University of Southern Queensland

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Study of effects of coarse grain contents on Atterberg limits and expansiveness of the clay

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Abstract

Expansive soil, which experiences significant volume change associated with change in water content, can cause severe distress to the structure build on them. It may be noted that repair of damaged infrastructures built on expansive soil costs billions of dollars annually. Especially roadways and small building are subjected to severe cracking and distress due to surface movements resulting from wetting and drying of expansive soil. Therefore geotechnical researchers have developed several measures to stabilise the soil by improving its expansive characteristics of soil. Soil stabilisation techniques aim at improving soil strength and increasing resistance to softening by water through bonding the soil particles together, water proofing the particles or combination of the two (Sherwood, 1993).

The research aims to study the effect of sand content on a selected, expansive clay type, namely kaolinite clay. In this study, kaolinite is mixed with a coarse grain material (fine sand) at various percentages by weight as a measure to stabilise/treat the expansive soil. Atterberg limits tests (liquid limit and plastic limit) and expansive properties (swelling potential and swelling pressure) tests are performed in the lab. The experiment results indicate that liquid limit, plastic limit, swelling potential and swelling pressure are reduced with increased fine sand content. The reduction of Atterberg limits is almost linear to the increment of sand content. However, change in swelling potential and swelling pressure is very significant when the sand content is increased from 25% to 50%. Further increase in sand content above 50% does not indicate significant changes in either swell pressure or potential. Therefore, further investigations need to be performed with more variation in percentage of sand content between 25 to 50%.

In addition to above Atterberg limits and expansive characteristics, this study aims to measure natural rebound or swell caused by unloading process and total swelling. Loading and unloading cycles were introduced to measure the swelling amount as well swell pressure. The literature does not provide adequate information on attempts to measure above two components on individual basis. However, the tests were carried out under ambient conditions or uncontrolled humidity, due to limited laboratory facilities. Nevertheless, this new contribution to knowledge can encourage future research in this direction to provide much useful information to geotechnical engineers.

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Limitations of Use

Certification of Dissertation

I certify that the ideas, designs and experimental work, results, analyses and conclusions set out in this dissertation are entirely my own effort, except where otherwise indicated and acknowledged.

I further certify that the work is original and has not been previously submitted for assessment in any other course or institution, except where specifically stated.

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1 Introduction

Soil is a complex material with different engineering properties that may vary due to many factors such as soil mineralogy, environment and stress. Expansive soils are the ones that undergo large amounts of heaving and shrinking due to seasonal moisture changes. Like any other soil deposits, expansive soils are usually heterogeneous in composition and each deposit is different from every other deposit.

Expansive soils are found in many parts of the world particularly in semi-arid regions where the evapotranspiration exceeds the precipitation. They are generally unsaturated and contain clay minerals that exhibit high volume upon wetting and drying. Expansive soil when wetted will apply a considerable pressure against the structure built on it. Structures such as roadways and small buildings built on expansive soil are often subjected to serious cracking and distress.

The solution to the problem of foundations on expansive soils cannot be achieved without an understanding of the fundamental characteristics of expansive soils and the variables involved that affect the swelling phenomena. Safe and economic designs of founds on expansive soils require determining the swelling indices such as swelling, pressure, swelling potential and swelling index (ElKholy, 2008).

The major problem in expansive soil is excessive volume change. Change of volume in foundation soil is usually accompanied by change in shear strength. Shear strength and volume change are important variables usually considered in the design and construction of building foundations, pavement (roads), embankments and retaining structures.

Due to expansive soil, the distressed infrastructure problems have resulted in billions of dollars of repair costs annually (Nelson and Miller, 1992). Therefore geotechnical researchers have developed many measures to reduce the expansiveness of soils. One of the techniques of treatment of expansive soil is with a wide range of additives. The current study seeks to improve expansive properties of soil by addition of non-expansive soil which is coarser than 0.425mm.

1.1 Project Aims and Objectives

Expansive soil is the one that experiences significant volume change associated with changes in waters contents. Extensive studies have been conducted regarding the properties of expansive soil. Due to the global distribution of expansive soils many different ways to tackle the problems have been developed and these can vary considerably. Full replacement of expansive soil layers and treatment of expansive soil with a wide of range of additives are of the two main techniques used in order to improve the properties of expansive soil.

The main aim of the research is to study the characteristics of commercially available expansive soil (i.e., kaolinite) and find the improvement in their engineering properties with the addition of non-expansive soil (i.e., fine uniform sand). The present study mainly focuses on Atterberg limits (liquid limit, plastic limit and plasticity index) and swelling indices such as swelling potential and swelling pressure of expansive soil.

An experiment based study is carried out to investigate and analyse the relationships of characteristics of expansive soil with and without coarse grain non-expansive soil.

The objectives of this project can be summarised as follows:

- To study plasticity properties (Atterberg limits) of soil
- To understand the nature of expansive soils and their properties
- To perform laboratory tests to study the characteristics of expansive soils when mixed with coarse-grained (non-expansive soil)
- To seek a correlation between Atterberg limits and swelling characteristics of expansive soils

2 Literature Review

Expansive soils, which undergo large volume changes when subject to the actions of wetting and dying, present significant geotechnical and structural engineering challenges all around the world costing several billions annually. The importance and necessity of knowledge of expansive soil properties and physical behaviours is reviewed in this literature review. A particular emphasis has been given to the following topics to gain a thorough understanding of the essential engineering properties of expansive soil.

- Basic soil properties
- Expansive soil background
- Swell-shrink properties
- Identification and classification of expansive soils

2.1 Basic Soil Properties

Soil is comprised of minerals, solid organic matter, water and air. The compositions of these components greatly influence soil physical properties, including texture, structure and porosity, the fracture fraction of pore space in a soil. These properties in turn affect air and water movement in the soil and thus the soil's ability to function. The understanding and knowledge of soil materials found in the construction site is the first step to be developed before the design stage. It is essential to know its basic characteristics as thoroughly as possible because soil behaves in a complex manner in different conditions. Only after the basic characteristics of soil are known, its engineering properties can be defined. In this section, brief description of soil and different soil classification systems that are used worldwide are presented.

Soil is a natural aggregate of mineral grains such as rocks, which can be separated by means of agitation in water (Murthy, 2003). It is a complex engineering material, which can be simply described as cohesionless or cohesive. Coarse grained are cohesionless soils in which the majority of the soil particles are greater than 75µm in size such as gravel, sand and boulders. These soils are also called granular soils, which are influenced by the comparative proportions of the different shape, size of particles and the density where gravitational forces determined their engineering characteristics. Fine-grained soils are cohesive soils in which the most of the soil particles are less than 75µm in size such as clay and silt.

Generally the smallest particle size that can be differentiated with the naked eye is one of about 75μm. The engineering behaviour of this soil type is dependent on the mineralogy of the fine soil particles and water content where interparticle forces are predominant (Murthy, 2003). Soil is formed due to the weathering of the parent rock, which takes place in arid climates (Murthy, 2003). The structure behaviour depends on the geotechnical properties of soil materials in which the structure builds on. Australian Standards (1993) use the index properties such as particle distribution and Atterberg Limits (plasticity) to classify coarse grained and fine-grained soils respectively.

2.1.1 Particle Size Distribution

Sieve Analysis is used to determine the particle size distribution of a coarse grained soil. A prepared dry soil sample is shaken thoroughly and passed through a stack of sieves that consist of different apertures. The percentage of soil particles passed through different sizes of sieves is calculated as a percentage of the total dry sample mass. In case of fine-grained soils, Hydrometer Analysis is used to determine the particle size distribution where soils are combined with distilled water to make 1000 ml of suspension. Then the hydrometer is used to measure the density of the solution for specific times. The time-density data is used to calculate the percentage of particle sizes for the required 48 hours period where observations are required to be made.

Both coarse and fine grains are quite commonly found in the soil, which makes it necessary to perform sieve and hydrometer analysis to determine the complete particle size distribution. The preferred way to carry out these tests is to perform sieve analysis first and then hydrometer test to the particles that passed the 75μm sieve. Then the particle size distribution is calculated cumulatively according to the percentage passing each sieve.

2.1.2 Atterberg Limits

The engineering properties of fine-grained soils vary significantly depending on the amount of moisture available within soils. A. Atterberg in early 1900 developed the limiting moisture contents for key physical states, which are known as Atterberg Limits, and this consists of liquid limit, plastic limit, and shrinkage limit (Das, 2006) as shown in figure 2. K. Terzaghi in the late 1920's and A. Casagrande in early 1930's refined these limits in order to make it suitable for geotechnical works. (Sivakugan, 2000).

The Atterberg limits are the behaviour of soil in a solid, plastic and liquid due to the variation in the range of the moisture content. The liquid limit (LL) of a soil is the percentage of water content above, which the soil behaves as a liquid. The plastic limit (PL) is defined as the percentage of moisture content above which the soil behaves as plastic. Plasticity Index (PI) is the difference between the liquid limit and plastic limits. The shrinkage limit is defined as the percentage of moisture content below which the soil will not shrink when dried. Figure 1 below summarizes the description of these states, limits and indices.

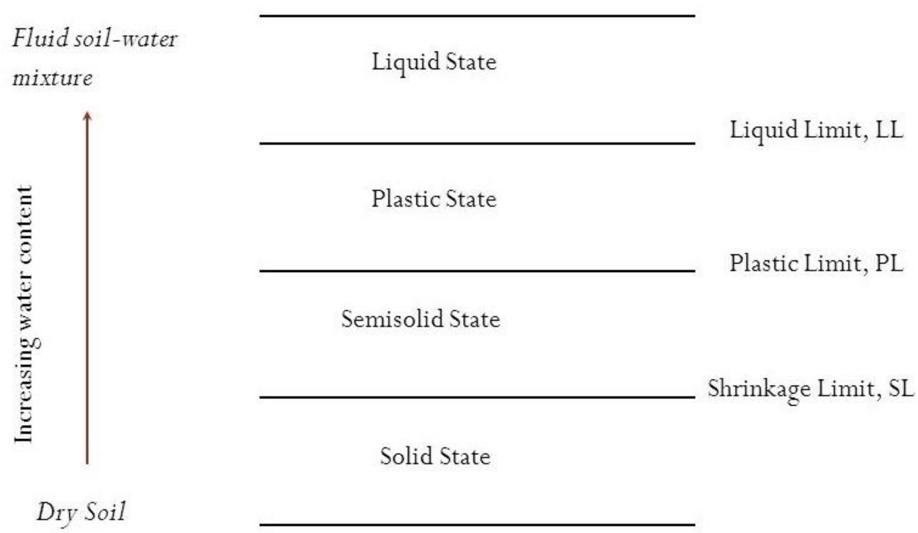

Figure 1. Different States and Limits (Das, 2006)

AS 1726 – 1993 stated that the fine-grained soils (clay and silt) can be described according to their plasticity, which is shown in below table 1.

Table 1. Plasticity in terms of liquid limits (Australian Standards, 1993)

Descriptive Term	Range of Liquid limit (%)
Of low plasticity	≤ 35
of medium plasticity	$> 35 \leq 50$
of high plasticity	> 50

2.2 Expansive Soil

2.2.1 Background

Expansive soil is the one that changes in volume in relation to changes in water content. The volume changes can either be in the form of swell upon absorption of water or in the form of shrinkage upon evaporation, and therefore they are sometimes known as swell/shrink soils or swelling soils or reactive soils.

All the infrastructures are built on foundations, which largely influence infrastructure structural performance. If the foundation soil tends to expand or contracts, it can cause failure in the structures. With naturally available soils, clays with high plasticity are classified as expansive soils as clays possess change in volume when subjected to moisture variations (Yang, H et al., 2007). Chen (1988) defined that Montmorillonite clay has the high swell shrink potential. Soil with swell potential can be often referred as vertisols, which contain clay minerals those possess a net negative electrical charge imbalance attracting the positive pole of dipolar water molecules and cations due to their natural physiochemical properties (Snethen, 1980). A huge numbers of infrastructure, particularly those with low self-weight, experience the problems created by reactive soils associated with serviceability performance mostly in the form of cracks and permanent deformation.

The expansive soils are found in humid environment, air or semi-arid regions of the tropical and temperate climate zones and are widely distributed over almost all geographical locations worldwide, Australia, Ethiopia, India and USA to name a few countries (Chen, 1988; Jones and Jefferson, 2015). In the United States, it was estimated that expansive soils affected structures worth billions of dollars, particularly to light building and pavements, more than any other natural hazard, including earthquakes and floods (Nelson and Miller, 1992). In Australia, expansive soils are widely distributed. It was estimated that 20% of the surface soils of Australia could be classified as expansive (Richards et al., 1983).

In Queensland, expansive or reactive soils are referred to by soil scientists as “Cracking Clays” or, more commonly, as “Black Soils” (Dept. of Main Roads QLD, 2000). The distribution of these Cracking Clays by land area covers approximately one third of the state. Figure *2* illustrates the extent of these types of soils within Queensland, based on geological soil mapping.

