University of Southern Queensland

Faculty of Health, Engineering & Sciences

The Use of Geopolymers for Stabilising Expansive Soils

A dissertation submitted by:

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In fulfillment of the requirements of

ENG 4111 and ENG4112 Research Project

Towards the degree of

Bachelor of Engineering (Civil)

Abstract

Expansive soils are common around the world and are characterised by their non-linear shrink-swell behaviour with changes in water content. Expansive soils create problems for engineers by damaging structures, pavements and foundations with this unpredictable movement. Along with these shrink-swell variations, expansive soils exhibit very low bearing capacities and strengths when moisture contents are high. Expansive soils vary in performance and identifying these soils can be difficult.

Lime stabilisation is the most common method for stabilising expansive soils in Australia. Lime stabilisation decreases plasticity, reduces shrink-swell, increases bearing capacity and shear strength of subgrade soils. Lime has disadvantages, namely the high financial and carbon cost of lime. Australia has an abundance of fly ash, a waste product from the production of electricity using coal. Fly ash can be used as a geopolymer binder when combined with sodium hydroxide and sodium silicate. These geopolymers can achieve the same results as lime and cement-based stabilisers, at a fraction of the carbon cost, and making use of waste fly ash that would otherwise go to landfill.

The most reliable methods of identifying expansive soils were examined and a series of laboratory tests were conducted to determine the characteristics of an expansive soil found in the South East Queensland region. A geopolymer treatment option was found using past research and applied to the expansive soil. The laboratory tests were repeated and the changes to the soil were recorded. These performance changes were examined and compared with the standards for subgrades as determined by Austroads pavement design guidelines. Pavements were then designed using these standards to determine if geopolymer soil stabilisation is viable from a technical performance perspective using the current Australian design guides for pavements. A cost analysis was also conducted to see if geopolymer stabilisation was viable from a financial perspective.

The research found that alkali activated fly ash geopolymers were effective at improving the engineering characteristics of expansive soils. The results obtained met the requirements for Austroads standards for bearing capacity and reductions in plasticity index, meaning that it was viable alternative from a technical perspective. Geopolymer stabilisation was found to be far more sustainable, producing roughly 10-20% of the carbon cost of lime stabilisation. Financially, geopolymer stabilisation is still expensive, costing roughly double what lime stabilisation does in Australia

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Limitations of Use

Certification

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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Acknowledgements

I would like to acknowledge and thank Professor Andreas Nataatmadja for his constructive feedback and supervision throughout the project. Without his help this project would not have been possible. I would also like to thank Daniel Eising and Piumika Ariyadasa for their assistance with the laboratory work and technical guidance. Lastly and most importantly I would like to thank my partner Abigail for her support throughout the course of this degree.

D.Daley

University of Southern Queensland

October 2019

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CHAPTER 1

INTRODUCTION

1.1 Background

Expansive soils are soils in which there is a large amount of variation in the volume of the soil as the moisture level changes. These soils shrink as the moisture levels decrease and swell as the moisture level increases. Their deformation usually exceeds elastic limits and therefore cannot be accurately predicted (Nelson & Miller 1992). This large, unpredictable swelling and shrinking often causes cracks and damage to the pavement, foundations or structures constructed over the soil. Expansive soils have been a problem for many years around the world with countries like Australia, New Zealand, India, USA, UK, South Africa and China spending large amounts of money and time fixing problems caused by expansive soils. Anusha and Ramakrishna (2016) highlight that the annual cost of damage to structures by expansive soils is estimated to be 150 million pounds in the UK and upwards of $1 Billion in the United states. This makes damage to structures from expansive soils second only to insect damage in the US, Anusha and Ramakrishna (2016). Floods, hurricanes and fire often capture the attention of governments and the general population due to their destructive power although expansive soils pose a far greater risk to infrastructure.

Figure 1- Cracking in Black Cotton Soil, (Kwan 2018)

1.2 Problem

Expansive soils pose many difficulties for governments, engineers, customers and residents. The unpredictability of expansive soils can result in premature degradation, cracking and excessive maintenance of pavements and structures built on them. Pipelines built through expansive soils may leak or burst and downtime or repairs can be very costly. The main problem with expansive soils is the large shrink and swell of the soil with a change in water content. Because this change drastically exceeds the elastic limits it can be impossible to predict. Expansive clays are frequently called 'Black Cotton Soils' As they have a dark, almost black appearance. These soils are a Montmorillonite group, which have a high potential for shrink swell. Stabilisation of these soils can be difficult, and require time, machinery and cost to remedy. In order to remediate these soils effectively a solution needs to be found that is cheaper and easier to apply. These soils are common in South East Queensland and there has been considerable interest in the potential for Fly Ash based geopolymers to stabilise these soils. Fly Ash is a viable solution as there are many coal fired power plants in Australia to source the waste material from, thus diverting it from landfill and disposal.

1.3 Project Aim

The broad aim of this project is 'To investigate the possibility of using geopolymers containing sodium silicate and sodium hydroxide to reduce the shrink-swell variation in expansive clay soils in the South East Queensland region and its use to stabilise the foundations of existing structures'.

1.4 Project Objectives

As per the project specification the specific objectives of the project are.

- Research information relating to the use of geopolymers for geotechnical stabilisation, geotechnical characteristics of expansive soils, appropriate testing procedures and contemporary stabilisation methods
- To determine the most reliable methods for identifying expansive soils in a field environment

- To determine the most effective and reliable preliminary test regime for confirming the soils chosen to meet the criteria for expansive soils
- Identify 1-2 locations likely to be suitable for testing. If soil tests are available use those, if not conduct preliminary soil tests to identify suitability for testing.
- Identify best geopolymer ratio and solution for testing, using past research results.
- Decide on appropriate testing procedures for samples using Australian Standards and past research.
- Prepare soil and samples in accordance with relevant Australian Standards and conduct testing on samples to gather data on stabilisation potential.
- Analyse test results and compare with current stabilisation methods for expansive soils.
- Determine on whether geopolymer stabilisation could be useful in stabilizing foundations of existing structures in South East Queensland.

If time permits the project will investigate application methods for this type of geopolymer stabilisation and attempt to make determinations on whether this can be used for both stabilisation of greenfield sites and for existing structures.

Objective one

'Research information relating to the use of geopolymers for geotechnical stabilisation, geotechnical characteristics of expansive soils, appropriate testing procedures and contemporary stabilisation methods'

Research has been conducted from a variety of resources- Hardcopy textbooks and geotechnical manuals, online academic journals, articles, conference papers and eBooks. Most research papers have been accessed through USQ's academic portal, thus ensuring that high quality resources have been used. The research conducted so far focuses primarily on recently published works, which ensures that the most up to date and industry standard techniques are used.

Objective two

'To determine the most reliable methods for identifying expansive soils in a field environment'

The author has conducted research of various textbooks, standards and academic papers and online resources in order to determine the most reliable test methods for identifying expansive soils in the field. Field identification is important as it gives a starting point for the geotechnical engineer or soil technician to narrow down what tests will be needed and can save a lot of time and expense by eliminating unnecessary laboratory tests. Considering soil types can vary dramatically from location to location, I have also consulted a local soil testing company and sought advice as to the most effective testing methods for the Ipswich region.

Objective three

> ***'To determine the most effective and reliable preliminary test regime for confirming the soils chosen meet the criteria for expansive soils '***

Textbooks, academic papers, journals and USQ Library resources were accessed to evaluate the most effective methods for testing the characteristics that best identify expansive soils. This was important as expansive soils have a number of engineering characteristics that can predict shrink-swell variation and ensuring that the right tests are conducted with detailed, correct procedures ensures the technical integrity of the experiments. These preliminary tests will give the base properties necessary to check whether the geopolymer stabilisation has been successful, and to what extent. Additional advice was sought from USQ technical staff and the project supervisor.

Objective four

> ***'Identify 1-2 locations likely to be suitable for testing. If soil tests are available use those, if not conduct preliminary soil tests to identify suitability for testing.'***

To do this, Ipswich City Council were approached for documentation and maps with soil types. A suitable location was found and soil test results for that location were requested from a local soil testing company. The company provided test results for the location that confirmed that the soil on the site was of highly expansive clays and suitable for the needs of the experiments. A location has been identified that may be affected by expansive soils, although this has not been confirmed by laboratory tests, the soils behaviour is consistent with that of expansive clay, so the soil will be tested to see if it meets the classification for expansive clays. The author considered it important to find two sites, one with documented oil properties aligning with expansive soils, and another undocumented site. This gave the author the opportunity to identify expansive soils in the field and test whether these observations were correct. In the event that the observed soil does not meet the classifications for expansive soils it means that at least one soil will be useful for polymerisation testing. If both meet the requirements, there is the advantage of having more data to substantiate the effects of treatment.

Objective Five

> '***Identify best geopolymer ratio and solution for testing, using past research results'***

Recent research was the most effective place to search for this information, as the field of geopolymer stabilisation relatively recent. Online journal articles, conference papers and dissertations were used. It was important to find the best ratio of geopolymer to activator solution, as well as the best geopolymer to soil ratio as the properties that make stabilisation attractive such as strength and affordability can be negatively impacted by the incorrect ratios. Many papers on the subject insist that the activator solution must be highly alkaline, molar concentrations in the 10-15 range. This highly concentrated alkaline solution does provide an excellent environment for polymerisation to occur, but also creates safety issues and increases the cost. Papers were found that reported negligible loss of strength and bearing capacity with molar concentrations of Sodium Hydroxide around the 5 MOL range. This substantially decreases the safety risk and cost of stabilisation.

Objective Six

> ***'Decide on appropriate testing procedures for samples using Australian Standards and past research'***

Research for this objective primarily focused on relevant Australian Standards and codes. With the characteristics that best predict the soils potential for activity already identified the approved tests that show these had to be found. The Australian Standards AS1289 series of tests provided detailed and accurate procedures for the preparation and testing of the samples. It is important to have a well-documented, approved method of testing to ensure that the results obtained are as accurate as possible and give the best results.

Objective seven

> ***'Prepare soil and samples in accordance with relevant Australian Standards and conduct testing on samples to gather data on stabilisation potential.'***

During this project, there was a strict adherence to the relevant Australian Standards, and all tests were conducted with integrity, under the routine supervision of USQ laboratory staff. This supervision ensured that mistakes were not made that could affect the data collected.

Objective Eight

> ***'Analyse test results and compare with current stabilisation methods for expansive soils.'***

The results obtained from testing were compared with results from similar research conducted. These results were then compared to performance characteristics obtained from other stabilisation methods to determine if alkali activated geopolymer stabilisation is as effective as other methods.

Figure 2-Cracking in pavement from swelling clay (Geoengineer.org 2018)

1.5 Scope

This project will focus on determining whether fly ash based geopolymers can provide an adequate increase in the performance of expansive soils in the Ipswich region. The project will first determine the test procedures that best identify expansive soils and then decide on the best way to treat the soil. The scope for the treatment will determine ratios and quantities of treatment materials and necessary curing times. There will be control tests run on untreated samples to determine the in-situ condition of the soil as well as repeating the tests on treated samples to determine the amount of improvement, if any.

Figure 3- Pavement cracking due to expansive soil (Author 2019)

1.6 Limitations

This project will be limited in scale, with only laboratory tests being conducted on the samples. In field testing and test structures will not be able to be conducted due to both time and budgetary restraints. The project is also limited by the test equipment available in the USQ geomechanics laboratory.

1.7 Dissertation Structure

In chapter two, the literature review presents the findings of the research conducted into expansive soils. This is broken up into five main parts. Firstly, the impact of expansive soils is examined, and methods of identifying expansive soils in the field. The properties and characteristics that determine how expansive a soil is are investigated, as well as the testing methods available to measure these properties. Finally, current stabilisation methods are explored and the effectiveness of these is examined.

Chapter three will outline the methodology chosen for conducting laboratory tests and explain the steps taken to ensure that the experiments were conducted in a way to ensure accuracy and reliability of results.

Chapter four presents the results of the experiments conducted. The results will be presented as a series of tables and graphs of the raw data collected during experimentation.

Chapter five will present pavement designs for three different roads, a lightly trafficked road, unsealed road and a heavy-duty road. Each of these applications will be designed using the relevant Austroads guide. Each application will have two designs, one using traditional stabilisation techniques, and another using geopolymer stabilisation.

Chapter six will analyse and discuss the results obtained during the research and compares it to information found in the literature review. The potential application for fly ash geopolymer stabilisation when using Austroads guides will be evaluated.

Chapter seven evaluates how well the project met the aims and objectives set out in the project specification, conclusions drawn from the research, and suggests any further work.

Chapter 2

Literature Review

2.1 Introduction

The literature review conducted to determine what has been discovered about the stabilisation of expansive soil from other researchers and clarify the objectives of my research. The literature review started with the problems that expansive soils can pose for structures and pavements. The individual properties that contribute to a soils expansiveness and instability were examined to determine which were most important factors. The commonly used field test and sampling methods have been examined so a suitable preliminary test schedule can be developed. Various stabilisation methods were examined to gain a better understanding of what mechanisms they used to achieve stabilisation. The chemical reactions behind the geopolymerisation processes were studied so that the process could be better understood. The geopolymer stabilisation research focused on determining optimum geopolymer to activator ratios as well as total fly ash content. Information for curing times and sample preparation were examined and coupled with appropriate testing methods. The results obtained from other research was noted to give a benchmark on results that could be obtained if the experiments are successful.

