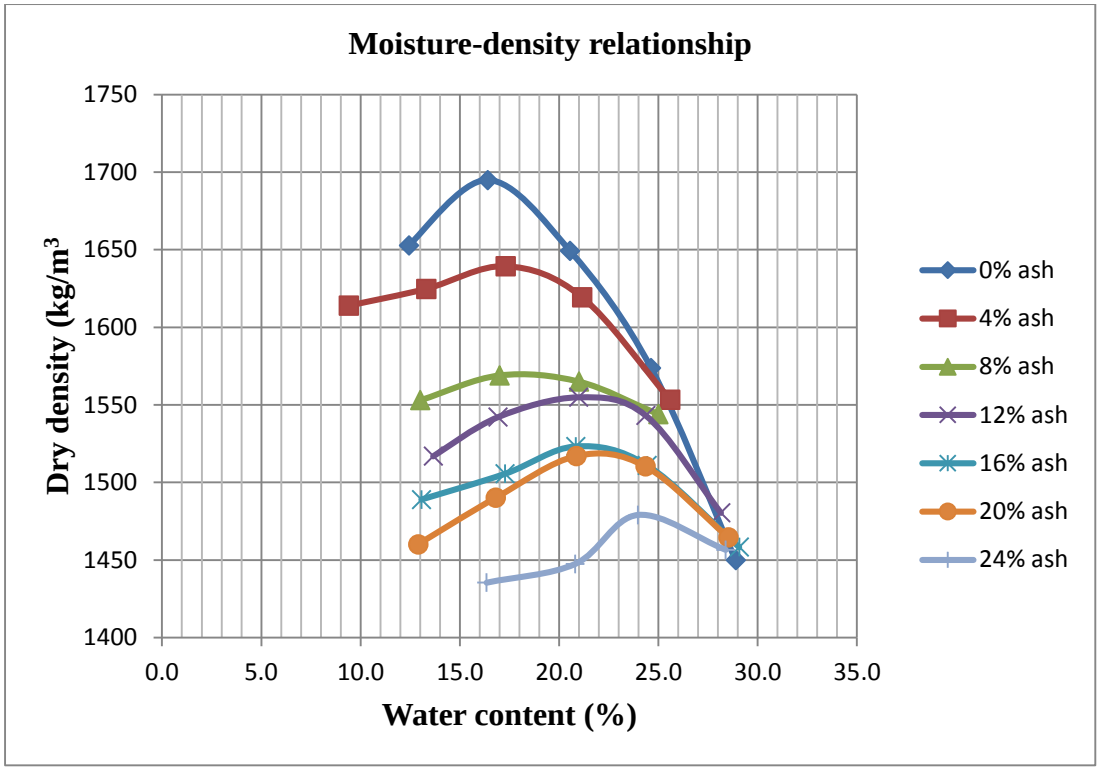

Figure 4.7. Moisture-density relationship for Clay 1 at different ash contents

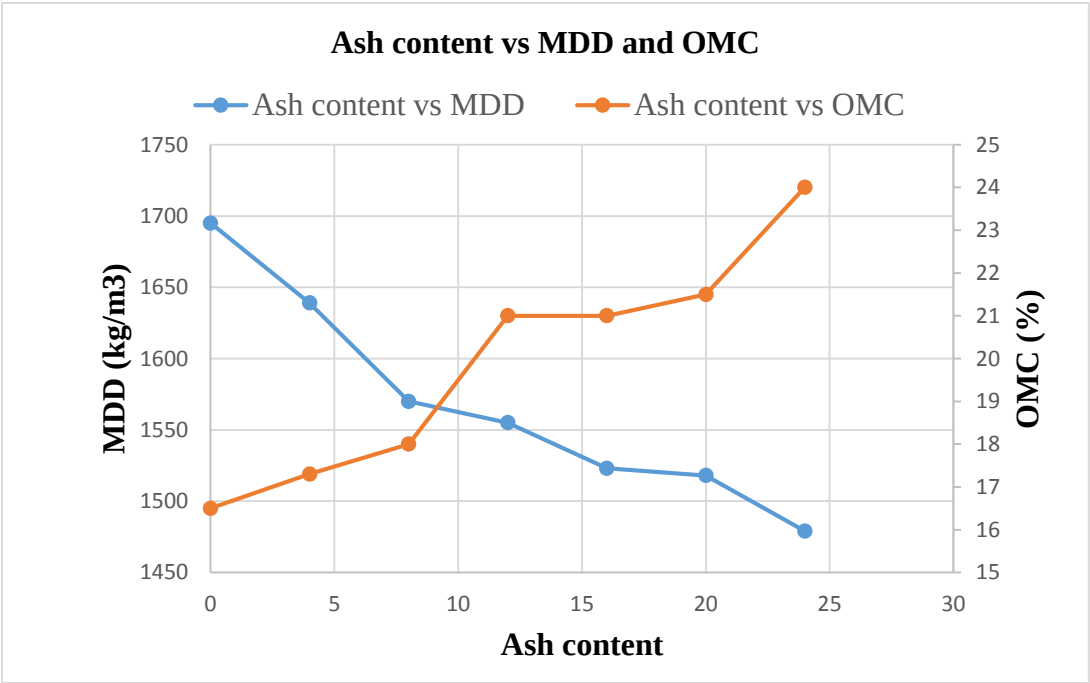

Figure 4.8. Effect of paper mill ash treatment on maximum dry density (MDD) and optimum moisture content (OMC) of Clay 1

4.4.2. Clay 2

In the same way as for Clay 1, the standard compaction test was conducted on Clay 2 treated at different ash contents. Based on the fact that 4% ash had no big effect on Clay 1, the starting ash content for Clay 2 was shifted to 8%. As mentioned in Section 4.2 of the chapter, Clay 2 was compacted at additional ash contents of 28% and 32% compared to Clay 1, in an endeavour to get the peak of ash content-UCS curve but the curve kept going up as shown in the following section related to UCS test. The results are shown in the figures and table here below.

From Figure 4.10, contrary to Clay 1, it seems that there is no correlation between ash content with OMC and MDD. It can also be seen that, from 20% ash, the change in optimum moisture content is very small. Another observation is that, no treated sample reached the OMC of the untreated sample, whereas for Clay 1, the OMC for the untreated material was the lowest.

Table 4.5. Optimum moisture content (OMC) and maximum dry density (MDD) of Clay 2 at different ash contents

Ash content (%)	0	8	12	16	20	24	28	32
OMC (%)	27.5	23.5	26.8	24.5	25.0	25.3	25.0	25.0
MDD (kg/m^3)	1404	1427	1424	1410	1418	1425	1452	1450

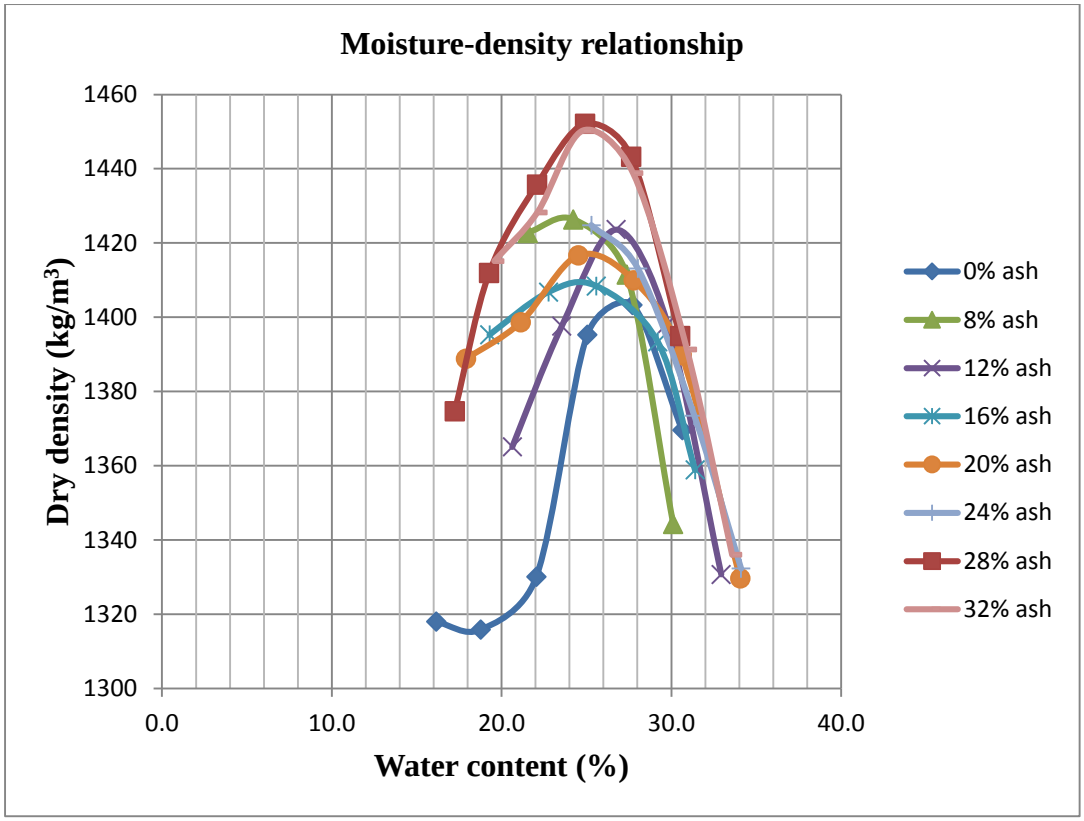

Figure 4.9. Moisture-density relationship for Clay 2 at different ash contents

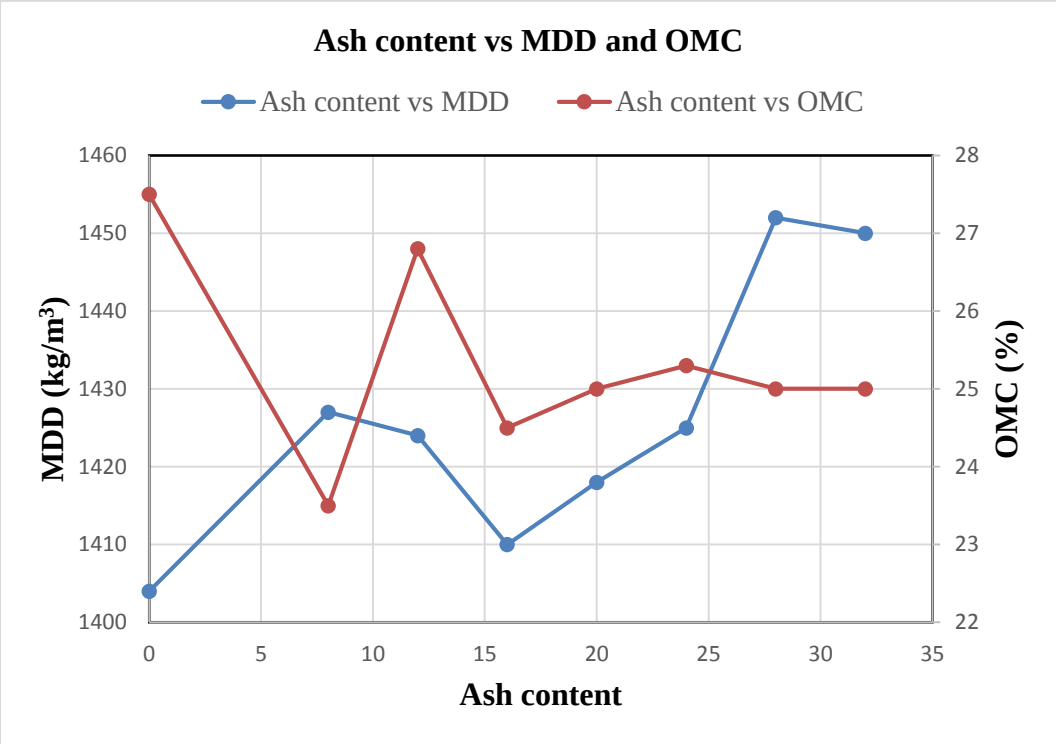

Figure 4.10 Effect of paper mill ash treatment on maximum dry density (MDD) and optimum moisture content (OMC) of Clay 2

4.4.3. Clay 3

The standard compaction results of Clay 3 are also presented in the same way as previous clay materials in the following. Figure 4.12 shows that, while the maximum dry density decreases with increasing ash content, OMC changes slightly with ash content increase.

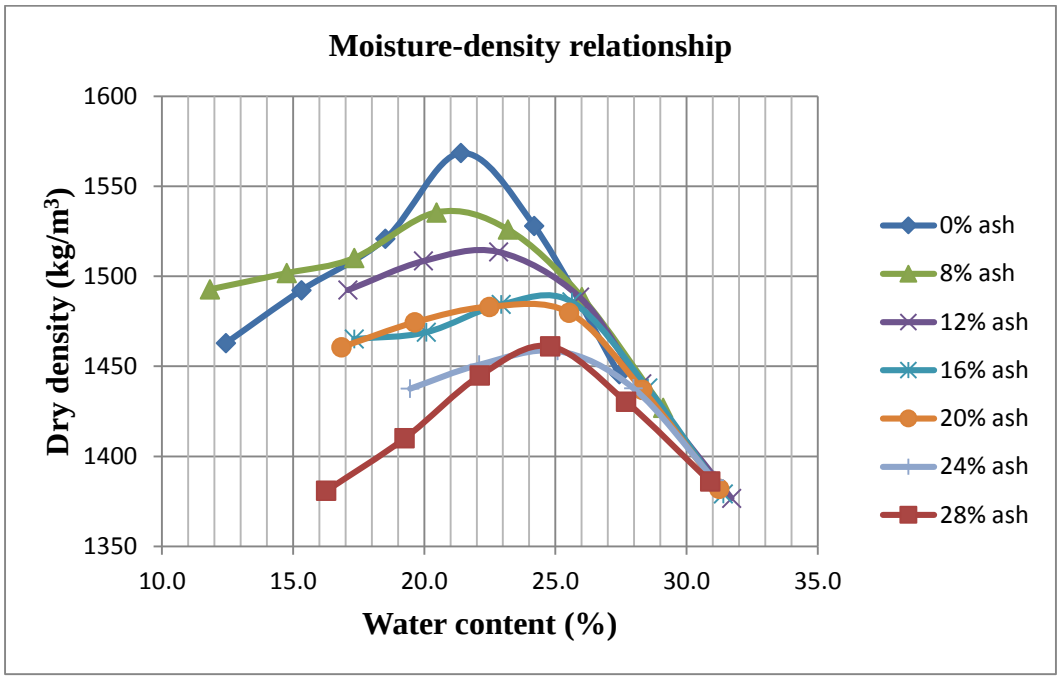

Figure 4.11. Moisture-density relationship for Clay 3 at different ash contents

Table 4.6. Optimum moisture content (OMC) and maximum dry density (MDD) of Clay 3 at different ash contents

Ash content (%)	0	8	12	16	20	24	28
OMC (%)	21.4	21	22	24.8	24	24.8	24.5
MDD (kg/m³)	1568	1536	1515	1488	1484	1459	1462

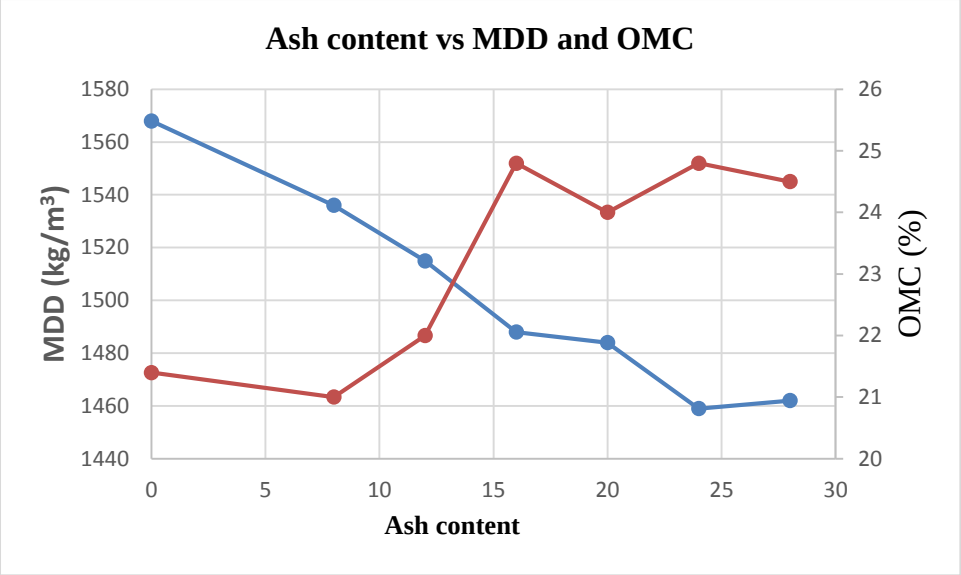

Figure 4.12. Effect of paper mill ash treatment on maximum dry density (MDD) and optimum moisture content (OMC) of Clay 2

The moisture-density relationship curves for ash contents used to study the effect of paper mill ash on soil properties (see Section 4.2) of Clays 1, 2and 3 are separately illustrated in Appendix A.

4.5. UNCONFINED COMPRESSIVE STRENGTH (UCS)

This test was conducted on both cured and uncured specimens prepared from different clay materials following the procedures mentioned in Section 4.2. of this chapter. For all clays, one set of specimens was tested immediately after compaction and another set was cured for 7 days in a water bath at a temperature of 25°C. The results are shown in the following sections.

4.5.1. Clay 1

The effect of paper mill ash treatment on unconfined stress-strain behaviour of Clay 1 for uncured and cured specimens is shown in Figure 4.13 and Figure 4.14, respectively. It can be observed that, for both cured and uncured specimens, while the maximum stress increases with increasing ash content up to 20% ash content, the strain corresponding to the peak or the maximum unconfined strength generally decreases. Therefore, the specimens treated with paper mill ash exhibit more britleness than untreated specimens.

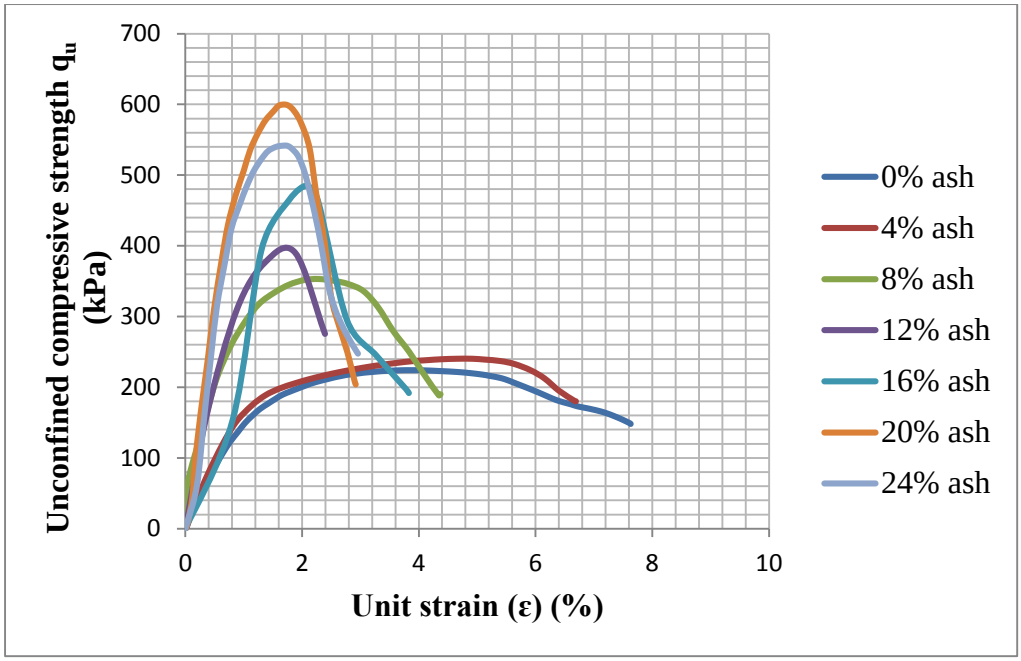

Figure 4.13. Effect of paper mill ash content on unconfined stress-strain behaviour of Clay 1, uncured specimens.

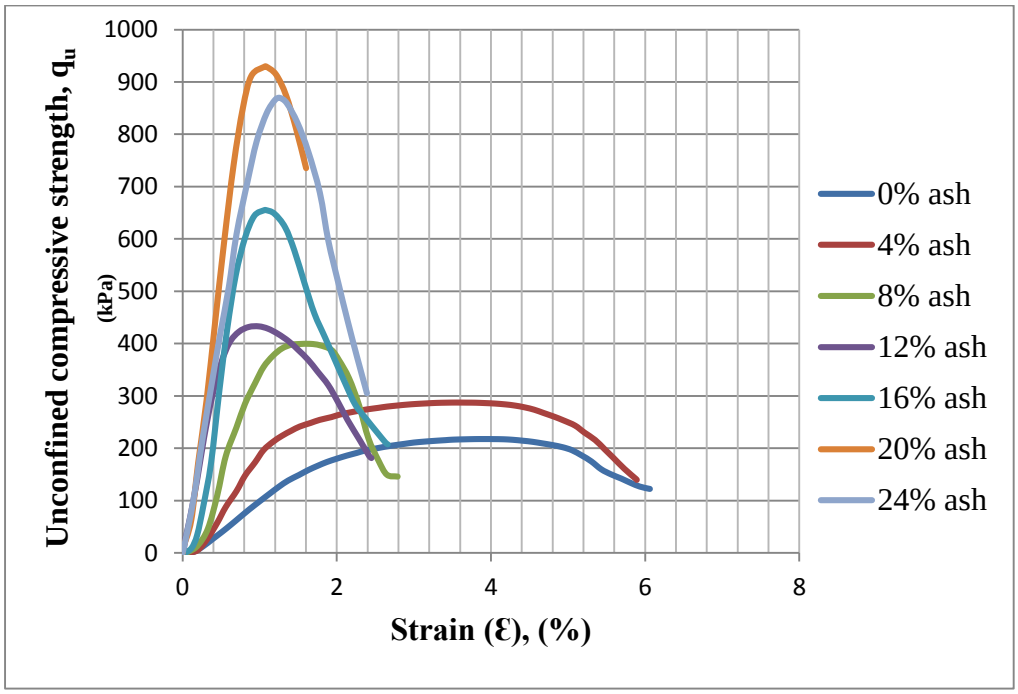

Figure 4.14. Effect of paper mill ash content on unconfined stress-strain behaviour of Clay 1, 7-days cured specimens.

Figure 4.15 reflects that the UCS increased with ash content up to 20% and then dropped for both uncured and cured specimens. This ash content corresponding to the maximum UCS was used to assess the effect of paper mill ash on other properties of Clay 1 mentioned in Figure 3.10. Figure 4.15 also shows that curing has a great effect on paper mill ash treated soil particularly at high ash contents.

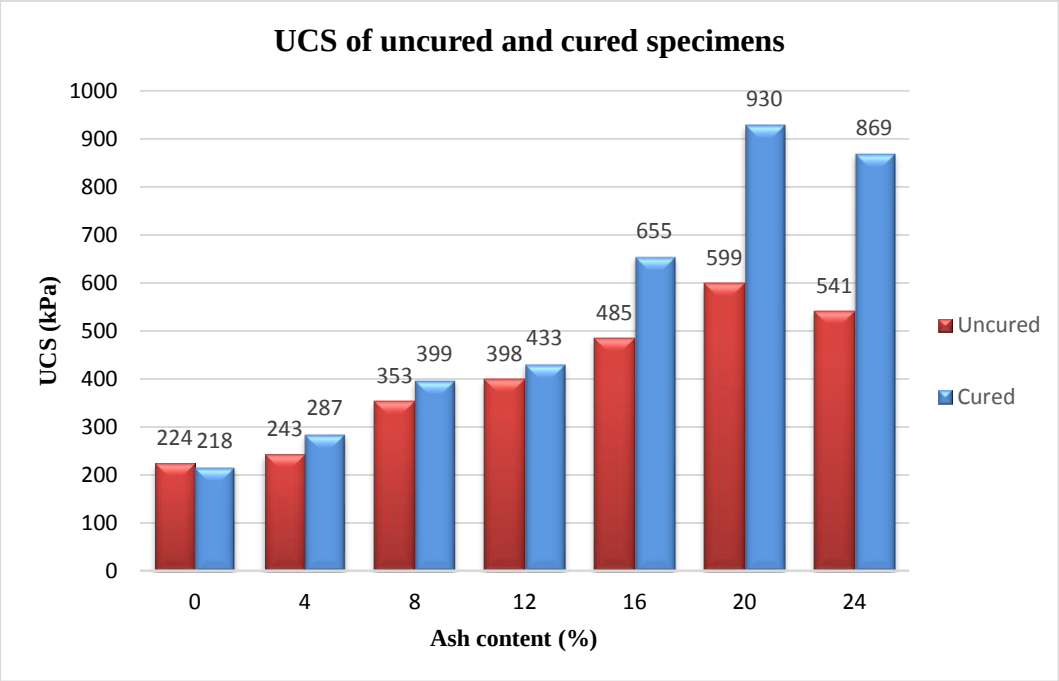

Figure 4.15. Effect of curing on Clay 1 specimens.

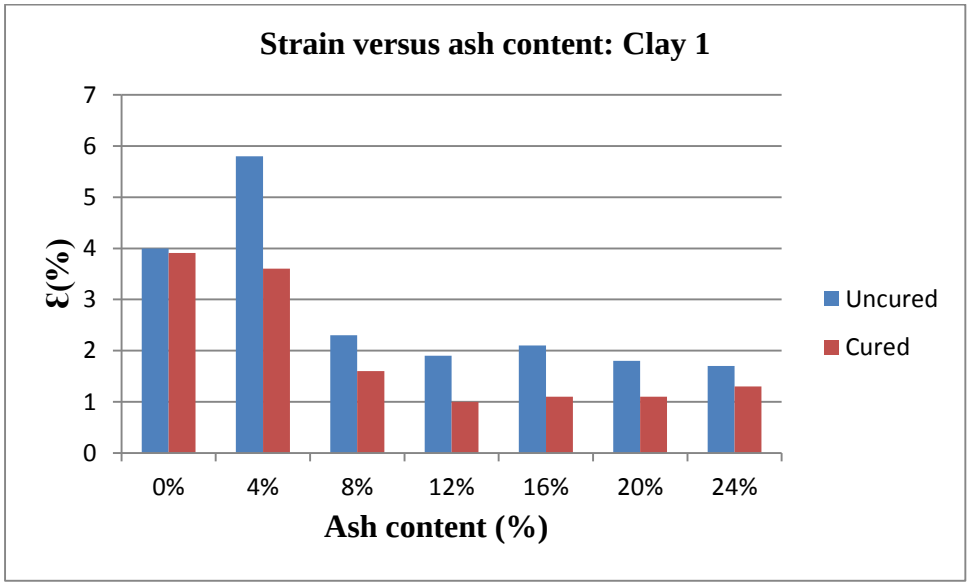

Figure 4.16. Variation of strain with paper mill ash content

Figure 16 shows that strain was reduced as the ash content was increased, with the exception of 4% ash content in the case of untreated material. This decrease in strain shows the stabilized material was becoming more brittle.

4.5.2. Clay 2

The UCS test results for clay 2 are also presented here below in the same way as for Clay 1. Brittle behaviour of spemens also increases with increasing the paper mill ash content.The effect of ash on peak stress-strain behaviour for Clay 2 is shown in Figures 4.16 and 4.17 respectively.

Contrary to Clay 1, no peak was observed on ash content-UCS curve of the cured specimens. As mentioned in Section 4.2, the ash content used to study the effect of paper mill ash on various properties of Clay 2 was the minimum ash content which produced an increase of 345kPa after curing. Figure 4.18 shows that the minimum ash content for wchich an incrase of 345kPa was reached is 24%. This paper mill ash content was used to assess the influence of the ash on some properties of Clay 2.