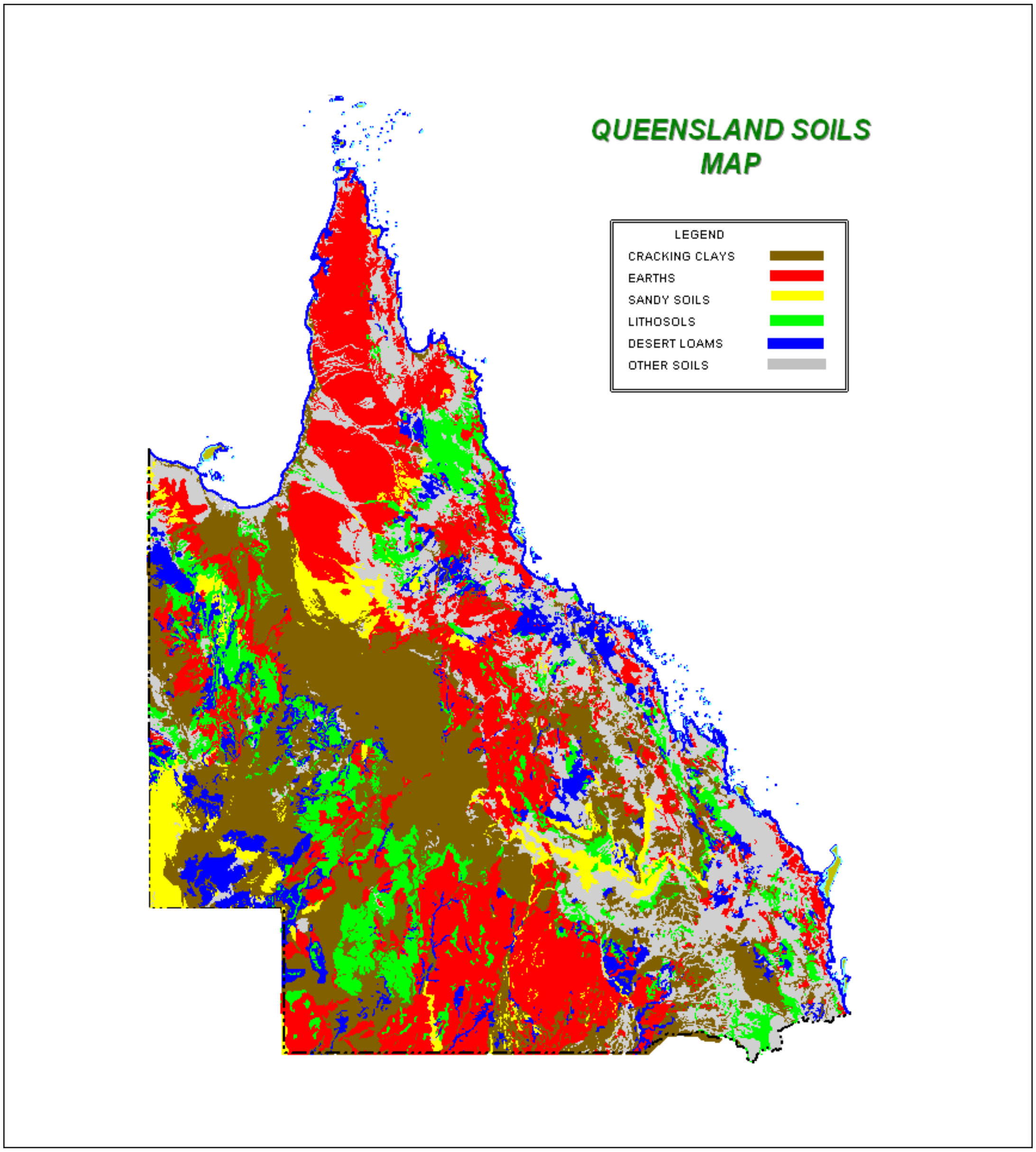

Figure 2 Distribution of Cracking Clays within Queensland (Queensland Department of Main Roads, 2000)

Six out of eight of Australia’s largest cities have clay foundation soils that consist of a higher proportion of expansive potential (Fityus et al., 2004). O’Malley and Cameron (n.d.) stressed that the western suburbs of Sydney and Brisbane, the western and northern suburbs of Melbourne, the foothills of Perth, almost the whole of suburban Adelaide and many regional centres of Australia are underlain by expansive soils in Australia.

Most of the light structures in Adelaide have cracks due to excessive foundation distortion created by expansive clays (Morgan and Kagawa, 1994). According to Harms (n.d.), 12% of the country is covered by Vertisols in Australia as shown in Figure *3*.

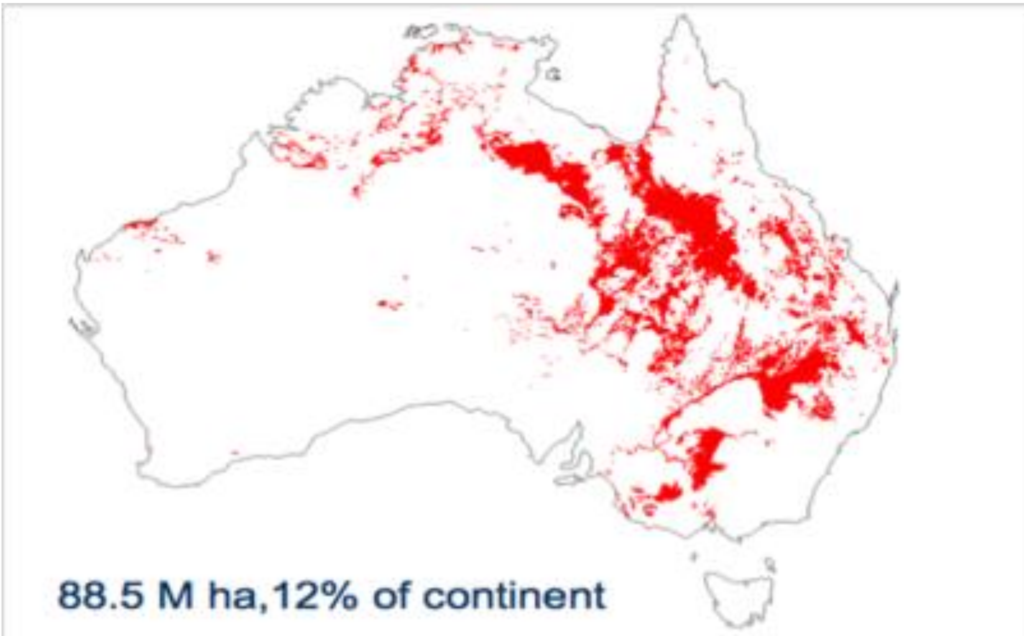

Figure 3. Distribution of Vertisols in Australia (Harms, n.d.)

2.2.2 Mechanism of Swelling

The extent of change in volume in expansive soils highly depends on its clay minerals. As the particle size distribution influence the engineering behaviour of coarse-grained soils, clay doesn't get influence by this distribution. However colloidal properties such as absorption of water due to large specific surface area of the soil particles influence the physical behaviour of the clay soils (Grim 1953). There are three important types of clay minerals, namely, Kaolinite, Illite and Montmorillonite as shown in figure 4 (Das, 2006).

As these clays are of plate like appearance, they have a high specific surface resulting in major impact on their properties (Craig, 2004). Most of the clay minerals consist of silicon-oxygen tetrahedron and aluminium-hydroxyl octahedral as their structural units from which, different types of clays are formed in different stacking sheets with various types of bond between these sheets (Craig, 2004).

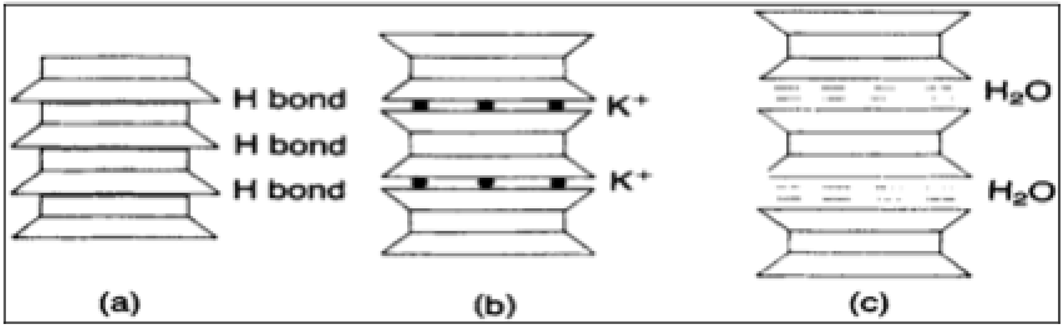

Figure 4. Clay Minerals: a) Kaolinite, b) Illite, c) Montmorillonite (Craig, 2004)

Kaolinite consists of the combination of a layer of a single sheet of silica tetrahedron with layer of a single sheet of alumina octahedron by hydrogen bonding. This strong hydrogen bonding minimizes the interlayer space between the sheets, which helps to reduce in expansion of clay mineral.

The Illite consists of an alumina octahedron sheet that is sandwiched between two silica sheets by potassium ions bonding. Das (2006) described that these potassium ions provide relatively weak bonding as it allows the higher amount of water cations to be absorbed. This causes increasing in expansion rate than that of Kaolinite but less than Montmorillonite. Montmorillonite clay has a structure similar to that of Illite except the presence of potassium ion bonding between the combined sheets (Das, 2006). Therefore, the large space between the combined layers attracts a large amount of water molecules and exchangeable cations. This resulting in weak bonding between the combined sheets which provides clay the freedom to swell in considerable amount due to the additional water being absorbed (Craig, 2004). Alternatively, the loss of moisture content during dry season causes substantial volumetric change.

2.2.3 Swell-Shrink Behaviour

The variation in moisture content causes the change in volume within soil, which is generally termed as swell-shrink potential. The volume of soil decreases or shrinks as it dries out and this causes desiccation cracks to appear due to internal stresses in the shrunken and dried soils mass. The soil increases its volume when it gets wet by swelling and this closed the open cracks resulting rises in the soil level.

The shrink-swell potential of expansive soils is assessed by water content, void ratio, internal structure, vertical stresses and the type and amount of clay minerals in the soil. The clay minerals that are present in the soils largely influenced the natural expansiveness of the soil, which includes smectite, montmorillonite and illite. Fine grained soils with higher amount of clay are prone to swelling as they can absorb large quantities of water during rainy season whereas these soils become very hard and dry during hot summer, resulting in shrinking and cracking the ground. This activity of hardening and softening of soil is referred as 'shrink-swell behaviour'.

2.3 Identification of Expansive Soils

The identification of expansive soils is essential before any design stages of infrastructures. According to Hamilton (1977) it is very important to identify the potential swelling or shrinking of subsoil problems for the selection of adequate foundation. It helps to determine the feasibility and selection of the construction site as well as subsequent performance of the structure.

Expansive soils are different from other soils and they can be distinguishable by their ability to swell from the imbibition of water with resulting volume change (Snethen et al., 1975). The knowledge of expansive soils will provide the indication of soil strata that possess the swell–shrink activity. Failure to identify these soils will result in extensive damage to structures.

According to Nelson and Miller (1992), there are various existing methods to identify the swelling potential of soils, which are listed as:

- Engineering Classification Test
- Mineralogical Methods
- Cation Exchange Method
- Free Swell
- Potential Volume Change (PVC)
- Expansion Index Test
- California Bearing Ratio (CBR)
- Coefficient of Linear Extensibility (COLE)

Engineering classification test, mineralogical methods and cation exchange methods are indirect methods, which involve the use of soil properties and classification schemes to estimate swell shrink properties whereas other remaining tests are direct methods to determine the actual swelling potential in the soil. These are briefly described in this section.

2.3.1 Engineering Classification Test

Index properties such as particle size distribution, clay content and plasticity are the most widely used for identifying and classifying expansive soils (Nelson and Miller, 1992). As the Atterberg limits define the consistency of fine-grained soils (clay) in four states depending on the water content, Plasticity Index is extensively used for classifying swelling potential which is given in Table 2 (Chen, 1988).

Table 2. 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

Snethen et al. (1977) discovered that liquid limit and plasticity index and in-situ soil suction are the best indicators of potential swell. The relationship between these properties is shown in Table *3*.

Table 3. Expansive soil classification based on Atterberg limits and in situ suction (Snethen et al., 1977)

LL (%)	PI (%)	In Situ suction (pF)	Swell	Swell Classification
>60	>35	>4	>1.5	High
50-60	25-35	1.5 -4	0.5 -1.5	Marginal
<50	<25	<1.5	<0.5	Low

New Zealand Standards ‘NZS 3604:1999 – Timber Framed Buildings’ also described expansive soils as those soils that have a liquid limit of more than 50 % and linear shrinkage of more than 15 %. The linear shrinkage is defined as the percent decrease in the length of a bar of soil dried in an oven from the liquid limit. If the linear shrinkage is high, it indicates a large potential shrinkage of the soil on drying which could pose the serious damage to the foundation.

The expansive soil characteristics are dependent on the amount of clay present in the soil. Nelson and Miller support this information saying the amount of colloidal particles (less than 0.001mm) directly influences the plasticity characteristics and volume change behaviour of soils. Skempton (1953) developed the relationship by combining Atterberg limits and clay content into a single parameter called the Activity. The activity is defined as follows:

$$Ac = \frac{\text{Plasticity Index}}{\text{Clay content in percentage}}$$

With the definition of activity, Skempton proposed three classes of clays according to the value of the activity, which is given in below table 4.