2.2 Problem with Expansive soils

Nelson and Miller(1992) Described expansive soils as soils that shrink and expand dramatically with a change in their moisture content. This large variation in shrink and swell can cause huge amounts of damage to structures and pavements built on these soils. Nelson and Miller note that the difficulty with expansive soils lies in the fact that the deformations can be significantly greater than elastic deformation, which means that they cannot be accurately predicted using traditional elastic theory methods. Along with the swelling of soil at high moisture contents, the bearing capacity of the soil reduces considerably.

Figure 4-Cracking due to expansive soil Marburg QLD, (Author 2019)

2.3 Clay Minerology

Expansive soils typically belong to the Montmorillonite group of clays. Clay soils are made up of two distinct units, an alumina octahedron and a silica tetrahedron. These silica tetrahedrons form crystalline silica sheets and the alumina octahedrons form what is known as a gibbsite sheet (Das 2006). Das also explains that in some clays the alumina is replaced by magnesium, which then forms brucite sheets, with the same crystalline structure as the gibbsite sheets. In montmorillonite clays the gibbsite sheet is in the middle of two silica sheets, with isomorphous substitution occurring in the gibbsite sheet, where the aluminium atom is replaced with magnesium or iron. Illite clays have this same silica-gibbsite-silica layered structure, with potassium ions bounding the silica layers together. In montmorillonite clays these potassium ions are not present and large quantities of water is attracted to the space between layers Das (2006).

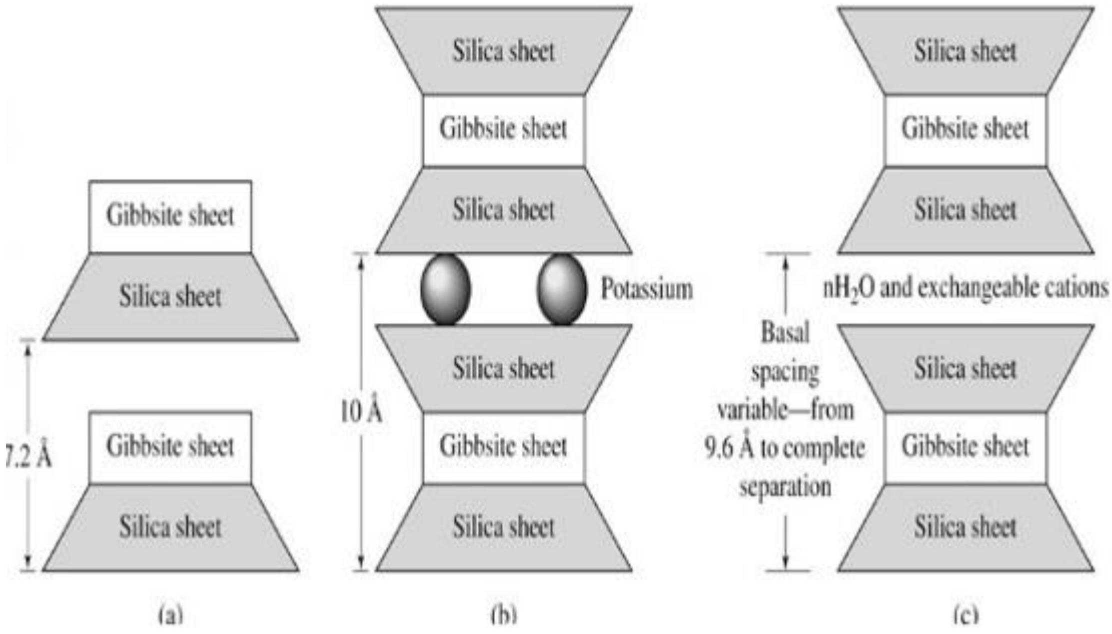

Figure 5- Clay Structures (Das 2006)

All clay particles carry a negative charge on their surfaces, and Das(2006) went on to explain that when clay is dry the negative charges are balanced by cations such as Mg^{2+},Na^{+},K^{+} and Ca^{2+}. When the clay becomes wet these cations float around the surface of the clay particle in what is known as a diffuse double layer. Water is a dipole, which means that it has a positive charge at one end and a negative charge at the other. In the case of clay particles this means that the water is attracted to both the positively charged cations in the diffuse double layer and the negatively charged surface of the clay particle. In montmorillonite clays the particles have a huge surface area as the particles are thin, flaky plates. This large surface area to volume ratio means that montmorillonite needs a huge amount of water between each particle to balance the large negative charges on its surface. This water around the clay particles is what gives clay its plastic properties, and in turn causes the dramatic loss of bearing capacity in expansive soils (Das 2006).

2.4 Soil Classification

There are two primary soil classification systems as noted by Das (2006), the Unified Soil Classification System (USCS) and the American Association of State highway and Transportation Officials (AASHTO). Both classification systems rely on plasticity (liquid limits and plastic limits) and texture (grain size). These two standards are important from an engineering perspective as earlier texture-based standards were only suitable for agricultural purposes and failed to consider the quantity and type of clay-based minerals present in fine grained soils. These mineral compositions are responsible for a great deal of a soil's physical properties (Das 2006).

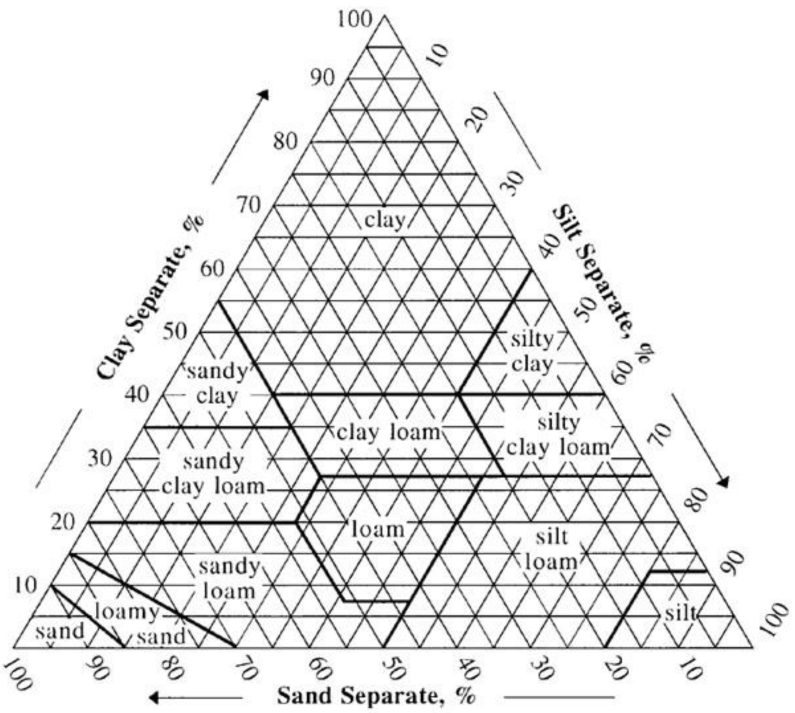

Figure 6-USDA classification chart (USDA 2019)

2.4.1 USCS classification system

The USCS system classifies soils into two broad categories, coarse grained soils and fine-grained soils. Fine grained soils are those where 50% or more passes through the No 200 sieve. The USCS makes an allowance for organic and inorganic silts and clays. Fine grained soils are either classified as

- M- Inorganic silts
- C- Inorganic clays
- O- Organic silts and clays

Criteria for assigning group symbols				Group symbol
Coarse-grained soils More than 50% of retained on No. 200 sieve	**Gravels** More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[a]	$C_u \geq 4$ and $1 \leq C_c \leq 3$[c]	GW
			$C_u < 4$ and/or $1 > C_c > 3$[c]	GP
		Gravels with Fines More than 12% fines[a,d]	$PI < 4$ or plots below "A" line (Figure 5.3)	GM
			$PI > 7$ and plots on or above "A" line (Figure 5.3)	GC
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[b]	$C_u \geq 6$ and $1 \leq C_c \leq 3$[c]	SW
			$C_u < 6$ and/or $1 > C_c > 3$[c]	SP
		Sands with Fines More than 12% fines[b,d]	$PI < 4$ or plots below "A" line (Figure 5.3)	SM
			$PI > 7$ and plots on or above "A" line (Figure 5.3)	SC
Fine-grained soils 50% or more passes No. 200 sieve	**Silts and clays** Liquid limit less than 50	Inorganic	$PI > 7$ and plots on or above "A" line (Figure 5.3)[e]	CL
			$PI < 4$ or plots below "A" line (Figure 5.3)[e]	ML
		Organic	$\frac{\text{Liquid limit — oven dried}}{\text{Liquid limit — not dried}} < 0.75$; see Figure 5.3; OL zone	OL
	Silts and clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line (Figure 5.3)	CH
			PI plots below "A" line (Figure 5.3)	MH
		Organic	$\frac{\text{Liquid limit — oven dried}}{\text{Liquid limit — not dried}} < 0.75$; see Figure 5.3; OH zone	OH
Highly Organic Soils	Primarily organic matter, dark in color, and organic odor			Pt

[a] Gravels with 5 to 12% fine require dual symbols: GW-GM, GW-GC, GP-GM, GP-GC.

[b] Sands with 5 to 12% fines require dual symbols: SW-SM, SW-SC, SP-SM, SP-SC.

[c] $C_u = \frac{D_{60}}{D_{10}}$; $C_c = \frac{(D_{30})^2}{D_{60} \times D_{10}}$

[d] If $4 \leq PI \leq 7$ and plots in the hatched area , use dual symbol GC-GM or SC-SM.

[e] If $4 \leq PI \leq 7$ and plots in the hatched area, use dual symbol CL-ML.

Figure 7-USCS Classification chart (Das 2006)

After the gain size distribution is complete the Atterberg limit results are used to further classify the soil. It is either classified 'L' for low plasticity (LL<50%), or 'H' for high plasticity (>50%). The fact that the USCS system makes an allowance for organic fines is important for the study of expansive soils because soils with a high level of organic matter tend to not behave expansively compared to montmorillonite clays.

2.4.2 AASHTO classification system

The AASHTO also categorises soils into fine grained and coarse-grained soils, although a soil is considered fine grained if 35% or more passes through the No 200 sieve. Just like the USCS system the Atterberg limits are determined after particle size is found. Das (2006) noted that a coarse-grained soil that has about 35% fine grains behaved like a fine-grained soil and this is because the fine grains fill the voids between the coarse ones and keeps them apart. It is for this reason that Das (2006) stated that the AASHTO is a better engineering classification system for soils with a fines percentage of 35%-50%.

Although the AASHTO system better classifies the fine-grained soils between 35%-50%, the fact that the USCS has categories is perhaps of more importance to studies in expansive soils, as the high level of organics can skew classifications.

General classification	Silt-clay materials (more than 35% of total sample passing No. 200)			
Group classification	A-4	A-5	A-6	A-7 A-7-5ª A-7-6ᵇ
Sieve analysis (percentage passing)				
No. 10				
No. 40				
No. 200	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 40				
Liquid limit	40 max.	41 min.	40 max.	41 min.
Plasticity index	10 max.	10 max.	11 min.	11 min.
Usual types of significant constituent materials	Silty soils		Clayey soils	
General subgrade rating	Fair to poor			

[a]For A-7-5, $PI \leq LL - 30$
[b]For A-7-6, $PI > LL - 30$

Figure 8-AASHTO Classification chart (Das 2006)

2.5 Characteristics that predict Shrink-swell potential

As Nelson and Miller (1992) noted, the behaviour of expansive soils is a complex subject and there can be many causes of movement, all of which can be broken down into three main categories.

Environmental factors

- Conditions in the environment around the site which influence the soil, such as groundwater, drainage, temperature, vegetation, climate.

Soil characteristics

- the physical qualities of the soil itself such as, grain size distribution, mineral composition, organic content etc.

State of Stress

- The loadings, both past and present that have contributed to the soil's consolidation, in-situ conditions, loading and soil profile.

2.5.1 Environmental Factors

Environmental factors are one of the most important factors to consider when dealing with expansive soils. This is because unlike soil characteristics and stress states, the environmental factors can be controlled to an extent relatively simply and cheaply. The characteristics of the soil determine the swelling capacity with moisture content so by controlling the moisture available to the soil it is not given the opportunity to change in volume. Due to the moisture content of an expansive soil being the cause of its expansion and contraction, controlling the amount of soil moisture can limit the degree to which the soil volume changes, allowing a degree of reliability to its expected behaviour. Nelson and Miller (1992) list some important environmental factors as Climate, Groundwater, Drainage and man-made water sources, vegetation, temperature and climatic variations. All these factors control the moisture level in expansive soils. Moisture levels for some sites can be kept at a relatively stable rate with proper attention to drainage and other factors. Snethan et al (1977) studied the 17 published indicators for expansive soil and placed environmental factors as important as a soil's liquid limit and plasticity index.