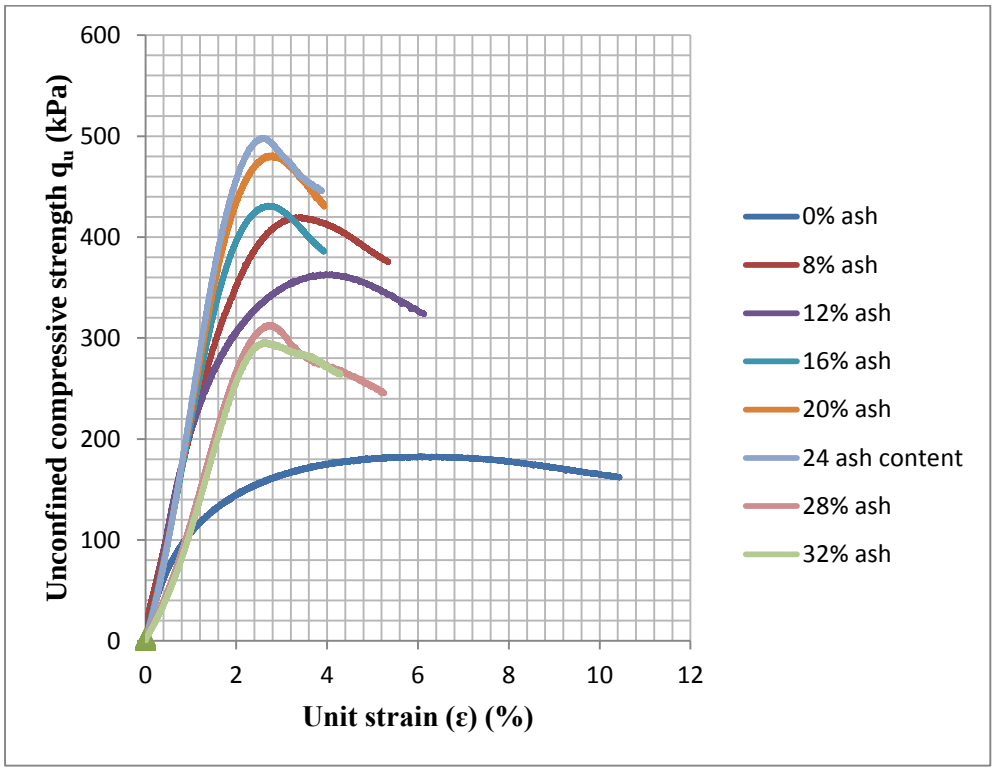

Figure 4.17. Effect of paper mill ash content on unconfined stress-strain behaviour of Clay 2, uncured specimens.

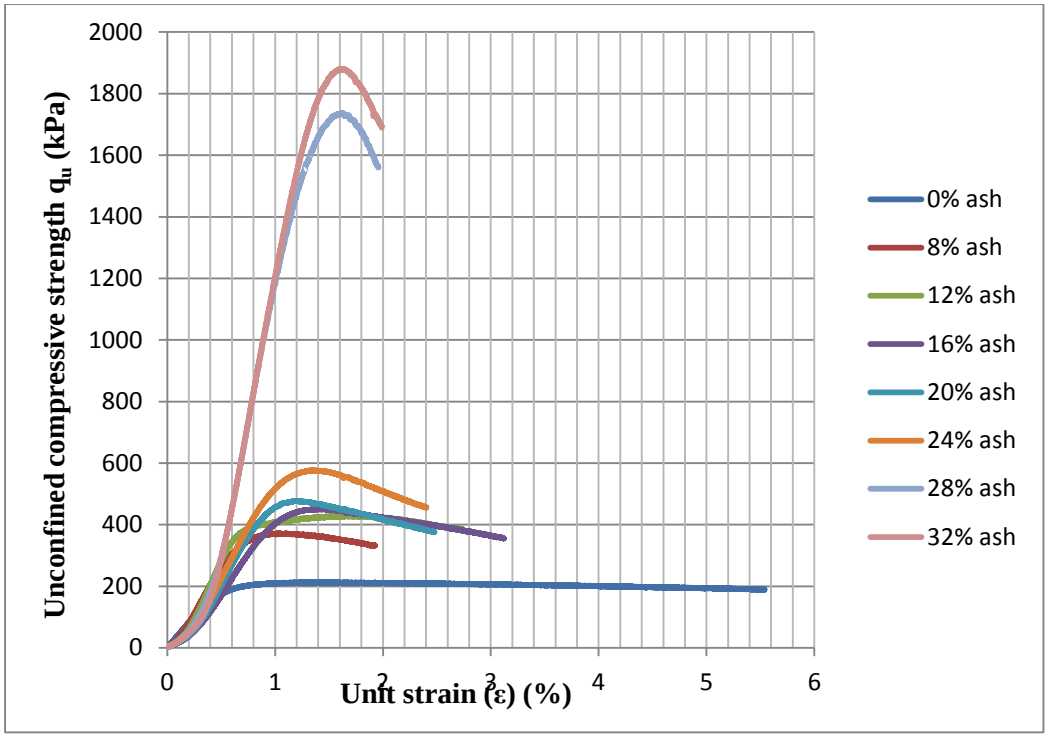

Figure 4.18. Effect of paper mill ash content on unconfined stress-strain behaviour of Clay 2, 7-days cured specimens.

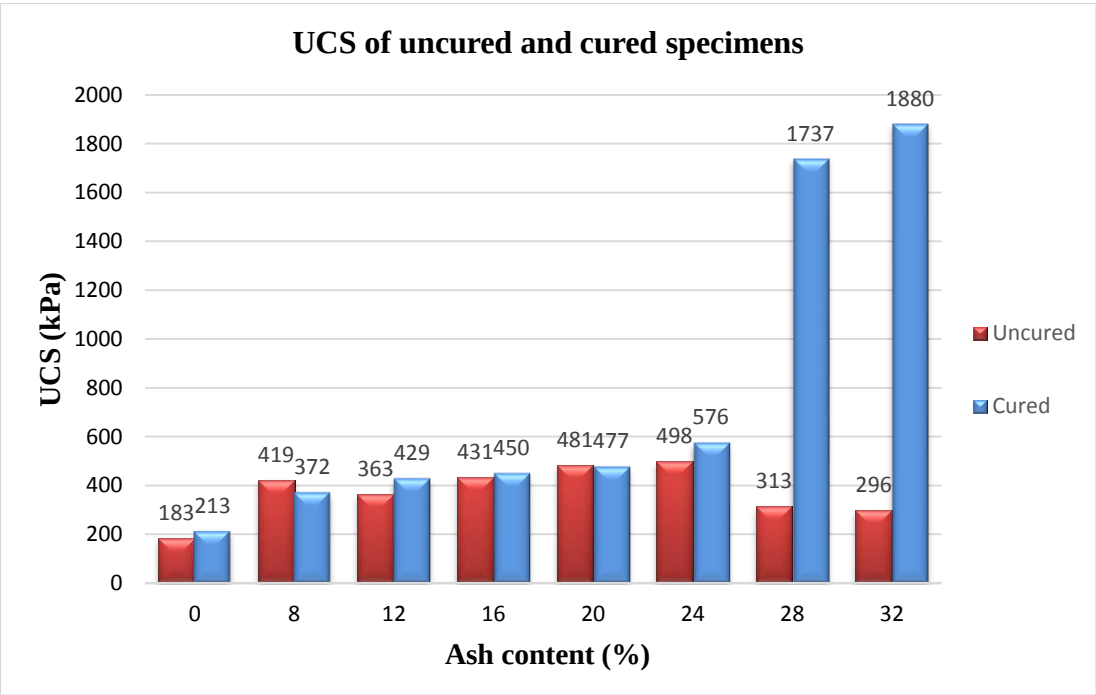

Figure 4.19. Effect of curing on Clay 2 specimens.

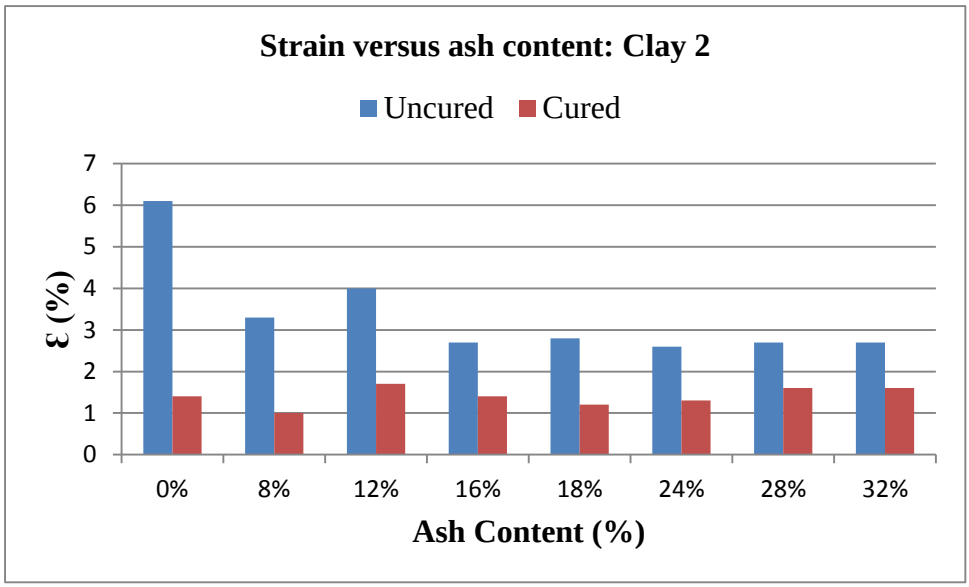

Figure 4.20. Variation of strain with paper mill ash content

4.5.3. Clay 3

The effect of paper mill ash content on UCS of Clay 3 is presented in Figures 4.19 and 4.20 for uncured and cured specimens, respectively. The same approach applied for Clay 2 to determine the ash content to be used for assessment of paper mill ash on properties of Clay 3, was also applied for Clay 3 since no peak on ash content-UCS curve was observed for cured specimens. From Figure 4.21 it can be seen that the minimum ash content to achieve an increase of 345kPa after curing was 8%. The results are shown in figures below.

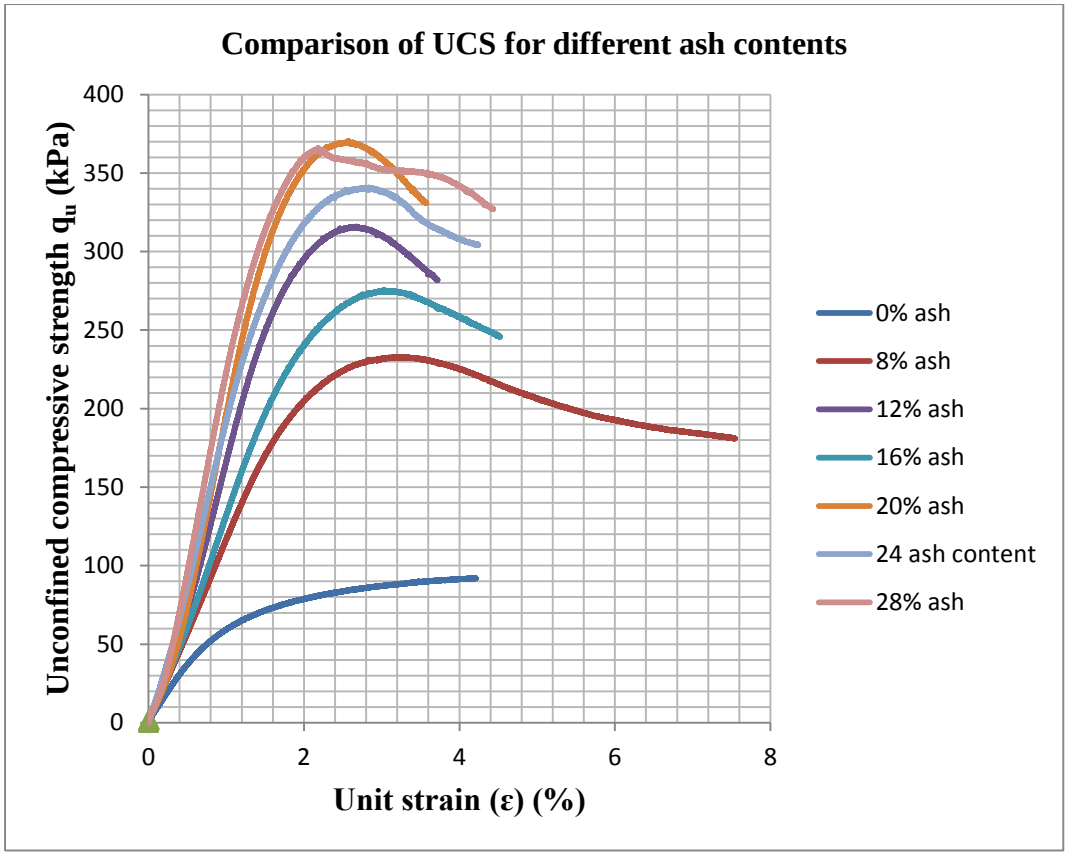

Figure 4.21. Effect of paper mill ash content on unconfined stress-strain behaviour of Clay 3, uncured specimens.

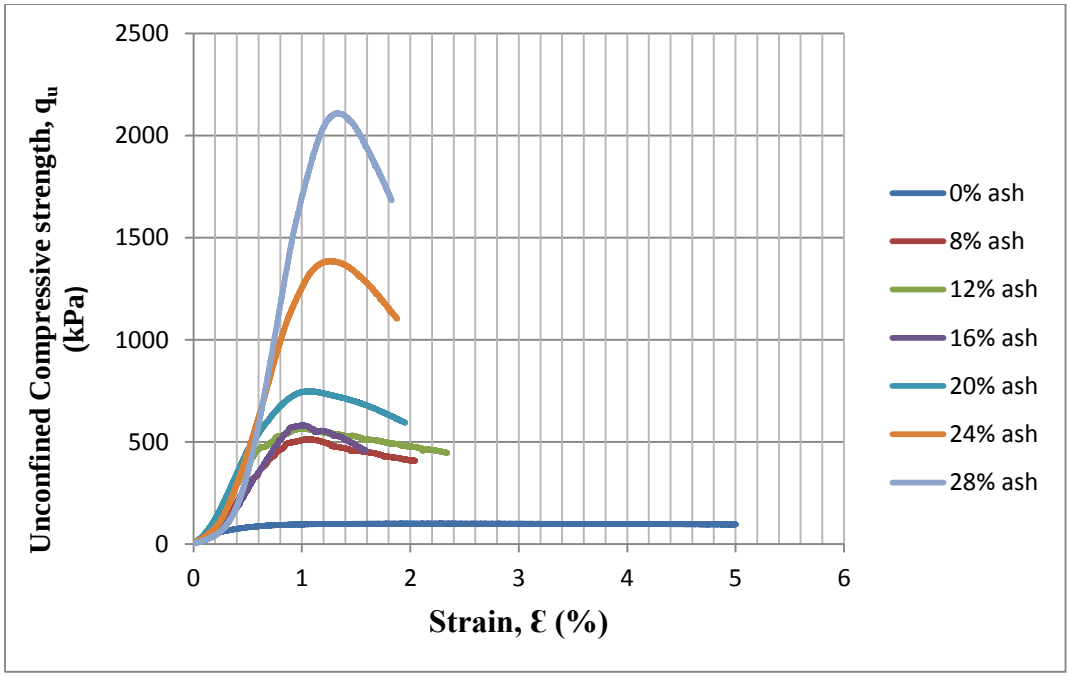

Figure 4.22. Effect of paper mill ash content on unconfined stress-strain behaviour of Clay 3, 7-days cured specimens.

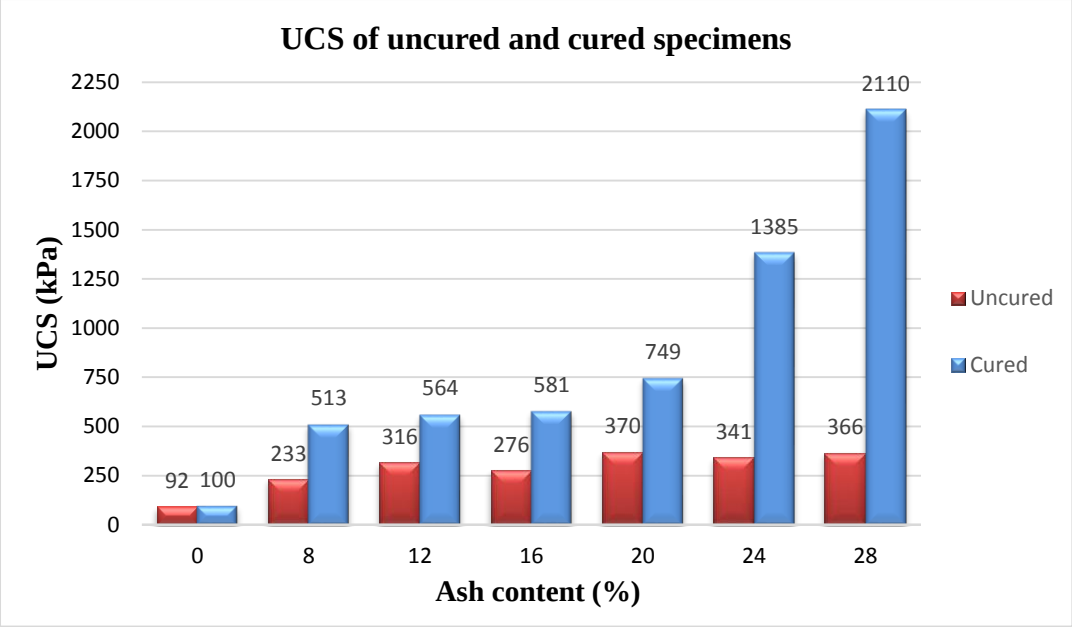

Figure 4.23. Effect of curing on Clay 3 specimens

Figure 4.23 shows that at high paper mill ash content, the uncured specimens exhibited lower UCS compared to low ash content specimens. However, the UCS for high paper mill ash content specimens became very high after 7-day curing. This might be due to the pozzolanic reactions between paper mill ash and clay materials. For instance, the UCS for 28% ash content was increased by 6 times after curing, from 366 kPa to 2110 kPa. In addition to the UCS-strain curves given in this section, other separated UCS-strain curves for the selected paper mill ash contents, used to assess the effect of the ash on swelling clay properties, as defined above, are given in Appendix B.

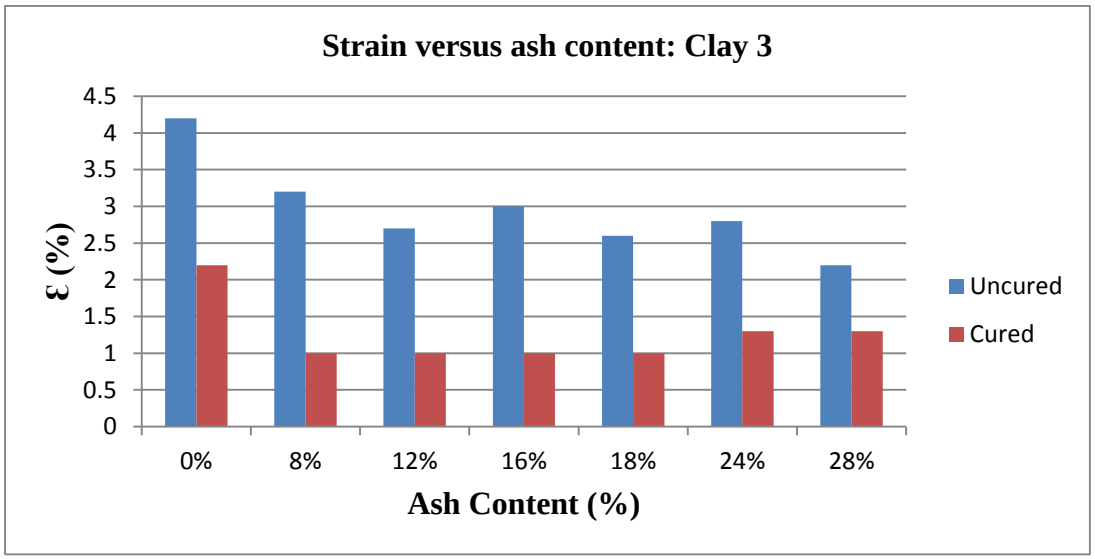

Figure 4.24. Variation of strain with paper mill ash content

4.6. SWELL TESTS

Swell tests were conducted according to ASTM D4546-08 (2008) on specimens compacted according to ASTM 698-07 (2007). The swell test specimens were prepared using a cutting ring of 20mm height and 71.5 mm diameter. These tests were conducted on both untreated and treated soil to determine free swell and swell pressure to determine the effect of paper mill ash on those parameters. To this end, a Method A, of ASTM D4546-08, consisting of applying different loads to different identical specimens and then inundating them, was used.

Vertical stress versus vertical strain curves and one typical time-swell curve are shown here below. Other time-swell curves are given in Appendix C. From Figure 4.25, it can be seen that the specimen underwent swelling of about 4mm under a stress of 1.2kPa. From Figure 4.26, it is observed that the free swell is approximately equal to 20% and the swelling pressure is 130kPa. Figures 4.27 to 4.29 show the effect of paper mill ash treatment on swelling behaviour of different clays used in this study.

Figure 4.33 shows that free swell dropped from 19.7% to 5.4% and the swell pressure from 130kPa to 85 kPa due to treatment of Clay 1 with 20% ash. The changes in free swell and swell pressure due to paper mill ash treatment of three clays are summarized in Table 4.7.

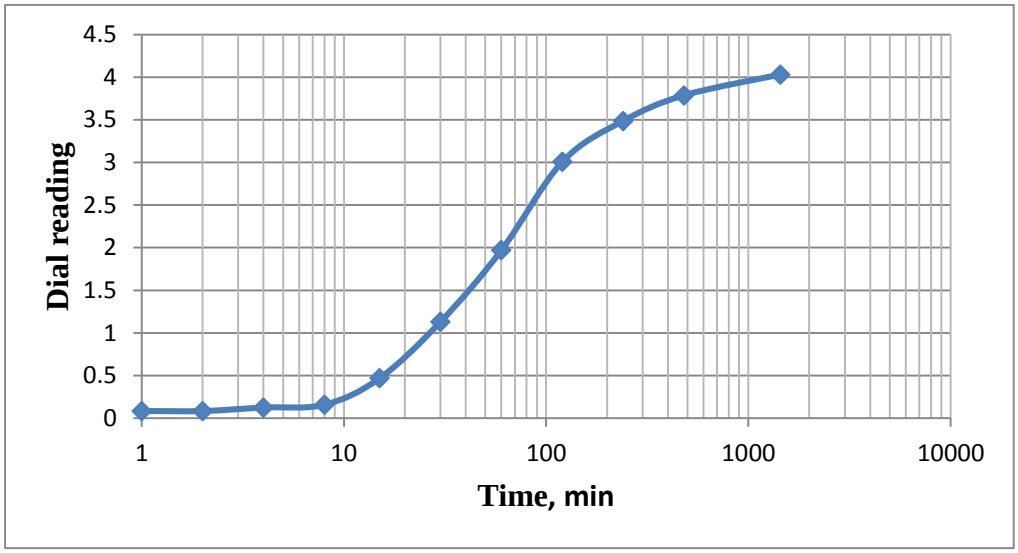

Figure 4.25. Time-swell curve for 1.2kPa stress on Clay 1 specimen.

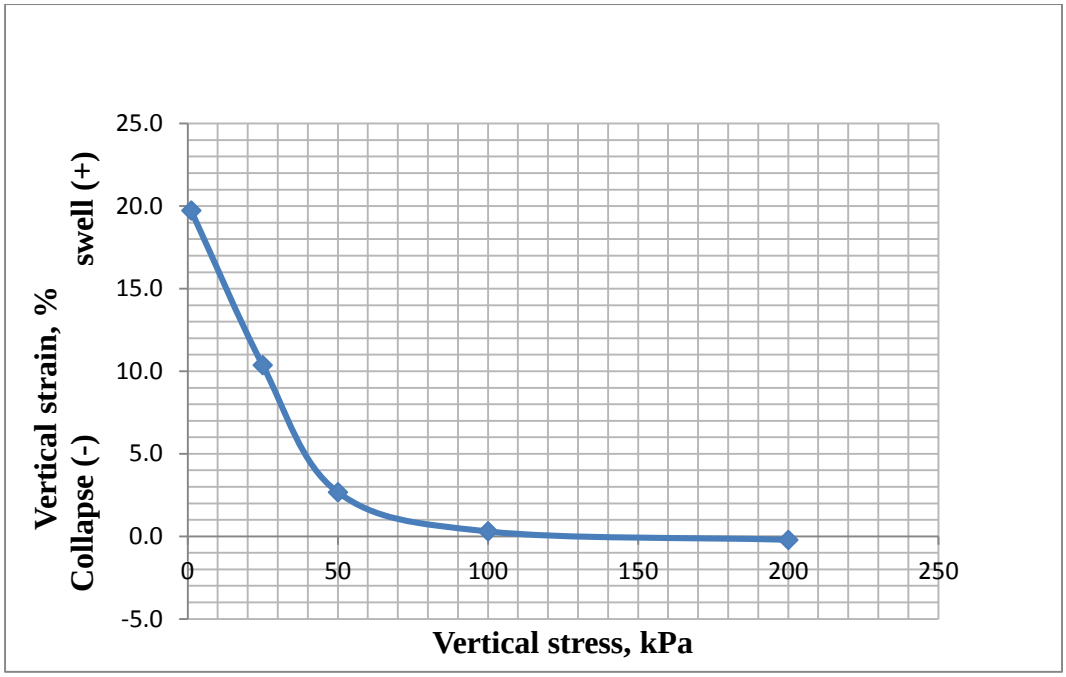

Figure 4.26. Stress versus wetting –induced swell/collapse strain, untreated Clay 1

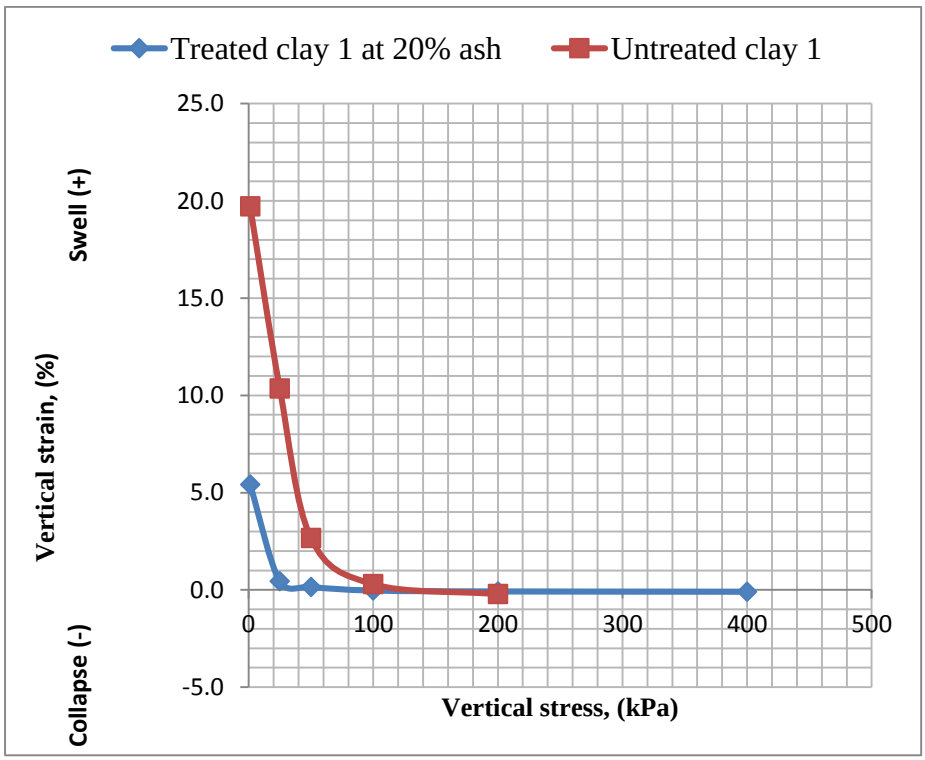

Figure 4.27. Stress versus wetting–induced swell/collapse strain, untreated and treated Clay 1