Table 4. Classes of Clay in terms of Activity (Skempton, 1953)

Value of Activity	Class of Clay
<0.75	Inactive
0.75≤1.25	Normal
>1.25	Active

Active clay acts as the most potential for expansion. Montmorillonite (Na) has the activity of 7.2 whereas Montmorillonite (Ca) has the activity of 1.5. Illite has the activity of 0.9 and Kaolinite has the activity value in between 0.33 to 0.46. But when the soils contain mixed mineralogy and Montmorillonite clay minerals, this classification did not seem to be précised according to Parker et al. (1977). Seed et al. (1962) developed a chart to determine the swelling potential based on percent clay sizes and activity as shown in figure 5. They also noted that the two soils with the same swell potential may exhibit very different amount of swell.

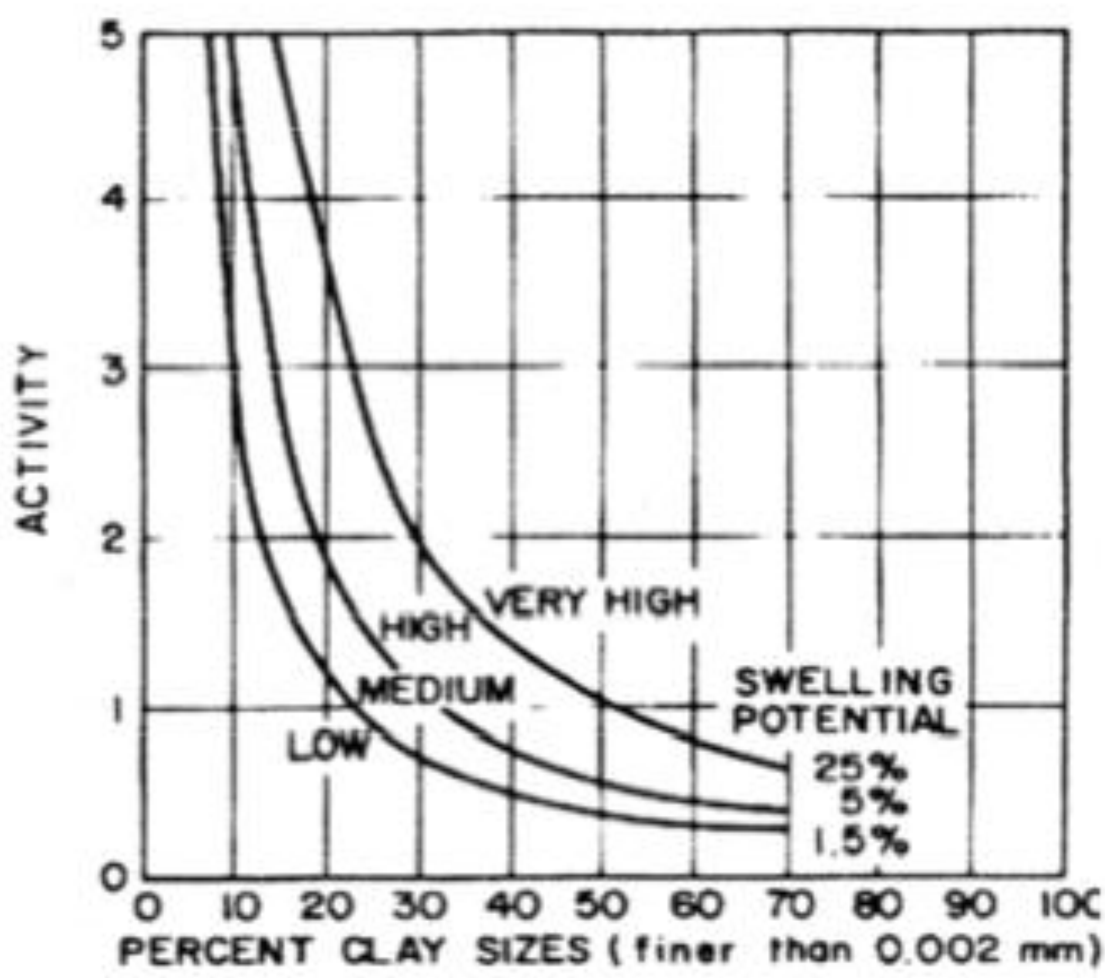

Figure 5. Swelling potential based on activity and clay (Seed et al, 1962)

2. 3.2 Mineralogical Methods

The expansive soil's swelling and shrinking behaviour greatly depends on the type and amount of clay minerals present in the soil (Ranjan and Rao, 2012). With variety of techniques available, X-ray diffraction methods are one of the most popular methods, which provides detailed information about the atomic structure of crystalline substances. As the different minerals with the various patterns of crystalline structures will diffract X-rays to yield different X-ray diffraction patterns, the types of minerals and proportion present in the soils can be known (Ranjan and Rao, 2012). Other methods to determine mineralogy in soils include differential thermal analysis (DTA) and electron microscopy (Nelson and Miller, 1992).

2.3.3 Cation Exchange Capacity (CEC)

Chen (1988) describes the CEC as the charge or electrical attraction for cation per unit mass measured in milliequivalents per 100 grams of soil. Excess salts present in the soil are removed first and adsorbed cations are replaced by saturating the soil exchange sites with a known species in this test procedure. Then the original cation complex composition is determined by the chemical analysis of the original extract (Nelson and Miller, 1992). CEC is related to the amount and type of clay present in a soil. As CEC increases, the swell potential increases because high CEC values are the indicator of a high surface activity. Specific ranges of CEC values of various clay minerals are shown in table 5.

Table 5. Cation exchange capacities of various minerals (Mitchell, 1976)

Clay Mineral	CEC (meq/100g)
Kaolinite	3-15
Illite	10-40
Montmorillonite	80-150

From table 4 above, it can be seen that montmorillonites are 10 times as active in absorbing cations as kaolinites due to the large net negative charge carried by the montmorillonite particle and its larger specific surface as compare of kaolinite and illite. Pearing (1963) and Holt (1969) developed a chart based on the mineralogy, activity ratio and CEC ratio, which are given in Figure *6*.

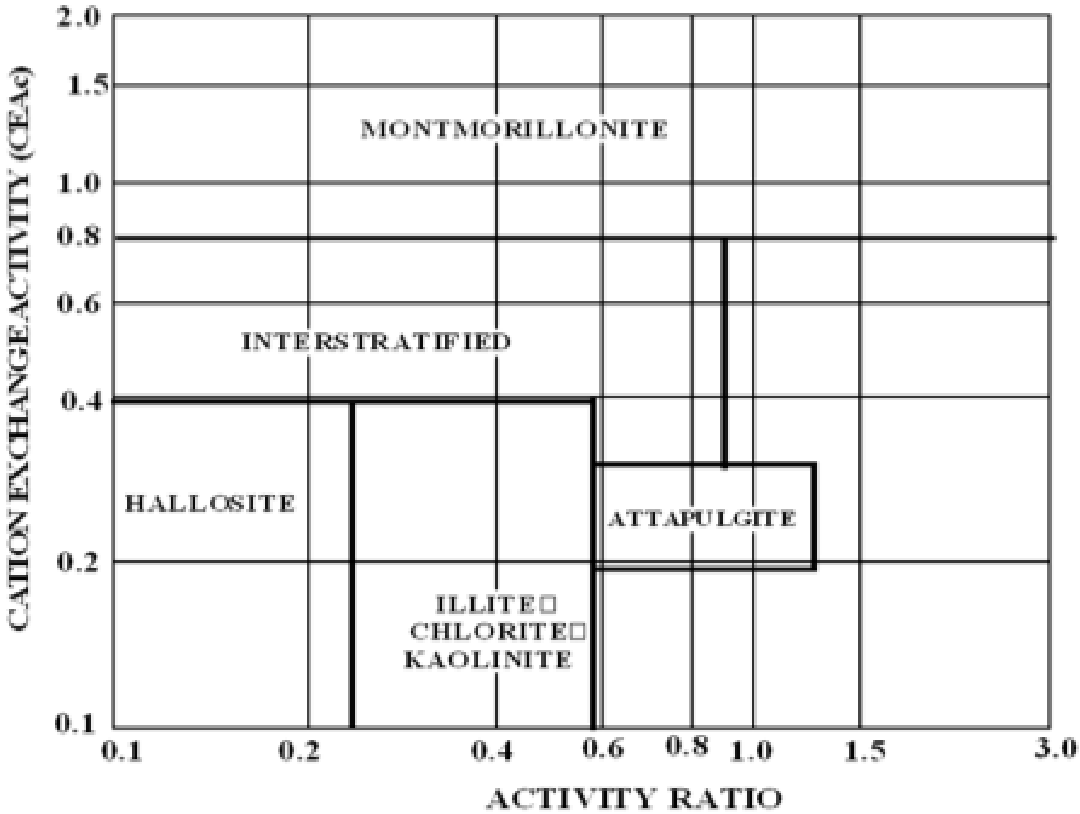

Figure 6. Mineralogical classification (Pearing, 1963; Hold, 1969)

The follow up research of the work done by Pearing (1963) and Holt (1969), produced the correlation between CEAc and activity ratio to indicate the swell potential soils, which is shown in Figure *7*.

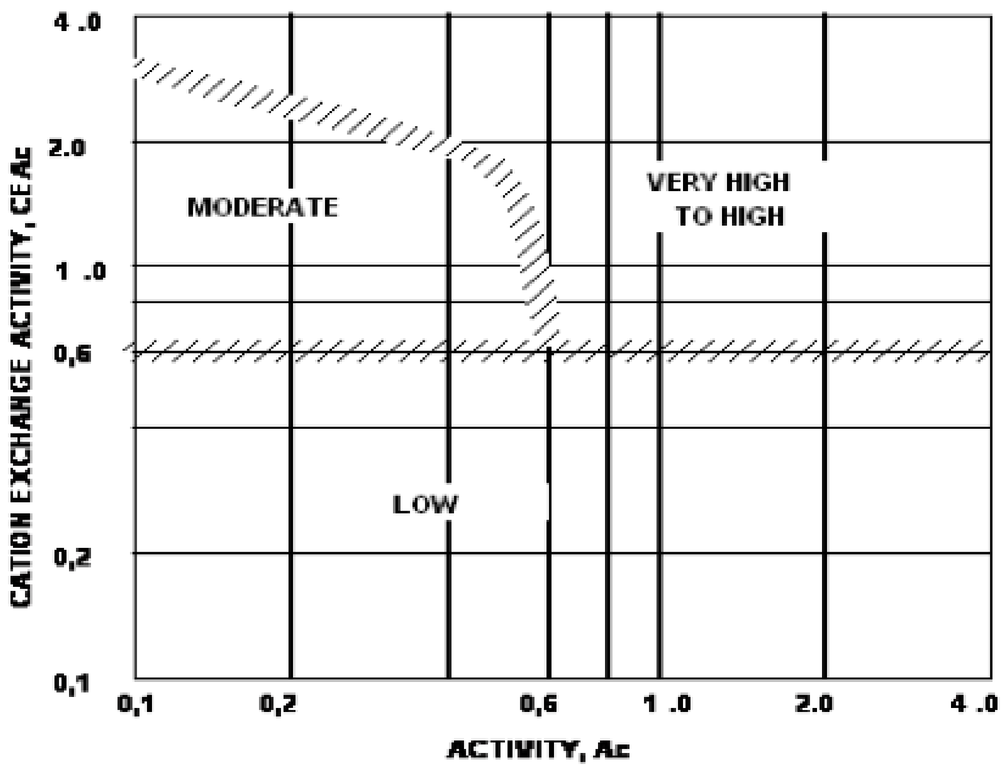

Figure 7. Expansive potential indication from CEAc and activity (Nelson & Miller, 1992)

2.3.4 Free Swell

In this test, the free swell is determined by pouring slowly 10 cm3 of oven dried soil (passing a 435 um sieve) into a 100 cm3 measuring jar filled with distilled water and recording the volume of the soil after it comes to rest at the bottom of the jar (Holtz and Gibbs, 1956). Then the increase in volume of the soil is written as a percentage of the initial volume, which is free swell. Nelson and Miller (1992) stated that the Montmorillonite (Na) of high grade consists of a free swell value in between 1200 to 2000%. The soils having free swell values greater than 100% are considered as expansive, whereas below 50% of free swell value of soil may not experience large volume of change. The drawback of this method is that it does not account for variation of density.

2.3.5 Potential Volume Change (PVC).

The Potential Volume Change (PVC) metre is used to measure the potential change in volume within clay. In this test, the remoulded soil sample is placed into the consolidometer ring and compacted at 2600 kJ/m3 at its natural moisture content. Water is added to the sample in the ring and allowed to swell against the ring.

Then the swell index is measured as the pressure on the ring, which is correlated to qualitative ranges of PVC. As the remoulded samples are used in this test, the results of PVC meter tests is only advantageous to estimate shrink-swell behaviour but cannot be used as design parameters for in place soils.

2.3.6 Expansion Index Test (EI)

The expansion index (EI) was developed in California and used to evaluate building sites (Nelson and Miller, 1992). According to American Society for Testing and Materials (ASTM) in accordance to the ASTM D4829 testing method, this method is used to determine the expansion potential of soils for practical engineering applications. In this test, the soil is passed through a No. 4 sieve (4.75mm) and bringing to achieve approximately optimum moisture content. Then the soil is compacted into a 10.2 cm diameter standard meld and a 6.9 kappa pressure is applied. Volume change is recorded for up to 24 hours.

The expansion index is calculated as follows:

Expansive Index = $\frac{\Delta H}{H}$ x 1000

Where ΔH = percent swell

Potential expansion of soils are classified by using EI is shown in Table *6*.