2.5.2 Soil characteristics

Although there are stabilisation standards available for both granular and cohesive soils, Khan (2016) explains that there is no stabilisation standard available for soils with a plasticity index greater than 35. This is because there have been several studies that have shown that soils with a high plasticity index will have a high swelling potential. Chen (1988) classifies soils with a plasticity index greater than 35 as those with a very high swelling potential.

Das (2006) confirmed that the plasticity index is extensively used for classifying expansive soils and should always be determined. He goes on to state that the two most important indicators of swell potential are the liquid limit and plasticity index.

Nelson and Miller (1992) noted that the grain size distribution, clay content and plasticity are all reliable indicators for identifying expansive soil. They mentioned that commonly the Atterberg limits and clay content results are combined to a single parameter called 'Activity'. Further to this Seed et al (1962) developed a chart based on activity and % clay sizes. Skempton (1953) conducted research which observed that the plasticity index of a soil increased at a linear rate with the percentage of clay particles (<2μm). It was this observation which led him to coin the term 'Activity', which is defined as the slope of the line correlating plasticity index and percentage finer than 2µm (Das 2006). Altmeyer (1955) recommended the elimination of the percent clay testing as many laboratories were not equipped with hydrometer testing equipment at the time. In place of this he suggested finding the shrinkage limit or linear shrinkage value. Snethen et al (1977) evaluated 17 of the published criteria for predicting swell and concluded that the Liquid Limit and Plasticity index are the best indicators, along with the soils natural condition and environment.

There has been considerable difficulty in deciding which attributes are the best indicators of swell potential but the two tests that return the most consistent results were found to be the liquid limit and the Plasticity index.

Thomas, Baker and Zelazny (2000) stated that there have been many studies conducted to find the best indicators of shrink swell potential but there still has not been a test method developed which can accurately determine this potential. During their research they conducted particle size distribution, Cation exchange capacity, Atterberg Limits and potential volume change testing. The results of this testing showed that plasticity index was a poor indicator of shrink-swell capacity and that cation exchange capacity and liquid limit were the best indicators of shrink-swell. Quite often, easy to conduct field tests are the preferred method for geotechnical engineers to identify problematic soils and it more complex laboratory testing such as CEC and hydrometer analysis may not be done unless it is a large project, or the soil is thought to be particularly problematic.

Nelson and Miller (1992), concluded that although many of these procedures can be reliable at times for identifying expansive soils, there are such a large number of potential causes for soil activity that even a well-considered approach may not provide reliable predictions. They go on to note that the best indicator of soil expansiveness is generally past observations in the local area, and that an engineer should use local knowledge of the soils for projects and not rigidly adhere to standards that may not be best suited for local conditions.

It is for these reasons that preliminary tests for identifying the properties of the test soils will comprise of wet and dry sieving to determine particle size, the determination of the Atterberg Limits using the Casagrande bowl apparatus to find the liquid limit and the determination of plastic limits using the standard procedure listed in AS 1289. The linear shrinkage tests will also be conducted to give a better indication of improvements after treatment.

2.5.3 State of Stress

Nelson and Miller (1992) Noted that the volume change is directly related to the change in the state of stress of a soil. The effective stress of a saturated soil is a combination of total stress minus the pore water pressure. The in-situ state of the soil must be considered, for example an over consolidated soil is more prone to expansion. Excavated soils can experience heave as the reduction in stress means more water can be absorbed. This is also true in unsaturated soils. The construction of a pavement or structure over expansive soil can change both the consolidation and moisture content of the surrounding soil, so changed in the soil's behaviour must be accounted for. This change on behaviour can be mitigated to an extent by the installation of adequate drainage or landscaping close to the structure.

2.6 Strength testing

The preliminary testing already outlined is useful for determining the properties exhibited by expansive soils. These properties are useful for predicting and measuring the expansivity of a soil. Although these are important, the strength and bearing capacity of the soil is what is important when determining if the soil is suitable for a structure of pavement to be built on it. There are a number of strength tests available for soils and each test examines different failure modes for the soil. Some tests such as the California Bearing Ratio test the soil's confined bearing capacity and is useful when designing pavements. Others such as the direct shear test are useful when determining the angle of friction between a soil and the material in which the foundation is constructed Das (2014). The suitability of each test to a particular application is determined by a number of factors including;

- Type of soil (Cohesive of non-cohesive)
- Expected in service moisture content (saturated or unsaturated)
- Application (Pavement, foundation, pilings, etc)
- Stress type (shear, compression, etc)

The two most commonly used tests for evaluating the strength of soil in Australia are the Unconfined Compression Strength test (UCS) and the California Bearing Ratio (CBR). The UCS test is commonly used for clay specimens, Das (2014). The UCS directly tests the soil's undrained shear strength. The Undrained shear strength is necessary for determining the bearing capacity of foundations, dams and pilings. The confining pressure during a UCS test is zero and thus is only suitable for cohesive soils. During the UCS test a cylindrical test specimen is subjected to an axial load by a piston until failure occurs. The failure mode of the sample is recorded (shear or bulge), and the maximum load on the piston.

Figure 9- UCS(left) and CBR(right) testing machines- VJ tech (2019)

The California Bearing Ratio test is commonly used to determine the bearing capacity of subgrades and basecourses for road and pavements. The CBR test involves a compacted sample in a mould being subjected to a vertical loading by a piston in a testing machine. The load at certain penetration distances is measured and compared to that of a granular crushed rock. This result determines the relative strength of the sample. This test directly measures the pressure required to penetrate a soil sample with a piston of a known area.

Auststab (2016) notes that when designing lime stabilised subgrades, there are three procedures currently used, these are

- Austroads method using CBR and imperial design charts
- CBR using CIRCLY (a pavement design software)

- UCS, by Queensland Transport and Main Roads (QMTR)

Although the CBR test is the most commonly used, due to how useful it is for pavement design, the UCS test is also useful as it gives an accurate shear test result. The shear test result is useful for virtually all applications, not just pavements. It is also one of the fastest and cheapest methods for determining shear strength. Because both of these test methods are used in Australia, and they show improvements in strength in two different methods it was determined that both test methods should be used.

2.7 Common stabilisation methods

2.7.1 Lime and cement

Currently the most common method of soil stabilisation in Australia is through the use of lime or cement. White (2010) noted that the addition of lime to soil stabilises through cementitious reactions, due to the lime reacting with natural Pozzolans in the clay. The efficiency of this reaction can be negatively affected by factors in the soil such as high levels of organic carbon or a lack of natural pozzolans, Auststab (2012) describes the primary reaction of cement stabilisation in soils as one that occurs independently of the soil itself. That reaction is the hydration of the cementitious binder with the moisture in the soil. This reaction forms calcium silicate and aluminium hydrates. The secondary reaction occurs when natural pozzolans in the soil react with hydrated lime that is released during the initial reaction. Unlike the primary reaction, this is a slow reaction and can take a number of weeks and depends on moisture levels and temperature. Auststab (2012) also confirms what White said, and the presence of sulphates and organic materials may slow or cease this reaction. This effect is something that needed to be considered during this project. Auststab (2012) noted that that best results from stabilisation with secondary stabilisation occurs with a ratio of one-part lime to two part fly ash. This is of course a cementitious reaction and not a geopolymer, although this could provide a useful starting point for evaluating the effectiveness of a geopolymer binder.

The Austroads series of pavement design guides gives guidance on different methods of stabilisation used in Australian pavements. The primary goal of subgrade stabilisation is to improve the design CBR or modulus of the top of subgrade prior to the construction of the pavement (Austroads (2019). Austroads breaks down stabilisation techniques into pavement material treatments and subgrade treatments. Only subgrade treatments are applicable to the research presented in this paper. Austroads (2019) notes that for subgrade treatment lime and cement have two very different uses.

Category of stabilisation	Indicative laboratory strength after stabilisation	Binders adopted	Anticipated performance attributes(5, 4)
Subgrade and formation treatments			
Stabilised earthworks materials	1 < UCS(2,3) ≤ 2 MPa or CBR(1)	• Lime and/or cementitious binder (high plasticity soils) • Cement and/or cementitious binder (low plasticity soils)	• Improved constructability • Improved subgrade CBR and modulus • Improved shear strength • Reduced heave and shrinkage

Figure 10-Subgrade treatment options (Austroads 2019)

Austroads notes that lime stabilisation techniques are most commonly used to stabilise high plasticity soil. It goes on to mention that lime treatment improves the handling properties of cohesive soils, such as clays, this is primarily noticeable in the reduction of the soil's plasticity, this reduction in plasticity also results in increases to CBR and strength modulus. If long term improvements to these characteristics are required Austroads (2019) recommends that a higher

Table 1- Binder type selection (Austroads 2019)

Particle size	More than 25% passing 75 µm sieve			Less than 25% passing 75 µm sieve		
Plasticity index (PI)	PI ≤ 10	10 < PI < 20	PI ≥ 20	PI ≤ 6 & PI x %passing 75 µm ≤ 60	PI ≤ 10	PI > 10
Binder type						
Cement and cementitious blends(1,3)	Usually suitable	Doubtful	Usually not suitable	Usually suitable	Usually suitable	Usually suitable
Lime	Doubtful	Usually suitable	Usually suitable	Usually not suitable	Doubtful	Usually suitable
Bitumen	Doubtful	Doubtful	Usually not suitable	Usually suitable	Usually suitable	Usually not suitable
Bitumen/ lime blends	Usually suitable	Doubtful	Usually not suitable	Usually suitable	Usually suitable	Doubtful
Granular	Usually suitable	Usually not suitable	Usually not suitable	Usually suitable	Usually suitable	Doubtful
Dry powder polymers	Usually suitable	Usually suitable	Usually unsuitable	Usually suitable	Usually suitable	Usually not suitable
Other proprietary chemical products(2)	Usually not suitable	Usually suitable	Usually suitable	Usually not suitable	Doubtful	Usually suitable

binder content be adopted. Table 2.4 of Austroads (2019) provides a guide of which stabilisation method should be employed based on the properties of the soil. Expansive soils usually have a plasticity index of greater than 20 and most clays have more than 25% passing the 75µm sieve. It can be seen from the table that cement, bitumen, granular and dry powder polymers are not suitable for stabilising these soils. Lime is the only option recommended by Austroads for stabilising expansive soils. Austroads lists the effects of lime stabilisation of subgrades as:

- Increasing bearing capacity
- Reducing plasticity and seasonal swell and shrinkage
- Reducing moisture sensitivity
- Improving compatibility
- Reducing in situ moisture content to improve trafficability for construction.

The stabilisation of expansive soils is primarily concerned with the first three points. Lime is available in two different forms, quicklime and hydrated lime. Quicklime has several advantages over hydrated lime in stabilising expansive soils. Austroads (2019) notes that quicklime is significantly cheaper per tonne than hydrated lime and has a higher available lime content per unit mass. (1.00-1.32). Quicklime is also significantly heavier than hydrated lime, so storage and transportation costs are less. Quicklime is effective at drying out moist soil, although requires additional water if the moisture content of the soil is low.

Table 2-Lime comparison chart (Austroads 2019)

Property	Hydrated lime	Quicklime	Slurry lime
Chemical composition	$Ca(OH)_2$	CaO*	$Ca(OH)_2$
Form	Fine powder	Granular	Slurry
Equivalent $Ca(OH)_2$ per unit mass (available lime)	1.00	1.32	0.56 to 0.33**
Bulk density (t/m^3)	0.45 to 0.56	1.05	1.25

In order to determine the lime content required to achieve long term strength gains, Austroads (2019) outlines the procedure for the Lime Demand Test. The test has two methods, one using UCS and another using the CBR test to gauge the effectiveness of treatment. When using the CBR method to determine the effective lime content the standard notes that the soaked swell should be recorded as well as the bearing capacity. The lime content that achieves the required design CBR and swell reduction is then adopted for construction.

Lime stabilisation for subgrades is usually achieved using machinery purpose built for stabiliser application. A soil stabiliser is a vehicle with a powered metal drum with rows of mixing blades that break up the subgrade and mix it with powdered binder and water. Some machines are capable of adequately mixing the subgrade with the binder in one pass, while others may require up to four passes, depending on the power of the machinery and the plasticity of the soil. The machinery is the same regardless of the powdered binder used, so the same machine can be used for lime or cement stabilisation.

Figure 11- A soil stabiliser applying lime (Wirtgen 2019)

2.7.2 Fly ash

Many treatment options for expansive soils have been trialled with varying success rates. Traditional options include removal and replacement, prewetting, moisture barriers, surcharge loading and chemical stabilisation. Stabilisation methods that have proven useful in the past include, lime, cement, blast furnace slag, gypsum, rice husk and fly ash. As with all solutions there are trade-offs. Many chemical stabilisers such as fly ash, blast furnace slag and rice husks are waste products from other manufacturing processes. In the case of fly ash, it is produced during the combustion of coal to make electricity. Fly ash is collected by electrostatic precipitators before the flue gases escape the chimney. Fly ash is predominantly made up of silicon dioxide (SiO_2), aluminium oxide (AlO_3) and calcium oxide

(CaO), Anusha (2017). As fly ash is a waste material, there is an environmental benefit if it can be reclaimed and used for other purposes, as the carbon footprint is significantly lower. The world Business Council for Sustainable Development put the CO_2 cost for a tonne of Portland cement at between 0.8-1.0 tonnes. This is compared to fly ash geopolymers, which have a CO_2 cost between 0.2-0.4 tonnes.