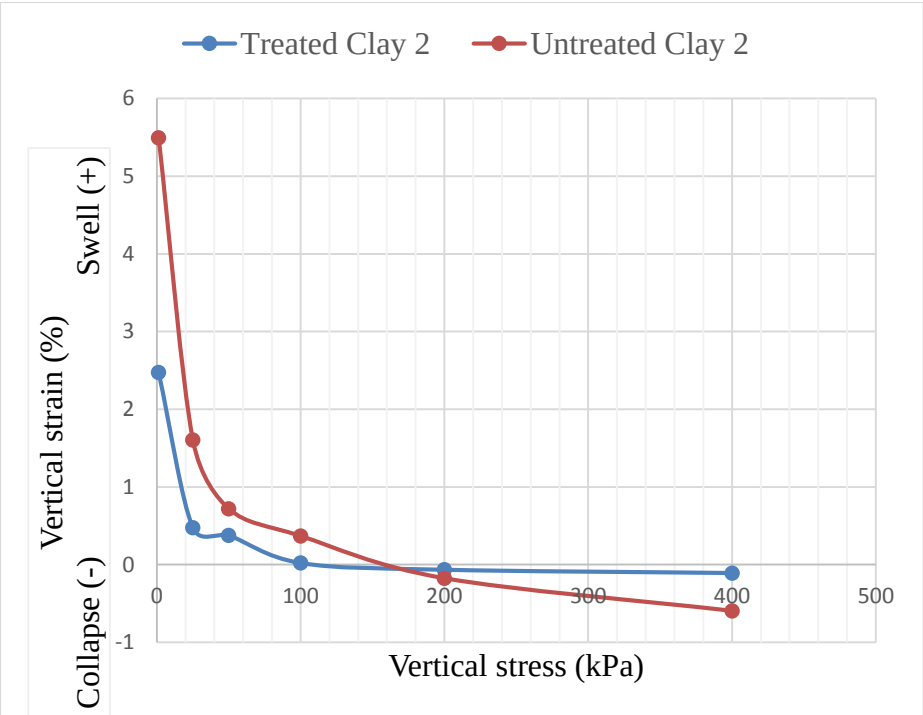

Figure 4.28. Stress versus wetting–induced swell/collapse strain, untreated and treated Clay 2

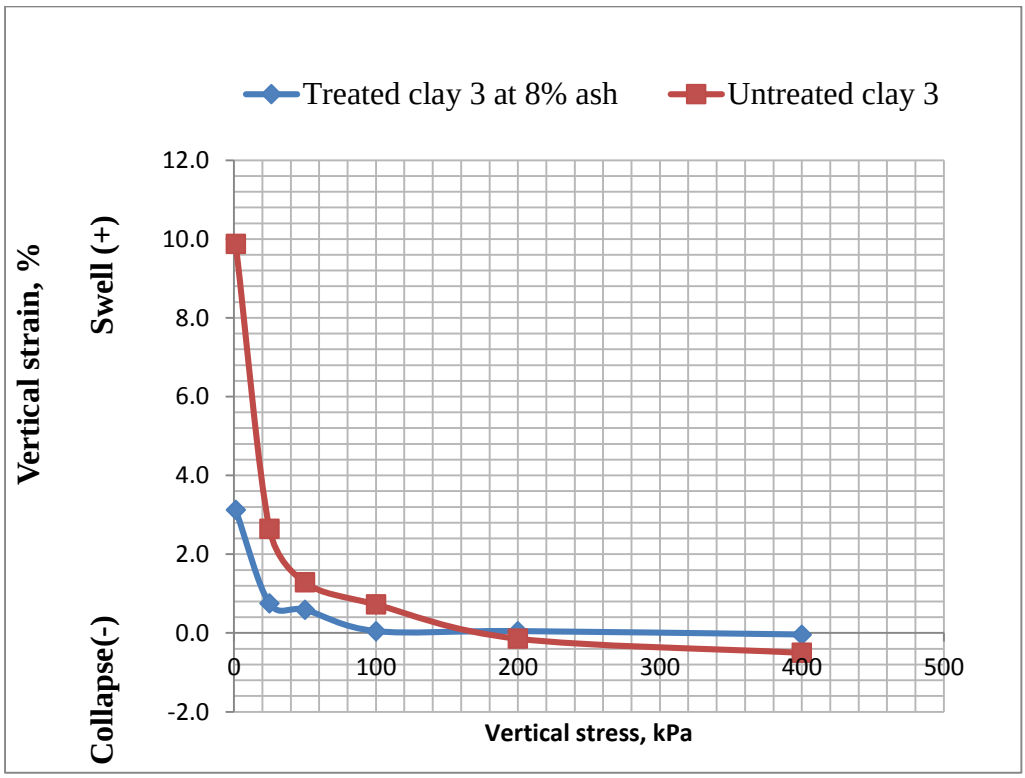

Figure 4.29. Stress versus wetting–induced swell/collapse strain, untreated and treated Clay 3

Table 4.7. Effect of paper mill ash on swelling behaviour of soils

	Ash content (%)	Free swell (%)	Swell pressure (kPa)
Clay 1	0 (untreated)	19.7	130
	20	5.4	85
Clay 2	0(untreated)	5.5	167
	24	2.5	110
Clay 3	0 (untreated)	9.9	175
	8	3.1	120

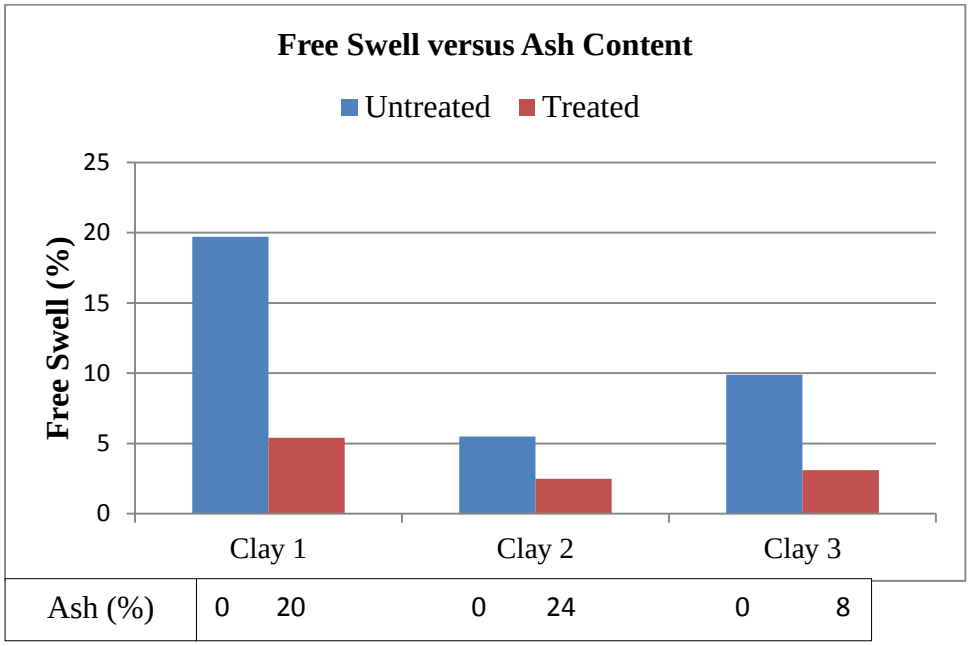

Figure 4.30. Effect of paper mill ash on free swell

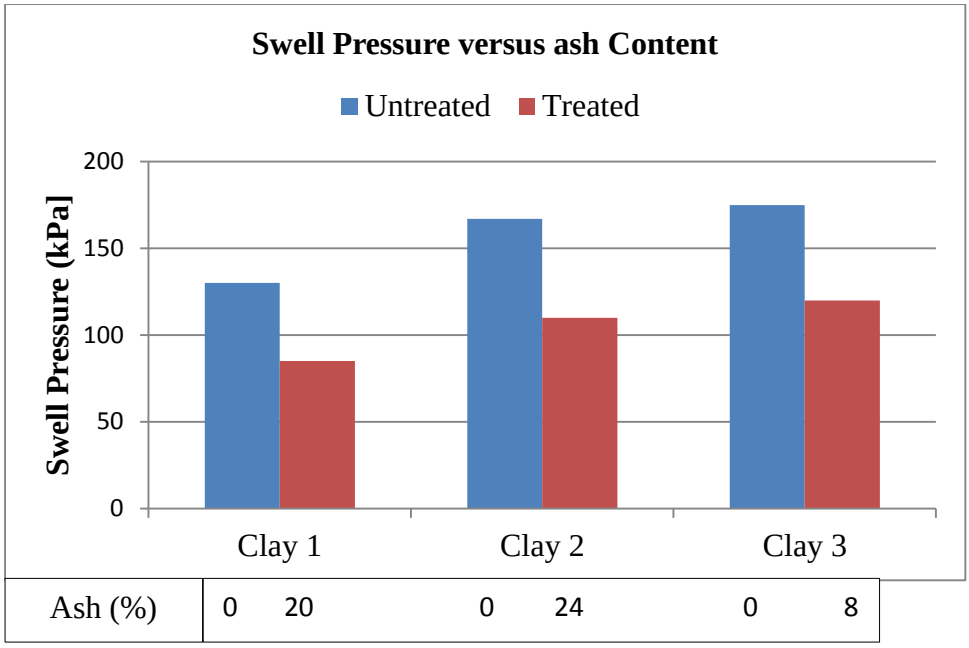

Figure 4.31. Effect of paper mill ash on swell pressure

4.7. CALIFORNIA BEARING RATIO (CBR)

The California bearing ratio test was carried out on untreated and treated clay specimens and on both unsoaked and 4-days soaked specimens to assess the effect of paper mill ash on CBR. The effect of paper mill ash on the CBR load-penetration curves is shown in Figure 4.36 to Figure 4.41 and the corresponding CBR values are summarized in Table 4.8.

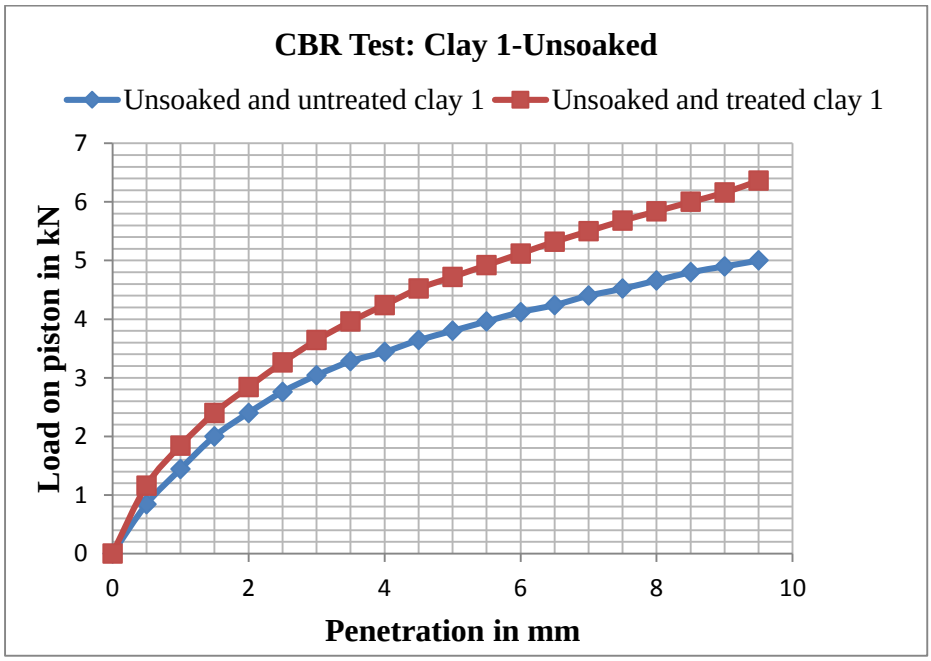

Figure 4.32. Effect of paper mill ash on CBR load-penetration curve of unsoaked specimens, Clay 1

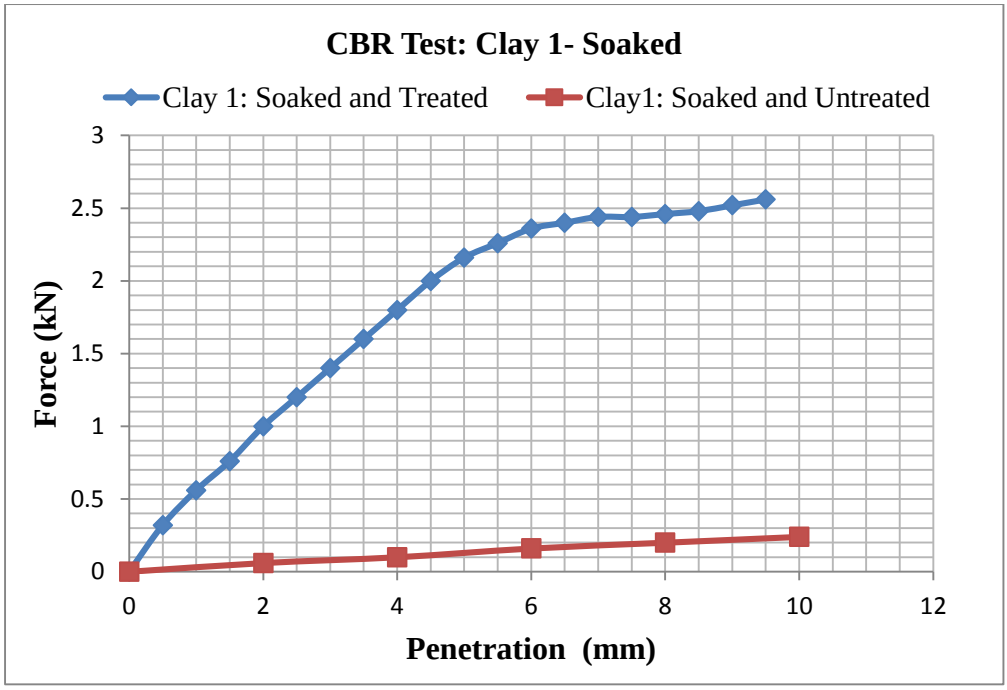

Figure 4.33. Effect of paper mill ash on CBR load-penetration curve of soaked specimens, Clay 1

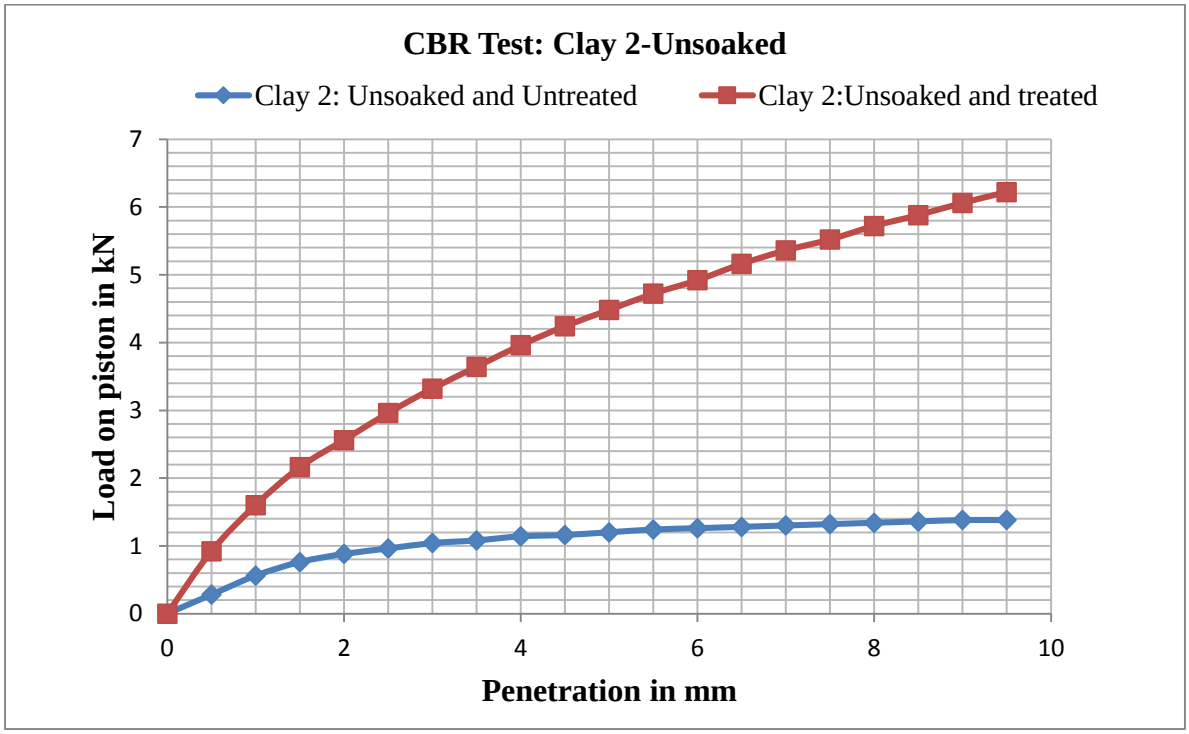

Figure 4.34. Effect of paper mill ash on CBR load-penetration curve of unsoaked specimens, Clay 2

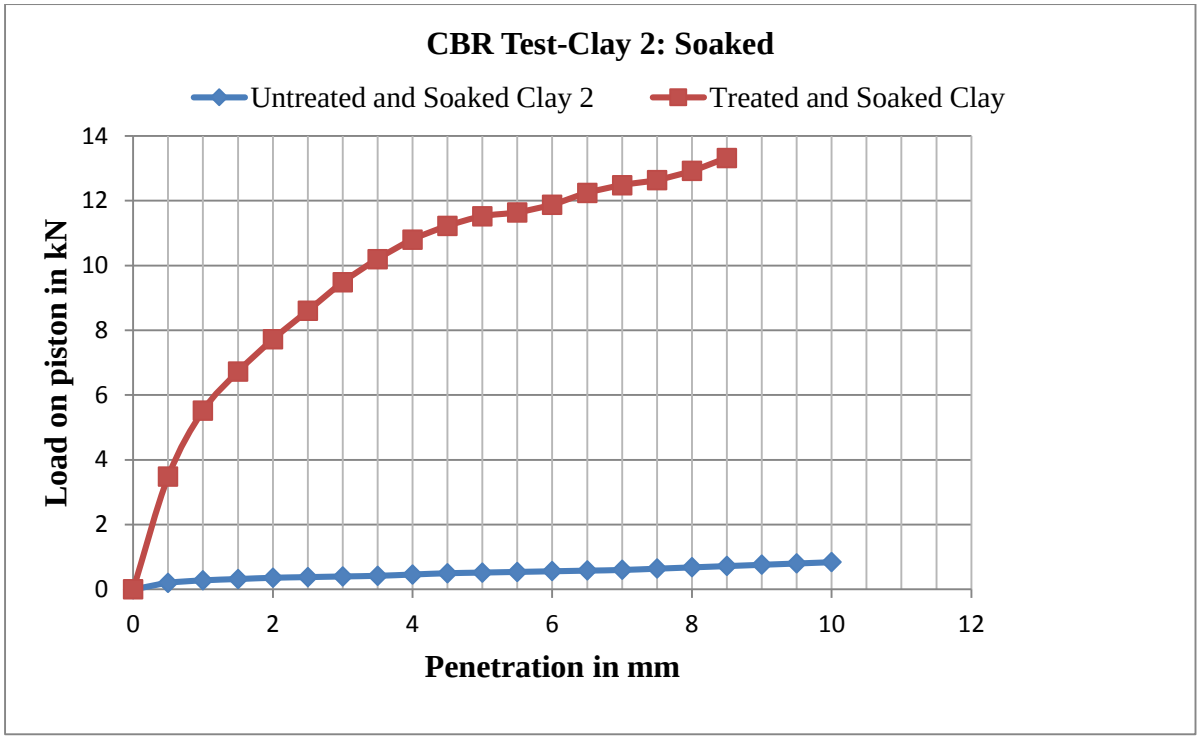

Figure 4.35. Effect of paper mill ash on CBR load-penetration curve of soaked specimens, Clay 2

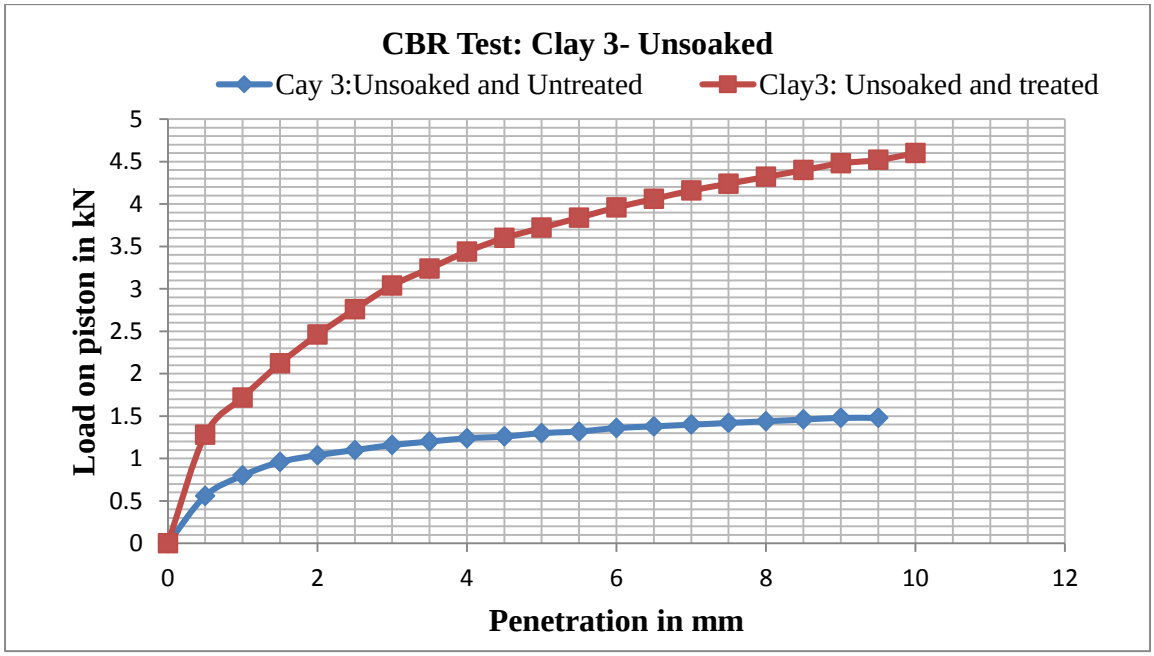

Figure 4.36. Effect of paper mill ash on CBR load-penetration curve of unsoaked specimens, Clay 3

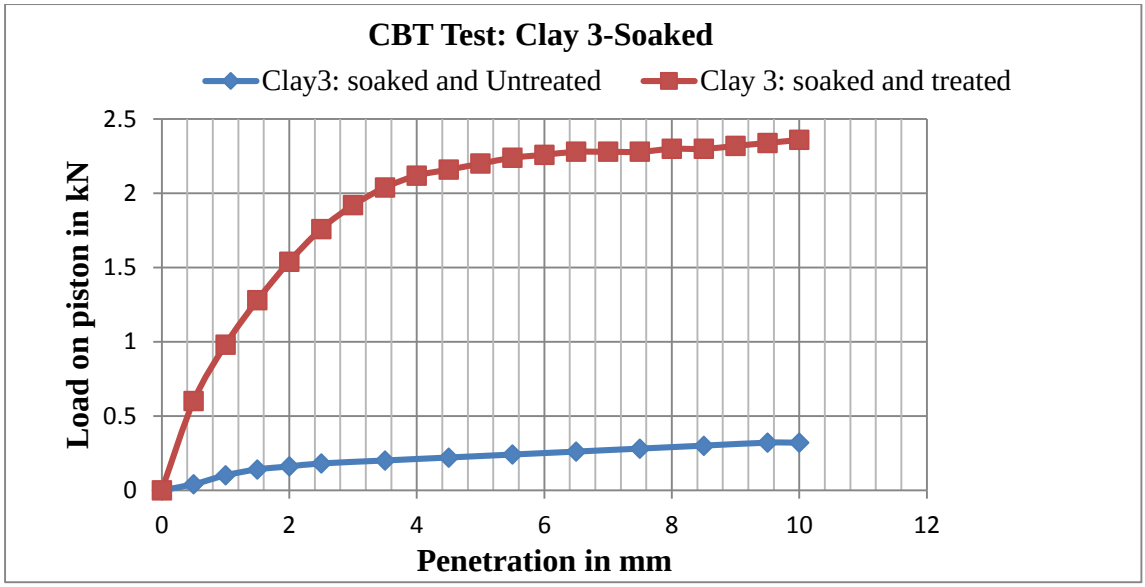

Figure 4.37. Effect of paper mill ash on CBR load-penetration curve of soaked specimens, Clay3

Table 4.8. CBR for different soils

Clay		**Clay 1**		**Clay 2**		**Clay 3**	
Paper mill ash content (%)		0	20	0	24	0	8
CBR	**soaked**	0.6	10.8	2.8	64.4	1.3	13.2
	unsoaked	20.7	24.4	7.2	22.4	8.2	20.7

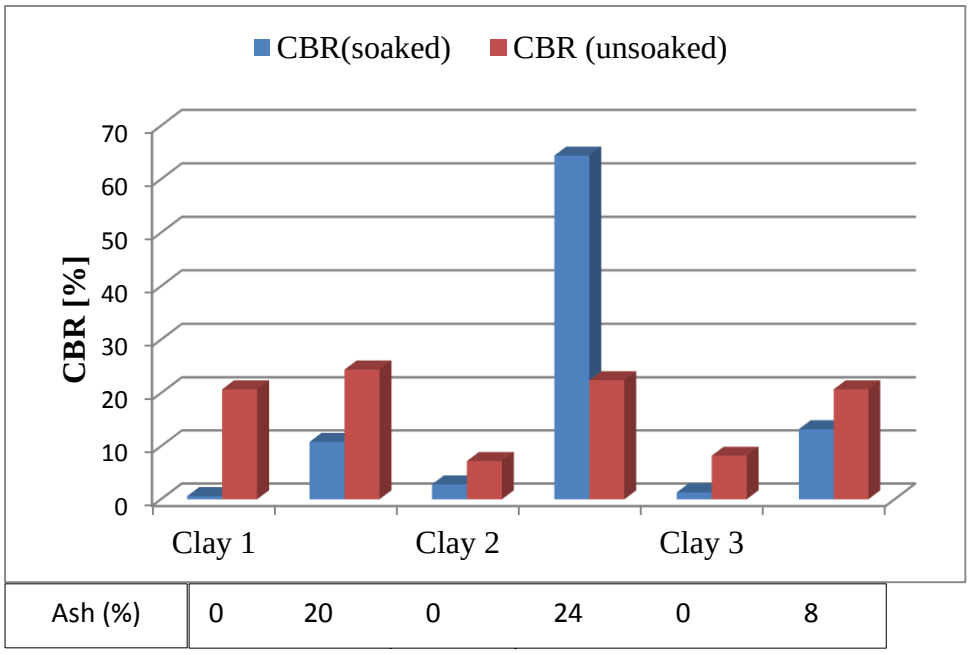

Figure 4.38. Effect of paper mill ash on CBR

4.8. SUMMARY OF FINDINGS

Based on the results presented in Chapter 4, the following findings were observed:

- Addition of paper mill ash led to improvement of workability for all three clays.
- Addition of paper mill ash caused an increase of the fraction of soil passing 75µm for Clay 1 and Clay 3, and decrease of that fraction for Clay 2 (Figures 4.1, 4.3 and 4.5).
- Paper mill treatment led to decrease in plasticity index (PI) for Clay 2 and 3 (Tables 4.2 and 4.3, and conversely, to increase in PI for Clay 1 (Table 4.1).
- Addition of paper mill ash resulted in dry density decrease and optimum moisture content increase for Clay 1.
- Addition of paper mill ash also resulted in increase in unconfined compressive strength immediately after mixing for all clays (uncured specimens) and in significant increase after 7 days curing.
- While the addition of paper mill ash in Clay 1 led to the ash content for which the UCS reaches maximum after curing, for Clay 2 and Clay 3, UCS kept increasing with increasing ash content for the proportions used in this study.
- Generally, brittleness increased with increasing ash content for all clays.
- Addition of paper mill ash resulted in free swell and swell pressure reduction for all clays.
- Treated clays exhibited significant improvement of CBR for both soaked and unsoaked specimens (Table 4.8).