Table 6. Classification of Potential Expansion of soils using EI (ASTM D4829)

Expansion Index, EI	Potential Expansion
0-20	Very low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

2.3.7 California Bearing Ratio (CBR)

The California Division of Highways prior to the World War II originally developed the CBR test. Since then, it is widely used around the world to assess the subgrade strength of the soil (Chakroborty and Das, 2005). According to Queensland Department of Transport and Main Roads (QTMR) testing method Q113C that follow the principles of AS1289.6.1.1, this test can be either in a soaked or unsoaked condition and the duration of soaking can be 4 or 10 days. This test is performed to determine vertical swell of fine-grained soils before measuring the penetration resistance (Nelson and Miller, 1992). In this test, Soils are compacted into 152 mm internal diameter CBR test cylinders at different moistures and densities that soaked in water under a surcharge load for 4 days for 4 days soaked CBR test. Then swell percentage is measured with the help of dial gage during and after the 4 days soaked period. Then the sample is drained for 15 minutes prior to the penetration test.

Austroads (2004) classified the extent of expansive soils based on Atterberg limits and potential swell, which is shown in Table *7*.

Table 7. Expansive soil classification with Atterberg limits and potential swell (Austroads, 2004)

Expansive Nature	Liquid Limit (%)	Plasticity Index PI (%)	PI x%<0.425mm	Potential Swell (%)
Very High	> 70	> 45	> 3200	> 5
High	> 70	> 45	2200-3200	2.5 – 5
Moderate	50 – 70	25 – 45	1200-2200	0.5 – 2.5
Low	< 50	<2 5	< 1200	<0 .5

2.3.8 Coefficient of Linear Extensibility

The coefficient of linear extensibility (COLE) is performed to assess the potential expansion of soils, which is used routinely by the U.S Soil Conservation Service. This test determines the linear strain of an undisturbed and unconfined soil sample, which goes on for drying from 33 kPa suction to oven dry suction of 1000 MPa (Nelson and Miller, 1992). In this test procedure, undisturbed soil samples (clods) are coated with a flexible plastic resin that is impermeable to liquid water but permeable to water vapour. The clods are brought to a soil suction of 33 kPa in a pressure vessel, which are then weighted in air and water to determine their volumes using Archimedes principle. The clods are oven dried and measurement of volumes is taken again.

The change in volume of sample from the moist to oven dry state is COLE and is given by,

$$\mathrm{COLE} = \frac{\Delta \mathrm{L}}{\Delta \mathrm{LD}} = \left[\left(\frac{\gamma \mathrm{dD}}{\gamma \mathrm{dM}} \right)^{0.33} - 1 \right]$$

Where

$\frac{\Delta \mathrm{L}}{\Delta \mathrm{LD}} =$ Linear strain relative to dry dimensions

γdD = dry density of oven dry sample

γdM = dry density of sample at 33kPa suction

3 Materials and Methodology

3.1 Material Description

This chapter describes the materials and methodology selected for laboratory tests. Soils used are commercially available Kaolinite and uniform fine sand. As mentioned, the purpose of the study is to study of effect of addition of non-expansive soil (fine sand) to expansive soil, i.e. kaolinite. The tests conducted are liquid limit, plastic limit, plasticity index and expansive characteristics, swelling potential (vertical swell) and swelling pressure.

The above tests are performed on original clay sample and after mixing the sample with fine sand at various percentage by weight, namely 25%, 50% and 75%. The soil tests are performed following the Australian Standards 1289 for general soil properties and free swell technique for swelling indices. Table 8 shows a total of 4 samples with various proportions of sand.

Table 8. Soil Mixture (proportions) used in the current testing program

Sample	S1	S2	S3	S4
Clay (soil)	100%	75%	50%	25%
Sand	0%	25%	50%	75%

3.1.1 Kaolinite

Kaolinite, also known as China clay, is a common phyllosilicate mineral. Kaolinite is the most common clay mineral and entire clay deposits can be composed of this mineral. It has a soft consistency and earthy texture. It is easily broken and can be molded or shaped, especially when wet. The clay used in present study is commercial Kaolin (Eckalite 1) available from Pottery Works Queensland. It is off white in colour.

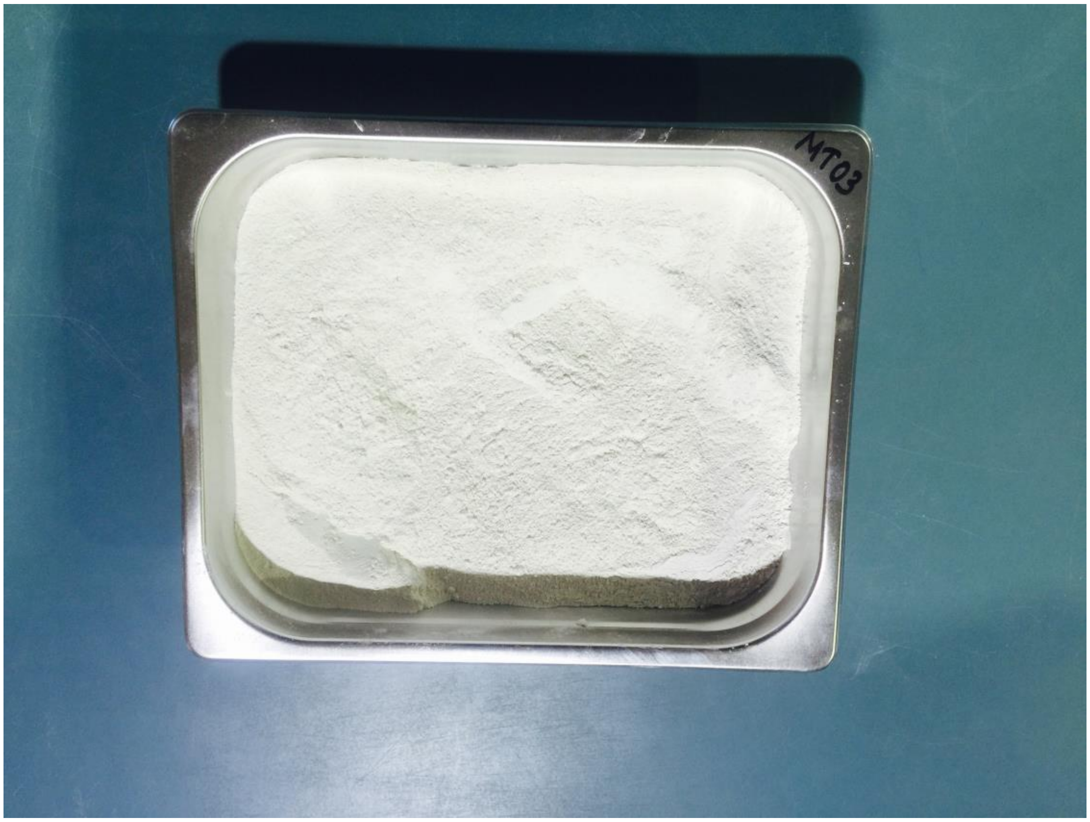

Figure 8. Kaolin (Eckalite 1) clay

3.1.2 Fine uniform sand

The non-expansive material used in the study is fine uniform washed river sand available from River Sand Pty. Ltd., Australia. This silica sand contains more than 99% quartz and has a specific gravity of 2.63. The particle size of the fine sand used in this study is shown in Table 9.

Table 9. Particle size distribution of fine sand

> 0.425mm	> 0.3mm	>0.150mm	>.075mm	<0.075mm
4%	63%	32%	0.8%	0.2%

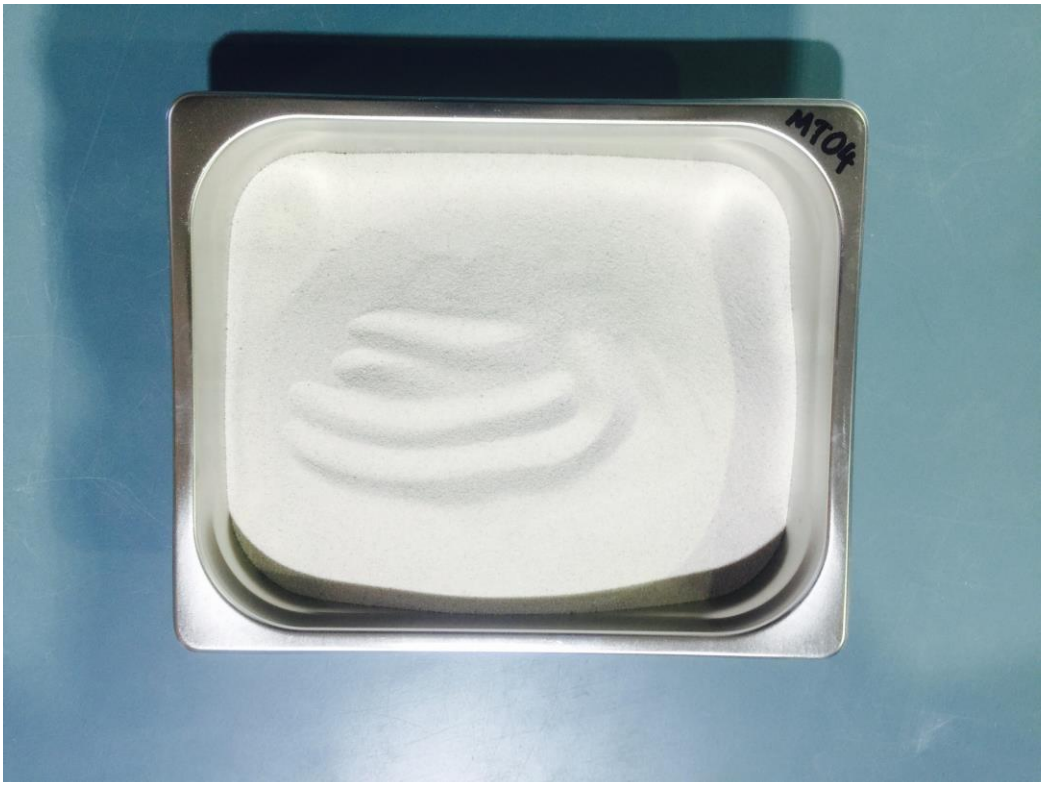

Figure 9. Fine uniform sand

3.2 Testing Program

Laboratory tests were designed to investigate the basic properties (liquid limit and plastic limit) and swelling characteristics (swelling potential and swelling pressure) of the soil sample that is in dry powder form. The sample was then mixed with fine sand material at different percentage by weight.

3.2.1 Liquid limit

The liquid limit is percentage moisture content that defines where the soil changes from the plastic to a fluid state. Liquid limit test was conducted using a cone penetrometer following Austrian Standard 1289.3.9.1. Liquid limit is the water content in percentage when penetration of the cone is 20mm. Dry powder clay and tap water was used. The dimension of the specimen cup used is 55mm in diameter and 45mm in height. A homogeneous paste is prepared of the sample by adding water and cone penetrometer test is performed with the range of penetration depth taken from 15 to 25 mm.

A minimum of 5 tests was performed with different water contents with at least two values below and two values above 20mm penetration. Moisture content of the soil is determined following AS 1289.2.1.1. Water content versus penetration depth graph is plotted and water content corresponding to 20mm penetration is noted, which is the liquid limit of the soil sample.

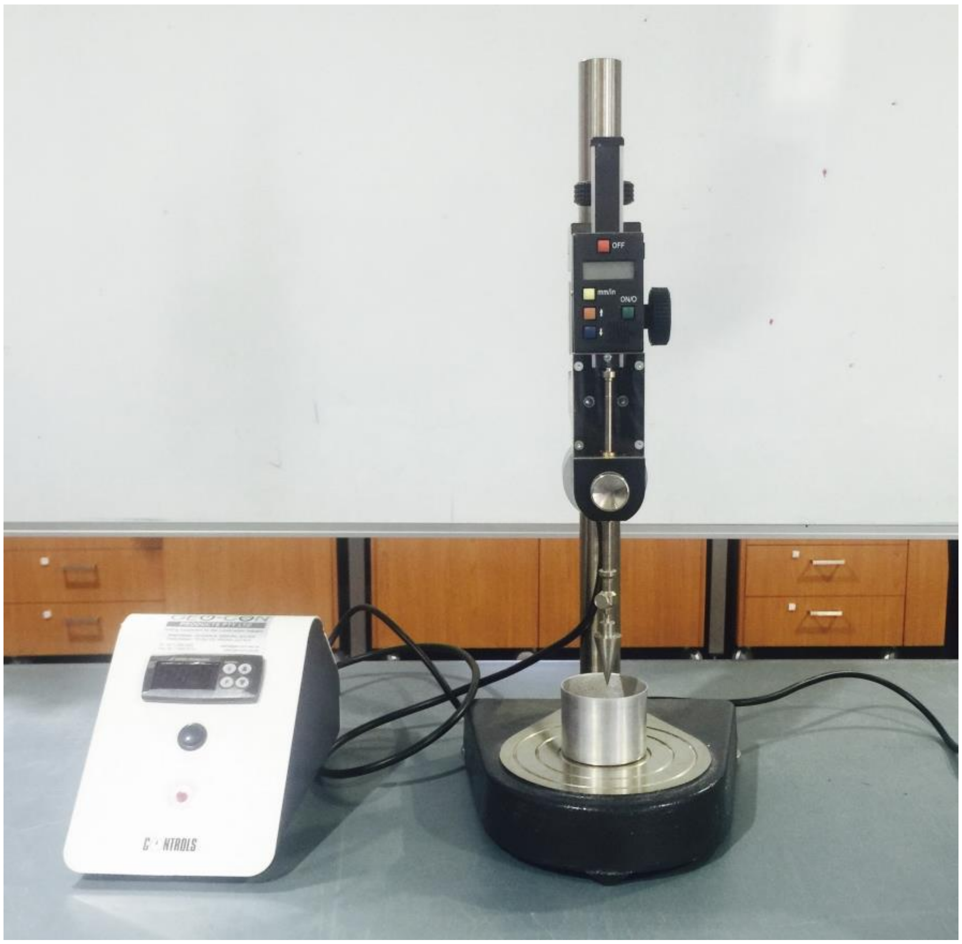

Figure 10. Cone penetrometer for determining liquid limit

3.2.2 Plastic limit

Plastic limit is the percentage moisture content that defines where the soil changes from semi-solid to a plastic (flexible) stage. Plastic limit test was conducted following AS 1289.3.3.1. As plasticity index is also to be calculated from this test result, the soil sample (paste) used for determination of liquid limit was used for the plastic limit test.