Fly ash comes in two recognised classes, class C and F. Class C fly ash is produced from burning younger, sub-bituminous coals. Class C fly ash usually contains greater than 20% lime (CaO), Anusha (2017). Because of the high lime content this type of fly ash is self-cementing when in hydrated.

Property	Type I/II cement	Class C fly ash	Class F fly ash
SiO_2 (%)	19.4	38.4	59.7
Al_2O_3 (%)	4.5	18.7	30.2
Fe_2O_3 (%)	3.2	5.1	2.8
CaO (%)	62.3	24.6	0.7
MgO (%)	3.4	5.1	0.8
SO_3 (%)	2.9	1.4	0.02
Na_2O (%)	0.52 eq.	1.7	0.2
K_2O (%)	See Na_2O	0.6	2.4
Loss on ignition (%)	2.7	0.3	0.8
Density	3150 ± 10 kg/m^3	2630 kg/m^3	2160 kg/m^3

Figure 12-Bentz 2014

Class F fly ash is produced through the burning of older, harder anthracite and bituminous coal. The lime content of class F fly ash is less than 7%. Because of this low lime content, the fly ash is not self-cementing. Class F fly ash is classes as a pozzolan. Pozzolans are siliceous or aluminous materials which react with calcium hydroxide and water at room temperatures to form cementitious materials.

Class F fly ash can also be made into a geopolymer through the use of a chemical activator. Sodium silicate is the most commonly used geopolymer. Geopolymers are described as ceramic materials formed of long covalently bonded amorphous networks. Geopolymer cements are capable of hardening at room temperatures and are becoming a viable alternative to Portland cement. This geopolymer reaction can potentially be used to improve the performance of expansive soils.

2.7.3 Alkali-Activated Fly Ash

Alkali-activated fly ash stabilisation relies on a different mechanism to improve the properties of the soil. Black (2012) explains a geopolymer concrete as one that results from the reaction of a source material that is rich in silica and alumina with alkaline liquids. Class C fly ash, as discussed earlier, contains typically around 20% lime (CaO) and is therefore Cementous. Class F fly ash contains less than 7% lime and as a result is not cementitious, but a pozzolan. Khan (2018) stated that through a series of laboratory tests Class C fly ash geopolymer performed well for stabilizing highly expansive clay, although cement was still the best option purely from a performance perspective. Anusha (2017) confirmed this by conducting uniaxial compression tests with black cotton soil stabilised with varying ratios of fly ash to soil, both with and without an alkali activator. Curing times of 3,7 and 28 days were applied. The results showed that chemically activated samples achieved a strength 2.7 times greater than a purely fly ash stabilised sample. They concluded that better results were achieved by reducing the activator/ash ratio, which while not only improving mechanical strength results, also improved the cost effectiveness of the process.

Black (2012) conducted research into alkaline-activated class F fly ash mix processes and found that when the alkaline activator to fly ash ratio was increased beyond a certain point, there was a decrease in compressive strength of the geopolymer. He found this ratio to be in the region of 0.5-0.65. The experiments conducted by Black (2012) did not involve soil but purely testing the strength of the geopolymer exclusively.

Murmu (2018) conducted a series of tests on the stabilisation of black cotton soil using class F fly ash, both with and without an alkali activator of sodium hydroxide. The soil samples were first tested for Atterberg limits and particle size distribution. The soil was then prepared with 5, 10, 15 and 20% fly ash by weight. The samples were put through uniaxial compression tests as well as soaked and unsoaked CBR tests, with a curing period ranging from 0 to 90 days. The results showed that the greatest strength increases came from samples treated with the alkali activator, and that soaked samples showed the largest increase in strength during CBR testing. This was hypothesised to be because of the additional gel formation available during soaking. This allowed the solution to geopolymerise more efficiently.

2.8 The process of Geopolymerisation

The geopolymerisation process differs considerably from a cementitious reaction. Yun-Ming (2016) notes that the term geopolymer was first used in 1982 by Davidovits. The 'geo' portion of the word was chosen to represent the inorganic aluminosilicate used in the reaction, which is always geologically based. Geopolymers consist of two parts, a solid Binding material and a liquid alkaline activator. The solid aluminosilicates are usually waste products such as fly ash, blast furnace slag or clays. The liquid activator solutions are made of soluble metal alkalis such as sodium hydroxide or potassium hydroxide. A soluble source of silicates such as sodium silicate is added to provide silicates for the polymerisation.

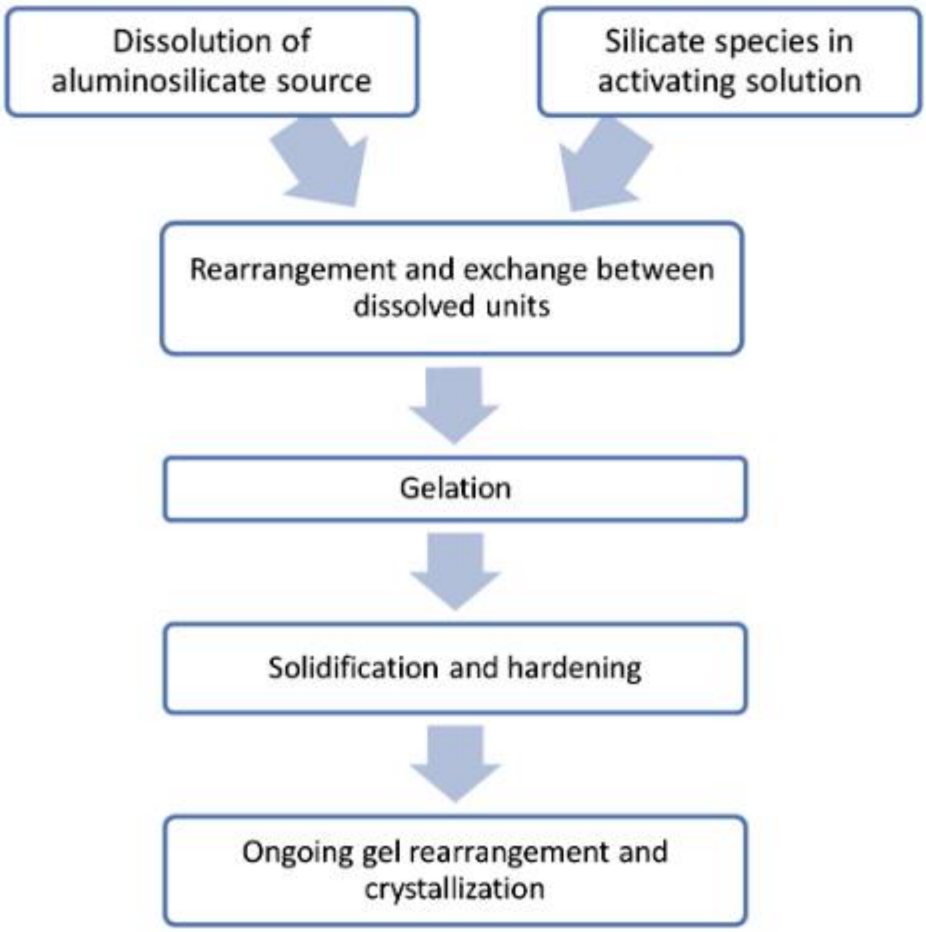

Figure 13- The geopolymerisation process (Yun-Ming 2016)

The geopolymer reaction begins with the Dissolution of aluminosilicates in the alkaline activator Yun-Ming (2016). The Hydroxyl ions in the alkali reactant facilitates this dissolution. These dissolved silica and alumina ions then form a gel as they coagulate into organised structures. This first gel phase, as noted by Yun-Ming (2016) consists of structures with a high Al content. This first gel phase continues to react as water is expelled from the solution during the reaction towards what is called the second gel phase. This second phase contains more Si than Al. After the second gel phase the structures start to link together in the crystallisation phase. The process is quite complicated and as Yun-Ming (2016) notes, the steps occur almost simultaneously, and it is impossible to isolate the steps of the reaction in experimental studies.

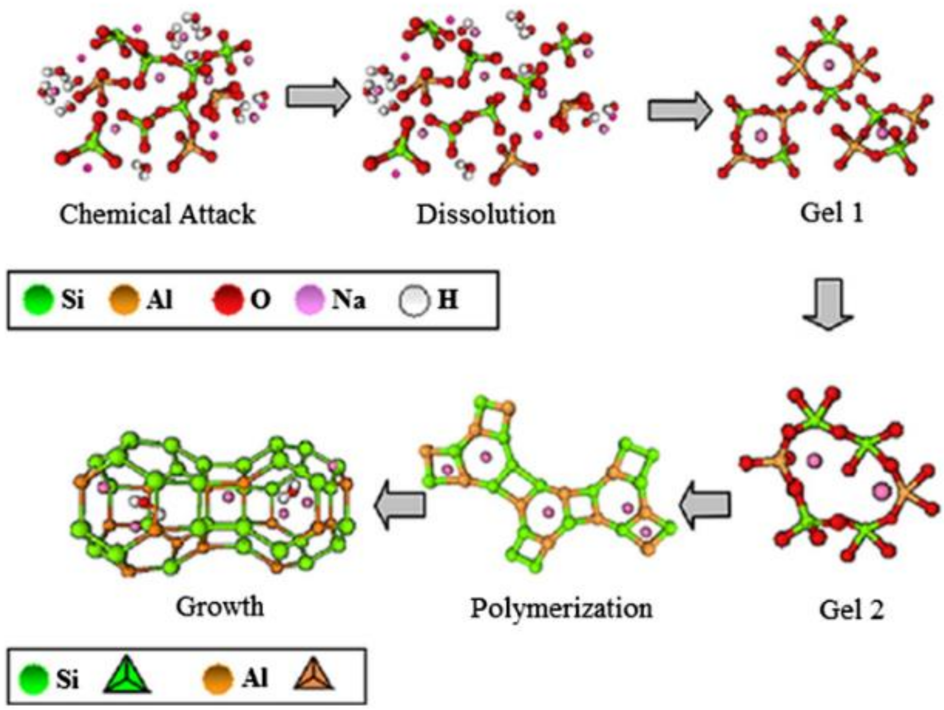

Figure 14- Molecular view of polymerisation (Yun-Ming 2016)

2.9 Fly Ash and Alkali Ratios

One of the most important aspects of the project is deciding on the ratios of Soil to Fly Ash, and the ratio of activator to Sodium Silicate. There are a number of reasons for this, including

- Strength Improvement
- Monetary Cost
- Difficulty of application
- Carbon Cost

In order for the geopolymer to be a viable solution for stabilizing expansive soils it must be a better alternative to other methods, such as cement or lime stabilisation. A literature review was undertaken to determine the correct ratios of fly ash to soil and sodium silicate to sodium hydroxide. The decision was made to determine these ratios via literature review rather than experimentation. This was due to the fact that there is substantial past research available on these ratios, giving a good level of reliability to the information provided. The other reason was that there was only a limited amount of testing that could be conducted in the time available. Each sample required several days curing time each time a moisture content was changed, as heavy clay has a very low permeability. The curing time for geopolymers also meant a substantial time between preparing and testing the samples.

Stabiliser	UCS (kPa)	PI	Dosage (%)	Reference
Cement	300–1200	65	8–16	Petry and Wohlgemuth (1987)
	1604	42.22	12	Amu, Fajobi, and Afekhuai (2005)
	970–1600	39, 11, 36	4–7	Kennedy, Smith, Holmgreen Jr, and Tahmoressi (1987)
	2500	19–41	3–5	Christensen (1969)
	750–2275	25, 42, 37	3–9	Bhattacharaj and Bhatty (2003)
Lime	170–720	16.8	6–14	Petry and Wohlgemuth (1987)
	230–2600	39, 11, 36	4–7	Kennedy et al. (1987)
	825–1175	25, 42, 37	3–9	Bhattacharaj and Bhatty (2003)
	841–188	50	3–19	Thyagaraj, Rao Sudhakar, Suresh, and Salini (2012)
	900	19–41	3–5	Christensen (1969)
	250–1500	54.7	3	Sridharan, Prashanth, and Sivapullaiah (1997)
Lime + FA	480–1880	35.53	4–8	Dahale, Nagarnaik, and Gajbhiye (2016)
	511–1830	54.7	3	Sridharan et al. (1997)
	34.73–63.38	13.34	8.5	Sharma, Swain, and Sahoo (2012)
	200–300	18.35	10	Kumar, Walia, and Bajaj (2007)
	2509–2570	7–37	25% (lime + FA)	Viskochil, Handy, and Davidson (1957)
Geopolymer	324–513	43	FA + 12M NaOH	Bagewadi and Rakaraddi (2015)
	6000–15000	34.8	VA + 30%KOH	Miao, Shen, Wang, and Luo (2017)
	1397–2706	46	FA + 5M NaOH	Present study

Figure 15- Geopolymer ratio results (Murmu 2018)

Murmu (2018) conducted a series of UCS and CBR tests on black cotton soil with varying ratios of soil to fly ash. Compared to other research, a much lower concentration of Sodium Hydroxide was used as an activator. They noted that the majority of research on alkali activated FA geopolymers focused on the use of high concentration (>10M) NaOH for strength development. Murmu (2018) noted that although the strength gains were substantial, the highly caustic nature of these concentrated solutions meant that they were uneconomical and unsafe to handle. It was for these reasons that Murmu (2018) used a 5M solution. They also noted that black cotton soils could be stabilised using 5%-20% FA to soil by weight. They concluded that FA geopolymer is effective at stabilising BCS even at low concentrations of NaO. They also found that the liquid limit and plastic limit improved with the addition of fly ash at ranges of 5%-20%. The key finding of the research was that the strength developed after 7 days curing was much higher than the minimum strength requirement for sub-base. This shows that suitable strength values can be obtained with lower concentrations of alkaline activators and fly ash, resulting in a cheaper and safer to apply treatment.