4.9. CONCLUSION

The effect of paper mill ash on different properties of clay materials used in this study were analysed in this chapter. Generally, the addition of paper mill ash has a positive effect on a number of expansive soils properties. From the results, the following can be observed:

- Addition of paper mill ash affected all properties of the studied clay materials.
- Particle size distribution curves of treated clays were shifted either to the coarser side or to the opposite side compared with untreated clays which substantiates the effect of paper mill ash on particle size distribution. The shift to the coarser side shows a decrease in percentage of correponding passing fraction while the shift to the other side shows increase in percentage of passing fraction. An increase of fines may lead to plasticity increase if the test is conducted immediately after mixing.

- Addition of paper mill ash had great positive effect on plasticity of Clay 2 and 3. After treatment, Clay 2 and Clay 3 changed to less plastic soils. However, the treatment of Clay 1 led to higher plastic soil due to the increase in fines from the ash.
- Compaction results showed different behaviour of treated clays. For Clay 1 and 3, the maximum dry density decreased with increase in paper mill ash and the optimum moisture content (OMC) increased with ash content, which is in accordance with lime treated soils. For clay 2, there was no noticeable change in OMC and the MDD slightly increased with ash content increase.
- Unconfined compressive strength and California bearing ratio of treated expansive clays showed high improvement which highlights the pozzolanic reactions between soil and paper mill ash. The immediate increase in CBR might be attributed to the reaction between soil and paper mill ash.
- Addition of paper mill ash leads to decrease of both free swell and swell pressure for all studied expansive clays. However it was observed that the values of free swell and swell pressure of different clays did not reflect the degree of expansiveness as showed by classification tests. The swell pressure and free swell would have been greater for Clay 2 classified as high swelling clay than other clays. This converse observation may be attributed to a number of factors involved in swelling behaviour of soils such as initial moisture content, clay mineralogy, stress path, etc. For instance, the classification tests done on Clay 2 stabilized at 24% ash has shown that the clay shifted from high swelling clay to low swelling clay according to Van der Merwe chart. However, the free swell and swell pressure determined on Clay 2 are smaller than the ones estimated for clay 1. The reason of low free swell of untreated Clay 2 might be due to the high OMC at which the specimens were prepared. This finding is in accordance with the study carried out by Murthy (2007) where it was found that capacity of water absorption of expansive soil decreases with the increase of its degree of saturation. In fact, the swelling potential decreases with an increase of the mixing water content. However the most important observation is that the addition of paper mill ash led to significant decrease of both free swell and swell pressure for all expansive clays.

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CHAPTER 5 APPLICATION OF THE STUDY RESULTS

As previously noted, expansive soils, due to their swelling-shrinking behaviour, cause damage to facilities built on them, especially light constructions. The most affected facilities and structures are among others low-rise buildings, retaining structures, channel and reservoirs linings, utility lines, railways and roads. A number of techniques commonly used in practice to fix those problems, were briefly described in Section 2.1.8 of this work. The study, especially Chapter 4 of this research, showed that the paper mill ash, a calcium-rich industry waste, can be used to improve both index and engineering properties of expansive soils. The aim of this chapter is to illustrate the benefit of using paper mill waste ash for expansive soil stabilization. To this end, the results obtained from the experimental work were used for two design applications namely design of a pavement structure and pier on expansive soil.

The structure analysis and design was carried out with both untreated and treated expansive soil properties obtained from the experimental work. The pavement structure design was carried out with untreated expansive soil as an in situ subgrade and then after treatment of the upper layer of the subgrade with paper mill ash.

5.1. APPLICATION ON A PAVEMENT DESIGN

A pavement can be defined as a composite system of layers which work in unison to withstand the traffic loads during its lifetime. A pavement is designed to carry the traffic spectrum forecast during its design period.

This application consisted of analysing a flexible pavement structure. According to Molenaar (2007), flexible pavements are those paved with other materials than cement concrete or concrete blocks. The design of flexible pavement encompasses a number of factors such as time, traffic, pavement materials, subgrade soils, environmental conditions, construction details and economics (TRH 4, 1994). A detailed description of each factor is beyond the scope of this study, however, a brief description of some of them, applicable to this application case, is provided in the following sections.

Many design systems for flexible pavements, namely empirical design systems, mechanistic design systems and mechanistic-empirical design systems, have been developed to provide adequate functional and structural service levels during the design period (SAPEM, 2013). For this design application, the mechanistic-empirical design system, currently used in South

Africa for flexible pavement design (SAPEM, 2013), was used. The main components of the design system are shown in Figure 5.2.

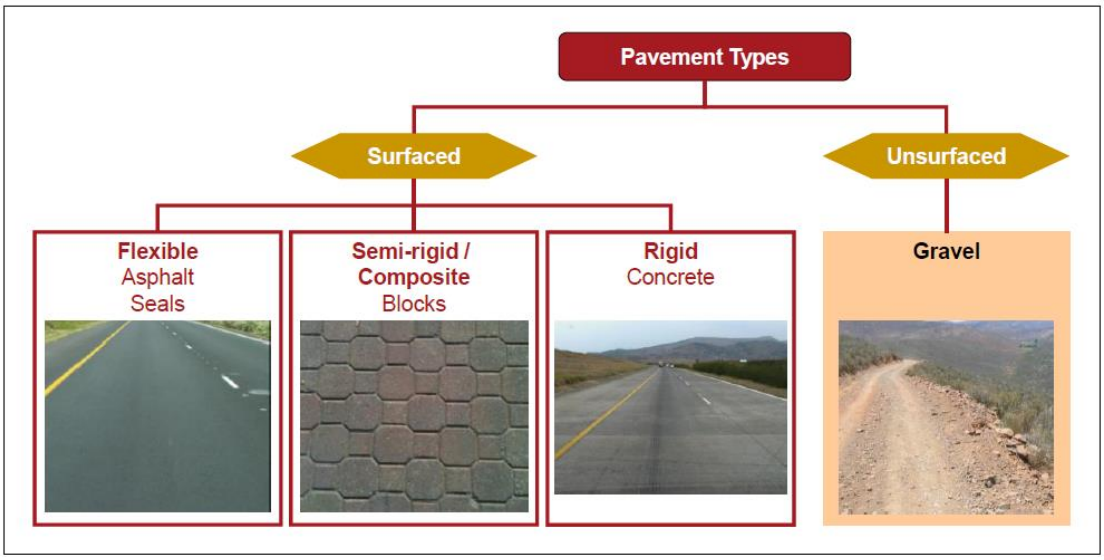


Figure 5.1. Classification of pavement types based on materials (SAPEM, 2013).

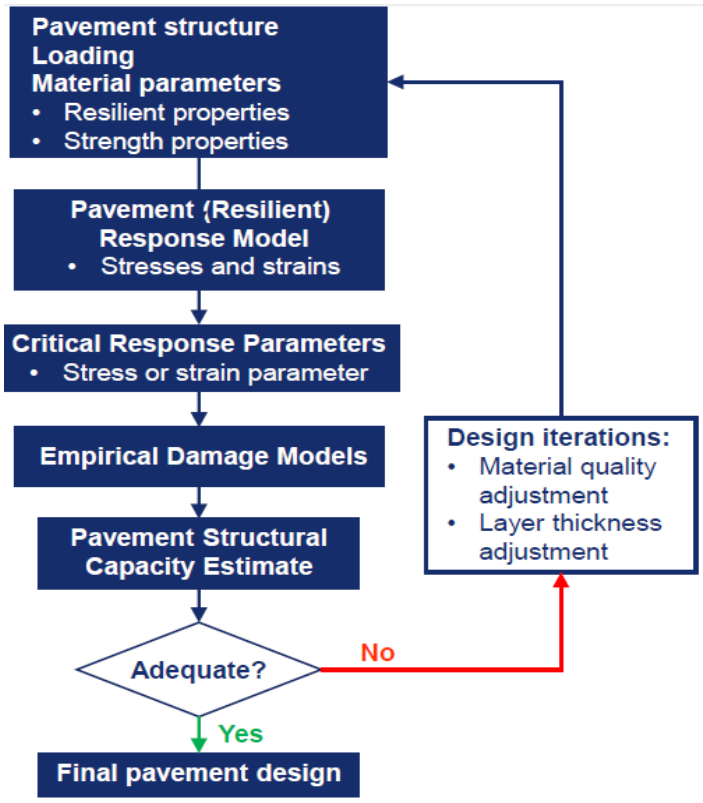


Figure 5.2. Main components of a Mechanistic-Empirical pavement design method (SAPEM, 2013).

5.1.1. Pavement structure

For the purpose of this chapter, a pavement structure consisting of layers overlying subgrade, defined in South African Pavement Engineering Manual (SAPEM) (2013) was used. The analysed pavement structure consisted of an inverted pavement system of a thin layer of hot mix asphalt (HMA) overlying a granular base. The granular base is underlain by a cemented subbase. According to SAPEM (2013) and Buchanan (2010), the inverted pavement systems are widely used in South Africa and often in high volume roadways. Below the subbase, a selected subgrade is laid to cover the in situ soil, thus providing a more or less strong platform to the overlying structural layers. A brief description of the function of each layer according to SAPEM is summarized in Table 5.1.

Figure 5.3. Typical pavement structure (SAPEM, 2013)

Table 5.1. Functions of the various layers in the pavement

Layer	Function
Surfacing	• Provide waterproofing, skid resistance, noise-damping, durability, visibility and adequate drainage.
Base	• Provide adequate support for the surfacing, • Distribute tire pressure and wheel load uniformly over the underlying structural layers and subgrade
Subbase	• Provide support and platform for the base, • Distribute load to protect underlying layer
Selected subgrade	• Cover the subgrade thus providing workable platform for imported materials layers • Increase the depth to subgrade thus reducing the stresses in subgrade by further spreading traffic loads • Enables subsequent structural layers to be adequately compacted during construction
Subgrade	• Provide foundation for overlaying structural pavement layers

The aim of this chapter is to study the benefit of using paper mill ash in expansive soil stabilisation. According to the TRH 14 (1985), expansive soils before or after treatment cannot meet the requirements for pavement structure materials. Therefore, during this design example, the material was used as subgrade material. For other layers, the thicknesses used in this application were chosen based on the current values used in South Africa. Based on the fact that two of the clays used in this study were collected along the National Road N1, this design example was applied on the category to which N1 belongs to, which is a Category A according to TRH 4 (1994). The thicknesses of the structural layers for this hypothetical design example were chosen in accordance with the Pavement Design Catalogue for Category A, and the typical values used in South Africa. However, for the foundation, only one selected layer was used so that the effect of the subgrade stabilization with paper mill ash could be easily noticed.

The pavement used in this study consisted of a layered system of a 40mm asphalt wearing course, a 150mm granular base, a 250mm cemented subbase, a 150mm selected subgrade and a semi-infinite subgrade in the case of the untreated subgrade. In the case of the treated subgrade, a 150mm soil-paper mill stabilized was incorporated into the system above the

untreated subgrade. The layer thicknesses were chosen based on the typical values used in different researches in South Africa. The asphalt thickness of 40mm was considered by Ebels (2008) as a typical value used in South Africa for flexible pavement design and, the base and subbase thicknesses are mentioned by Molenaar (2007) as typical values for flexible pavements in South Africa.

5.1.2. Traffic loading

The traffic load is one of the key parameters for a pavement design. According to Molenaar (2007), the wheel loads are of various sizes and shapes as shown in Figure 5.4. For the design input, the contact area is assumed to be circular and the contact pressure is commonly assumed to be equal to the tyre pressure (Molenaar, 2007).

Figure 5.4. Different tyre types (Molenaar, 2007). (From left to right: dual wheel, small size dual wheel, super super single tyre and wide base or super single tyre).

For this application, a single wheel configuration was used. The traffic survey conducted since 1995 have shown that the value of 520 kPa used to develop the catalogues in TRH 4 is no more a representative value for tyre pressure (SAPEM, 2013). To take into consideration

that traffic loading change, Theyse et al. (2011) recommend a value of 650kPa and a value of 750kPa is also used to account for the current traffic loading (SAPEM, 2013). In addition to this, other typical half-axle configurations for analysis were proposed in Theyse et al. (2011). For this hypothetical design example, a 40 kN single wheel load and a uniform contact pressure of 900 kPa was considered. The configuration is shown in Figure 5.5.

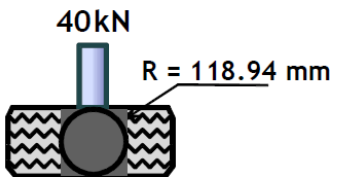


Figure 5.5. Typical half-axle configuration for pavement analysis (Theyse et al., 2011).

5.1.3. Material characteristics

As mentioned above, only the subgrade characteristics were determined from laboratory testing. The characteristics of materials used in structural pavement layers of this design example were taken from South African standards or research works as specified in the following.

Asphalt layer: A continuously graded hot mix asphalt layer, 40mm thick, was used. The values of 4000MPa for modulus and 0.44 for Poisson's ratio, recommended in South African Mechanistic-Empirical Design Method (SAPEM, 2013 and Theyse and Muthen, 2000) were used.

Base: Four main base pavements viz granular, cemented, hot-mix asphalt and concrete base are used in South Africa (TRH4, 1996). For this design application, an unbound base consisting of a high quality crushed stone, with a material code G1, was used as a base layer. For this material, a value of 0.35 for the Poisson's ratio recommended in Theyse and Muthen (2000) and the value of 450MPa (base over a cemented subbase) used in the development of TRH 4 (1996) were used. The ranges of shear strength parameters were given in Theyse (2008, cited in SAPEM, 2013). Values of 110kPa and 55° were chosen for cohesion and friction angle respectively. These values correspond to the dry condition.

Subbbase: A cemented subbase was used in this design example. A cemented subbase was chosen due to its great role to protect both the overlying granular base and underlying subgrade. A stabilized subbase exhibits a greater stiffness thus providing a greater load

spreading ability which results in effective protection of the subgrade from high stresses. In addition, a stabilized subbase assists in compaction by providing a rigid base for overlying layers thus reducing the risk of excessive deformation of the pavement (DFID, 2000). Since the high cemented materials viz C1 and C2 are no longer used for pavement layers (SAPEM, 2013, a cemented natural gravel, C3, with a modulus of 2000MPa and a Poisson's ratio of 0.35 was used.

Selected subgrade: This layer was incorporated into this design example to protect the subgrade which consists of swelling clay materials. This layer consists of a G7 material in accordance with SAPEM (2013). This material is classified as a gravel-soil material according to TRH 14 (1985). A ferricrete, G7 material, with cohesion of 85 kPa and friction angle of 44° (dry condition) was considered for this design example. A value of 120 MPa was considered for the elastic modulus (Theyse et al., 1996).

Subgrade: Subgrade materials consist of expansive clays studied in the laboratory during this work. However, only two materials out of three, were studied. The reason for this is that, the Clay 1 was sampled below a depth of 2.5 m and the clay layer is located below sand and boulder layers as shown in Figure 3.1 (Chapter 3) whose soil characteristics were not investigated.

For subgrade materials, the required parameter is the resilient modulus. As the modulus was not determined during laboratory work, the available relationships with investigated parameters were considered. Different relationships correlating the determined engineering properties with the resilient modulus have been developed. Some of them are summarized in the Table 5.2 below. It is to be noted that, the paper mill ash, a lime-rich product, was studied in accordance with the procedures applied for lime stabilization. Therefore, the same relationships used to correlate soil-lime properties were applied for soil-paper mill ash stabilization.

Table 5.2. Correlation of engineering properties with stiffness modulus

Equation	Reference	Note
$M_r = 0.124(q_u) + 9.98$	Mallela, Von Quintus and Smith (2004)	M_r = resilient modulus, ksi. q_u = unconfined compressive strength (UCS), psi (28-days cure).
E =10+0.124 UCS	Little et al. (1987)	E in ksi, and UCS is the Unconfined Compressive Strength in psi

Another relationship to estimate the resilient modulus, based on the repeated load CBR (California Bearing Ratio) was developed by Molenaar (2010).

$M_r=0.211\sigma_3^{0.563}M_{repCBR,}$

Where: M_r = resilient modulus,

σ_3 = confining stress [kPa],

M_{repCBR} = resilient modulus obtained from the repeated load CBR test

Once one of the parameters is missing, some rules of thumb, given in Table 5.3 can be used to estimate the modulus of unbound materials.

Table 5.3. Equations to estimate the subgrade modulus [E] = [MPa] and [CBR]=[%], (Molenaar, 2007).

Organization	Equation
Shell	E = 10 CBR
US Army Corps of Engineers	$E = 37.3\ CBR^{0.711}$
CSIR South Africa	$E = 20.7\ CBR^{0.65}$
Transport and Road Research Laboratory UK	$E = 17.25\ CBR^{0.64}$
Delft University, Ghanaian laterite	$E = 4\ CBR^{1.12}$

In addition to those relations, Terrel et al. (1979, cited in Little, 1995) developed a range of values and another relation to evaluate the improvement in resilient modulus for lime treated soils once it was not determined in the laboratory.

Table 5.4. Approximation of Resilient Modulus from Unconfined Compression Strength Data (Terrel et al., 1979, cited in Little, 1995).

Unconfined compressive strength, psi	Approximate value of resilient modulus, psi
100-200	25,000-100,000
200-400	100,000-300,000
> 400	300,000 +
For values of unconfined compressive strength exceeding 400 psi, the resilient modulus may be approximated by the relationship: E_r = 1.15 (UCCS) – 140, where UCCS is the unconfined compressive strength in psi and E_r is calculated in ksi	

For this application, the equations developed by Mallela et al. (2004) and Little et al. (1987), which are quite the same, were used to correlate the UCS (determined in laboratory) and the stiffness of the subgrade (see Table 5.5). It should be noted that, the UCS used in this design is a 7-days cured strength. The correlations between CBR and the stiffness were not considered since they require repeated load CBR test whereas normal CBR testing was done

instead, however, they were computed to give an idea about how the stiffness changes with the addition of the paper mill ash (see Table 5.5). To assess the effect of the paper mill ash on the subgrade, the mechanistic design was applied with both untreated and treated subgrades.

The estimated values of the stiffness modulus are given in Table 5.5 here below.

Table 5.5. Estimated stiffness for subgrade

Material	UCS (after 7 days curing) (kPa)		Estimated E (MPa)	CBR (%)		Estimated E (MPa)
Clay 2	Untreated	213	95	unsoaked	7.2	74.7
				soaked	2.8	40.4
	treated	576	140	unsoaked	22.4	156.2
				soaked	64.4	310.3
Clay 3	Untreated	100	81	unsoaked	8.2	81.3
				soaked	1.3	24.5
	treated	513	132	unsoaked	20.7	148.4
				soaked	13.2	110.7

5.1.4. Pavement response and damage models

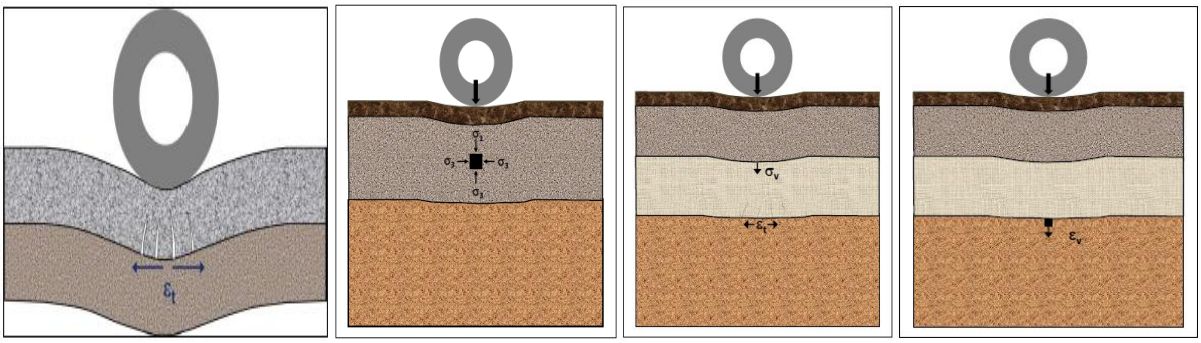
Due to traffic loading, the pavement structure experiences stresses, strains and deflections. In addition to traffic loading, these engineering parameters also depend on the the material properties. The relationship between engineering parameters and their causes viz load and material properties, is descibed by a model. For this design example, the homogeneous, isotropic, linear-elastic model was considered. In a layered elastic model, the layers are characterized by the elastic modulus or stiffness and the Poisson's ratio of each layer.

This pavement response model allows to determine stresses and strains at any point in the pavement structure resulting from the application of load on the surface. The damage in different pavement layers is evaluated based on the stresses or strains developed in some specific locations of the pavement structure. “These stresses and strains at specific locations in the pavement are referred to as critical parameters and serve as the primary, load related, input to the damage model” (SAPEM, 2013). It should be noted that each material exhibits a particular mode of failure. The damage model, also called transfer functions, relate the value of the critical parameter and the number of load applications that can be sustained at that value before the concerned material type will fail in a specific mode of failure (Theyse et al. 1996).

The critical parameters and locations for different pavement layers are as follows (Theyse et al. 1996):

- **Asphalt layer**: The critical parameter for the asphalt layer is the maximum horizontal tensile strain at the bottom of the layer**.** Due to repeated loading, the asphalt layer fails because of the fatigue cracking.
- **Granular material:** This material is prone to deformation and progressive shear under repeated loads. The deformation results from the densification of the material. The critical parameter for this material is the deformation which is controlled by the factor of safety against shear failure. The critical location is the midpoint of the granular layer.
- **Cemented material:** This material is prone to both crushing and effective fatigue. The critical parameters are the maximum tensile strain at the bottom of the layer and the vertical compressive stress at the top of the layer.
- **Selected and subgrade layers:** The permanent deformation or rutting is the critical parameter for these layers. The critical location for these layers is the top layer.

The critical parameters and locations are illustrated in Figure 5.6.



(a) (b) (c) (d)

Figure 5.6. Critical parameters and location for different layers (SAPEM, 2013): (a) Asphalt layer; (b) Granular layer; (c) lightly cemented layer; (d) selected and subgrades

The critical parameters and the analysis positions for a typical pavement structure under a dual-wheel loading are summarised in Figure 5.7.

Under a dual wheel loading, the critical parameters are analysed at different layer points on the same vertical as the wheel load application point and between the two wheels, and the bigger is used to compute the transfer functions. On the other hand, for a single wheel

loading, the critical parameters are calculated at every critical location on the same vertical as the application point of the load at the surface.

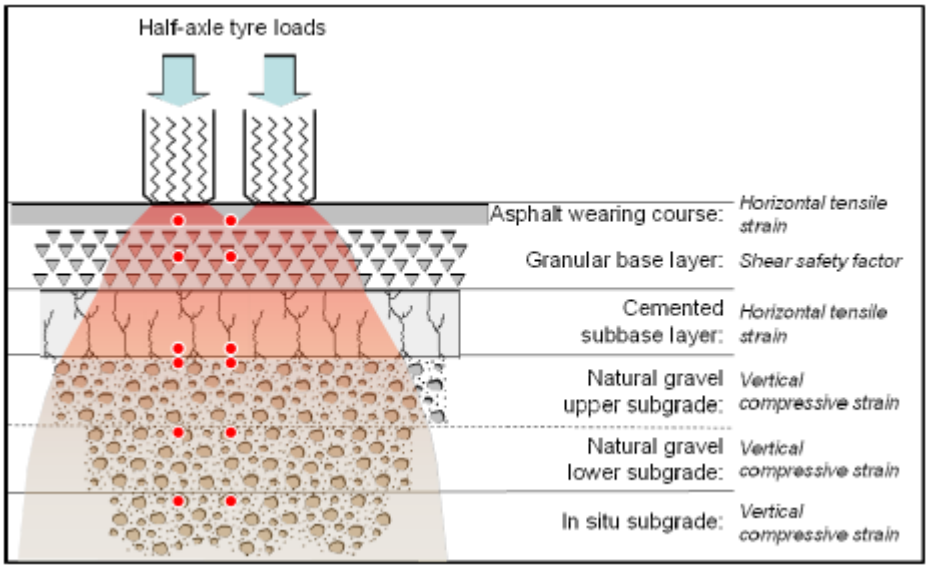


Figure 5.7. Analysis positions for critical parameters (SAPEM, 2013).

It is to be noted that Figure 5.7 was given to illustrate the critical locations for a dual wheel loading for a typical road structure. For this application a single wheel load was used and the pavement structure given in Figure 5.3 was considered.

5.1.5. Structural analysis

The structural analysis for the hypothetical pavement structure was done using BISAR, a static, linear elastic multilayer analysis program. The material properties and layer thicknesses used in the analysis are summarized in Tables 5.6 and 5.7.

Table 5.6: Material properties and layer thicknesses

Layer & material	Thickness (mm)	Shear parameters		Stiffness (MPa)	Poisson's ratio
		C	Φ		
Asphalt	40	-	-	4000	0.44
Granular base (G1)	150	110	55	450	0.35
Cemented subbase (C3)	250	-	-	2000	0.35
Selected subgrade (G7)	150	-	-	120	0.35

Table 5.7: Subgrade characteristics

Subgrade material		Thickness (mm)	Stiffness (MPa)
Clay 2	Treated with paper mill ash	150	140
	Untreated (in situ clay)	Semi-infinite	95
Clay 3	Treated with paper mill ash	150	132
	Untreated (in situ clay)	semi-infinite	81

From Table 5.7, it can be seen that the stiffness of Clay 2 was increased by 47% and by 63% for Clay 3. This increase will have a significant effect on the bearing capacity of the subgrade.

Note: According to TRH 13, the treated clays lie below the cemented zone, therefore the treated subgrades will be analysed as selected lower subgrades with a Poisson's ratio of 0.35.

Stresses and strains in the pavement layers as determined with BISAR

A mechanistic, layered-elastic analysis of the hypothetical pavement results in calculation of the mechanistic parameters (stresses and strains) summarized in Table 5.8.

Table 5.8. Summary of important mechanistic parameters resulted from the analysis

Layer	Critical parameter	Clay 2		Clay 3	
		Before treatment	After treatment	Before treatment	After treatment
Asphalt	ε_t (με)	247	247	245	246
Granular base	σ_1 (kPa)	549	549	549	549
	σ_3 (kPa)	96.7	96.5	97.3	97.1
Cemented subbase	σ_v (kPa)	358	358	357	358
	ε_t (με)	88.2	86.5	90.3	88.1
Selected subgrade	ε_v (με)	235	242	230	238
Treated subgrade	ε_v (με)	-	141	-	145
In situ non treated subgrade	ε_v (με)	186	139	202	152

From the analysis results, it can be seen that, after stabilization of the upper layer of the in situ subgrade, the deformation at the top of the in situ non treated subgrade was reduced by 25% for both Clay 2 and clay 3. The deformation was reduced from 186 με to 140 με and from 202 με to 152 με for clay 3.

In addition to the critical parameters, the mechanistic parameters were determined at all critical locations. The results are shown in figures below.