About 8 g of soil is rolled between the hand and a glass plate using sufficient pressure to form the soil into a 3mm diameter thread. If the soil thread crumbles before reaching 3mm diameter, more water is added but if the soil thread rolls down to 3mm diameter without crumbing, it is kneaded and re-rolled again. Crumbed threads of between 5 g to 20 g are collected and moisture content determined in accordance with AS 1289.2.1.1. Two tests are performed with moisture contents difference no more than 2% between them and average of two moisture contents give the resultant plastic limit of the soil.

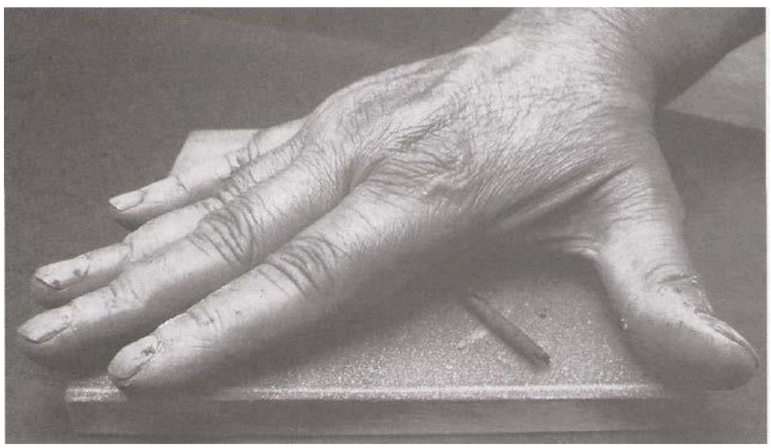

Figure 11. Rolling of soil mass on ground glass to determine plastic limit (Das, 2008)

3.2.3 Plasticity Index

The plasticity index is a measure of the plasticity of the soil. The plasticity index is the difference between the liquid limit and the plastic limit.

Plasticity Index (PI) = Liquid Limit (LL) – Plastic Limit (PL)

The most popular approach for determining the swelling potential of a soil is by the use Atterberg limits. Chen (1988) presented a single index method of predicting swelling potential solely based on plasticity index (Table *2*).

3.2.4 Swelling potential and swelling pressure

Swelling potential and swelling pressure are two important characteristics in determining the behaviour of expansive soils. The lack of a standard definition of swelling potential is the most confusing aspect of expansive soil classification (Nelson & Miller, 1992). In general, swelling potential of a soil is a measure of its ability to swell. Hold and Gibbs (1956) defined the swelling potential of a soil as the total volume change of when saturated under 6.9 kPa load.

The swelling pressure is the pressure required to hold the soil or restore the soil to its initial void ratio when given access to water. Cimen et al. (2012) defined swelling pressure as the pressure required to compress the fully swollen soil sample back to its initial volume in free swell test.

A number of factors influence the swelling characteristics of expansive soils. Some of these are gradation of the soil particles, texture, structure, density, applied loadings, load history, mineralogical composition, temperature, etc.

In this study swelling potential and swelling pressure of expansive soil as well as the soil mixtures were determined using one dimensional consolidation apparatus (oedometer) following free swelling odometer test method. At least two replicated lab tests were performed for each sand-clay mixture and average value from the experiments were taken. The classification of swelling potential for various sand-clay mixtures was made following expansive index method according to ASTM D4829 (Table *6*).

Figure 12. One dimensional oedoemeter for determining swell potential and swelling pressure

In an oedometer test method, the swelling/consolidation of soil is investigated by restricting lateral and axial deformations under oedometric conditions.

The soil was oven dried for more than 24 hours. The dry soil was thoroughly mixed with a calculated amount of water necessary to reach 22% initial moisture content. It was then further kneaded to form a homogeneous mixture. The soils were tested in fixed ring oedometer using stainless steel ring which had an inside diameter of 50.47 and a height of 20mm.

The specimen was moderately compacted in the oedometer ring in 3 layers using a small hammer weighing 344.3 gm. The excess soil was trimmed using a palette knife. Filter papers were placed on top and bottom of the soil specimen to prevent the tiny particles entering into the pores of the porous stones placed on both sides of the specimen. As the room was not temperature controlled, the space around the specimen ring was enclosed with a loose-fitting plastic wrap to minimise change in specimen water content. After positioning the specimen in the oedometer, the sample in the oedometer was compacted applying static force of 1000 kPa for 24 hours. Then compaction force was gradually reduced and the soil sample was subjected to a seating pressure of 7 kPa for 24 hours and natural rebound is measured as the height difference of the specimen under 1000 kPa and 7 kPa pressure. Thereafter the soil specimen was inundated with water and allowed to swell freely under a nominal pressure of 7 kPa for further 24 hours. The percent change in height of the specimen before and after adding water under 7 kPa is the free swelling potential value. In the study, as the water is added only after the soil sample has been kept under a seating load of 7 kPa for 24 hours after the removal of compaction force, natural rebound (swell) as a result of unloading cycle is also measured. This rebound swelling would be helpful in determining the required preload and the natural swell after the removal of preload which is used to reduce or eliminate the secondary settlement that occurs when actual construction takes place.

After the soil sample has reached a maximum swelling with addition of water under 7 kPa, it is loaded applying incremental loads starting at 25 kPa until specimen reaches its initial height. The swelling pressure can be taken as the pressure that brings the specimen back to its original height before adding the water under 7kPa. The total pressure required to bring back the specimen to the height after consolidation under 1000 kPa load was also determined.

4 Results and discussion

The results obtained from the laboratory experiments performed are presented and discussed in this section. Firstly the results of index properties are discussed followed by expansive properties and thereafter relationship between Atterberg limits and swelling characteristics.

4.1 Liquid limit, Plasticity Limit and Plasticity Index Tests

This section examines the variation of Atterberg limits with different percentage of sand. Liquid limit, plastic limit and plasticity index, which are very important elements in determining the expansive properties of a soil, are determined. These tests were performed to analyse how the plasticity characteristics of clay behaves with the addition of coarse material and to try and relate these plasticity characteristics with expansive properties of the soil specimen. In this study, liquid limit was determined using a cone penetrometer test. The test results for kaolinite clay mixed with various percentages of fine sand is presented in **Error! Reference source not found.**.

Table 10. Liquid Limit, Plastic Limit & Plasticity index of Kaolinite clay mixed with fine sand

Sample	**S1**	**S2**	**S3**	**S4**
Clay (soil)	100%	75%	50%	25%
Sand	0%	25%	50%	75%
Liquid limit (%)	75.7	55.5	38.8	23.7
Plastic limit (%)	30.4	24.6	16.51	10.9
Plasticity index	45.4	31.2	21.99	12.8

The original kaolinite soil specimen sample had a liquid limit of 75.8%. With the addition of sand, there was significant decrease in liquid limit value of the specimen. Liquid limit value reduced from 75.8% to 38.57% with addition of 50% sand. In other words, liquid limit reduced by 50% when sand content is increased by 50%. This result agrees with White (1949) who noted that liquid limit of kaolinite increased with a decrease in particle size which is consistent with the result in this study, i.e. decrease in liquid limit with increase in particle size.

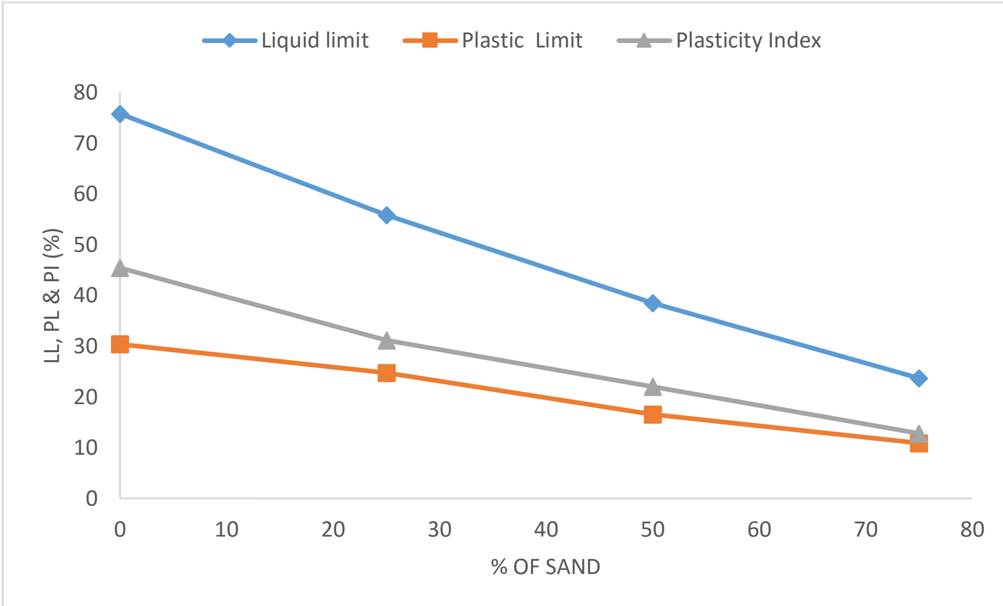

Figure 13. Variation of Liquid limit, plastic limit and plasticity index with % of sand

Table 13 shows that plastic limit of original specimen was 30.4% which reduced to 16.51% when sand content is increased by 50% sand. Vembu and Vipulanandan (2011) and KC (2014) also observed similar trend in results of liquid limit and plastic limit values of kaolinite mixed with sand. In the study performed by KC (2014), the liquid limit and plastic limit values of kaolinite were 75.84% and 28.49% respectively and with addition of 50% fine sand, the liquid limit and plastic limit values reduced to 32.05% and 15.42% respectively.

Similar results were observed in the value of plasticity index, which is a numerical difference between liquid limit and plastic limit values. Sridharan & Gurtug (2004) noted that the percent swelling is larger for soils having higher liquid limit and plasticity index. From table 13 we can observe that the plastic index of original clay reduced from 45.4% to 21.99% when sand content is increased by 50%. The results show that addition of sand reduces the plasticity characteristics of the expansive soil. The significant reduction in liquid limit, plastic limit and plasticity index values with addition of sand may be attributed to the non-cohesive nature of the sand. The pure clay exhibits high Atterberg limits due to the high kaolinite content which increased the intake of water molecules by the clay, facilitated by the negatively charged clay surfaces and the large specific surface area of the clay mineral.

Al-Shayea (2001) stated that addition of sand to clay reduces liquid limit and plastic limit because the sand particles act as an inert filler and do not interact electrochemically with water.

Figure 13 shows that as the amount of sand is increased in the mixture, PL, LL and PI decrease almost linearly with respect to the sand content. These results are similar to those obtained by Skempton (1953) for four clayey soils, Seed et al. (1964) for kaolinite and bentonite mixtures, by Nagaraj et al. (1991) and Han (1998) for bentonite-sand mixtures.

4.2 Swelling potential tests

Swelling potential is an indicator of magnitude of the swelling. It is defined as the equilibrium vertical volume change obtained from oedometer test, expressed as a percent of the original height. In this study, the vertical swelling percentage was calculated in accordance with the above definition by inundating the consolidated soil specimen under a seating pressure of 7 kPa. Many researchers have used the vertical swelling value to identify expansive soils and accordingly they have classified the soils as having low, medium, high and very high degrees of swelling potential. In this study, expansive index method according to ASTM4829 was followed to classify the swelling potential of the soil specimen (Table 6). Expansive index is the ratio of 1000 times the difference between final height and initial height divided by initial height and can be calculated as follows:

$$\text{Expansive Index} = \frac{\Delta H}{H} \times 1000$$

Where, ΔH = change in height and H = original height of specimen

It may be noted that expansive index is 10 times the vertical swell percent of the soil specimen.

Table 11. Expansive Index and swelling potential classification of clay-sand mixtures

% of Sand	0%	25%	50%	75%
Expansive Index	213	149	33.5	6.3
Swelling potential	Very High	Very high	Low	Very Low

Table 11 indicates that the original soil sample has an expansive index of 213 which is classified as very high swelling potential. As expected, the addition of coarse material to the original clay reduces the expansive capacity of the soil sample. With the addition of 25% sand, the expansive index reduces to 149 which is a reduction of 30%. However the clay-sand mixture still classified into very high swelling potential category. The addition of 50% sand reduces the expansive index to 33.5 and hence classifies into the low swelling potential category.