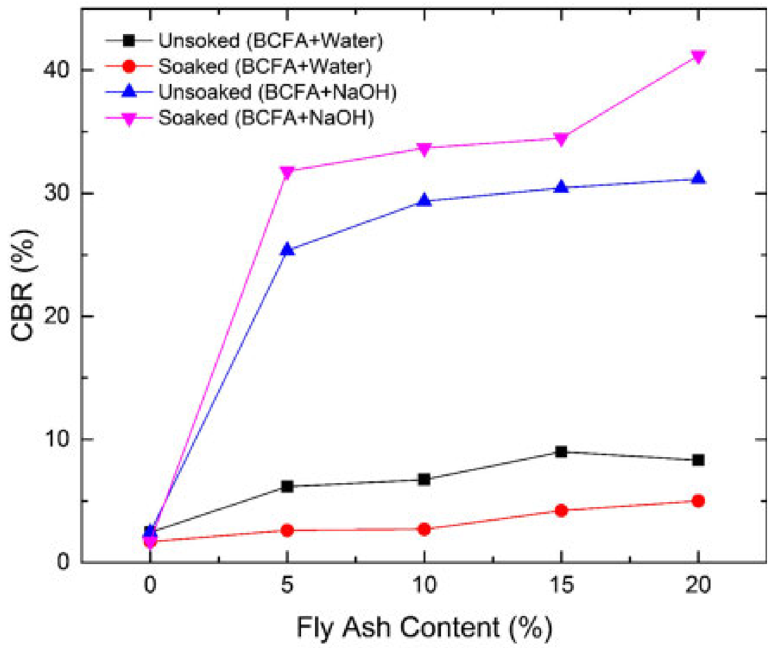

Figure 16- CBR with fly ash content (Murmu 2018)

Anusha (2017) used higher percentages of FA to soil , with 20%-40% FA by weight, with strength increasing with FA content. Although the strength increased, it was noted that it was not linear, and the addition of 20% FA provided adequate strength improvements for most applications. They also noted that there was a strong dependency between activator to ash ratio and strength and found it was advantageous to reduce it, which in turn lowers the cost.

Morsy et al (2014) Conducted research into the Effect of Sodium Silicate to Sodium Hydroxide Ratios on fly ash geopolymers. They experimented with ratios of Na_2SiO_3:NaOH between 0.5 and 2.5. They found that the compressive strength was at a maximum when the ratio was 1:1. The increase was sharp between 0.5 and 1.0, then dropped off at a slower rate between 1.0 and 2.5. Their experiments used a mixture of fly ash and sand, at a ratio of 0.5. Although the strength of the 1.0 ratio was the highest after a 28 day curing time, it was noted that all other ratios had a higher initial strength gain. They attributed this slow gain in strength to the silica and alumina in the fly ash being dissolved in the alkaline activator solution, which accelerated the polymerisation process. Morsy et al (2014) also found that this gave a more gradual release of the silica during the reaction in the gel phase. They found that the increase in Na_2SiO_3:NaOH from 0.5 to 1.0 resulted in the increase of sodium in the mixture, which is important for the creation of geopolymers as it acts as charge balancing ions. They also noted that after the Na_2SiO_3:NaOH ratio increased beyond 1.0, the excess sodium slowed water evaporation and geopolymer structure formation.

2.10 Gaps in Knowledge

Soil stabilisation of expansive soils has been extensively studied, as the potential damage from underestimating their impact to pavements and structures can be devastating. There has been large amount of research into what causes a soil to be expansive and methods of controlling this volume change. Different methods have been trialled to alter the characteristics that causes these changes. Given that lime stabilisation is easily applied, and lime production is a heavily established industry, it has become the industry standard for stabilisation of these difficult soils. Lime stabilisation does have disadvantages, namely lime is a relatively expensive material when needed in large quantities. With the heavy focus on increasing the sustainability, the high carbon cost of lime and cement production has become a concern for governments and industry. The production of electricity from coal has also resulted in a large amount of fly ash being produced as waste. There has been considerable research into potential uses for this waste product, and one of them is geopolymer cements and binders.

The use of geopolymers to stabilise expansive soils has been researched in recent years, and promising results have been obtained. In Australia, there has not been much research done on fly ash geopolymer stabilisation and the national guidelines for pavement design are focused on traditional methods of stabilisation such as lime, cement and granular options. As there is no guidance on geopolymer stabilisation, Australian expansive soils should be tested to see if treatments are suitable from a technical perspective. If treatments are successful, pavements should be designed using geopolymer stabilisation instead of the methods outlined in the standards. This paper will examine the effects of geopolymer stabilisation in an expansive soil found in the South East Queensland region and design pavements for a variety of applications using the current Austroads standards. The viability in Australia of fly ash geopolymer stabilisation of expansive soils will be examined from a technical, practical and financial perspective.

2.11 Summary

The literature review provided a lot of information into the identification of expansive soils and their treatment options. The characteristics of expansive soils were explored and the best tests to identify them were found. It was found throughout the literature that the behaviour and identification of expansive soils can be hard to predict, with a huge number of variables finally dictating how a soil will behave. Given this, there are a couple of indicators that were found to be more representative than others. These were the following

- Plasticity index
- Liquid Limit

The preliminary testing regime chosen will determine these properties. Changes in these properties will be noted after treatment as be used to examine whether the treatment was successful.

This literature review also examined the treatment options available for expansive soils. By gaining an understanding of these practices, the effectiveness of the fly ash treatment could be properly evaluated. The performance of these options, as well as their respective costs and difficulty of application gave a greater understanding of the current state of stabilisation technique available. This information will be useful in determining if the alkali activated fly ash treatments will be practically viable.

The literature review also provided a large amount of information regarding the optimum mixtures for alkaline activators and fly ash to soil ratios. This was important information as it was vital to designing the most effective experiment with the time available for testing. It was important to balance the performance increases associated with polymerisation, while keeping the costs to a minimum. The literature indicated that good results could be obtained by using a 20% by weight Fly Ash to Soil Dry weight. It was important to determine the minimum amount of fly ash that would achieve a good result as costs would be reduced in the real-world application. Costs would be reduced via two mechanisms, the reduced amount of fly ash that needed to be purchased and also the reduced amount of original soil that would have to be removed to account for the addition of fly ash.

There are design standards available in Australia that deal with the stabilisation of expansive soils for pavements. These standards outline various methods but lack any guide for stabilisation using geopolymers. Using these design guides, pavements should be designed using geopolymers, but following the performance indicators set when using other methods. This will ensure that the design process is as rigorous as other methods, and pavements built over these stabilised soils perform just as well over time.

The research also indicted that although a highly concentrated alkaline activator solution does result in higher strength, the increase was not sufficient to warrant its use. Good strength increases were found with lower concentrations (5M) concentrations of Sodium Hydroxide. This lower concentration meant that costs could be further reduced, but more importantly, the risk associated with handling caustic materials could be reduced. This increased safety level was important not just for the project, but also in real world application of the technique. The safety consideration would be a limiting factor for many countries that place a high importance on Workplace Health and Safety (WH&S), such as Australia. The ratio of sodium silicate to sodium hydroxide was determined to be most effective at a 1:1 ratio.

CHAPTER 3

METHODOLOGY

3.1 Introduction

This section will outline the methodology chosen for the laboratory testing. It will explain the collection and preparation of the initial soil and preliminary testing. The treatment method for the soil will be explained as well as tests for the treated samples.

3.2 Location of Samples

The sample soil was taken from an address in Marburg, a small township in the Ipswich City Council. The coordinates of the sample location were 27”33’46 South, 152”35’45 East. The elevation was 77m above sea level. The location was from an open park 50m from a floodway and natural creek. The site was chosen was well known to the author and presented with many symptoms of expansive soil. Nelson and Miller (1992) note that one of the most reliable methods of determining soil is through observation of the soil in its natural state and the effects on structures built on the soil.

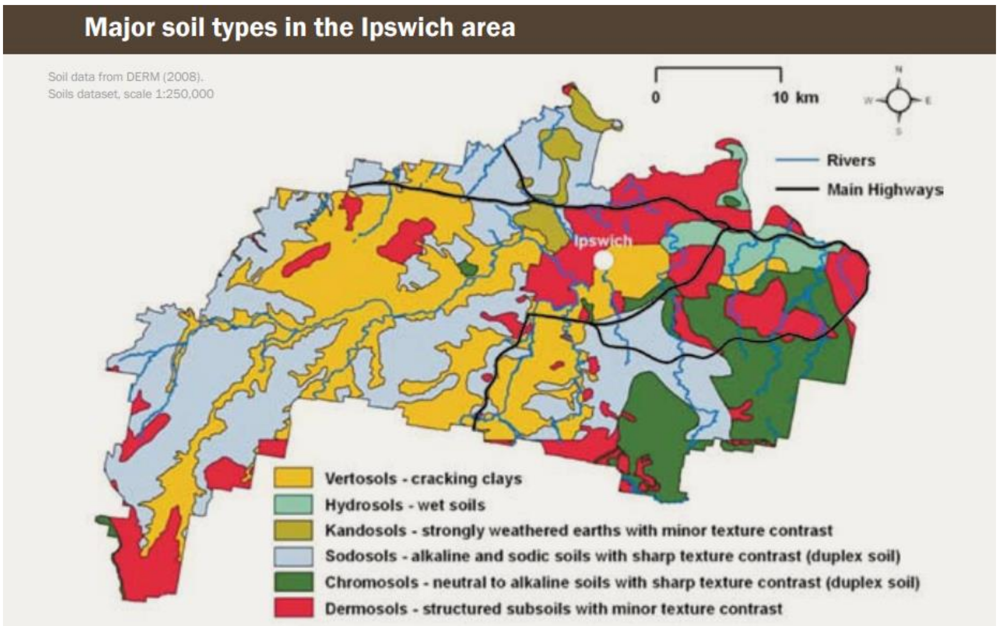

Figure 17- Soil types in Ipswich (ICC 2016)

Figure 18-Location of Sample, Google Earth (2019)

The soil was in a very dry state, with large cracks across the surface. On inserting a ruler into the cracks, some were found to be at least 50cm deep. The author also noted that the house built on the site has suffered extensive movement as the soil's moisture content changes throughout the seasons. This movement has resulted in cracking of concrete slabs, movement of a masonry fireplace and the need for the house to be re levelled annually. The public infrastructure in the area also has noticeable damage from soil movement. Footpaths, guttering and roads are extensively cracked, and more worryingly, the local highway overpass recently had to be repaired after less than 20 years of operation. The repairs to the overpass were required due to moisture ingress around the foundations after cracking caused by soil expansion. According to the Ipswich city council the area is dominated by Vertosols, a type of cracking clay. This was consistent with the observations of the author.

3.3 Collection of Samples

The samples were collected from the test site using a petrol-powered auger with varying flute sizes. The collection was performed in accordance with AS1289 1.2.1. The samples were taken from a depth of 200-800mm and during drilling the state of the soil was closely monitored to ensure that the soil type and condition was homogeneous and different layers were not collected in the same sample. For the entirety of the sample depth, the soil was one layer. This layer was a hard, black and dense soil with no visible larger particles. During drilling the cut surfaces were smooth shiny and came off in flakes. There was difficulty in drilling the sample due to how hard and sticky it was, and great care had to be taken to ensure the auger did not get stuck. A total of approximately 40kg of soil was taken to ensure enough was available for testing and additional samples would not be required. This ensured that all soil used for testing would have the same in situ moisture content and condition. The collected soil was sealed in in airtight plastic containers to ensure the moisture content remained stable.