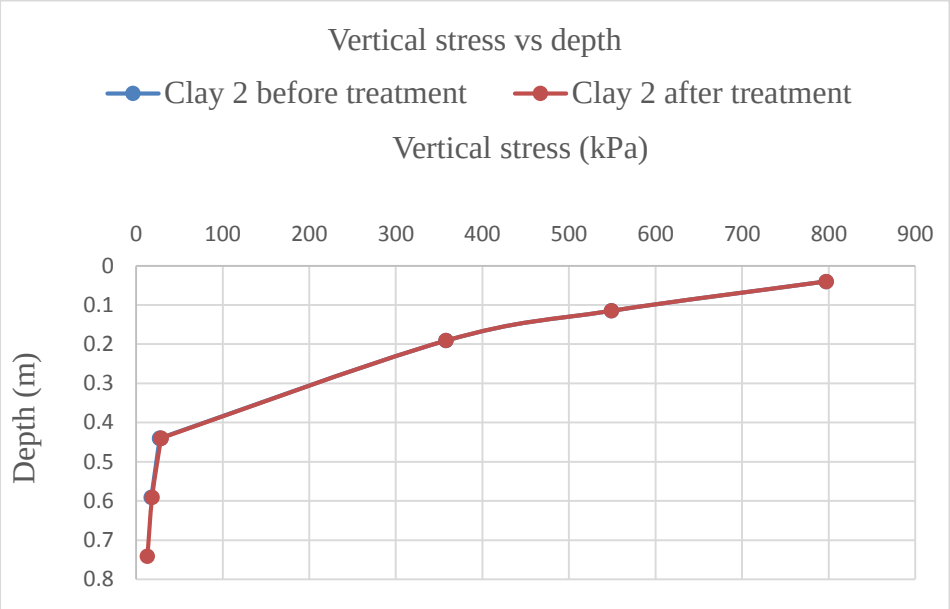


Figure 5.8: Variation of vertical stress within pavement layers, Clay 2

From Figure 5.8, it can be seen that, for both untreated and treated subgrade, the vertical stress was considerably reduced from the surface to the bottom of the cemented subbase. The vertical stress was reduced to 3.4% from 900 kPa at the surface to 27.1 kPa at the top of the selected subgrade. This shows that the chosen structural layers were able to spread the applied traffic load efficiently. It is also clear that the stabilization of the top layer of the subgrade had no effect on the vertical stress within the overlying layers. However, the vertical stress at the top of the untreated subgrade kept reducing due to the stabilization of the top layer of the subgrade.

Figure 5.9 shows signficant decrease of the horizontal stresses from the surface to the bottom of the cemented layer and from the top of the selected subgrade the horizontal stress becomes almost zero.

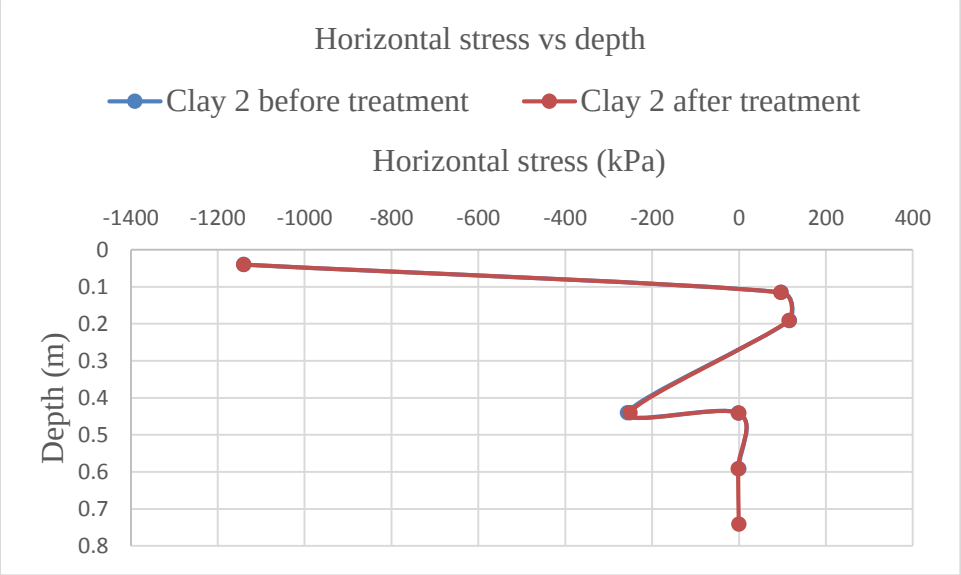


Figure 5.9: Variation of horizontal stress within pavement layers, Clay 2

As regards to the variation of vertical and horizontal strains through the pavement, Figure 5.10 and Figure 5.11 show that the maximum vertical strain was located in the middle of the granular base. These figures also show the reduction of the vertical and horizontal strains due to subgrade stabilization with paper mill ash.

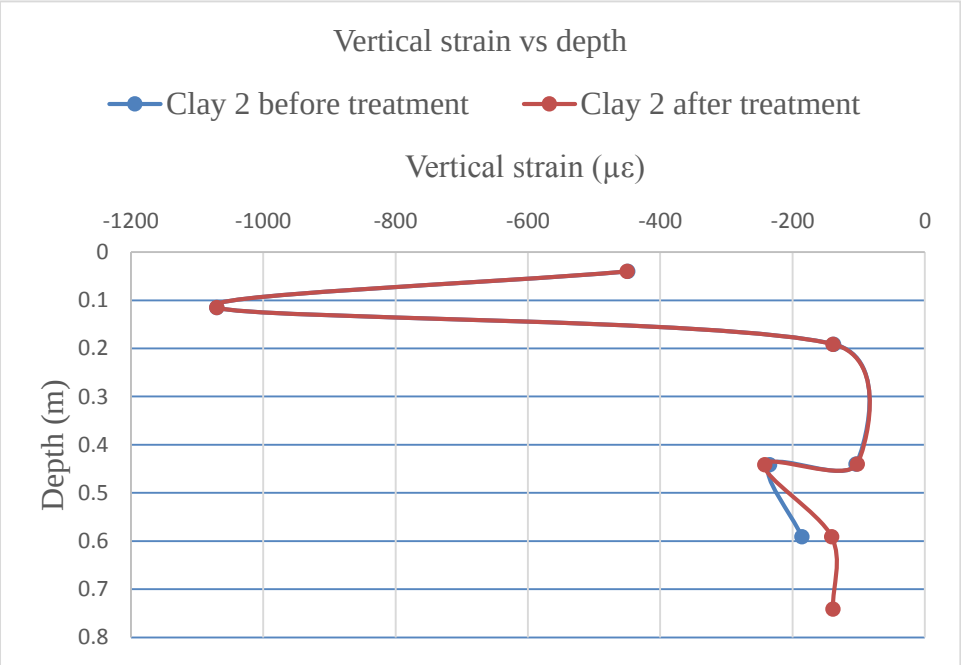


Figure 5.10: Variation of vertical strain within pavement, Clay 2

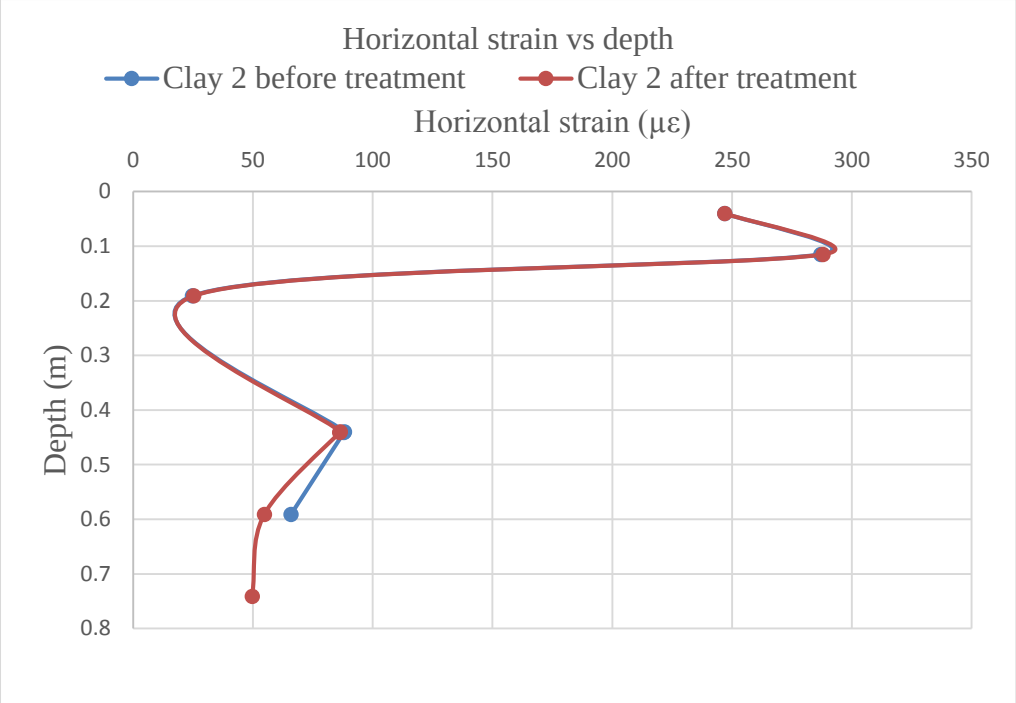


Figure 5.11: Variation of horizontal strain within pavement, Clay 2

Similar observations can be made for Clay 3 for both strains and stresses as shown in Figures 5.12 to 5.14. In fact, the change of stresses and strains through the pavement layers change in the same way for both clays.

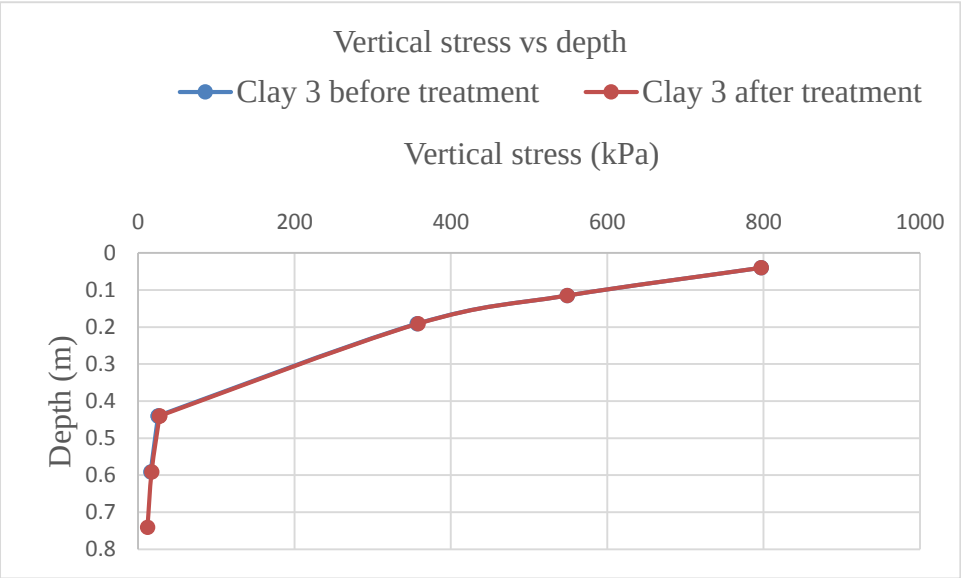


Figure 5.12: Variation of vertical stress within pavement layers, Clay 3

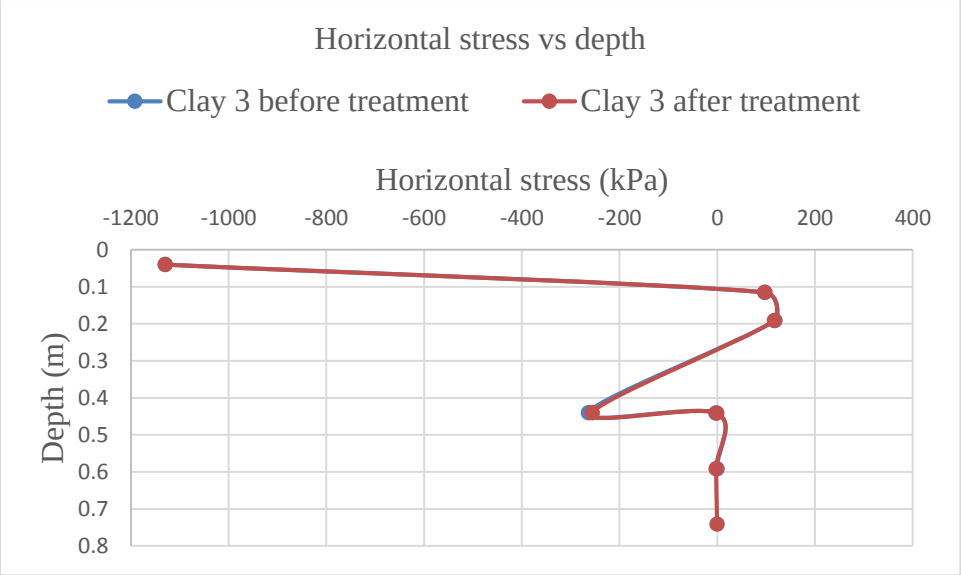


Figure 5.13: Variation of horizontal stress within pavement layers, Clay 3

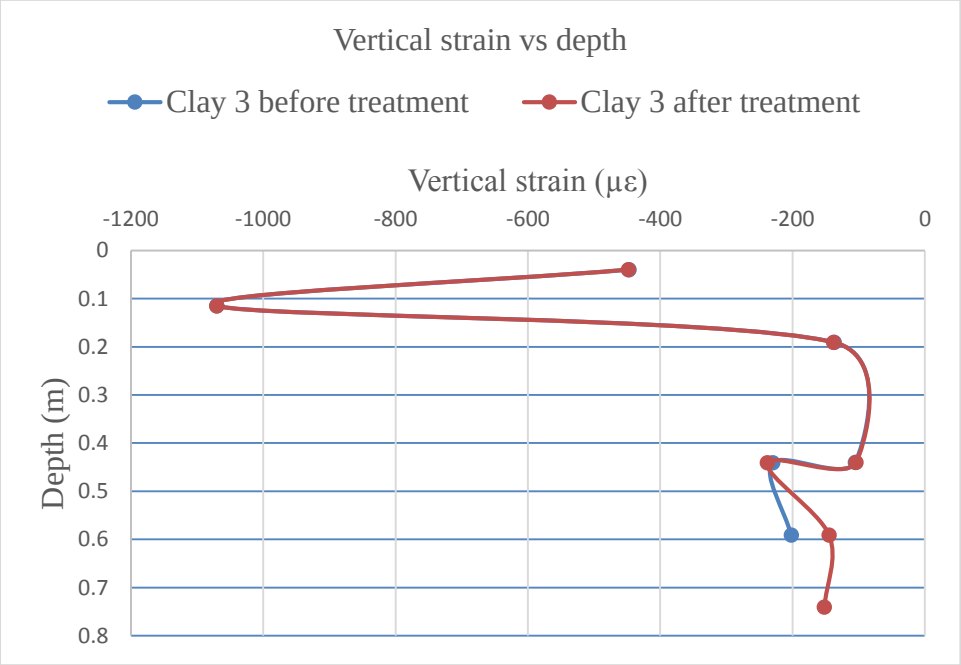


Figure 5.14: Variation of vertical strain within pavement, Clay 3

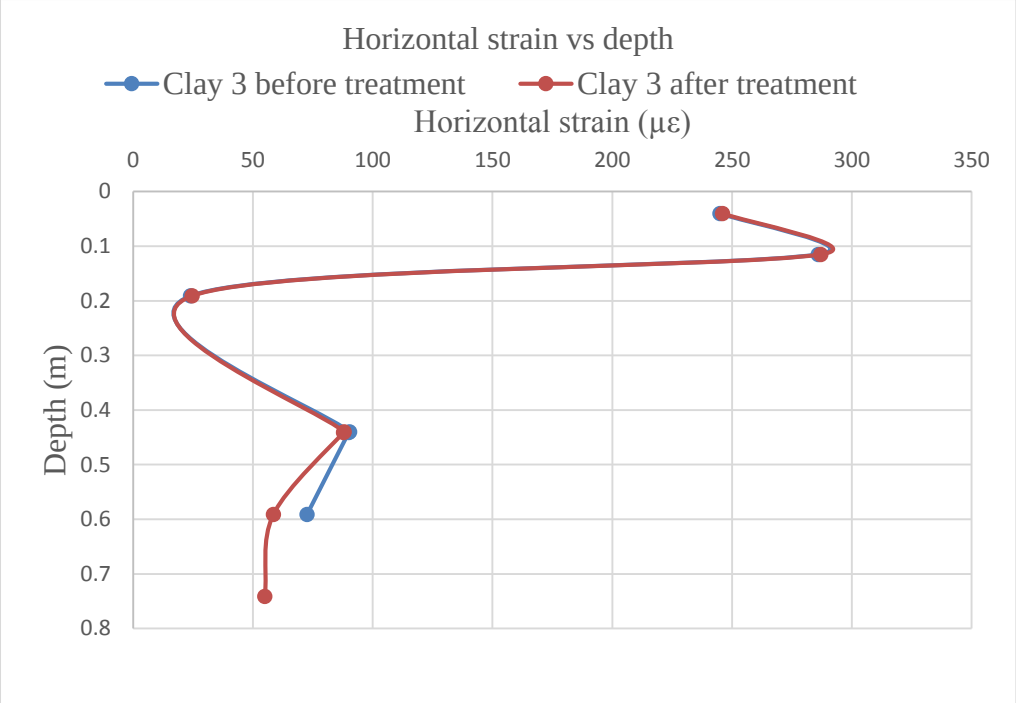


Figure 5.15: Variation of horizontal strain within pavement, Clay 3

5.1.6. Pavement life prediction

The pavement life prediction consists of two phases. Each layer life is determined individually, in terms of the number of load repetitions, and the pavement structural capacity is determined by the layer with the shortest life. The transfer functions used to predict the life of each layer, as difined in Theyse et al. (1996), Theyse and Muthen (2000) and SAPEM (2013) are given here below.

- **Asphalt layer**

The asphalt layer is analysed for fatigue. Its structural capacity of fatigue life is determined based on the percentage of the cracked road surface depending on its category. As defined above, this design example concerns the road of Category A with a reliability of 95% (TRH 4, 1996). For this category, the fatigue is reached when 5% of the road surface has cracked. The transfer function for a continuously graded asphalt surfacing, Category A, is given by:

$$N_f = 10^{17.40(1-\frac{\text{Log}\varepsilon_t}{3.4})} \qquad (5.1.1)$$

Where N_f = Fatigue life in number of quivalent standard axles.

ε_t = Horizontal tensile strain at bottom of asphalt layer

- **Granular material**

The transfer function is given in terms of the safety factor against shear failure of the material. The general equation for the transfer function of the granular material is given by:

$$N = 10^{(\alpha F+\beta)} \qquad (5.1.2)$$

Where N = number of quivalent standard axles to safeguard against shear failure

α, β = constants which depend on the reliability level, and

F = safety factor, expressed in terms of stress ratio as follows.

$$F = \frac{\sigma_3\left[K\left(\tan^2\left(45+\frac{\Phi}{2}\right)-1\right)\right]+2KC\tan\left(45+\frac{\Phi}{2}\right)}{(\sigma_1-\sigma_3)} \qquad (5.1.3)$$

Which may be written as:

$$F = \frac{\sigma_3\ \Phi_{term} + C_{term}}{(\sigma_1-\sigma_3)}$$

(5.1.4)

Where σ_1 and σ_3 = major and minor princiole stresses acting at a point in the granular (with the sign convention defined as: compressive stress positive and tensile stress negative).

C = cohesion,

Φ = angle of internal friction,

C_{term} and Φ_{term} = constants for granular materials which depend on moisture condition (given in literature). Considering dry conditions, C_{term} and Φ_{term} for G_1 are equal to 392 and 8.61 respectively.

K = constant for moisture

- 0.65 for saturated conditions
- 0.8 for moderate moisture conditins and
- 0.95 for normal moisture conditions.

The transfer function for category A roads is then given by:

$$N_A = 10^{(2.605122F+3.480098)} \qquad (5.1.5)$$

- **Cemented material**

As mentioned above (Section 5.1.4), the cemented layer is prone to two modes of failure viz effective fatigue and crushing. For that, two transfer functions taking into account those modes of failure have been developed. Nonetheless, it was found that crushing is not a terminal condition and as result, it is not used for the calculation of the cemented layer life (SAPEM, 2013). The general equation for the effective fatigue is given by:

$$N_{eff} = SF.10^{a(1-\frac{\varepsilon}{d\varepsilon_b})} \quad (5.1.6)$$

Where Neff = Effective fatigue life

ε = horizontal tensile strain at bottom of layer in microstrain

ε_b = strain at break (which depend on type of cemented material)

c, d = constants which depend on the road category

SF = shift factor for crack propagation (depends on the thickness of the layer)

For the thickness comprised in the range of 102mm to 319mm, including the thickness of 250mm used in this design example, the shift factor is given by:

$$SF = 10^{(0.00285t-0.293)} \quad (5.1.7)$$

Where t = layer thickness.

For this design case for Category A, c=6.72, d=7.49 and the calculated SF=2.6. The transfer function is then given by:

$$N_{eff} = 2.6 \text{ x } 10^{6.72(1-\frac{\varepsilon}{7.49\varepsilon_b})} \quad (5.1.8)$$

- **Subgrade material**

The selected subgrade and subgrade layers fail in permanent deformation which affect the road surface. The critical parameter is the vertical strain at the surface of these layers. Two transfer functions have been developed for two terminal conditions viz 10mm and 20mm rutting in the layers. However it has been demonstrated that the 20mm transfer function is conservative and the 10mm one is recommended, especially for road Category A and B (SAPEM, 2013). The transfer function is given by:

$$N_{PD} = 10^{(A-10\log\varepsilon_v)} \quad (5.1.9)$$

Where N_{PD} = standard axles to set level of permanent deformation

ε_v = vertical compressive strength at the top of the layer

A = constant which depends on the road category and the terminal condition

For this pavemnt analysis case, for a terminal condition of 10mm rutting, A= 33.30.

The individual layer life for each layer of the studied pavement is given in table 5.9.

Table 5.9. Individual layer life

	Layer life			
Layer	Clay 2		Clay 3	
	Before treatment	After treatment	Before treatment	After treatment
Asphalt	1.43E+05	1.43E+05	1.49E+05	1.46E+05
Granular base	3.41E+10	3.31E+10	3.74E+10	3.63E+10
Cemented subbase	3.18E+06	3.27E+06	3.07E+06	3.18E+06
Selected subgrade	3.88E+09	2.90E+09	4.82E+09	3.42E+09
In situ subgrade	4.03E+10	7.41E+11	1.76E+10	3.03E+11

From Table 5.9, it can be seen that the asphalt layer life is the shortest compared to the other pavement layer lives.

5.1.7. Impact of subgrade stabilization with paper mill ash on the pavement layer life

In order to assess the effect of the stabilization of the subgrade on the performance of pavement layers, the pavement life was determined firstly for the untreated in situ subgrade and then for the subgrade with the upper layer of 150mm thickness stabilized with paper mill ash. The results of the calculations are summarised in Table 5.9. From the above results it can be seen that:

- The stabilization of the top layer of the sugrade has a significant effect on the subgrade life. For Clay 2, the number of load repetitions was increased by 18 times and for Clay 3, 17 times.
- Whereas the in situ subgrade life was significantly increased with the soil-paper stabilization, this treatment had a small negative effect on the selected subgrade life. For Clay 2, the layer life was reduced by 25% and by 41% for Clay 3.
- For the rest of the layers, their lives in terms of load repetitions were the same or almost the same. For instance, the cemented layer was slightly improved for both clays whereas the granular base layer life was slightly reduced after stabilization of the subgrade with paper mill ash. The asphalt wearing course life was unchanged before and after stabilization for both clays.

5.1.8. Summary

This aim of this chapter was to examine the influence of the stabilization of the upper layer of the in situ subgrade with paper mill ash on pavement performance. To this end, a hypothetical pavement consisting of asphalt wearing course, granular base, cemented subbase, selected subgrade and in situ expansive subgrade was considered. The pavement structure and dimensions were selected based on the current standards used in South Africa in road construction or researchers' works For the pavement analysis, a mechanistic-empirical pavement design method, widely used in South Africa, was used during this study. The BISAR program was used to compute the mechanistic critical paramaters induced in pavement due to traffic loading. These mechanistic parameters were then used in the estimation of the life of each layer of the pavement in terms of the number of load repetitions that can be sustained by the layer before reaching its critical condition. During this design example, a single wheel load of 40kN with contact pressure of 900kPa, greater than the current pressure used in South Africa, was used to take into consideration the high traffic loading observed during traffic survey on some national roads.

From Table 5.9, it can be seen that the stabilization of the top layer, 150mm tthick, of the in situ subgrade has a significant positive influence on the life of the subgrade and a small detrimental effect on the adjacent layer viz selected subgrade whereas the life remains more or less constant for the other layers.

5.1.9. Conclusion

"…it is the native soil which really supports the load." John Loudon McAdam.

This chapter sought to study the impact of the subgrade stabilization with paper mill ash on the pavement life. From the results, the following conclusions can be made:

- The stabilization of the upper layer of the expansive soil subgrade has a significant positive effect on the subgrade life. This improvement of the subgrade perfomance can be attributed to the increase of the stiffness of the subgrade due to ash-clay stabilization.
- The treatment of the expansive soil subgrade also caused the reduction of the deformation of the subgrade top layer. Even if this did not affect the life of the wearing course, the reduction of the permanent deformation of the subgrade may

have a positve impact on the rutting of the road surface. This may also reduce the flexural fatigue cracking in the surface.

- Due to the gain in stiffness, the treated subgrade layer may be considered as an additional structural layer for the pavement. For that, it will provide an improved support to the upper layers thus allowing suitable compaction.

Briefly, taking into account the role played by the in situ soil for pavement structure and the significant increase of the stiffness and the life of the subgrade, it can be concluded that the paper mill ash, currently considered as waste, can be used as stabilizer for in situ swelling clay for pavement design and construction purposes.

5.2.DESIGN EXAMPLE OF PIER FOUNDATION ON EXPANSIVE CLAYS

Expansive soils cause damage to structures due to volume change. For that, the designer must provide the foundations capable of supporting the structure within tolerable movement limits. Many types of foundations have been recommended to protect the structures from the swelling caused by expansive soils. Those foundations include pier foundations, piled foundations, split foundations, stiffened raft foundations, etc. (Nelson and Miller, 1992; Williams, Pideon and Day, 1985). This design application concerns the design of pier foundation.

5.2.1. Soil profile description

For the purpose of assessing the effect of paper mill ash on swelling clay through stabilization, a single pier, without a grade beam, founded in a hypothetical subsoil profile was designed. The soil profile was qualified as a hypothetical one since no subsoil investigation was carried out during this study. However a part of the soil profile consists of the studied swelling clay materials. The design example was carried out on two out of three sites representing the swelling clays studied during this study. As described in Chapter 3, Section 3.2.1, the studied clays were collected from different locations and depths from the ground surface. Clay 1 was sampled at depth comprised between 2.5 and 3m. Given the depth at which Clay 1 was sampled, it seems to be impractical to stabilize this swelling clay. In addition to this, the upper boundary of the clay was just above the underground water level which means that the heave will not be significant due to saturation conditions. On the other hand, soil samples for Clay 2 and Clay 3 were collected slightly below the ground surface.

Clay 1 was overlaid by a boulder layer above which lies a sand layer extending to the surface and the material beneath swelling Clay 1 was not investigated. For Clay 2 and Clay 3, there was no information about the underlying materials. Therefore, it is assumed that all clay layers extend to the underlying bedrock.

5.2.2. Design considerations

During the design of pier foundations, the main concern is the uplift force exerted by swelling soil along the pier shaft within the active zone. To minimize that force, the diameter of the piers is kept relatively small. Nelson and Miller (1992) recommends the diameter comprised between 300 and 450mm. For this application, a diameter of 400mm was selected. An active zone, also called zone of seasonal fluctuation, is defined as the upper layer of heaving ground

influenced by climatic environmental factors (Nelson and Miller, 1992). According to Chao (2007), for safe design, the ultimate depth of the active zone is assumed to be equal to the depth of potential heave (Z_p). Two types of piers viz. straight shaft and belled piers, are commonly used in swelling soils (Nelson and Miller, 1992). For this design example, the straight shaft pier was considered. Depending on the soil profile characteristics, the pier can be designed as a rigid shaft pier or elastic shaft pier (Nelson and Miller, 1992). For this application, a rigid shaft pier, assumed to be constructed starting from the ground surface, was considered.

The principle of rigid pier design is that the negative, or downward, skin friction below the depth of potential heave, plus the dead load, are equal to the uplift pressures exerted on the pier by the swelling soil. It is assumed that the pier heave will be equal to zero (Nelson, Schaut and Overton, 2012).