Figure 15 shows the results of swelling tests in the form of vertical swell percent as a function of elapsed time (minutes). The curve for soil with sand content of 0% and 25% sand can be divided into three stages, initial, primary and secondary swelling whereas curve for soil with sand content of 50% and 75% is almost linear. The swelling rate is low in the initial stage due to the low permeability of the soil sample which reduces the rate of flow of water. The rate of swelling is very high in primary stage followed by a low rate in the secondary stage. This can be attributed to high water adsorptive forces during the primary stage (Adbduljauwad, 1993). Primary swelling occurs between 1 to 100 minutes where the time for secondary swelling varies between 100 to 1440 minutes.

The swelling percent for soil with sand content of 50% and 75% is very small compared to the soil containing lower percentage of sand. This is due to the fact that the swollen clay particles just occupy the voids between the sand grains causing a relatively non-significant amount of swelling.

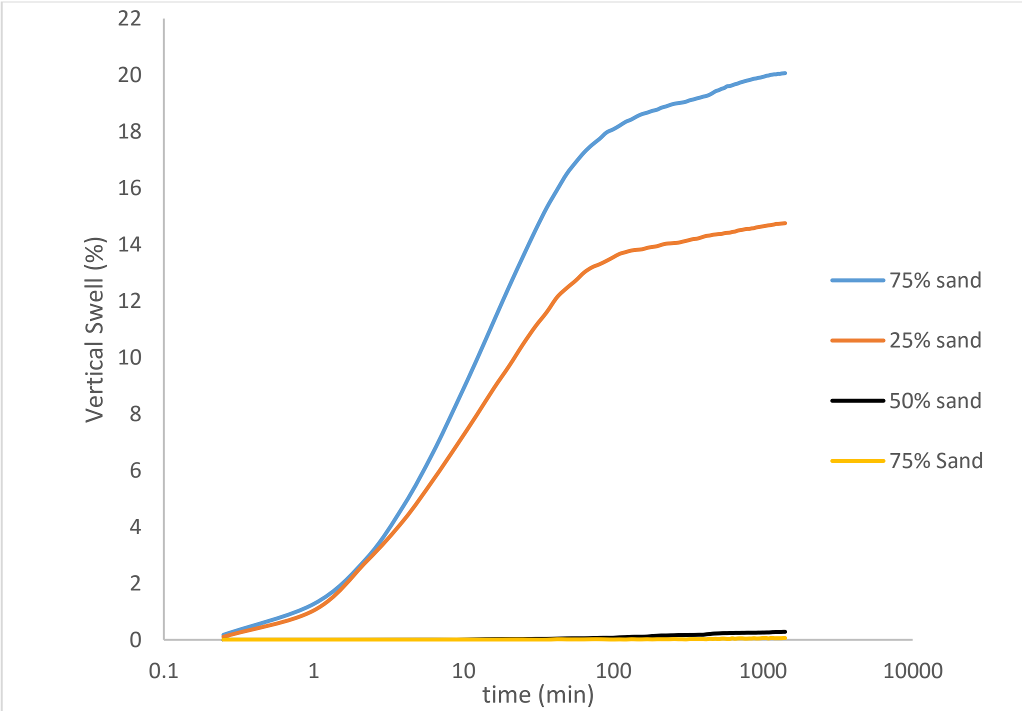

Figure 144. Time-swell curve

In addition to the vertical swell after addition of water, natural rebound swell 24 hours after unloading of compaction force of 1000 kPa to 7 kPa was also measured. Table 12 shows the vertical swell (displacement) before and after adding water in various clay-sand mixtures.

Table 12. Swelling of clay-sand mixture before and after addition of water

		% of Sand			
		0%	25%	50%	75%
Natural Rebound (mm)	Before adding water	0.403	0.57	1.2	0.436
Swelling (mm)	After adding water	3.279	2.127	0.073	0.012
Total swell (mm)		3.682	2.697	1.273	0.448

In a real life practical setting, this rebound swelling would be helpful in determining the natural swell after removal of preload which is used to reduce or eliminate the secondary settlement that occurs when actual construction takes place.

Table 12 indicates that total change in height of specimen after addition of water decreased significantly with the addition of fine sand. However, the natural rebound after consolidation increased with the increment of sand percentage till 50% sand and then decreased slightly with addition of 75% sand. This could be attributed to the variation in compacted density of the soil specimens. The original kaolinite clay after consolidation had a natural expansion of 0.403mm in 24 hours and a further swelling of 3.279mm after addition of water giving a total swelling of 3.682mm in 48 hours. With the addition of 50% sand, the natural rebound increased to 1.2mm but with the addition of water further expansion was only 0.073mm which gives a total swelling of 1.273. It is noted that the original clay has significantly less rebound compared to expansiveness after adding water whereas with the addition of higher percentage of sand, the vertical swell is relatively nonsignificant.

4.3 Swelling Pressure tests

As defined earlier, swelling pressure is the pressure required to compress the fully swollen sample back to its initial volume in a free swell test. Figure 15 shows that increase in the percentage of fine sand mixed from 0% to 75% reduced the swelling pressure of the soil specimen 366 kPa to 9 kPa.

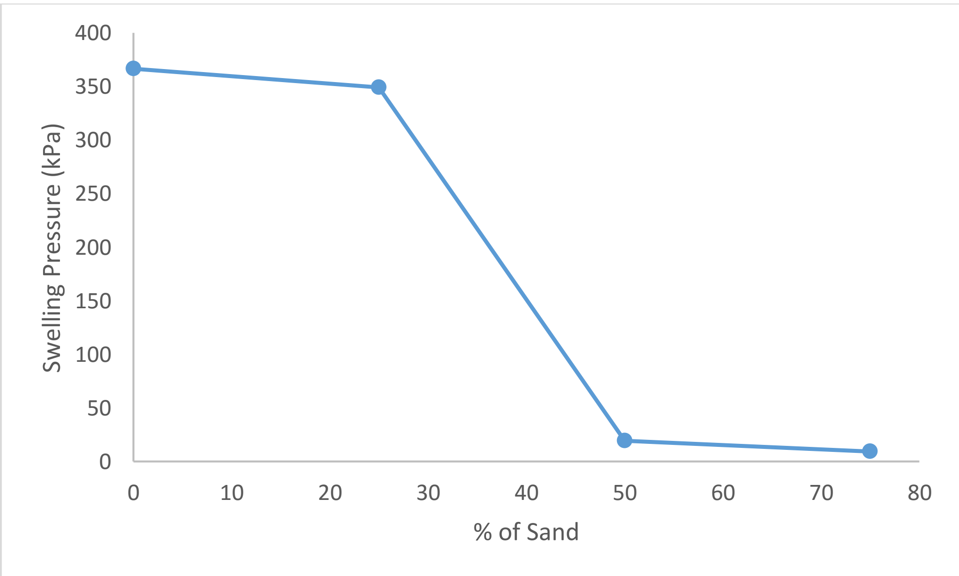

Figure 15. Variation of swelling pressure with % of sand

It may be noted that the rate of reduction of swelling pressure is very high between sand percentage of 25% and 50%. This rate of reduction is insignificant with more than 50% sand content. The swelling pressure reduces at a similar ratio of between 4 to 6% between 0 and 25% and 50% to 75% of sand whereas the rate of reduction between 25 to 50% is more than 90%. Therefore, further laboratory investigations need to be carried out with more variations in sand content between 25 and 50%. This result agrees with ElKholy (2008) and Abdelrahman, et al. (2004) who noted that an increase in the percentage of coarse fraction of sand mixed with clay reduced with swelling pressure.

In this study, as the soil specimen was unloaded from 1000 kPa to a seating load of 7 kPa and left for 24 hours before adding the water. Also the total pressure required to bring the specimen height back to the initial height after full static compaction under 1000 kPa was determined, which of course would be higher than the swelling pressure. It may be noted that while the swelling pressure reduced with addition of fine sand, the total pressure increased with the addition of the same. While it took 563 kPa pressure to bring the original kaolinite clay to the initial height under 1000 kPa load, the required pressure increased to 704 kPa for the soil specimen with 50% sand content. The rate of increment in the total pressure decreases with increase in the sand content.

Table 13. Swelling pressure and total pressure of clay-sand mixtures

% of Sand	Swelling Pressure (kPa)	Total Pressure (kPa)
0.00	366.51	563.38
25.00	349.11	649.47
50.00	19.44	703.98
75.00	9.29	732.68

No previous literature could be found to compare these studies regarding total swelling pressure. It may be noted that swelling characteristics of soil is more dependent on dry unit weight than initial water content but in this study the compacted dry density of tested soil specimens was not constant but varied between 1.4 to 1.9 g/cm^3. Therefore, the result of total pressure is inconclusive and further lab investigations need to be carried out and correlated with the current test results.

4.4 Correlation between Atterberg limits and expansiveness of soil

From the results discussed above, we know that both Atterberg limits and expansiveness of clay reduce with increase in sand content. Figure 16 shows the variation in swelling potential and swelling pressure with respect to plasticity index of clay-sand mixtures. All the three parameters, plasticity index, swelling potential and swelling pressure values are highest for original kaolin clay with 0% sand, which gradually reduces with increment in sand content. The pure clay sample had a plasticity index of around 45%, swelling potential of 21% and swelling pressure of around 360 kPa. With the addition of 50% sand, the plasticity index, swelling potential and swelling pressure reduced to 31%, 0.3% and 19 kPa respectively. It may be noted that the swelling potential and pressure are higher for soils having higher plasticity index. As there is an almost linear relationship between plasticity index, liquid limit and plastic limit, the above expansive characteristics will vary similarly with respect to liquid limit ad plastic limit. This result agrees with Cimen et al. (2012) who stated that plasticity characteristics and volume change behavior of soils are related to the amount of clay particles in the soil and that swelling properties of clay minerals are directly proportional to their plasticity properties.

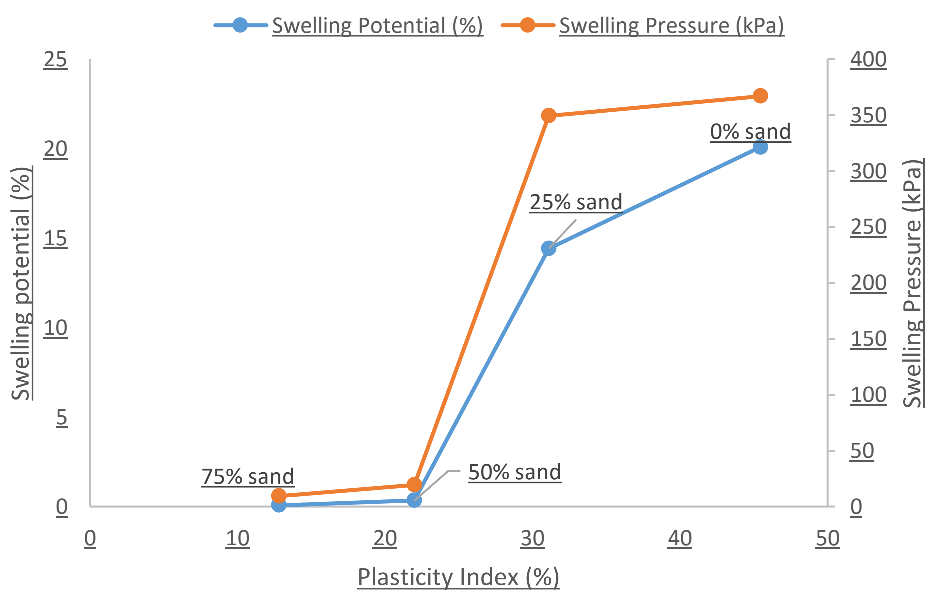

Figure 16. Variation in swelling potential and swelling pressure with respect to plasticity index

As stated earlier, Atterberg limit is the most common method for determining the swelling capacity of a soil. Chen (1988) derived a criteria for identifying swelling potential solely based on plasticity index of a soil, which is presented in Table *2*. From the table, it can be noted that soil having a plasticity index of greater than 35 is considered to have very high swelling potential. Therefore, the original clay used in this study which has a plasticity index of 45.4 is categorised as having a very high swelling potential. With the addition of sand, the plasticity index value decreases and so does the ability of the soil to expand. The mixture of clay and fine sand (50% of each) has a plasticity index of 21.99 which is categorised on Table 2 as having a medium swelling potential. It may be noted that swelling potential reduces from very high to high with addition of 25% sand, to medium with 50% sand and to low with 75% sand.

Dakshanamurthy and Raman (1973) used modified plasticity chart to determine swelling potential of the soil. The plasticity chart is a plot of plasticity index against liquid limit. It has two basic lines as follows

1. LL = 50 line. This line is used to divide silts and clays into high plasticity (LL > 50) and low to medium plasticity (LL < 50) categories.
2. A-line, defined as PI = 0.73 (LL -20). This line is used to separate clays, which plot above the A-line, from silts which plot below the A-line.

An additional U-line is defined as PI = 0.9 (LL -8) represents the uppermost boundary of the test found thus far for natural soils. The U-line is a good check on erroneous data and any test results that plot above this line should be checked.