3.4 Laboratory Testing of Untreated Samples

The testing of the collected samples was conducted in the soil laboratory of the USQ Toowoomba campus. This laboratory contained all the necessary tools and equipment for the testing of samples to be conducted in accordance with the relevant Australian Standards or procedures. The tests to be conducted will include

- Particle size analysis
- Determination of moisture content
- Liquid Limit
- Plastic Limit
- Plasticity Index
- Linear Shrinkage
- Optimum Moisture Content & Maximum Dry Density
- California Bearing Ratio
- Uniaxial Compressive Strength Testing

3.4.1 Determination of In-Situ Moisture Content

The moisture content was determined by following the procedure outlined in AS 12892.1.1. Both the oven and microwave method were used to determine if both were accurate methods. For the oven method a sample was taken and dried in an oven not exceeding 55 degrees Celsius. This lower temperature was to ensure that any delicate organics present did not breakdown. After the sample was dry it was then weighed. This process was repeated until the sample weight showed no further change. The moisture content was then calculated by taking the difference in the wet and dry soil and dividing that by the mass of the wet soil. The microwave method involved drying the sample in a microwave in two-minute intervals and weighing the sample. This process was repeated until there was no further change in weight. The same calculation for moisture content as the oven drying method was then used to determine moisture content.

3.4.2 Particle Size Analysis

The preparation of samples for laboratory testing was conducted in accordance with AS 1289.1.1, and the particle size analysis was conducted in accordance with AS 1289.3.6.1. The dried soil sample was sieved to determine particle size distribution. A 200g sample was analysed for particle size and 100% of the sample passed through the 2.36mm sieve. Due to the small particle size, wet sieving was used to further analyse the sample, as hydrometer analysis would have taken too long and the USQ Toowoomba labs did not have the equipment available to conduct the test. The wet sieving resulted with almost 100% of the sample passing the 425um sieve. According to AS 1289.1.1, if all the sample passes the 425um sieve than the soil is suitable to be used in its natural state for all Atterberg limits, as well as all other testing required by this project.

3.4.3 Liquid Limit Testing

Determination of the liquid limit (LL) was conducted in accordance with AS 1289.3.1.1, using the four-point Casagrande method. Water was added to a 300g sample of soil and it was mixed with spatulas to ensure that the sample was a completely homogeneous mix. After the desired consistency was established the soil was cured for four days in an airtight container. This curing time is established for clay samples to ensure that the moisture content is consistent throughout the entire sample.

The Casagrande bowl liquid limit apparatus was first inspected for serviceability in accordance with the standard. The tip of the spatula was measured to ensure the tip was less than 2.0mm thick. A small pat of soil was put in the bowl at a depth less than 10mm and parallel to the base. A groove was cut into the sample with the spatula and the crank handle was turned to start knocking the bowl against the rubber base. The amount of turns it took to close the groove in a 10mm length was recorded for the sample.

Figure 19- Casagrande bowl liquid limit test (Author 2019)

The four-point test required four data points, evenly spaced between 40 and 15 blows, so water was added to the sample until grove closure occurred at 40 blows. The moisture content of that sample was determined and recorded. This was the starting point for the recording. Water was added to another sample from the cured soil and mixed until the consistency was adequate. The test was the repeated until the soil sample attained a 10mm groove closure with 32 blows of the apparatus. When this result was obtained twice in a row with no more than one blow change in result the moisture content was obtained for this sample and the test was repeated. In order to obtain even spacing for the test, two more samples were taken, at 23 blows and 16 blows. The moisture contents at these points were calculated and recorded. With a complete data set the results were plotted on a semi-log graph, with the number of blows recorded on a logarithmic scale and the moisture recorded on a linear scale. The moisture content for a groove closure of 25 blows was determined from the chart and this point was deemed to be the Liquid Limit (LL) of the sample.

3.4.4 Plastic Limit Testing

The plastic limit of the soil was determined in accordance with AS 1289.3.2.1. A small sample of cured soil (8g) was taken and rolled in the tester's hands until small cracks appeared on its surface. This ball was then rolled on a frosted glass plate to form threads with a diameter of 3mm. If the threads crumbled before reaching 3mm more water was added. If the threads formed threads with a diameter less than 3mm without crumbling they were worked by hand until the moisture content reduced enough for the sample to crumble at the 3mm diameter. When 5-20 grams of suitable threads were collected, they were weighed and then the moisture content was determined as in section 3.4.1. The moisture content was recorded and the average across three collections of threads was determined to be the plastic limit.

3.4.5 Plasticity Index

The plasticity index was calculated using AS 1289.3.3.1. This was a simple calculation, where the plasticity index is the difference between the liquid limit and the plastic limit. This is described as the range of water contents that a soil exhibits plastic behaviour.

$$PI=LL-PL$$

3.4.6 Linear Shrinkage

Linear shrinkage was determined in accordance with AS 1289.3.4.1. The soil that had been cured for liquid limit testing had water added and was again mixed to a smooth consistency, as with the liquid limit tests. Water was added until the sample took 25±3 blows for groove closure. This moisture content is consistent with the liquid limit of the soil. The sample was then placed into brass moulds, of a semi cylindrical shape with a length of 100mm and an internal diameter of 15mm. The moulds were greased with petroleum jelly before placement of the sample to avoid the sample sticking to the mould. The sample was then air dried for three days at room temperature until shrinkage stopped and a colour change was noted. The change in sample length was measured and then the sample was placed into an oven at 105 degrees Celsius. It was then measured again to check that further shrinkage had not occurred. The final length of the dry sample was taken, and the percentage shrinkage was calculated to find the shrinkage limit.

3.4.7 Optimum Moisture Content and Maximum Dry Density

To find the optimum moisture content and maximum dry density AS 1289 5.2.1 and 5.2.2 respectively were used to find these characteristics. These values were determined by taking samples of soil at varying moisture contents either side of the estimated optimum level and compacting them into a cylindrical mould of dimensions 115mm (H) x 105mm (dia). The soil samples for each moisture content were mixed thoroughly and left to cure for a period of four days, in ensure the moisture content was even across the entire sample. In order to estimate the optimum moisture content for the sample, Australian Standards 1289 recommended that a starting point of 2-3% less than the plastic limit. Nelson and Miller (1992) notes that the highly plastic inorganic clays usually have an OMC of 36-19. This is quite a broad range and was only used as a guide. With a plastic limit of 22%, the testing started by estimating the OMC at 20% moisture content. Samples were prepared at moisture contents of 17% , 20%, 22%, 25% and 28%. These samples were compacted into layers, with each layer being measured to ensure that the three layers are as effectively compacted as possible. This measurement is important to ensure that each sample is equally compacted. After the compaction was completed the sample, mould and baseplate were weighed and the mass of soil was recorded.

Table 3- Specifications for standard compaction (AS1289 2019)

Specification for Standard Compaction	
No of Layers	3
Rammer mass (kg)	2.7 ± 0.01
Height of drop (mm)	300 ± 2.0
No. of uniformly distributed blows per layer	25 for one litre mould

This process was repeated for each of the moisture contents and the results recorded. The results were graphed with moisture on the horizontal axis and dry density on the vertical axis. According to AS 1289, the OMC can be determined when there is at least two measurements wetter and two drier than the maximum dry density. The OMC was determined by finding the moisture content at which the dry density was at a maximum.

3.4.8 California Bearing Ratio Test

The CBR testing was conducted in accordance with AS 1289.6.1.1. To ensure reliability and redundancy three samples were prepared and tested. The soil for use in the test was bought to the calculated optimum moisture content of 20% and thoroughly mixed to ensure the moisture was evenly distributed throughout the sample. The soil was then cured for four days. This curing time was stipulated in the standard and was to ensure the moisture content was uniform. This long curing time was due to the low permeability of the heavy clay.

After curing the soil, the moisture content was checked to ensure that it was at the OMC. After this was confirmed the samples were prepared in the same manner as the maximum dry density samples. To ensure consistency, the samples were weighed to confirm they had been compacted to within 2% of the maximum dry density. Because expansive clays are particularly weak at higher moisture contents, only soaked CBR tests were conducted, as this was the soil condition critical to failure modes.

Figure 20- Zeroing dial gauge for swell reading (Author 2019)

Before soaking, a tripod and dial gauge were zeroed on each sample, to record vertical swell after soaking. The prepared samples were soaked in a water bath for four days before being drained for 15 minutes and tested. Before testing the dial gauge was checked and the vertical swell for each sample was recorded to the nearest 0.1mm. The tests were conducted in the manual CBR test machine in the soil laboratory at USQ's Toowoomba campus. The load on the test piston and the penetration depth were recorded in accordance with the test procedure and plotted to obtain the maximum bearing capacity and therefore the CBR of the sample. The moisture content of the soaked samples were taken after testing and recorded.

3.4.9 Uniaxial Compression Testing

UCS testing was conducted using AS 1289.6.4.1. A soil sample was prepared at the optimum moisture content, as in section 3.4.2. After being left to cure for four days the sample was compacted into a cylindrical mould of 51mm diameter. The soil was compacted in three equal layers using a specified number of hammer blows and then cut off to a height of 100mm.

Table 4- Specifications for standard compactions (AS1289 2019)

Specification for Standard Compaction	
No of Layers	3
Rammer mass (kg)	2.7 ± 0.01
Height of drop (mm)	300 ± 2.0
No. of uniformly distributed blows per layer	25 for one litre mould

The prepared samples were weighed and placed into a LoadTrac 2 test machine. The sample was seated and pre-loaded with no more than 50N of force. This preloading ensured that the sample was properly seated in the test rig. The test was then started and ran at the rate of compression set in the standard. The results were plotted, and the maximum uniaxial strength was recorded as the point where the maximum load was detected on the loading cylinder. The test was repeated three times to ensure accuracy.

3.4.10 Free Swell Testing

The free swell index for the soil was determined using the free swell index test, as per IS:2720. Two 10g samples of both the treated and untreated soil were oven dried and sieved through a 425µm sieve. Each dried sample was poured into a 100ml graduated glass cylinder. The volume of each of these cylinders was recorded. One each of the treated and untreated samples were covered with distilled water up to the 100mm mark, and entrapped air was removed by gently stirring with a glass rod. The remaining two samples were topped up to the 100ml mark with kerosene and entrapped air bubbles were removed in the same manner. The samples were left for 48 hours and then the final volumes of each of the samples was recorded.

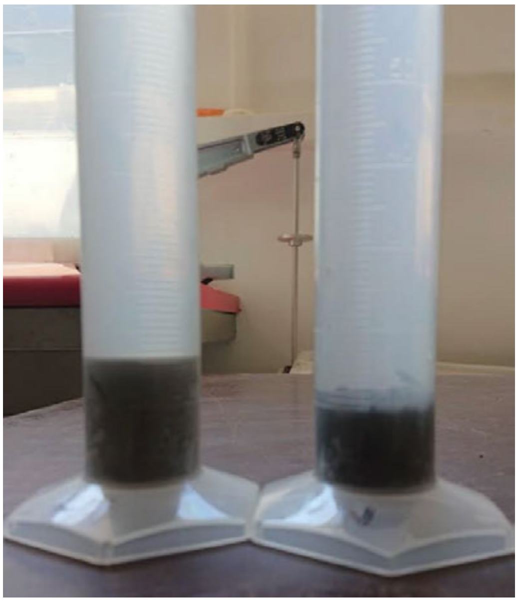

Figure 21- Free swell test (Author 2019)

3.5 Treatment and Testing of Soil

In order to determine if fly ash geopolymer stabilisation is a viable alternative to other methods, the methods for determining the long-term strength of lime stabilisation needed to be examined. Austroads (2017) notes that for lime stabilised subgrades, the structural thickness design procedures are based on the design CBR and the design modulus assigned to the stabilised subgrade. For pavement design using the Austroads methods, the stabilised subgrade can only be assigned a design CBR value not exceeding 15%. Although lime stabilisation of soil can result in higher CBRs, pavement design only allows for a maximum of 15%. In order for the testing to be determined as successful, a design CBR of 15% should be achieved. Further improvements in bearing capacity are desirable from a practical point of view, there is no difference in the design standards and will be considered suitable as an alternative to lime stabilisation. It is for this reason that lime stabilisation was not tested during the research. The aim of the project was to determine whether fly ash geopolymers were a viable alternative to other methods of stabilisation, and if they met the 15% CBR result they were considered viable from a technical perspective.

The treatment of samples required the addition of fly ash, sodium silicate and sodium hydroxide to the soil. The addition of fly ash would alter the soil's optimum moisture content as well as the maximum dry density. Before CBR tests could be conducted the optimum moisture content and maximum dry density of the fly ash and soil mixture had to be found. Class F fly ash was added to the soil, at a 20% of dry soil weight ratio. The fly ash was thoroughly mixed through the soil until the mixture was consistent. The literature indicated that the addition of 20% fly ash usually resulted in a 2-3% drop in OMC so this was used as a starting point for determining the OMC and MDD. The fly ash soil mixture was divided into five samples, and each was bought to a different moisture level to determine the OMC. The percentages used were 16%, 18%, 20%, 22% and 24%. The OMC procedure outlined in 3.4.7 was repeated until the OMC and MDD was found.