A typical rigid pier in expansive soil as well as forces acting on it are shown in Figure 5.15. The required length of a rigid straight shaft pier is given by (Nelson, Overton and Chao, 2003):

$$L = Z_p + \frac{1}{f_s}\left[\alpha_1 \sigma'_{CV} Z_p - \frac{P_{dl}}{\pi d}\right] \qquad (5.2.1)$$

Where,

Z_p = the depth of potential heave

f_s = the negative skin friction below the depth of potential heave

α_1 = a coefficient of uplift between the pier and the soil

σ'_{CV} = the swelling pressure from the constant volume pressure

P_{dl} = minimum dead load on the pier

d = diameter of the pier

The swelling pressure from the constant volume pressure is defined as the required pressure to keep the volume of a specimen in eodometer cell constant subsequently to inundation of the specimen. This procedure is one of various methods used to determine the swelling pressure. According to Nelson and Miller (1992), the constant volume test procedure was assessed to be more accurate in determining expansive soil heave. The depth of potential heave which represents the maximum depth of active zone that could occur is defined as the depth to which the overburden vertical stress equals or exceeds the swelling pressure of the soil.

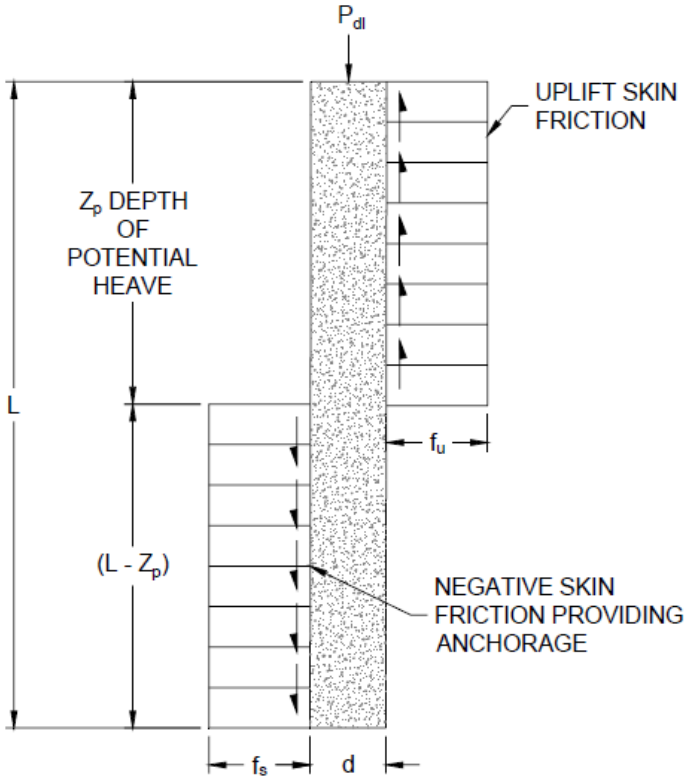


Figure 5.16: Forces acting on a rigid pier in expansive soil

From figure 5.15, it is clear that, due to two forces acting on the pier, one upward another one downward, a tensile force will be developed in the pier. The maximum tensile force, $P_{max,}$ generated in the pier is given by:

$$P_{max}=P_{dl}-f_u Z_p \pi d \qquad (5.2.2)$$

Where

f_u = the uplift skin friction given by:

$f_u = \alpha_1 \sigma'_s$ where α_1 is a coefficient of uplift between the pier and the soil. Different values for α_1 have been recommended by different researches (Chen, 1988; Nelson and Miller, 992; Benvenga, 2005 cited in Chao). Chao (2007) found the range proposed by Bevenga, 0.3 to 0.8, to be more realistic. A value of 0.5 was assumed in this application.

5.2.3. Soil properties

Since no site investigation and testing were conducted, some in situ soil properties used in this application were assumed to be equal to the values determined in laboratory. These properties are summarized in Table 5.10.

Table 5.10: Soil properties for heave prediction

Soil type		Water content (%)	Total density (Mg/m^3)	Swell pressure, σ'_{cs}, (kPa)
Clay 2	Untreated	27.5	1.404	167
	Treated	25.3	1.425	110
Clay 3	Untreated	21.4	1.568	175
	Treated	21.0	1.536	120

The swelling pressure mentioned in Table 5.10 was determined in accordance with Method A of ASTM D4546-08 as described in Chapter 3. It is to be noted that, while for untreated soil the pier will be founded into a ground assumed to be of the same type i.e. swelling clay, for the case of treated clay, the pier will be founded into a two-layer material where the upper layer consists of stabilized swelling clay with paper mill ash.

For this design application, the dead load to be supported by the pier is assumed to be equal to 50kN. In the following, the required length of a rigid straight pier with no movement as well as the free-field heave at the bottom of the superstructure were determined for both untreated and treated clays to evaluate the effect of the stabilization of the swelling clay with paper mill ash. To this end, the acceptable structure movement was assumed to be 50mm. According to Nelson, Chao & Overton (2007) the differential movement between adjacent piers is comprised between a half and one time the total heave.

5.2.4. Determination of the free-field heave

The total free-field heave or the heave of the ground surface with no applied loads is obtained by summing up the heave of different layers within the depth of potential heave. The heave ρ at the midpoint of a soil layer of thickness Δz, at a depth z is given by (Nelson et al., 2007):

$$\rho = C_H.\Delta Z log[\frac{\sigma'_{CV}}{(\sigma'_{VO})_z}] \qquad (5.2.3)$$

Where

C_H= the heave index

σ'_{CV}= swelling pressure from the constant volume oedometer test

$(\sigma'_{VO})_z$ = the overburden pressure at the depth z

The swelling pressure from the constant volume oedometer test is given by (Nelson et al., 2007):

$$\sigma'_{CV} = \sigma'_i + \lambda(\sigma'_{cs} - \sigma'_i) \quad (5.2.4)$$

Where σ'_i= inundation pressure defined as the pressure on the specimen before inundation

σ'_{cs}= swell pressure (obtained using other method than constant volume test procedure)

λ = constant, defined by Nelson, Chao, Overton and Schaut (2012) as follows:

λ = 0.36 to 0.90 (avg = 0.62) for claystone

λ = 0.36 to 0.97 (avg = 0.59) for all soil types

After determining those parameters, the depth of potential heave, Z_p is determined by equating the overburden pressure ($(\sigma'_{VO})_z$) to the swelling pressure (σ'_{CV}). After obtaining the depth of potential heave, the latter is subdivided into several layers and the individual layer heave is calculated using Equation (5.2.3).

5.2.5. Determination of the pier length

The length of the pier is obtained by equating the uplift forces to the sum of the applied dead load and the negative skin friction forces to fulfill the principle of no pier movement.

- The uplift forces are given by:

$$F_u = f_u Z_p \pi d \text{ [kN]} \quad (5.2.5)$$

Where f_u= uplift skin friction (kPa)

Z_p= depth of potential heave [m]

d= diameter [m]

- The negative skin friction is given by:

$$f_s = \alpha_s \sigma'_h \quad (5.2.6)$$

Where α_s=coefficient of negative friction between the pier and the soil, assumed equal to α_1 and

σ'_h= the lateral stress acting on the pier in the anchorage zone assumed equal to the swelling pressure.

The negative (anchorage) skin friction force is given by:

$$F_s = f_s(L-Z_p)\pi d \quad (5.2.7)$$

Where Z_p= depth of potential heave [m]

d= diameter of the pier [m]

5.2.6. Calculation results and pier design

Based on the equations mentioned above, the obtained results were summarized in Table 5.11. For the case of soil treatment, a layer of 2m from the surface was stabilized with paper mill ash.

Table 5.11: Heave index and depth of potential heave

Soil material		Total density (Mg/m^3)	Swell pressure, σ'_{cs} (kPa)	Swelling pressure for a constant volume, σ'_{cv} (kPa)	Heave index, C_H	Depth of potential heave, Z_p, (m)
Clay 2	Untreated	1.404	167	98.94	0.028	7.2
	Treated	1.425	110	65.31	0.014	4.7
Clay 3	Untreated	1.568	175	103.66	0.049	6.7
	Treated	1.536	120	71.21	0.017	4.7

Table 5.11 shows that the depth of potential heave is reduced due to soil stabilization of both Clays 1 and 2. This indicates the reduction in heaving due to the decrease of the active zone.

Table 5.12: Free-field heave

Soil type		Water content (%)	Total density (Mg/m^3)	Swell pressure, σ'_{CV} (kPa)	Free-field heave (mm)
Clay 2	Untreated	27.5	1.404	98.94	86
	Treated	25.3	1.425	65.31	49
Clay 3	Untreated	21.4	1.568	103.66	141
	Treated	21.0	1.536	71.21	71

Table 5.12 shows that the free-field heave is significantly reduced by soil stabilization with paper mill ash. For clay 2, the free-field heave was reduced by 43% and 49.6% for clay 3.

This reduction shows how the damage caused by swelling clay can be reduced by paper mill ash treatment.

Table 5.13: Required length of a rigid pier

Soil material		Swelling pressure for a constant volume, σ'_{cv}	Depth of potential heave, Z_p, (m)	F_u (kN)	F_s(kN)	L(m)
Clay 2	Untreated	98.94	7.2	447.4	62.2L-447.6	13.6
	Treated	65.31	4.7	137.4	62.2L-292.2	6.1
Clay 3	Untreated	103.66	6.7	436.2	65.1L-435.9	12.6
	Treated	71.21	4.7	149.8	65.1L-305.8	6.2

This table shows that the required length of pier founded in Clay 2 is reduced by 55% and, by 51% for the pier founded into Clay 3.

5.2.7. Conclusion

Based on the results of this application the following conclusions can be made:

- The treatment of swelling soil with paper mill ash cause significant decrease of the free-field swell. For that, paper mill ash can be used to protect light structures from ground heave.
- It was also found that the free heave can be reduced to an acceptable value by stabilizing swelling clay with paper mill ash. For instance, the free-field heave of Clay 2 was reduced from 86mm to 49mm which is less than the allowed value of 50mm.
- Once the pier foundation is used, considerable amount of materials, thus, money can be saved by stabilizing swelling clay with paper ash depending on the project size because the stabilization process also involves additional cost. The construction of piers requires various construction materials such as concrete and steel bars. By reducing the length of piers by 50% results in project cost reduction.
- The stabilization of swelling clay with paper mill also results in heaving reduction which means that the risk of damage caused by ground heaving is reduced.

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CHAPTER 6 CONCLUSIONS AND RECOMMENDATIONS

6.1. INTRODUCTION

The review of existing scholarship that was conducted earlier in this study has clearly showed that expansive soils are encountered on all continents and they cause a number of damages to structures particularly light structures, thus placing a large financial burden on builders. For that, many techniques have been developed and applied to prevent or remediate problems caused by expansive soils. Soil stabilization is one of the techniques used for the purpose of preventing expansive soil problems.

Many soil admixtures, traditional as well as non-traditional, have been applied worldwide to modify and/or stabilize problematic soils in order to meet projects' requirements. The literature review also showed that lime as well as lime-rich products have a great application in expansive soil stabilization.

Based on the reviewed successful application of lime-rich products for expansive soil stabilization, this study was carried out to investigate the applicability of a paper mill waste ash to stabilize expansive soils. To achieve this objective, three different clays classified into low, medium and high degrees of potential expansiveness according to classification index properties, were used. Series of laboratory soil mechanics tests including gradation, Atterberg limits, standard compaction, unconfined compressive strength, swell tests and California bearing ratio were performed on both non-treated and treated materials to assess the effect of the ash on those expansive soils' properties. The amount of paper mill ash used for treatment was determined on basis of unconfined compressive strength enhancement as described in chapters three and four.

During this study, as the stabilizing material used is new and is classified as a lime-rich product, testing conditions for lime-treated soils were considered.

Conclusions drawn from the findings and recommendations are given in the following paragraphs.

6.2. CONCLUSIONS

From the results and the applications of the results as well as the literature review, following conclusions were drawn:

- The reduction in plasticity index, thus greater workability compared to untreated clays indicates the immediate cation exchange, flocculation and agglomeration as in the case of lime stabilization.
- The results show that paper mill can be used efficiently to reduce the swell potential as well as swelling pressure of expansive soils
- The immediate increase of CBR substantiates the reaction between paper mill ash and soil like in the case of soil-lime mixture.
- Paper mill ash can be used to save money because it is now considered as a waste, and also time by soil modification as in the case of lime treatment.
- The results also showed that paper mill ash can be substantially used to improve the unconfined compressive strength (UCS) of treated soils. Following the correlation of UCS with other various soil properties (Section 3.3.3), paper mill ash can be used to enhance a number of soil engineering properties.
- Paper mill ash can be used to stabilize swelling clay subgrade provided that adequate measures are taken regarding drainage. The pavement design example showed that the life of subgrade can be significantly increased by addition swelling clays with the ash.
- The length of a pier foundation can also be reduced by stabilizing the upper layer of swelling ground.

Generally, based on the above observations, the paper mill ash from multi fuel boiler (MFB) from Mondi, Merebank paper mill, used for this study can be used affectively to improve different properties of expansive soils in the same way as lime. However, it can be found that the amount of paper mill ash to be used is greater than the amount of lime required for expansive soil treatment. It can also be concluded that paper mill ash works well with soils with high percentage of fines.

6.3. RECOMMENDATIONS

This section provides recommendations for beneficial use requirements and additional tests required for more effective use of the studied paper mill ash and recommendations for further research.

6.3.1. Beneficial use requirements and additional required tests

From the results of this study it can be seen that the used paper mill can be used for expansive soil treatment in plasticity reduction, strength improvement, swell potential reduction, etc. However, before its effective use the following should be considered:

- As remarked in chapter 3, the used ash results from the incineration of sawdust, coal ash, coal and bark. This diversity of raw materials may lead to variability of the resulting ash thus affecting the expected stabilization results. Therefore, the construction industry interested in using this waste as construction material should work closely with Mondi Merebank paper mill so that ash with constant properties be produced.
- Immediate Atteberg limits after soil-stabilizer mixing may show an increase in plasticity because of the increase in fines; for that, Gautrans recommends the determination of Atterberg limits on UCS cured specimens after testing. Therefore, Clay 1 which exhibited an increase in PI, its Atterberg limits should be carried out on UCS cured specimen soils to check this behavior of Clay 1 after treatment with paper mill ash.
- In order to simulate site conditions where soil behavior is highly controlled by confining pressure and water content, shear strength of paper mill stabilized expansive soils should be studied using triaxial tests.
- Due to the role currently played by resilient modulus in pavement design, the resilient modulus test should be performed on paper mill ash stabilized expansive clays.
- Other tests that should be conducted are: resistance to moisture, durability and fatigue, etc.

6.3.2. Recommendation for further studies

The following studies should be conducted for optimum use of the paper mill ash from multi fuel boiler (MFB) of Merebank paper mill:

- Comparison of cost implications between the use of paper mill ash and lime as well as results comparison. This comparison should be conducted on a specific project like stabilization of pavement subgrades and/or subbases and foundations
- Study of the soil stabilization using combination of paper mill ash with lime or cement.
- Environmental impact on the use of paper mill ash for soil stabilization
- Study of influence of environmental factors such as wetting-drying cycles on the engineering properties of paper mill ash stabilized expansive soil,
- Stabilization of dispersive soil with paper mill ash
- Potential use of Merebank Mondi paper mill ash for various purposes mentioned in section 2.9.

APPENDICES

APPENDIX A: Compaction characteristics

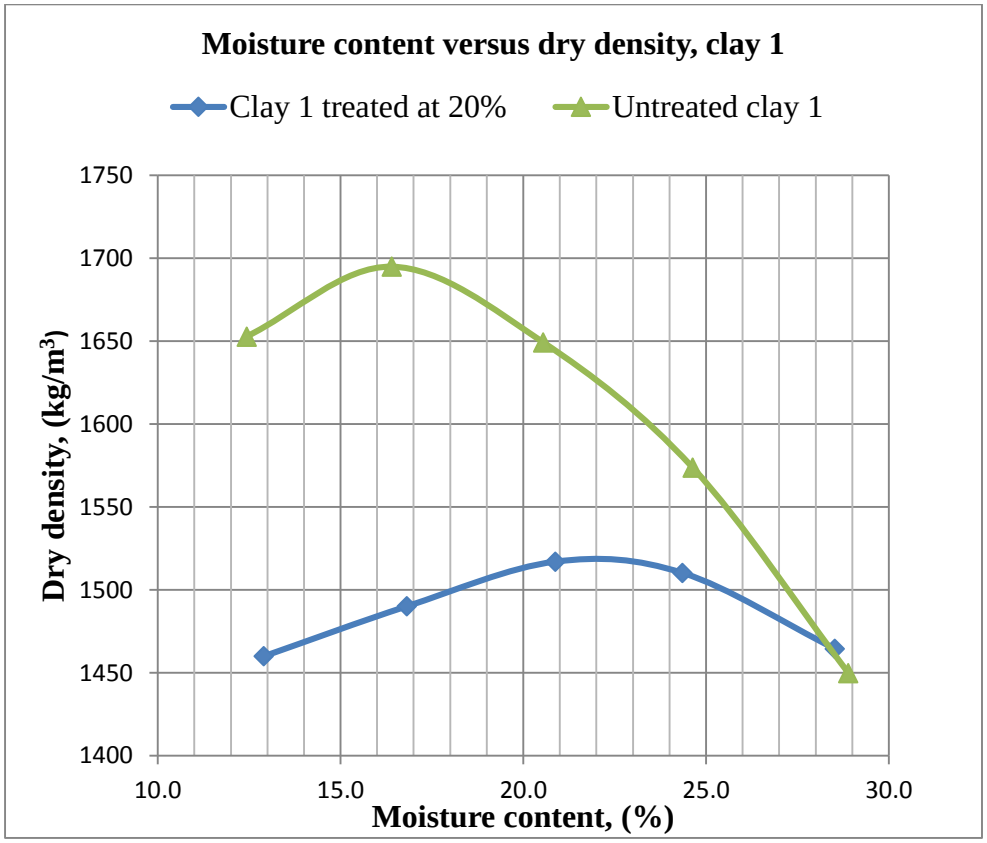

Figure A.1: Moisture content versus dry density for untreated and 20% ash-treated clay 1

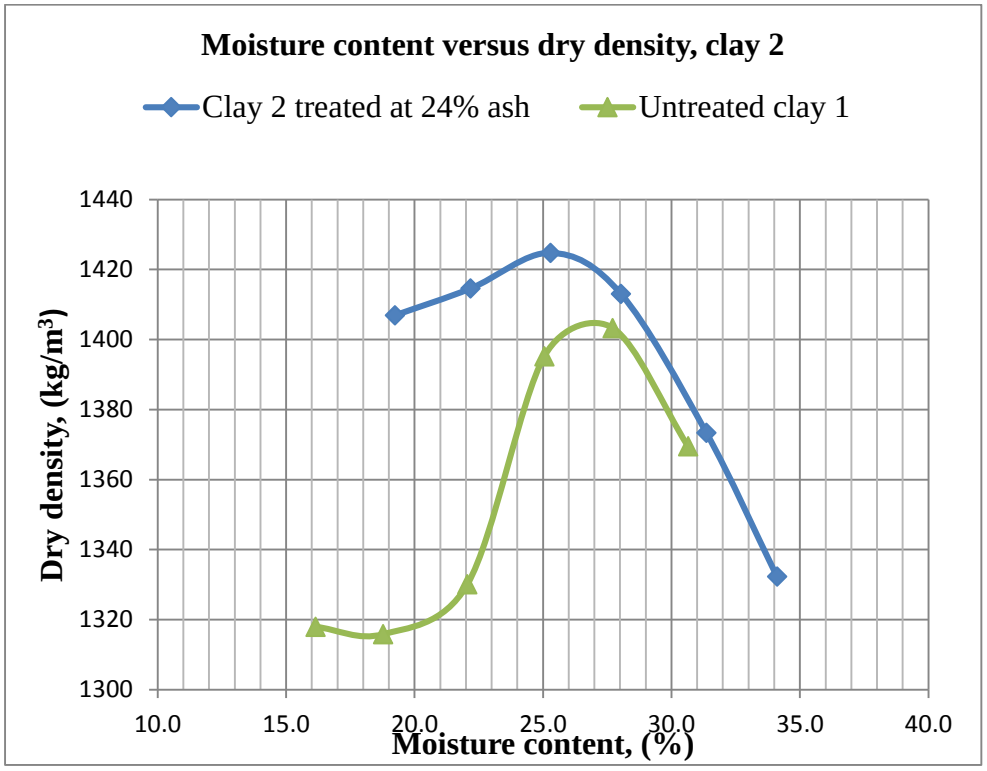

Figure A.2: Moisture content versus dry density for untreated and 24% ash-treated clay 2

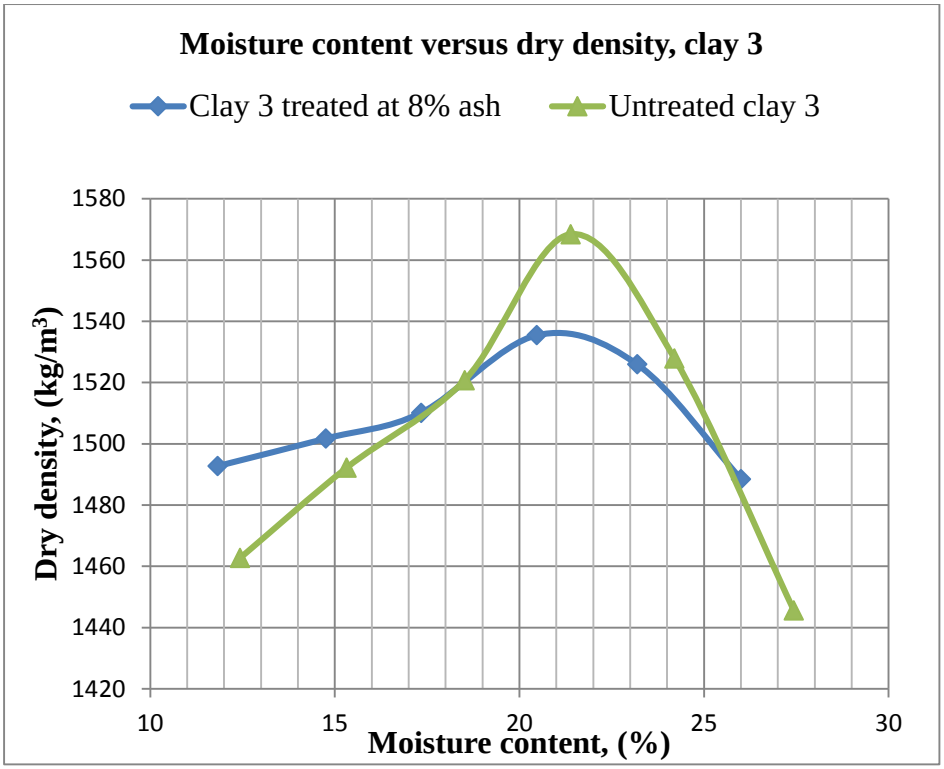

Figure A.3: Moisture content versus dry density for untreated and 8% ash-treated clay 3

APPENDIX B: Unconfined compressive strength versus unit strain

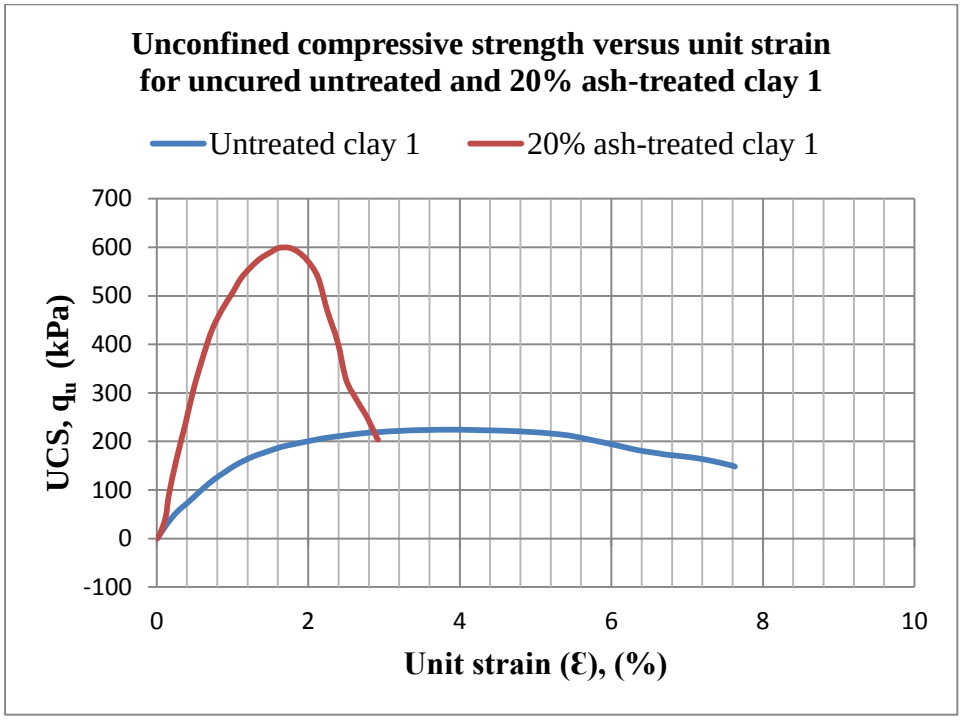

Figure B.1: Unconfined compressive strength (UCS) versus unit strain for uncured untreated and 20% ash-treated clay 1 specimens.

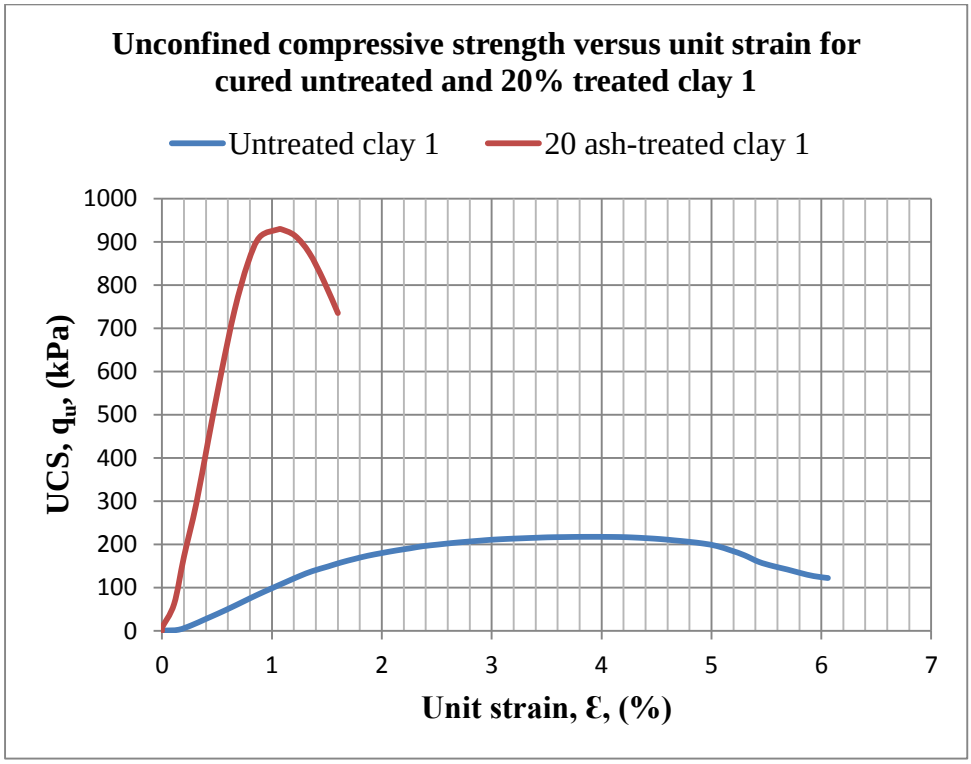

Figure B.2: Unconfined compressive strength (UCS) versus unit strain for cured untreated and 20% ash-treated clay 1 specimens.