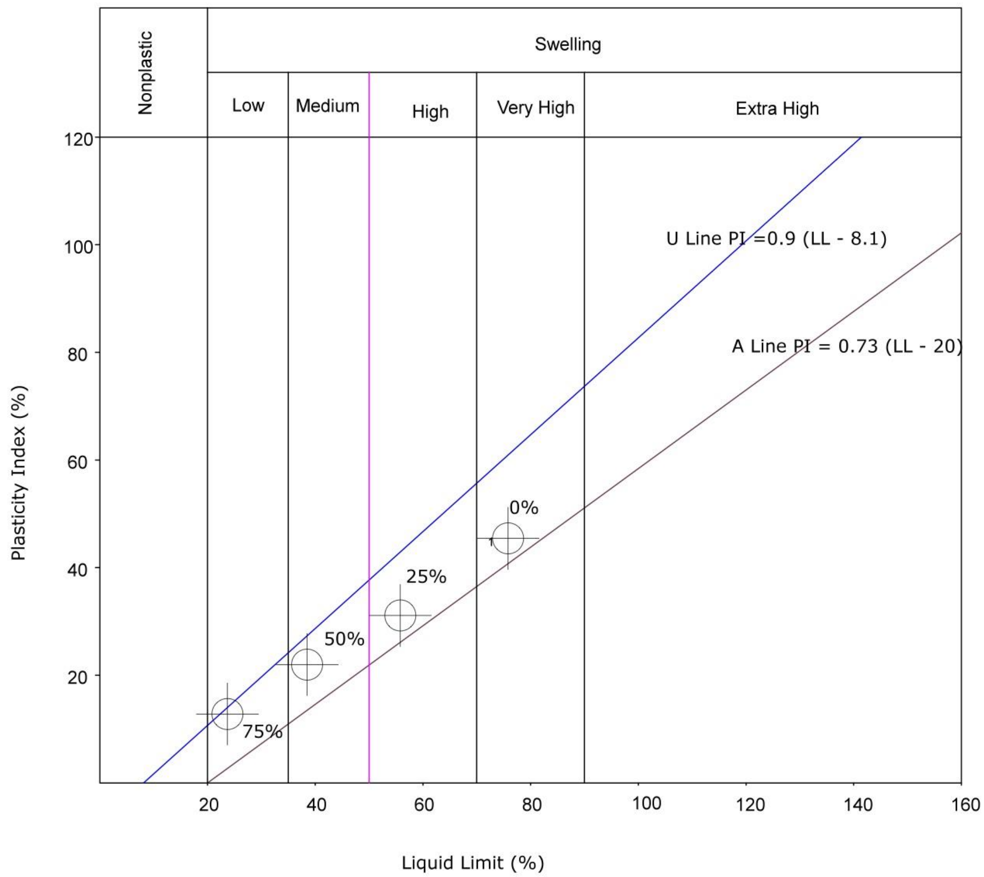

Figure 17. Modified Plasticity Chart (Cimen et al., 2012)

Figure 16 shows the plasticity index and liquid limit presented on the modified plasticity chart. All the plasticity index values plotted against liquid limit values lie within A-Line and U-line. The original clay and 25% sand mixed clay have high plasticity whereas clay with mixed 50% and 75% sand have low plasticity. We can observe from Figure 15 that original Kaolinite clay has very high swelling potential. The swelling potentially gradually decreased with the increment of sand to the original clay. With the addition of 25, 50 and 75% sand to the original clay, the swelling potential of soil reduced from very high to high, medium and low respectively.

The results for swelling potential based on Atterberg Limits using modified plasticity chart (Cimen, et al., 2012) and Chen's (1988) method is very similar with both methods identifying the original clay used in the study as having very high swelling potential and with increment of sand to 50%, the clay-sand mixture had a medium swelling potential.

It may be noted that as the plasticity of the soil increases from non-plastic to very high plastic the swelling of soil also increase in the same range of plasticity. This verifies that swelling properties of clay minerals follow the same trend as their plasticity properties.

5. Conclusion

The engineering properties of clay-sand mixtures are highly influenced by the sand content. The Atterberg limits (liquid limit, plastic limit and plasticity index) are found to reduce with increase in sand content. This variation is found to be almost linear with respect to the sand content which could be attributed to non-cohesive nature of the sand.

The free swell percentage and swelling pressure clay-sand mixtures also reduce with the increase in sand content. However, results indicate a significant change in both swell pressure and swell percentage when the sand content is increased from 25% to 50%. Further increase in sand content above 50% does not indicate significant changes in expansive properties. In other terms, addition of 50% of coarse content should have treated the expansive nature of the kaolinite clay.

This rate of reduction of the swelling and swelling pressure is very high between 25% and 50% sand content but with further addition after 50%, the rate of change of the above swelling characteristics is nonsignificant.

It may be noted that as the plasticity of the soil increases from non-plastic to very high plastic the swelling of soil also increase in the same range of plasticity. This verifies that swelling properties of clay minerals follow the same trend as their plasticity properties.

While the free swell after addition of water decreased with increase in sand content, the rebound swelling before adding water increased with increase in sand content up to 50% but then slightly reduced at 75% sand. Also the total pressure required to bring back the soil specimen to its original height after being statically compacted under 1000 kPa is greater for clay-sand mixture containing higher percentage of sand.

Therefore it can be concluded that addition of sand improves the overall characteristics of soil by reducing its plasticity, swelling potential and swelling pressure but the results obtained for natural rebound and total pressure is inconclusive due to the unavailability of previous work for comparison. Therefore, further lab testing are needed to refine and improvement the findings.

6. Recommendations for further work

The main problem faced in this study was related to the method of compaction to reach the desired dry density of the soil specimen. Due to the unavailability of suitable equipment and apparatus for standard compaction method, the soil specimen was statically compacted in the oedometer itself which made it impossible to get constant dry density for all soil specimens after compaction. Also each test took at least a week to complete and there were many failed tests, therefore the lab investigations could be done with only 3 variation in sand contents. A large number of data points is required to obtain more accurate and generalised relationships to predict the swelling behaviour of soil. Therefore further lab investigations could be performed:

- Using different method of compaction
- Using different static load for compaction to see how the swelling properties change with the change in degree of consolidation.
- With more variation in sand content especially between 25% and 50%.

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Appendix A – Project Specification

ENG4111/4112 Research Project

Project Specification

For: Suman Shrestha

Title: Study of effects of coarse grain contents on Atterberg limits and expansiveness of the clay

Major: Civil Engineering

Supervisor: Dr Buddhi Wahalathantri

Enrolment: ENG4111 – EXT S1, 2016
ENG4112 – EXT S2, 2016

Project Aim: To see how the expansive properties to soil change when coarse grain contents (various percentages by weight) are added to it.

Programme: Issue A, 24th March 2016

1. Write an initial literature review on expansive soils.

2. Perform Atterberg's limits (Liquid limit, Plastic limit & plasticity index) of the soil sample and then perform oedometer test to find its expansive characteristics (swell potential, swell pressure & swelling index).

3. Add coarse content (probably fine sand because of the size of the equipment) by 20, 50 & 80 percentages by weight and repeat the above step 3 for the each of the mixtures.

4. Compare the obtained results with the available literature. Seek a correlation between Atterberg limits and expansive characteristics of samples.

5. Conclude whether the added contents have a positive or negative affect on the expansive properties of soil and the ratios of each mixture.

Appendix B – Tabulated test results for Liquid limit & Plastic limit

Table B1. Liquid limit test result of clay-sand mixture (0% sand)

Tin number	1	2	3	4	5
Mass of tin (Ma)(gm)	36.60	37.00	37.30	37.00	36.70
Mass of tin + wet soil (Mb)(gm)	51.60	64.90	61.20	58.70	62.90
Mass of tin + dry soil (Mc)(gm)	45.60	53.20	50.90	49.20	51.10
Mass of water (Mw = Mb-Mc)(gm)	6.00	11.70	10.30	9.50	11.80
Mass of dry soil (Ms = Mc-Ma)(gm)	9.00	16.20	13.60	12.20	14.40
Moisture content [(Mw/Ms)*100] (%)	66.67	72.22	75.74	77.87	81.94
Penetration(mm)	14.98	18.52	20.22	21.23	23.01
Liquid Limit (LL) (%)			**75.7**		

Table B2. Liquid limit test result of clay-sand mixture (25% sand)

Tin number	1	2	3	4	5
Mass of tin (Ma)(gm)	37.00	37.40	36.80	37.00	37.00
Mass of tin + wet soil (Mb)(gm)	61.00	60.20	59.50	58.10	59.40
Mass of tin + dry soil (Mc)(gm)	53.00	52.30	51.40	50.40	51.00
Mass of water (Mw = Mb-Mc)(gm)	8.00	7.90	8.10	7.70	8.40
Mass of dry soil (Ms = Mc-Ma)(gm)	16.00	14.90	14.60	13.40	14.00
Moisture content [(Mw/Ms)*100] (%)	50.00	53.02	55.48	57.46	60.00
Penetration(mm)	15.77	17.93	20.01	21.30	22.90
Liquid Limit (LL) (%)			**55.48**		

Table B3. Liquid limit test result of clay-sand mixture (50% sand)

Tin number	1	2	3	4	5
Mass of tin (Ma)(gm)	36.60	36.90	37.10	36.90	37.00
Mass of tin + wet soil (Mb)(gm)	54.90	51.60	53.80	55.30	55.30
Mass of tin + dry soil (Mc)(gm)	50.00	47.60	49.20	50.00	49.90
Mass of water (Mw = Mb-Mc)(gm)	4.90	4.00	4.60	5.30	5.40
Mass of dry soil (Ms = Mc-Ma)(gm)	13.40	10.70	12.10	13.10	12.90
Moisture content [(Mw/Ms)*100] (%)	36.57	37.38	38.02	40.46	41.86
Penetration(mm)	17.16	18.29	18.34	23.17	23.99
Liquid Limit (LL) (%)			**38.9**		

Table B4. Liquid limit test result of clay-sand mixture (75% sand)

Tin number	1	2	3	4	5
Mass of tin (Ma)(gm)	36.70	37.00	37.00	37.40	37.00
Mass of tin + wet soil (Mb)(gm)	58.90	56.50	64.80	73.70	59.40
Mass of tin + dry soil (Mc)(gm)	54.80	52.85	59.55	66.40	54.80
Mass of water (Mw = Mb-Mc)(gm)	4.10	3.65	5.25	7.30	4.60
Mass of dry soil (Ms = Mc-Ma)(gm)	18.10	15.85	22.55	29.00	17.80
Moisture content [(Mw/Ms)*100] (%)	22.65	23.03	23.28	25.17	25.84
Penetration(mm)	17.70	18.65	19.08	22.96	24.29
Liquid Limit (LL) (%)			**23.7**		

Table B5. Plastic limit test result of clay-sand mixture (0% sand)

Tin number	1	2
Mass of tin (Ma)(gm)	37.20	37.10
Mass of tin + wet soil (Mb)(gm)	42.70	43.20
Mass of tin + dry soil (Mc)(gm)	41.40	41.80
Mass of water (Mw = Mb-Mc)(gm)	1.30	1.40
Mass of dry soil (Ms = Mc-Ma)(gm)	4.20	4.70
Moisture content [(Mw/Ms)*100] (%)	30.95	29.79
Plastic (PL) (%)	**30.37**	

Table B6. Plastic limit test result of clay-sand mixture (25% sand)

Tin number	1.00	2.00
Mass of tin (Ma)(gm)	37.00	37.20
Mass of tin + wet soil (Mb)(gm)	43.60	43.20
Mass of tin + dry soil (Mc)(gm)	42.20	42.10
Mass of water (Mw = Mb-Mc)(gm)	1.40	1.10
Mass of dry soil (Ms = Mc-Ma)(gm)	5.20	4.90
Moisture content [(Mw/Ms)*100] (%)	26.92	22.45
Plastic (PL) (%)	**24.69**	

Table B7. Plastic limit test result of clay-sand mixture (50% sand)

Tin number	1	2
Mass of tin (Ma)(gm)	36.60	37.60
Mass of tin + wet soil (Mb)(gm)	43.50	44.10
Mass of tin + dry soil (Mc)(gm)	42.50	43.20
Mass of water (Mw = Mb-Mc)(gm)	1.00	0.90
Mass of dry soil (Ms = Mc-Ma)(gm)	5.90	5.60
Moisture content [(Mw/Ms)*100] (%)	16.95	16.07
Plastic (PL) (%)	**16.51**	

Table B8. Plastic limit test result of clay-sand mixture (75% sand)

Tin number	1	2
Mass of tin (Ma)(gm)	37.10	36.60
Mass of tin + wet soil (Mb)(gm)	44.50	42.30
Mass of tin + dry soil (Mc)(gm)	43.70	41.80
Mass of water (Mw = Mb-Mc)(gm)	0.80	0.50
Mass of dry soil (Ms = Mc-Ma)(gm)	6.60	5.20
Moisture content [(Mw/Ms)*100] (%)	12.12	9.62
Plastic (PL) (%)	**10.87**	

Appendix C – Test Data Oedometer Test

Table C1 Test data for oedometer test of clay-sand mixture (0% sand)

Diameter of ring	5.047	cm
Height of specimen after compaction	1.495	cm
Area	20.006	cm^2
Volume	29.909	cm^3
Mass of ring	60.10	g
Initial water content (added)	0.22	%
Ring + specimen	114.50	g
Mass of specimen	54.40	g
Dry mass of specimen	44.590	g
Bulk density	1.819	g/cm^3
Dry density	1.491	g/cm^3
After completion of test		
Mass of Ring + specimen	123.1	g
Mass of specimen	63.00	g
Dry mass of Specimen	44.30	g
Water content at the end	0.422	%
Corrected initial water content	0.228	%

Table C2 Test data for oedometer test of clay-sand mixture (25% sand)