With the OMC of 20% calculated, the additional water required to reach this moisture content was calculated and a 5 molar concentration of sodium hydroxide solution was prepared using the risk assessed safety procedure prepared as part of the project specification (see annex A) The 1:1 ratio of sodium hydroxide to sodium silicate was difficult to dissolve as the highly caustic solution caused the dissolved sodium silicate to crystalize out of solution. This problem had been noted in other research so was not unexpected. The undissolved sodium silicate was mixed through the soil where it would be available for the polymerisation reaction during curing.

Figure 22- Clay sample before and after addition of fly ash (Author 2019)

After the preparation of the treated soil, a suitable amount was set aside for curing and use for Atterberg limit testing. The rest of the treated soil was prepared for CBR and UCS testing. These were prepared using the same methods as the untreated samples. After preparation, the samples were placed in airtight containers and allowed to cure for 28 days. During the curing stage, the temperature was not strictly controlled, although the temperature ranges were not extreme as the laboratory is heated and air conditioned, therefore changes to curing time from extreme cold or heat were ruled out. The CBR test

After the 28 day curing time, the treated samples were visually inspected and then retested using identical test methods to the untreated samples. Using the same test methods ensured that the results were as consistent as possible. The liquid limit, Plastic limit, Linear shrinkage, CBR testing and UCS testing were repeated and results recorded.

3.6 Cost Benefit Analysis

The cost benefit analysis was performed for both the fiscal cost, and the greenhouse gas cost. The fiscal cost benefit was conducted in order to ascertain if the treatment would be economically viable, provided the improvements in strength are sufficient. The greenhouse gas cost comparison was conducted to determine any benefit sustainability that this treatment could provide. These two factors were determined using past research and are discussed in Chapter six.

3.7 Laboratory Safety

To meet Workplace Health and Safety requirements a risk assessment was created using the university Risk management procedure (RMP). The RMP has been designed to comply with the *Work Health and Safety Act 2011 (QLD).* The risk management process focuses on-

- Identifying hazards
- Understanding the likelihood and potential consequences of the hazards (Risks)
- Reviewing current or planned approaches to controlling risks
- Adding new control measures where required.

This project had two distinct phases where risks could be present. The sample collection phase and the laboratory testing phase. The sample collection phase involved the collection of soil using hand tools and a petrol-powered auger. There was a risk when using hand tools which was mitigated by using appropriate personal protective equipment (PPE). This PPE included gloves, goggles, steel capped shoes and long pants. The petrol-powered auger was operated by a qualified technician with appropriate PPE, which included hearing protection.

The second phase of the project involves the laboratory testing of samples. As this phase was more complicated, a Risk management Plan was raised on the university's RMP Share point register. Several hazards were identified, and their risks were evaluated using the risk management matrix on the site.

Table 5- Risk assessment matrix (USQ 2019)

Risk Matrix					
	Consequence				
Probability	Insignificant No Injury 0-$5K	Minor First Aid $5K-$50K	Moderate Med Treatment $50K-$100K	Major Serious Injury $100K-$250K	Catastrophic Death More than $250K
Almost Certain 1 in 2	M	H	E	E	E
Likely 1 in 100	M	H	H	E	E
Possible 1 in 1,000	L	M	H	H	H
Unlikely 1 in 10,000	L	L	M	M	M
Rare 1 in 1,000,000	L	L	L	L	L
Recommended Action Guide					
Extreme:	E= Extreme Risk – Task ***MUST NOT*** proceed				
High:	H = High Risk – Special Procedures Required (Contact USQSafe) Approval by VC only				
Medium:	M= Medium Risk - A Risk Management Plan/Safe Work Method Statement is required				
Low:	L= Low Risk - Manage by routine procedures.				

There were two broad categories, the first being mechanical hazards such as crushing and dropping while preparing samples and using the test equipment. The second were chemical hazards, from the use of sodium silicate and sodium hydroxide. Safety Data sheets were consulted so the risks and mitigation measures could be accurately assessed. Sodium hydroxide is highly caustic and as a result needed to be handled carefully, using the appropriate PPE. The alkalinity of the activator solution could be minimised by finding the lowest effective molar concentration for polymerisation. This lowered the risk of chemical burns when mixing. Sodium silicate is safe when in a liquid solution, although skin contact should be avoided as it can be an irritant.

Risk Register and Analysis

	Step 1	Step 2	Step 2a	Step 3				Step 4					
	Hazards: From step 1 or more if identified	*The Risk:* What can happen if exposed to the hazard with existing controls in place?	*Existing Controls:* What are the existing controls that are already in place?	*Risk Assessment:* Consequence x Probability = Risk Level				*Additional Controls:* Enter additional controls if required to reduce the risk level	*Risk assessment with additional controls:* Has the consequence or probability changed?				
				Consequence	*Probability*	*Risk Level*	*ALARP*		*Consequence*	*Probability*	*Risk Level*	*ALARP*	
	Example												
	Working in temperatures over 35^0 C	*Heat stress/heat stroke/exhaustion leading to serious personal injury/death*	*Regular breaks, chilled water available, loose clothing, fatigue management policy.*	*catastrophic*	*possible*	*high*	*No*	*temporary shade shelters, essential tasks only, close supervision, buddy system*	*catastrophic*	*unlikely*	*mod*	*Yes*	
1	Sodium Silicate	Causes eye irritation	wear eye protection, gloves and all PPE in accordance with SDS. Eye wash station available.	Minor	Rare	Low							!
2	CBR and shrink swell machine	Drop injury to feet from heavy sample mould	Wear steel capped boots, prepare sample in accordance with standards	Minor	Rare	Low							!
3	sodium hydroxide	Cause eye and skin burns	Wear eye protection, gloves and full body coverings as recommended by SDS.	Moder	Rare	Low							!
4	Preparing samples with compression equipment	Getting fingers of other body parts jammed in compression tool	Be careful when conducting experiment	Minor	Unlik	Low							!

CHAPTER 4

RESULTS

4.1 Introduction

This chapter will present the results from the laboratory tests. Results for the treated and untreated samples will be presented together for the same test, in order to show the effects of treatment on the soil. Due to time restraints some experiments were only conducted once, while other more important tests, such as UCS and CBR were conducted multiple times, to ensure accuracy.

4.2 In-Situ Moisture Content

The moisture content of the soil was tested using AS 1289.2.1.1. Two samples were taken at depths of 300mm and 700mm, to check if the moisture level differed for the same soil type at different depths. The average in-situ moisture content was 17.5%.

4.3 Particle Size Distribution

Particle size distribution for the sample was conducted according to Australian Standards 1289.3.6.1. Initially a dry sieve was conducted, but after the entire sample passed the 2.00mm sieve it was determined that wet sieving was necessary due to the small particle size of the natural soil. Wet sieving was conducted through the No 40 sieve (425 micron). The sample had >98% of material passing the 425 micron sieve, and as such it was suitable for use in all laboratory tests in its natural state. Further analysis to determine the percentage clay content could not be conducted as there was not time of resources available to conduct hydrometer testing.

4.4 Liquid Limit Testing

The liquid limit testing was conducted in accordance with AS1289.3.1.1, the untreated samples had a liquid limit of 44%. This is within the range normally agreed upon for expansive clays. It is perhaps in the low range, but it is still classified as an expected LL for soils with a high degree of expansivity. After treatment the liquid limit decreased to 36%. This is a decrease of 8%, which is significant from a performance perspective. Both of these results were obtained with a high degree of certainty due to the choice of the four-point method rather than the single point. During the liquid limit testing of the treated samples it was noticed by the technician conducting the test that the soil was much easier to work with, exhibiting better workability and increased permeability.

Table 6- Liquid limit testing results (Author 2019)

Untreated	
Blows	Moisture content
41	42.5
32	43.2
23	44
16	45

Liquid Limit	44%

20% fly ash+5mol NaOH and NaSIO3	
Blows	Moisture content
41	35.5
32	35.9
20	36.2
15	37

Liquid Limit	36%

4.5 Plastic Limit Testing

The plastic limits were found using AS 1289.3.2.1. This method can be prone to errors, mainly due to the person conducting the test not being experienced enough to correctly gauge the crumbling point of the rods. The treated and untreated samples were tested using the same equipment in the same lab under supervision of the technical staff in order to reduce the chance of error. The untreated sample had a plastic limit of 20%. After treatment the plastic limit increased to 25%. This increase of 5% is significant and showed a discernible difference in the performance of the soil. During testing of the treated sample there were large differences in the physical appearance and performance of the soil. The treated sample was noticeably harder and more granular than the soil in its natural state. The workability of the soil was better, prior to treatment the soil was very sticky and slick when wet and took a lot of working to ensure the water added was distributed evenly. After treatment the soil was less sticky and had noticeable larger grains throughout the sample. On addition of water it more readily mixed through the sample and was far more user friendly. Although these observations are not objective, the soil was found to be far easier to work with.

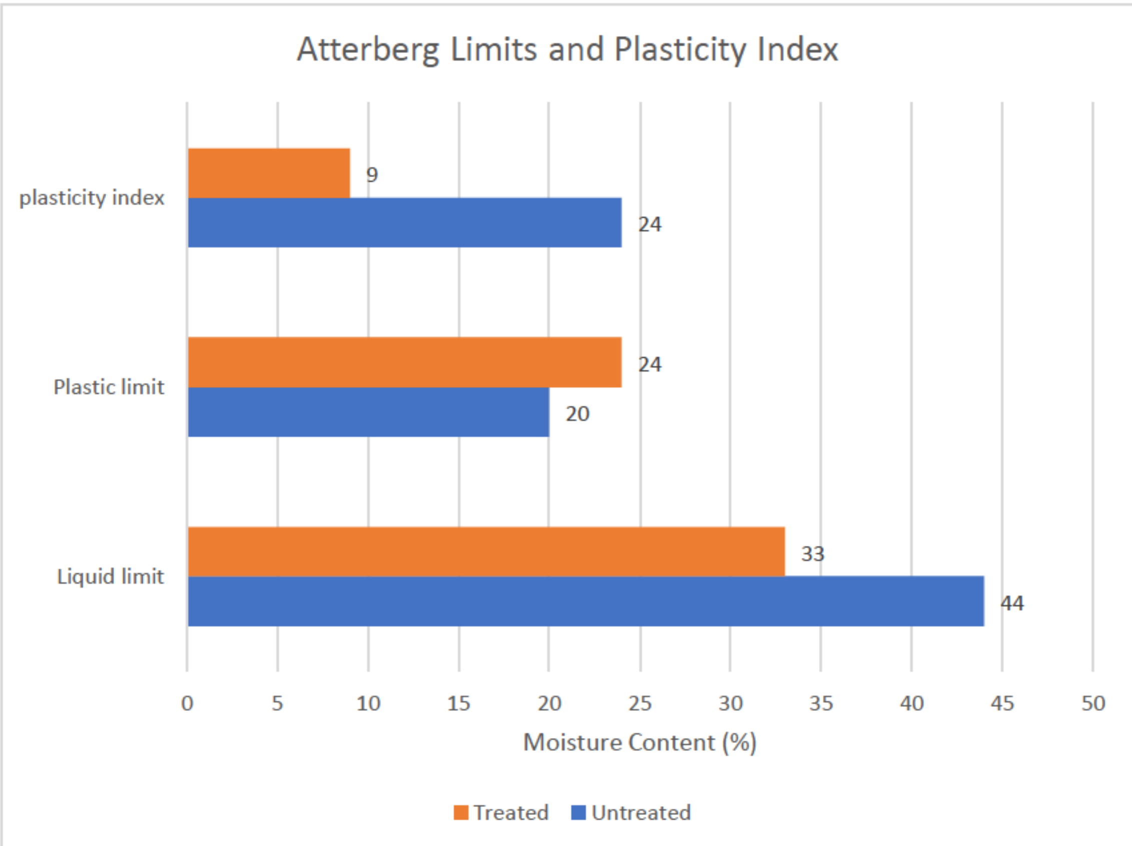

Figure 23- Atterberg limit results (Author 2019)

4.6 Linear Shrinkage Limit

The shrinkage limits were conducted in accordance with AS 1289.3.4.1. The untreated samples shrunk considerably, with an average of 21%. This is on the high end of the shrinkage scale and represents a soil with a critical level of expansion, according to Altmeyer (1955). After treatment the linear shrinkage reduced to an average of 11%. This is still considered to be a high level of shrinkage, although the treatment did result in almost halving the linear shrinkage. This substantial reduction shows a large improvement in the soil.

Figure 24-Linear shrinkage comparison

4.7 Maximum Dry Density/Optimum Moisture Content

The maximum dry density and optimum moisture content was conducted in accordance with AS 1289 5.2.1 and 5.2.2. The maximum dry density came to a value of 1559.6 kg/m^3 . This maximum dry density coincided with a moisture content of 20%.