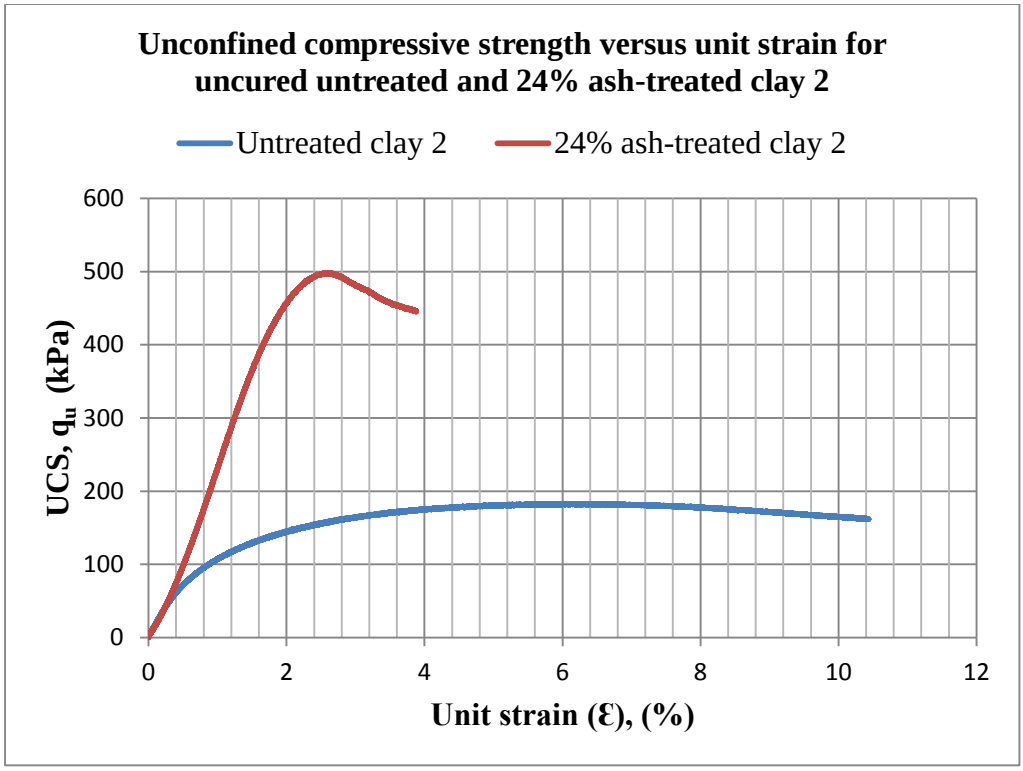

Figure B.3: Unconfined compressive strength (UCS) versus unit strain for uncured untreated and 24% ash-treated clay 2 specimens.

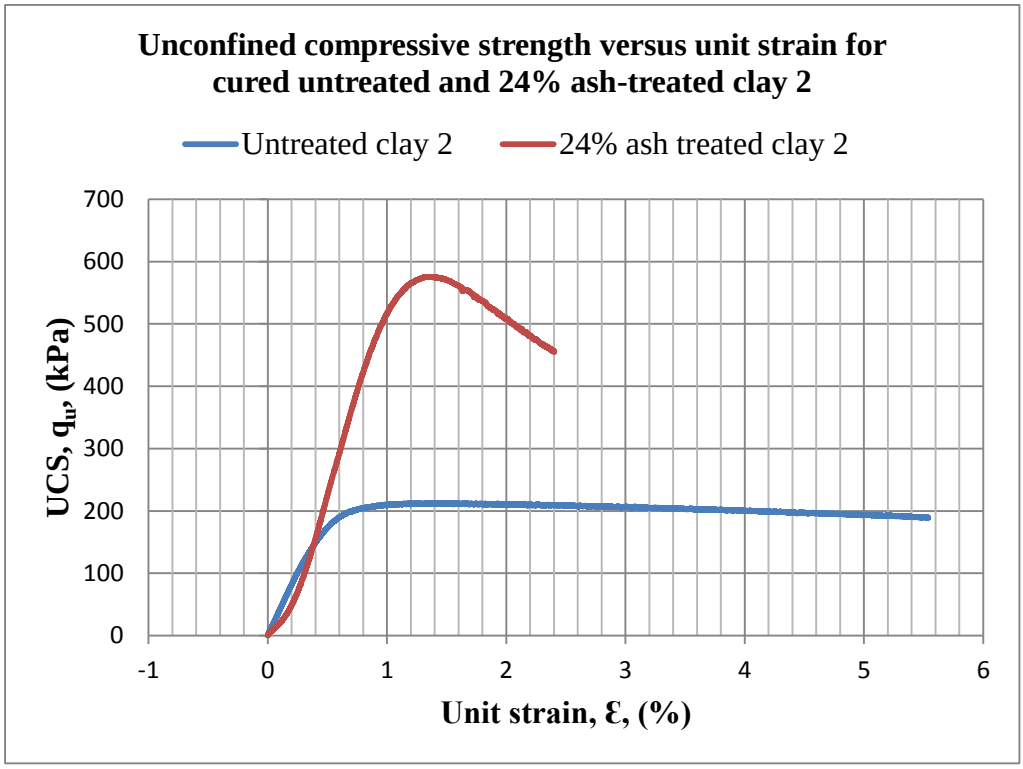

Figure B.4: Unconfined compressive strength (UCS) versus unit strain for cured untreated and 24% ash-treated clay 2 specimens.

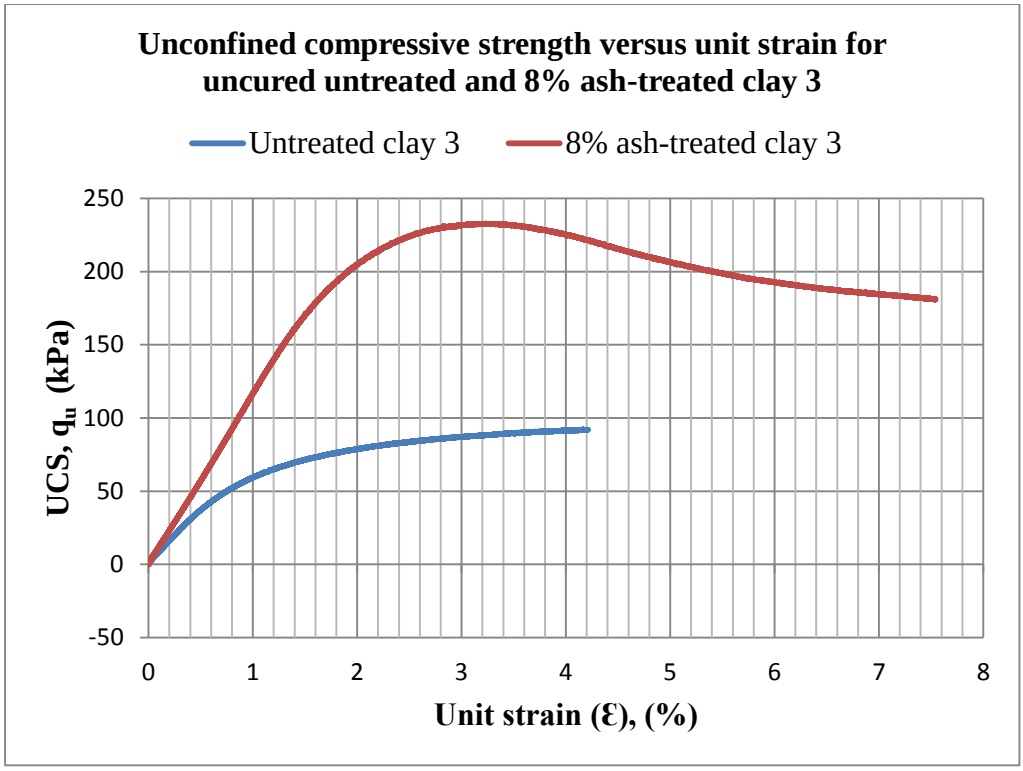

Figure B.5: Unconfined compressive strength (UCS) versus unit strain for uncured untreated and 8% ash-treated clay 3 specimens.

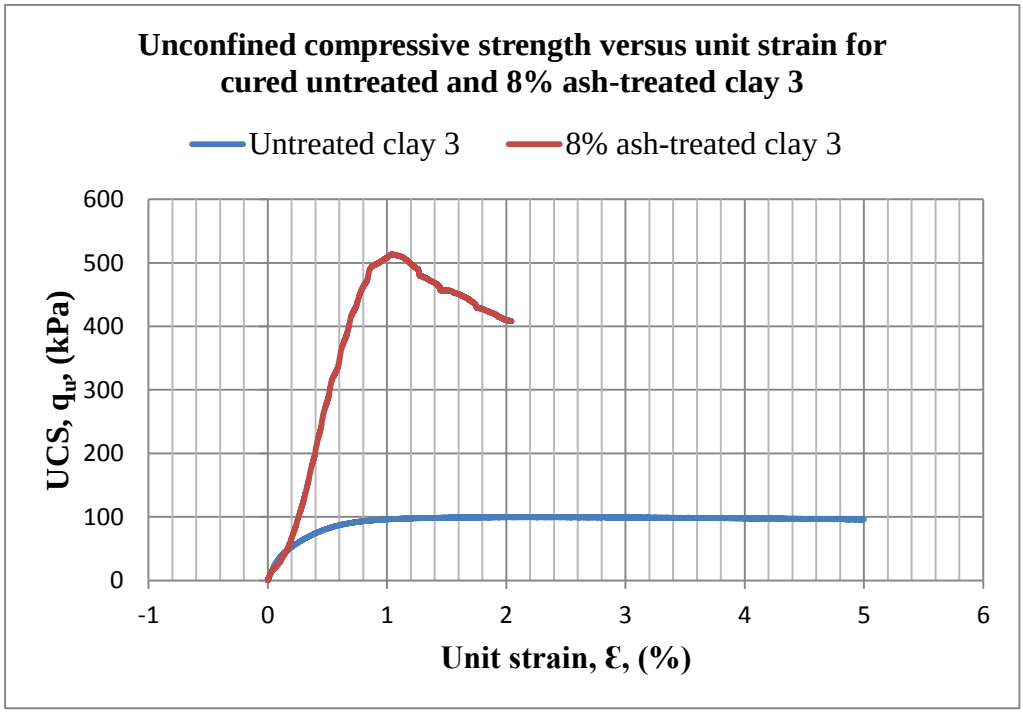

Figure B.6: Unconfined compressive strength (UCS) versus unit strain for 7 days cured untreated and 8% ash-treated clay 3 specimens.

APPENDIX C: Swell test results

C-1. CLAY 1

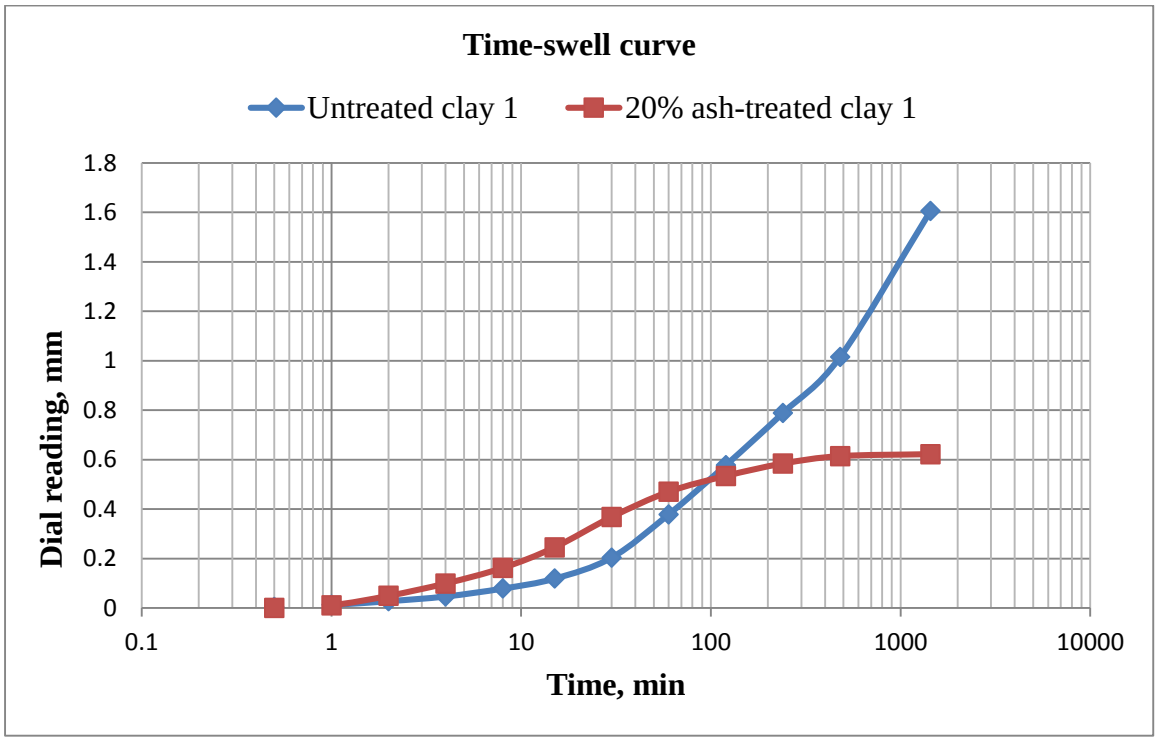

Figure C-1.1: Time-swell relationship for untreated and 20% ash-treated clay 1 under 1.2 kPa pressure

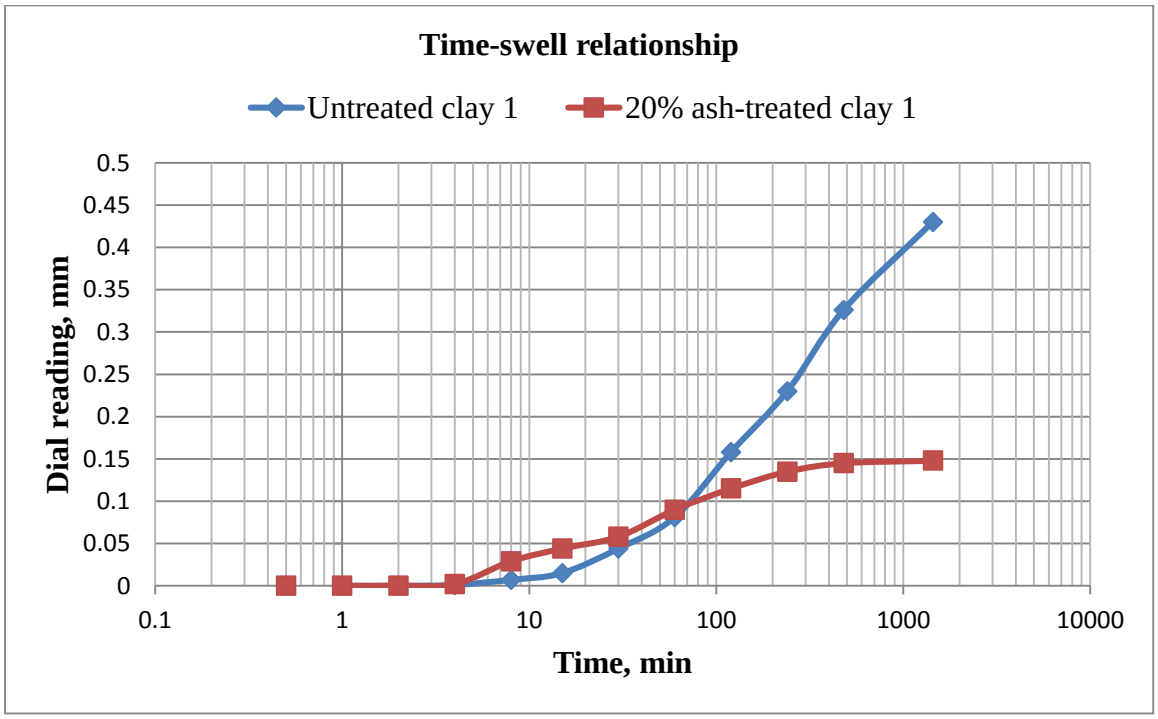

Figure C-1.2: Time-swell relationship for untreated and 20% ash-treated clay 1 under 25 kPa pressure

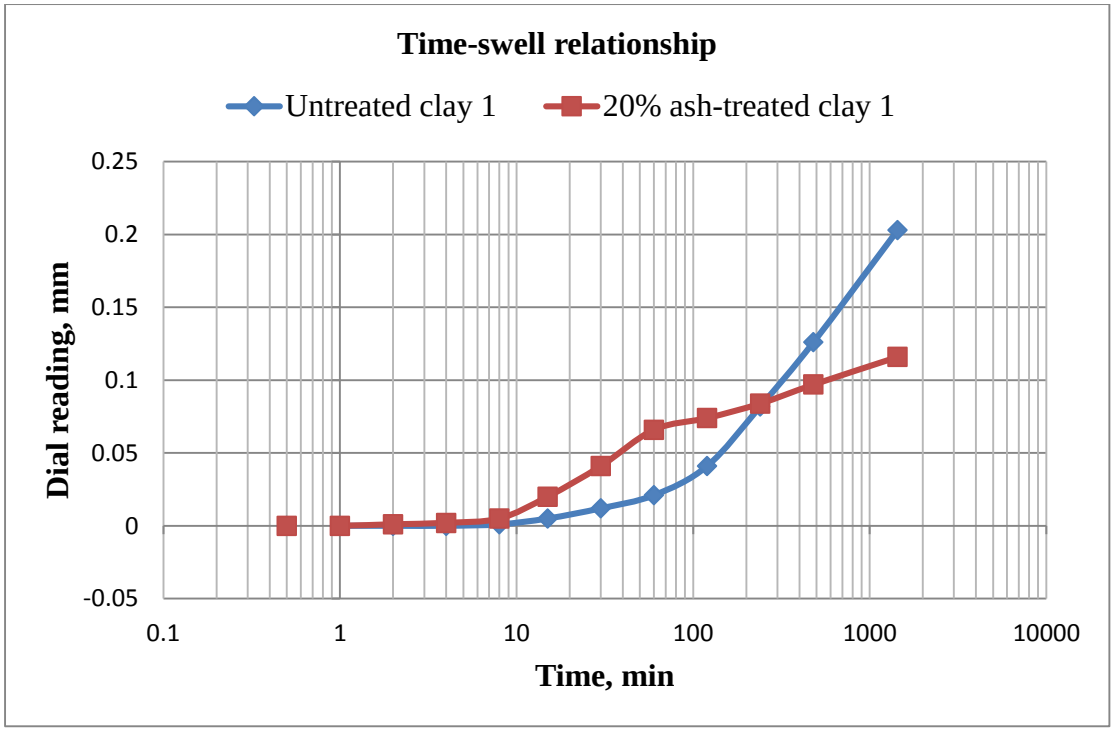

Figure C-1.3: Time-swell relationship for untreated and 20% ash-treated clay 1 under 50 kPa pressure

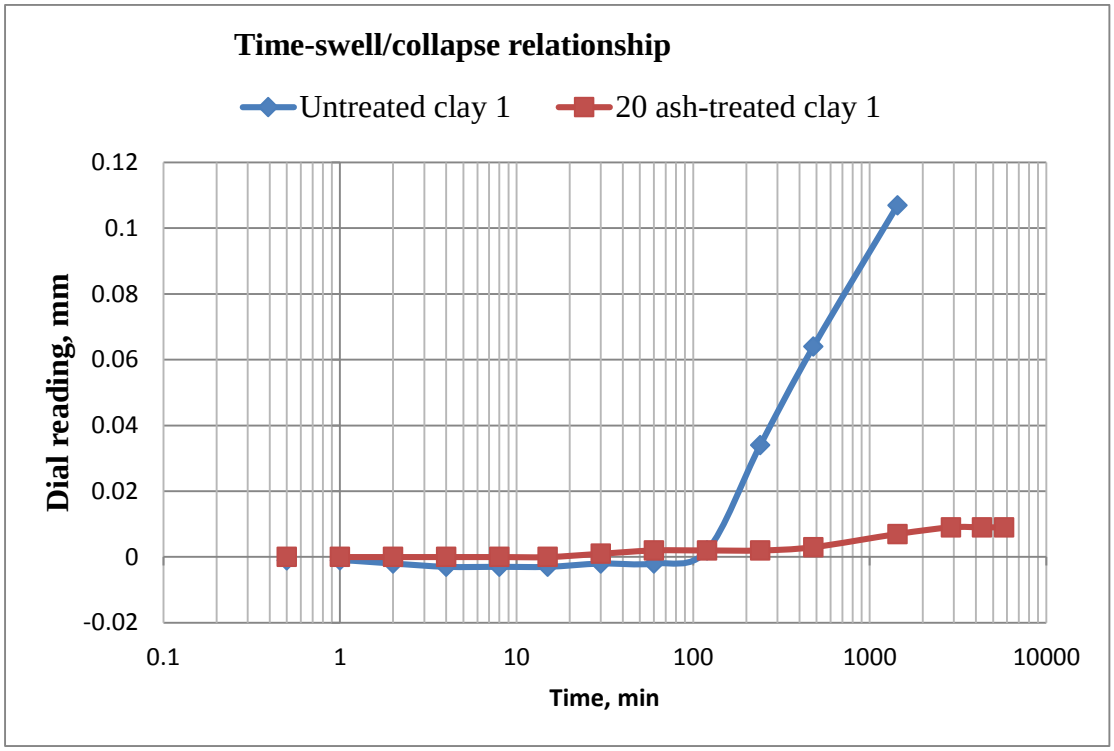

Figure C-1.4: Time-swell relationship for untreated and 20% ash-treated clay 1 under 100 kPa pressure

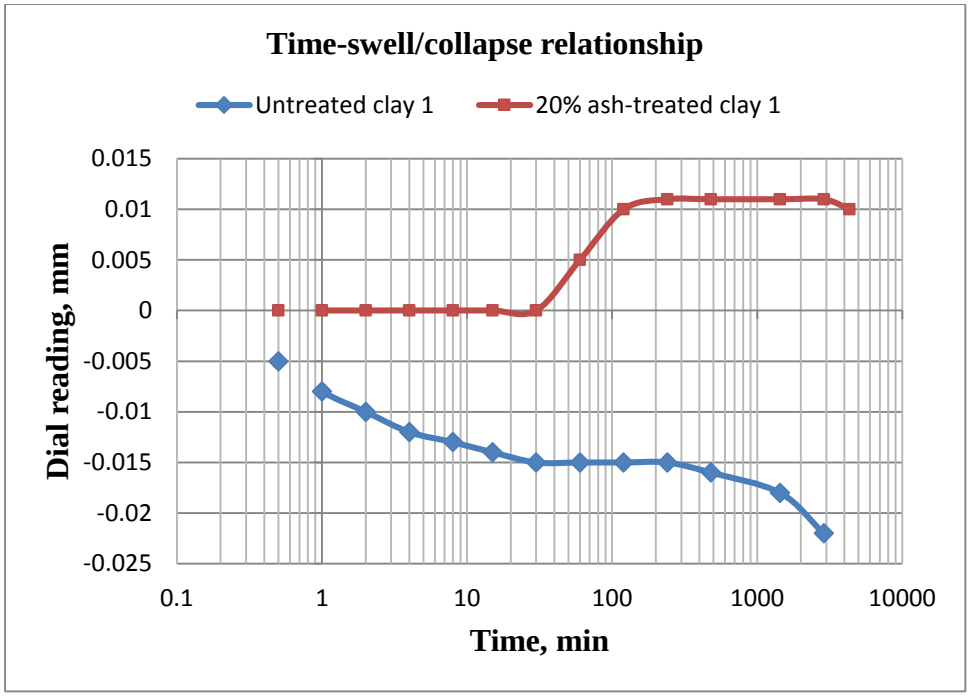

Figure C-1.5: Time-swell relationship for untreated and 20% ash-treated clay 1 under 200 kPa pressure

C-2. CLAY 2

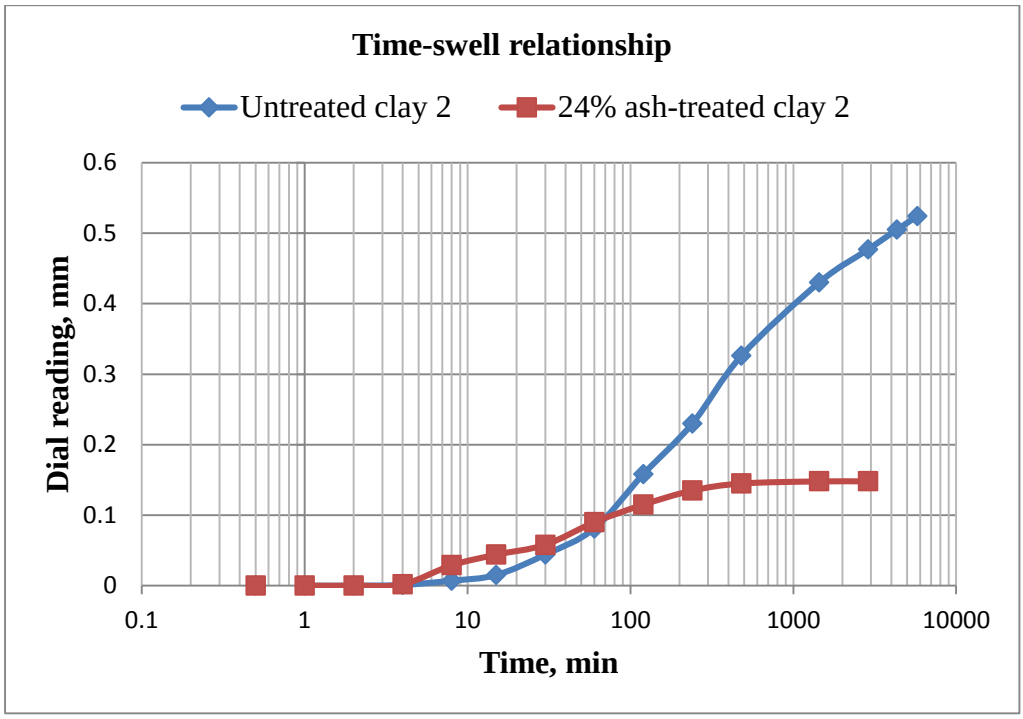

Figure C-2.1: Time-swell relationship for untreated and 24% ash-treated clay 2 under 1.2 kPa pressure

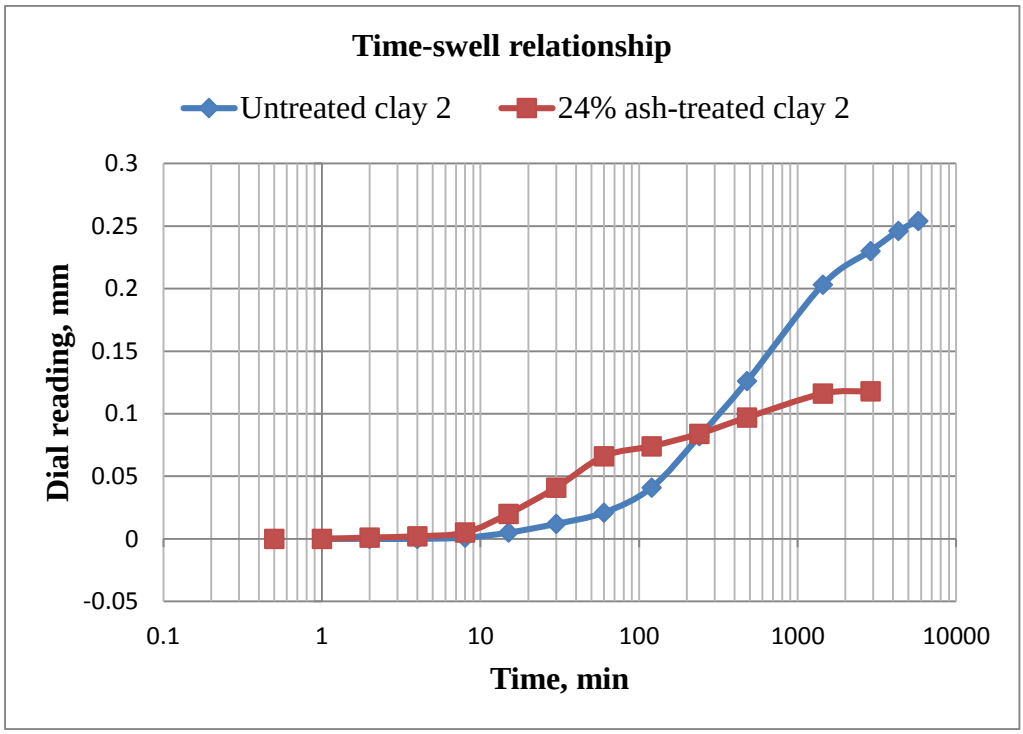

Figure C-2.2: Time-swell relationship for untreated and 24% ash-treated clay 2 under 25 kPa pressure