Diameter of ring	5.047	cm
Height of specimen after compaction	1.367	cm
Area	20.006	cm^2
Volume	27.338	cm^3
Mass of ring	60.100	g
Initial water content (added)	0.22	%
Ring + specimen	117.90	g
Mass of specimen	57.80	g
Dry mass of specimen	47.377	g
Bulk density	2.114	g/cm^3
Dry density	1.733	g/cm^3
After completion of test		
Mass of Ring + specimen	143.3	g
Mass of specimen	63.20	g
Dry mass of Specimen	47.40	g
Water content at the end	0.333	%
Corrected initial water content	0.219	%

Table C3 Test data for oedometer test of clay-sand mixture (50% sand)

Diameter of ring	5.047	cm
Height of specimen after compaction	1.789	cm
Area	20.006	cm^2
Volume	35.788	cm^3
Mass of ring	60.10	g
Initial water content (added)	0.22	%
Ring + specimen	140.90	g
Mass of specimen	80.80	g
Dry mass of specimen	66.23	g
Bulk density	2.26	g/cm^3
Dry density	1.85	g/cm^3
After completion of test		
Mass of Ring + specimen	143.9	g
Mass of specimen	83.80	g
Dry mass of Specimen	65.80	g
Water content at the end	0.274	%
Corrected initial water content	0.228	%

Table C4 Test data for oedometer test of clay-sand mixture (75% sand)

Diameter of ring	5.047	cm
Height of specimen after compaction	1.842	cm
Area	20.006	cm^2
Volume	36.847	cm^3
Mass of ring	60.10	g
Initial water content (added)	0.160	%
Mass of Ring + specimen	145.50	g
Mass of specimen	85.40	g
Dry mass of specimen	73.621	g
Bulk density	2.318	g/cm^3
Dry density	1.998	g/cm^3
After completion of test		
Mass of Ring + specimen	150.0	g
Mass of Specimen	89.90	g
Dry mass of Specimen	72.10	g
Water content at the end	0.247	%
Corrected initial water content	0.184	%

Appendix D – Tabulated Results of Free swell tests

Table D1 Free swell test results of clay-sand mixture (0% sand)

Initial specimen height after compaction = 15.358		
Time (min)	Change in height (mm)	Vertical Swelling (%)
0.00	0.00	0.00
0.25	0.08	0.53
1.00	0.34	2.18
2.25	0.67	4.39
4.00	0.99	6.47
6.25	1.34	8.69
9.00	1.70	11.10
12.25	1.95	12.72
16.00	2.17	14.14
20.25	2.33	15.18
25.00	2.47	16.08
30.25	2.59	16.89
36.00	2.67	17.41
42.25	2.75	17.90
49.00	2.81	18.30
56.25	2.85	18.58
64.00	2.89	18.83
72.25	2.94	19.12
81.00	2.96	19.29
90.25	3.00	19.51
100.00	3.01	19.59
110.25	3.02	19.68
121.00	3.06	19.90
132.25	3.07	19.97
144.00	3.09	20.09
156.25	3.10	20.16
169.00	3.10	20.20
182.25	3.11	20.26
196.00	3.12	20.33
210.25	3.13	20.41
225.00	3.13	20.41
240.25	3.14	20.43
256.00	3.14	20.46
272.25	3.14	20.47
289.00	3.15	20.48
306.25	3.15	20.52

324.00	3.17	20.62
342.25	3.17	20.63
361.00	3.17	20.66
380.25	3.18	20.69
400.00	3.18	20.73
420.25	3.19	20.75
441.00	3.20	20.82
462.25	3.20	20.82
484.00	3.20	20.86
506.25	3.21	20.89
529.00	3.21	20.93
552.25	3.22	20.96
576.00	3.22	20.99
600.25	3.22	20.99
625.00	3.23	21.03
650.25	3.24	21.08
676.00	3.24	21.08
702.25	3.25	21.14
729.00	3.25	21.17
756.25	3.25	21.19
784.00	3.26	21.20
812.25	3.26	21.21
841.00	3.26	21.23
870.25	3.26	21.25
900.00	3.26	21.25
930.25	3.27	21.26
961.00	3.27	21.27
992.25	3.27	21.27
1024.00	3.27	21.27
1056.25	3.27	21.28
1089.00	3.27	21.28
1122.25	3.27	21.28
1156.00	3.27	21.30
1190.25	3.27	21.30
1225.00	3.27	21.30
1260.25	3.27	21.31
1296.00	3.27	21.31
1332.25	3.27	21.32
1369.00	3.27	21.32
1406.25	3.27	21.32

Table D2. Free swell test results of clay-sand mixture (25% sand)

Initial specimen height after compaction = 14.263 mm		
Time (min)	Change in Height (mm)	Vertical swelling (%)
0.00	0.00	0.00
0.25	0.02	0.11
1.00	0.13	0.93
2.25	0.46	3.25
4.00	0.71	4.98
6.25	0.94	6.57
9.00	1.12	7.88
12.25	1.30	9.11
16.00	1.43	10.04
20.25	1.55	10.85
25.00	1.64	11.49
30.25	1.72	12.08
36.00	1.78	12.47
42.25	1.84	12.90
49.00	1.87	13.14
56.25	1.90	13.35
64.00	1.92	13.45
72.25	1.94	13.63
81.00	1.96	13.77
90.25	1.97	13.85
100.00	1.98	13.92
110.25	2.00	14.01
121.00	2.00	14.08
132.25	2.01	14.11
144.00	2.01	14.13
156.25	2.02	14.17
169.00	2.02	14.18
182.25	2.02	14.20
196.00	2.02	14.21
210.25	2.03	14.23
225.00	2.03	14.26
240.25	2.03	14.28
256.00	2.04	14.30
272.25	2.04	14.32
289.00	2.04	14.34
306.25	2.05	14.37
324.00	2.05	14.39

342.25	2.05	14.39
361.00	2.05	14.41
380.25	2.06	14.44
400.00	2.06	14.45
420.25	2.06	14.46
441.00	2.06	14.49
462.25	2.07	14.53
484.00	2.07	14.57
506.25	2.08	14.62
529.00	2.08	14.64
552.25	2.09	14.70
576.00	2.09	14.71
600.25	2.10	14.72
625.00	2.10	14.73
650.25	2.10	14.73
676.00	2.10	14.74
702.25	2.11	14.79
729.00	2.11	14.79
756.25	2.11	14.82
784.00	2.11	14.82
812.25	2.11	14.83
841.00	2.11	14.84
870.25	2.11	14.85
900.00	2.11	14.85
930.25	2.12	14.88
961.00	2.12	14.88
992.25	2.12	14.88
1024.00	2.12	14.90
1056.25	2.12	14.91
1089.00	2.12	14.91
1122.25	2.12	14.91
1156.00	2.12	14.92
1190.25	2.12	14.92
1225.00	2.12	14.92
1260.25	2.12	14.92
1296.00	2.13	14.93
1332.25	2.13	14.93
1369.00	2.13	14.93
1406.25	2.13	14.93

Table D3. Free swell test results of clay-sand mixture (50% sand)

Initial Specimen height after compaction = 19.089mm		
Time (mm)	Change in height (mm)	Vertical Swelling (%)
0.00	0.0000	0.0000
0.25	0.0002	0.0010
1.00	0.0007	0.0037
2.25	0.0009	0.0047
4.00	0.0010	0.0052
6.25	0.0020	0.0105
9.00	0.0030	0.0157
12.25	0.0050	0.0262
16.00	0.0050	0.0262
20.25	0.0050	0.0262
25.00	0.0070	0.0367
30.25	0.0070	0.0367
36.00	0.0080	0.0419
42.25	0.0100	0.0524
49.00	0.0110	0.0576
56.25	0.0110	0.0576
64.00	0.0120	0.0629
72.25	0.0140	0.0733
81.00	0.0140	0.0733
90.25	0.0140	0.0733
100.00	0.0190	0.0995
110.25	0.0200	0.1048
121.00	0.0210	0.1100
132.25	0.0220	0.1152
144.00	0.0220	0.1152
156.25	0.0240	0.1257
169.00	0.0280	0.1467
182.25	0.0310	0.1624
196.00	0.0330	0.1729
210.25	0.0340	0.1781
225.00	0.0340	0.1781
240.25	0.0350	0.1834
256.00	0.0360	0.1886
272.25	0.0360	0.1886
289.00	0.0360	0.1886
306.25	0.0370	0.1938
324.00	0.0370	0.1938

342.25	0.0380	0.1991
361.00	0.0380	0.1991
380.25	0.0380	0.1991
400.00	0.0430	0.2253
420.25	0.0450	0.2357
441.00	0.0490	0.2567
462.25	0.0500	0.2619
484.00	0.0510	0.2672
506.25	0.0520	0.2724
529.00	0.0530	0.2776
552.25	0.0530	0.2776
576.00	0.0540	0.2829
600.25	0.0540	0.2829
625.00	0.0550	0.2881
650.25	0.0550	0.2881
676.00	0.0550	0.2881
702.25	0.0550	0.2881
729.00	0.0550	0.2881
756.25	0.0560	0.2934
784.00	0.0560	0.2934
812.25	0.0560	0.2934
841.00	0.0560	0.2934
870.25	0.0560	0.2934
900.00	0.0570	0.2986
930.25	0.0570	0.2986
961.00	0.0570	0.2986
992.25	0.0570	0.2986
1024.00	0.0580	0.3038
1056.25	0.0580	0.3038
1089.00	0.0590	0.3091
1122.25	0.0610	0.3196
1156.00	0.0610	0.3196
1190.25	0.0620	0.3248
1225.00	0.0620	0.3248
1260.25	0.0620	0.3248
1296.00	0.0630	0.3300
1332.25	0.0640	0.3353
1369.00	0.0640	0.3353
1406.25	0.0640	0.3353

Table D4. Free swell test results of clay-sand mixture (75% sand)

Initial Specimen height after compaction = 18.908mm		
Time (mm)	Change in height (mm)	Vertical Swelling (%)
0.00	0.00	0.00
0.25	0.00	0.01
1.00	0.00	0.01
2.25	0.00	0.01
4.00	0.00	0.01
6.25	0.00	0.01
9.00	0.00	0.01
12.25	0.00	0.01
16.00	0.00	0.01
20.25	0.00	0.01
25.00	0.00	0.01
30.25	0.00	0.01
36.00	0.00	0.01
42.25	0.00	0.02
49.00	0.00	0.02
56.25	0.00	0.02
64.00	0.00	0.02
72.25	0.00	0.02
81.00	0.00	0.02
90.25	0.00	0.02
100.00	0.00	0.01
110.25	0.00	0.02
121.00	0.00	0.01
132.25	0.00	0.02
144.00	0.00	0.02
156.25	0.00	0.02
169.00	0.00	0.02
182.25	0.00	0.02
196.00	0.00	0.02
210.25	0.00	0.02
225.00	0.00	0.02
240.25	0.00	0.02
256.00	0.01	0.03
272.25	0.00	0.02
289.00	0.00	0.02
306.25	0.00	0.02
324.00	0.01	0.03

342.25	0.01	0.03
361.00	0.01	0.03
380.25	0.01	0.03
400.00	0.01	0.03
420.25	0.00	0.02
441.00	0.01	0.04
462.25	0.01	0.03
484.00	0.01	0.04
506.25	0.01	0.03
529.00	0.01	0.04
552.25	0.01	0.04
576.00	0.01	0.03
600.25	0.01	0.03
625.00	0.01	0.05
650.25	0.01	0.04
676.00	0.01	0.05
702.25	0.01	0.05
729.00	0.01	0.05
756.25	0.01	0.04
784.00	0.01	0.04
812.25	0.01	0.05
841.00	0.01	0.05
870.25	0.01	0.06
900.00	0.01	0.05
930.25	0.01	0.05
961.00	0.01	0.05
992.25	0.01	0.06
1024.00	0.01	0.06
1056.25	0.01	0.06
1089.00	0.01	0.05
1122.25	0.01	0.06
1156.00	0.01	0.07
1190.25	0.01	0.06
1225.00	0.01	0.06
1260.25	0.01	0.06
1296.00	0.01	0.06
1332.25	0.01	0.06
1369.00	0.01	0.06
1406.25	0.01	0.06

Appendix E – Risk Assessment

Risk Assessment

For all the projects, it is very essential that the associated risks be identified, assessed and controlled. This is only possible in a controlled work environment and by following safe workplace procedures.

Risks are associated with hazards and while hazards are cannot be avoided, risks associated can certainly be reduced and controlled by proper management.

A risk management chart summarizing hazard identification, potential risks, and control methods is presented in the table below.

Activity	Hazard	Risks	Risk Level	Control measures
Lab experiments	Falling over of objects/parts of equipment (oedometer, etc)	Potential injury to the user/damage to the equipment	Moderate	Avoid placing objects at the edge of the table. Place them properly so they don't roll over and fall
Lab experiments	Fine sand particles flying in air	Potential irritation of eyes, nose and throat and/or serious problems	Moderate	Wear facemask and goggles Do not turn on fan in high speed near the sand
Lab Experiment	Tables with wheels	Injury to people and/or damage of equipment	Low	Make sure the wheels are locked
Research and reporting	Sitting at a desk for prolonged periods	Potential neck and back injuries	Moderate	Follow the ergonomics (correct siting posture) Take regular breaks and do some stretches
Research and reporting	Viewing computer screen for prolonged periods	Potential eye problems and headaches	Low	Take a few minutes breaks every hour. Regularly look away from computer, possibly out of the window during