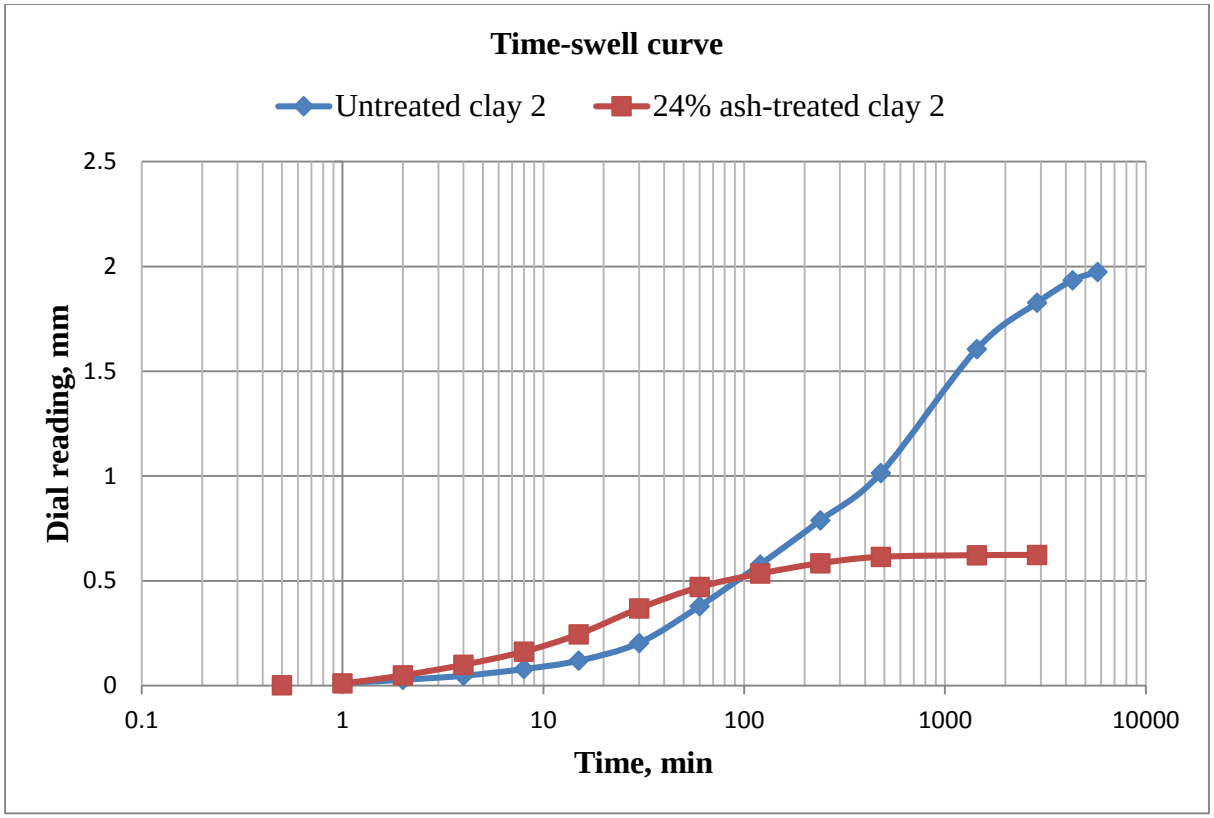

Figure C-2.3: Time-swell relationship for untreated and 24% ash-treated clay 2 under 50 kPa pressure

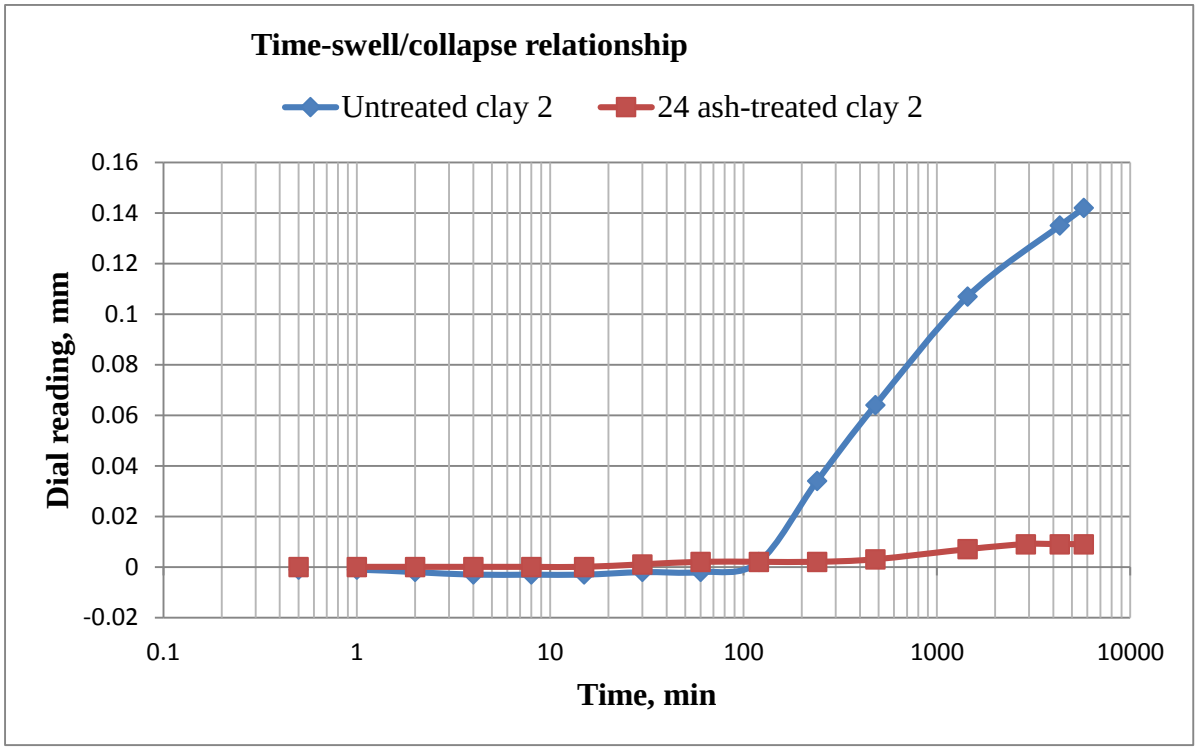

Figure C-2.4: Time-swell relationship for untreated and 24% ash-treated clay 2 under 100 kPa pressure

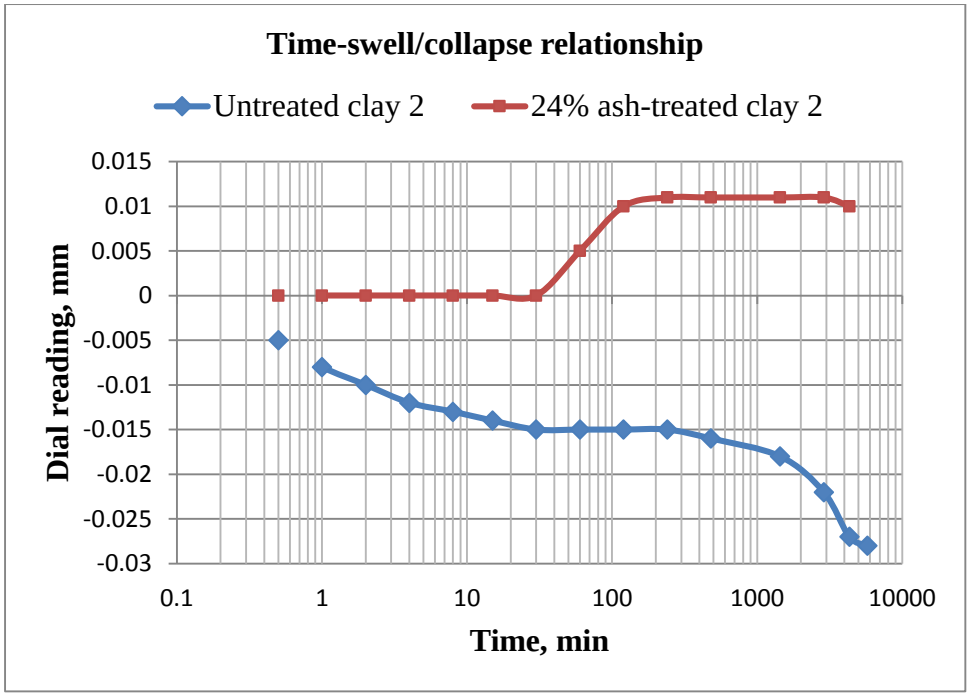

Figure C-2.5: Time-swell relationship for untreated and 24% ash-treated clay 2 under 200 kPa pressure

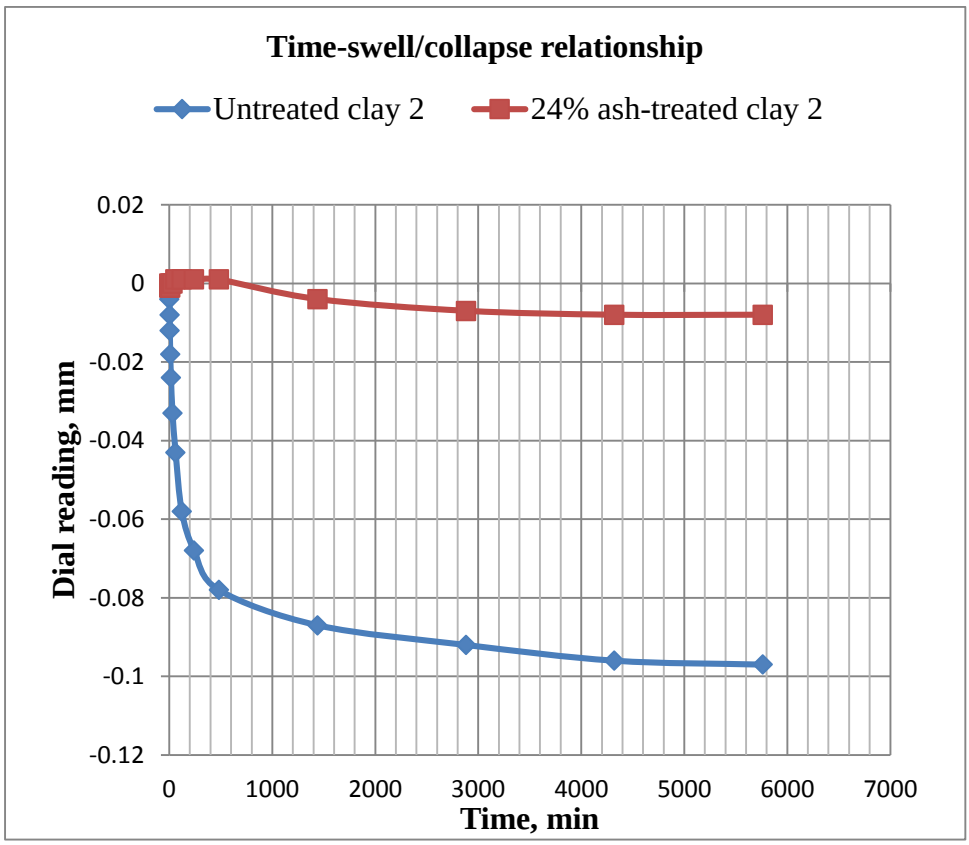

Figure C-2.6: Time-swell relationship for untreated and 24% ash-treated clay 2 under 400 kPa pressure

C-3. CLAY 3

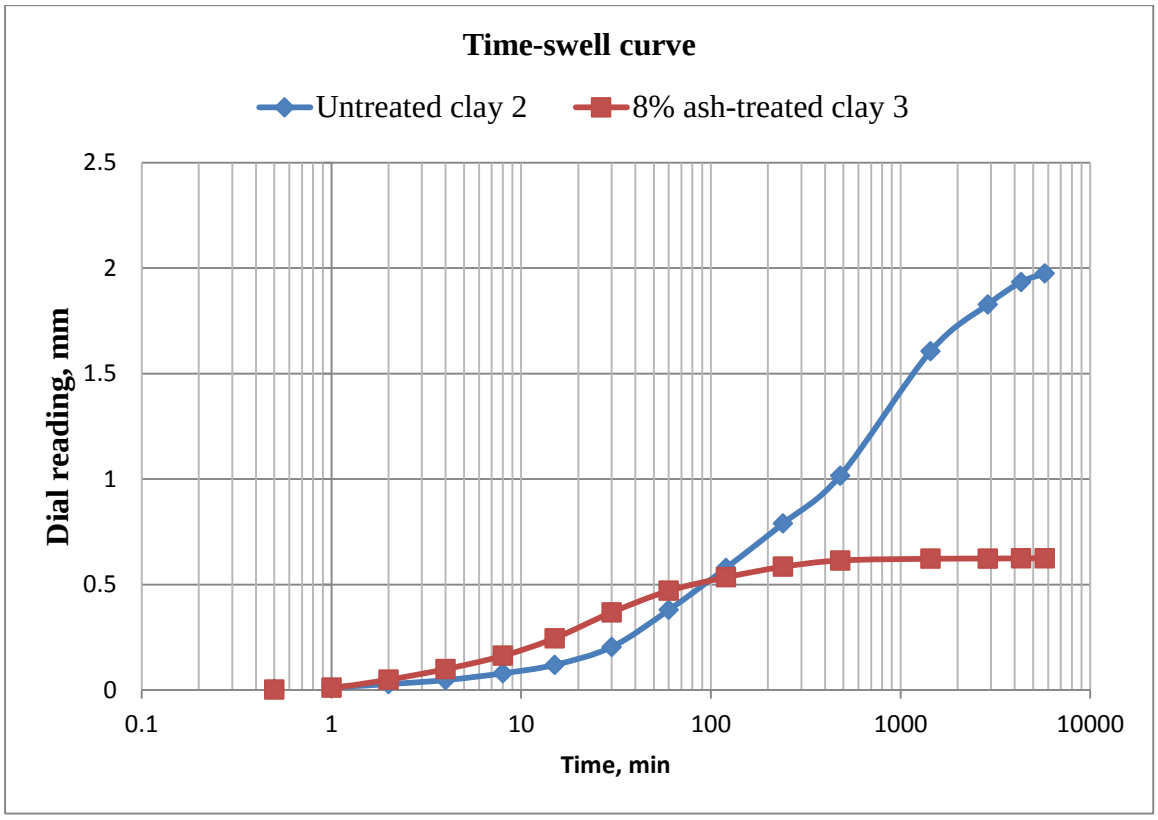

Figure C-3.1: Time-swell relationship for untreated and 24% ash-treated clay 3 under 1.2 kPa pressure

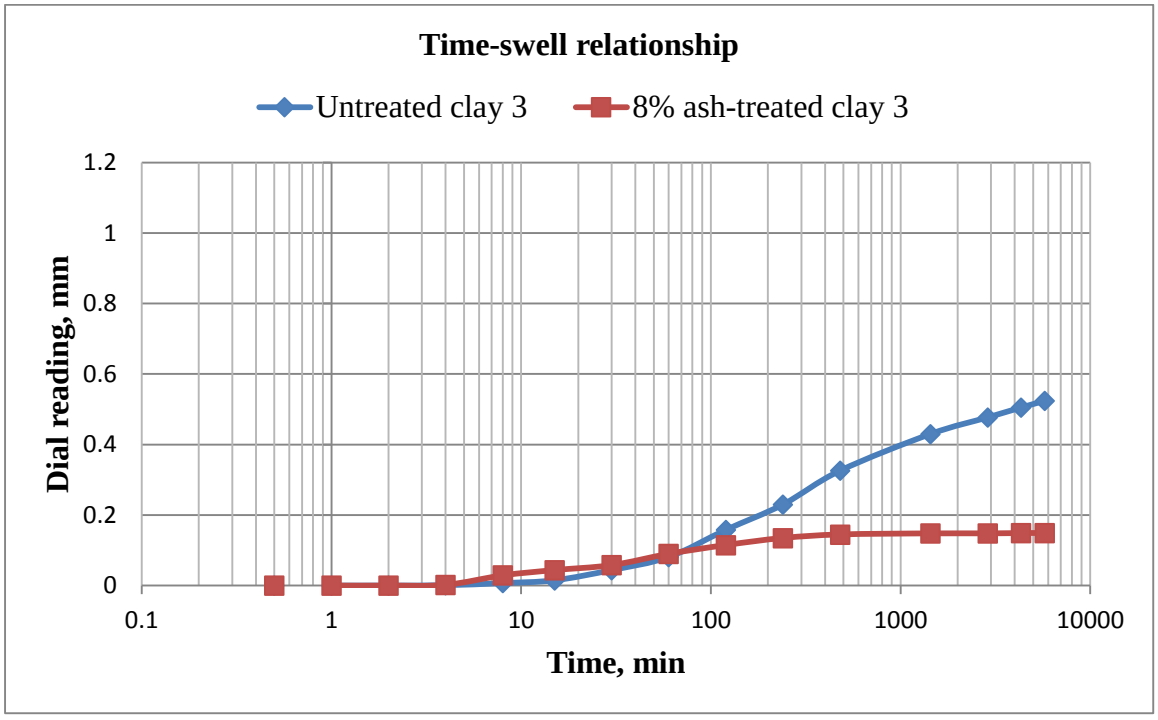

Figure C-3.2: Time-swell relationship for untreated and 24% ash-treated clay 3 under 25 kPa pressure

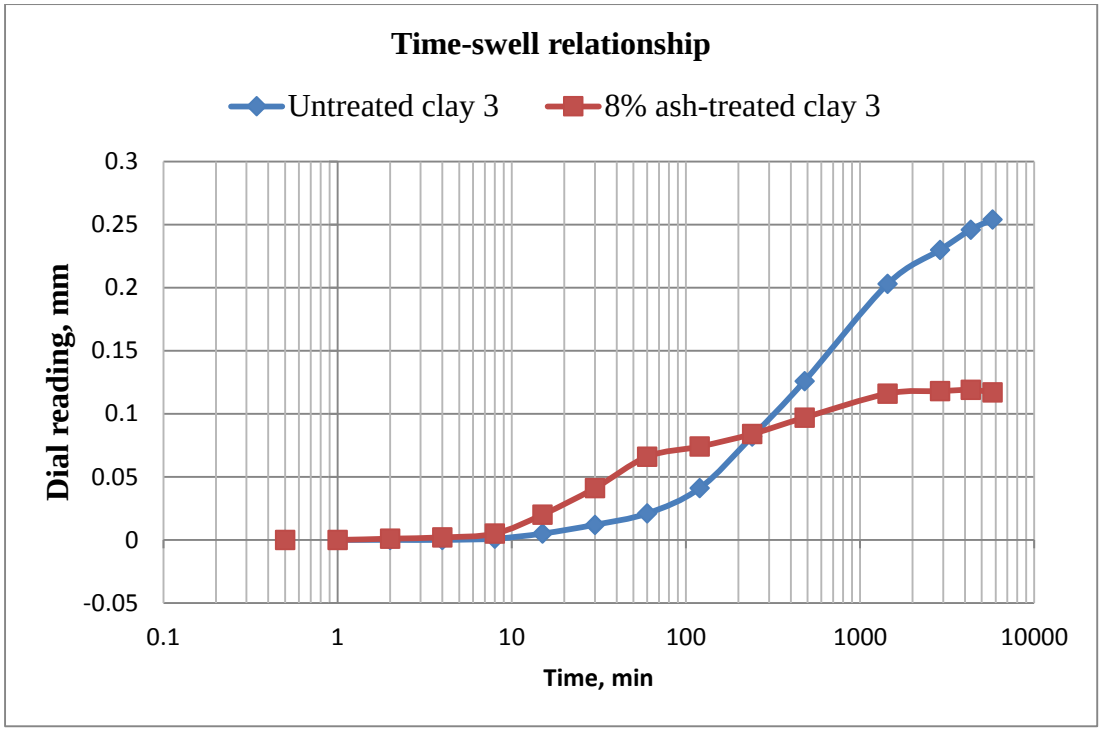

Figure C-3.3: Time-swell relationship for untreated and 24% ash-treated clay 3 under 50 kPa pressure

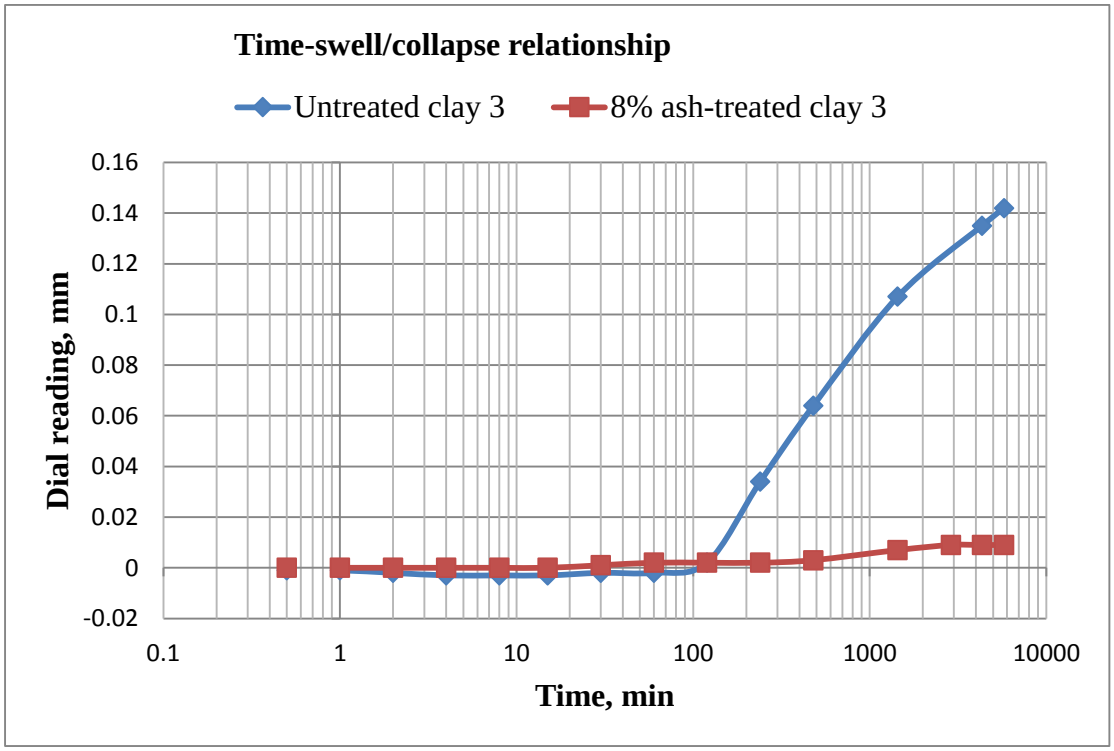

Figure C-3.4: Time-swell relationship for untreated and 24% ash-treated clay 3 under 100 kPa pressure

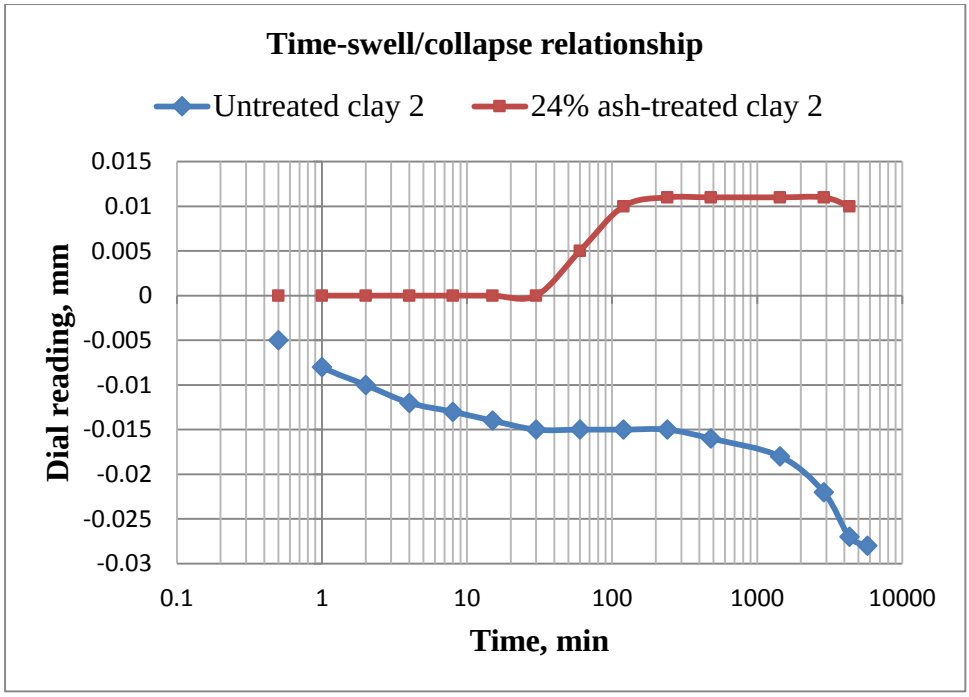

Figure C-3.5: Time-swell relationship for untreated and 24% ash-treated clay 3 under 200 kPa pressure

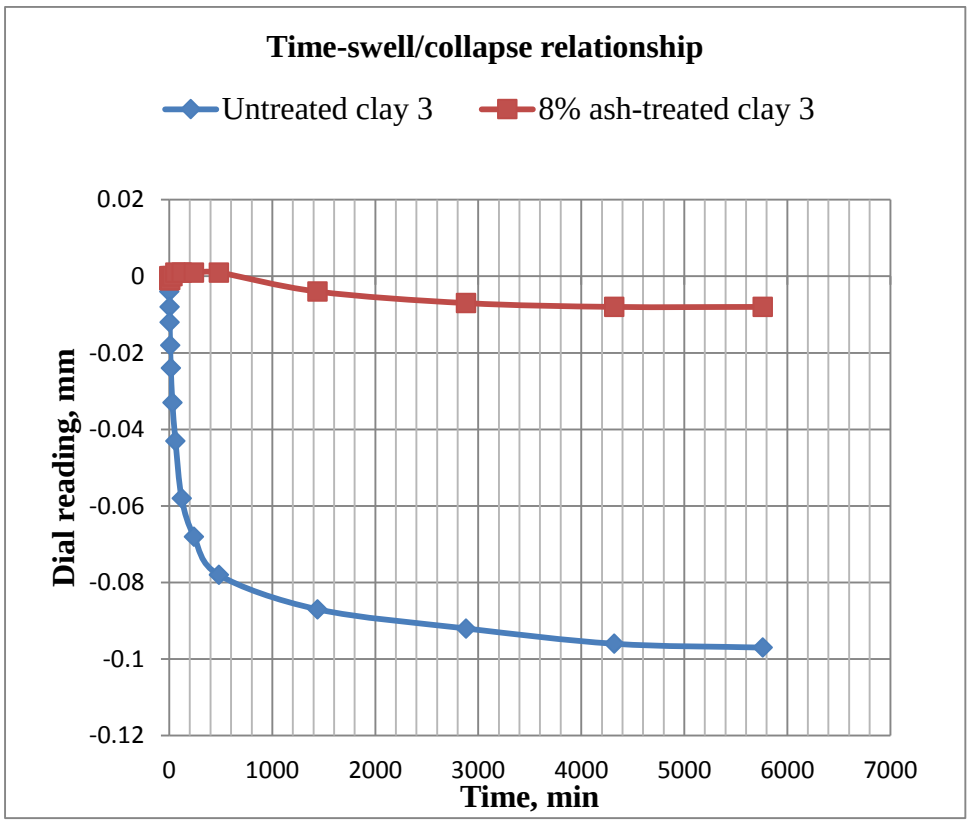

Figure C-3.6: Time-swell relationship for untreated and 24% ash-treated clay 3 under 400 kPa pressure