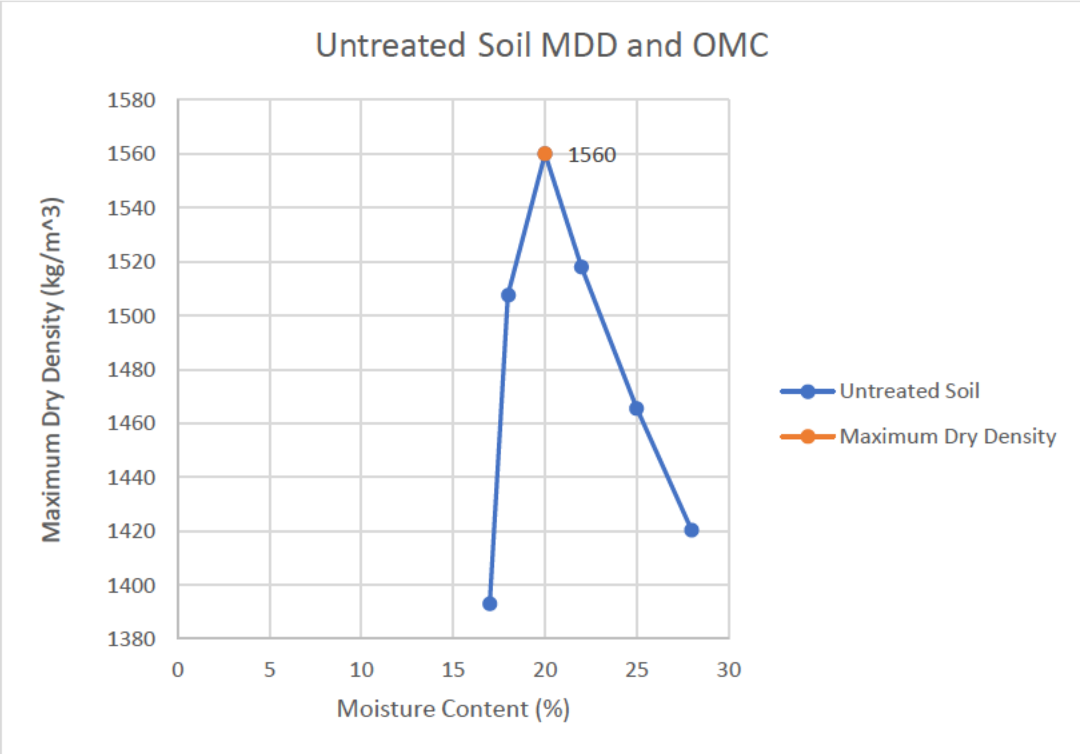

Figure 25- Untreated MDD and OMC

For the treated sample the procedure was repeated, and a value of 1433kg/m^3 was found at a moisture content of 21%. This gain in optimum moisture content was consistent with the findings of Anusha (2017) and Murmu (2018) which both found that the optimum moisture content of black cotton soils increased by 1-3% after the addition of fly ash for stabilisation. Although the OMC and maximum dry density may be affected by the addition of sodium silicate and sodium hydroxide, it was assumed that this would be negligible, and due to time constraints, the effects of these additions on OMC and MDD were not investigated.

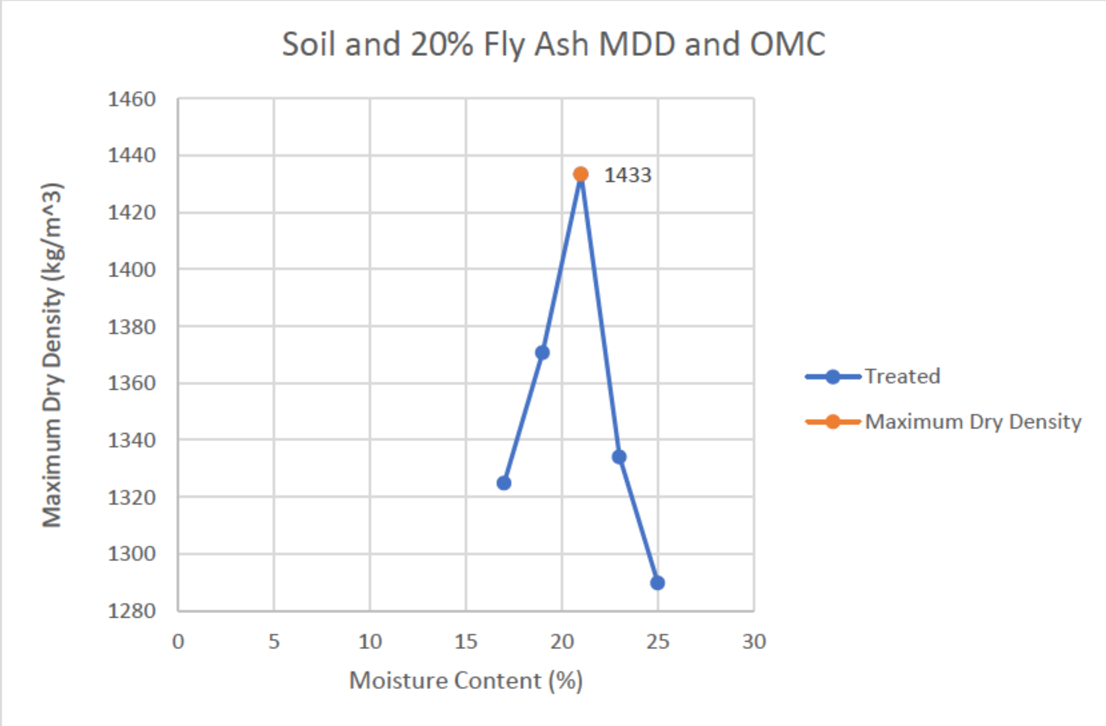

Figure 26- MDD and OMC of treated soil

4.8 California Bearing Ratio

The California bearing ratio tests were conducted in accordance with AS 1289.6.1.1. For the soaked tests, the vertical swell was recorded as well as the bearing capacity. Three untreated samples were prepared, to ensure a greater degree of accuracy. Upon testing it was found that the first sample was prepared incorrectly and did not test well. When the sample was inspected after testing it was found to be incorrectly compacted and as such, it had broken up during soaking. Therefore, the results from this sample were discarded. The average from the two remaining tests were taken and the maximum bearing ratio was obtained. The results showed an average CBR of 2.4%. This is extremely low and would not be suitable for any load bearing structure without treatment. If a road were to be constructed over this soil, either the base coarse material would have to be significantly thicker to compensate for the weak subbase or the entire subgrade would have to be removed and replaced, which would come at a significant cost. The vertical swell was recorded and was significant with an average of 5% swell. The Department of Transport and main Roads (2018) dictates that an expansive soil with a vertical swell of 5% is classified as a soil with a 'Very High' expansive nature.

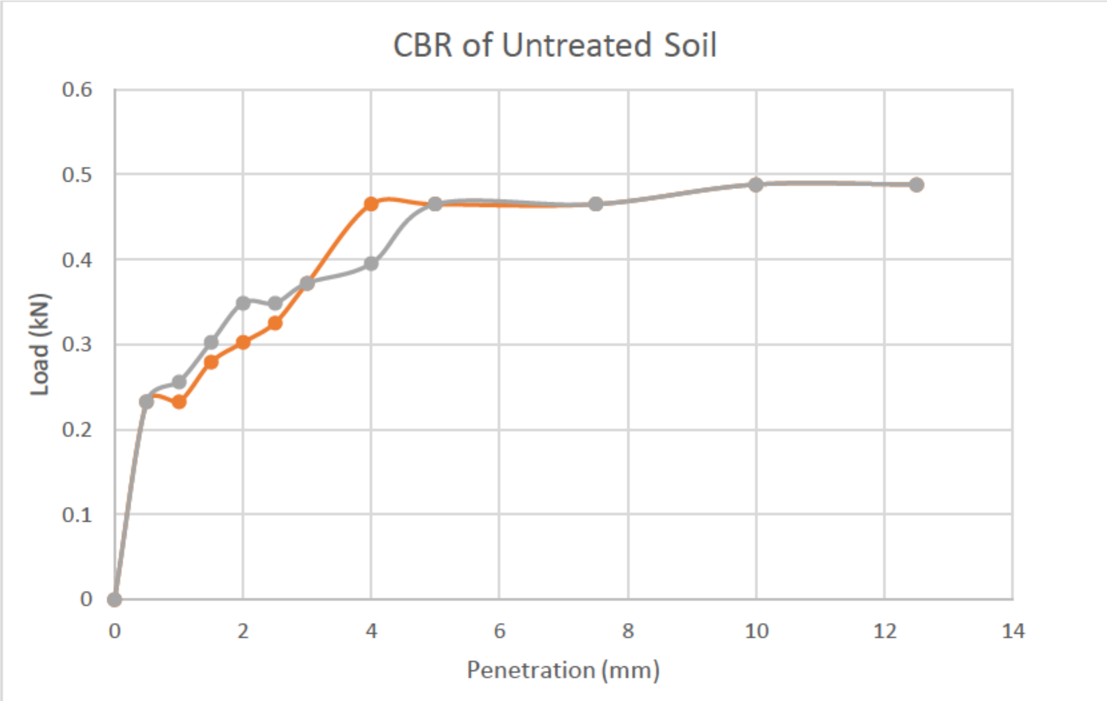

Figure 27- CBR of untreated soil (Author 2019)

Three treated samples were prepared for testing and after soaking were tested in a Load Trac 2 machine. All three samples were tested successfully. The results for the treated samples showed a huge improvement over the untreated samples. It can be seen in the load penetration curve that the sample one curve requires correction of slightly less than one millimetre. After doing this it aligns closely to samples 2 and 3. The new CBR came to 15%. The vertical swell for the treated samples was measured before testing and across the three samples the average swell had reduced to 0.83%. This represents approximately an 85% reduction in vertical swelling. The Department of Transport and Main Roads (2018) classes soils with a soaked CBR swell of 0.5-2.5% as having a Moderately expansive nature. This is an improvement over the untreated soil.

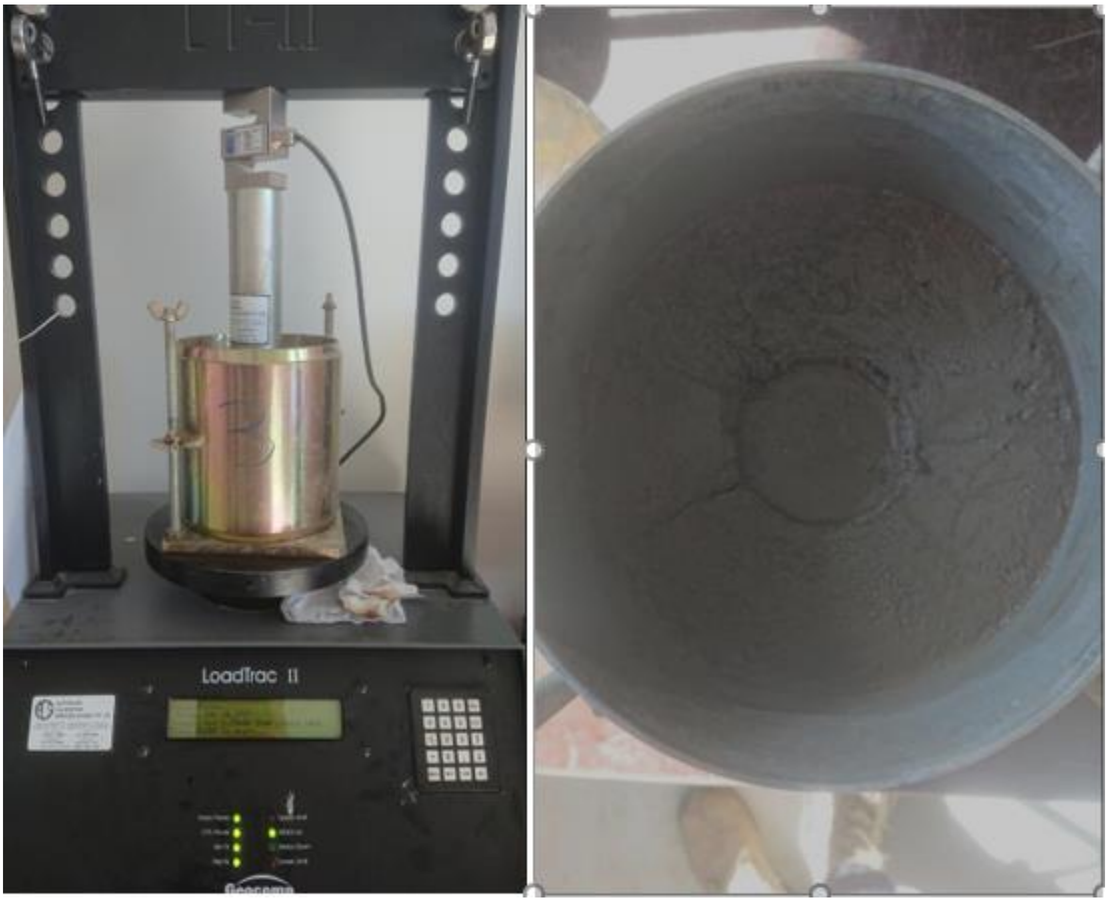

Figure 28- CBR testing of fly ash geopolymer treated samples (Author 2019)

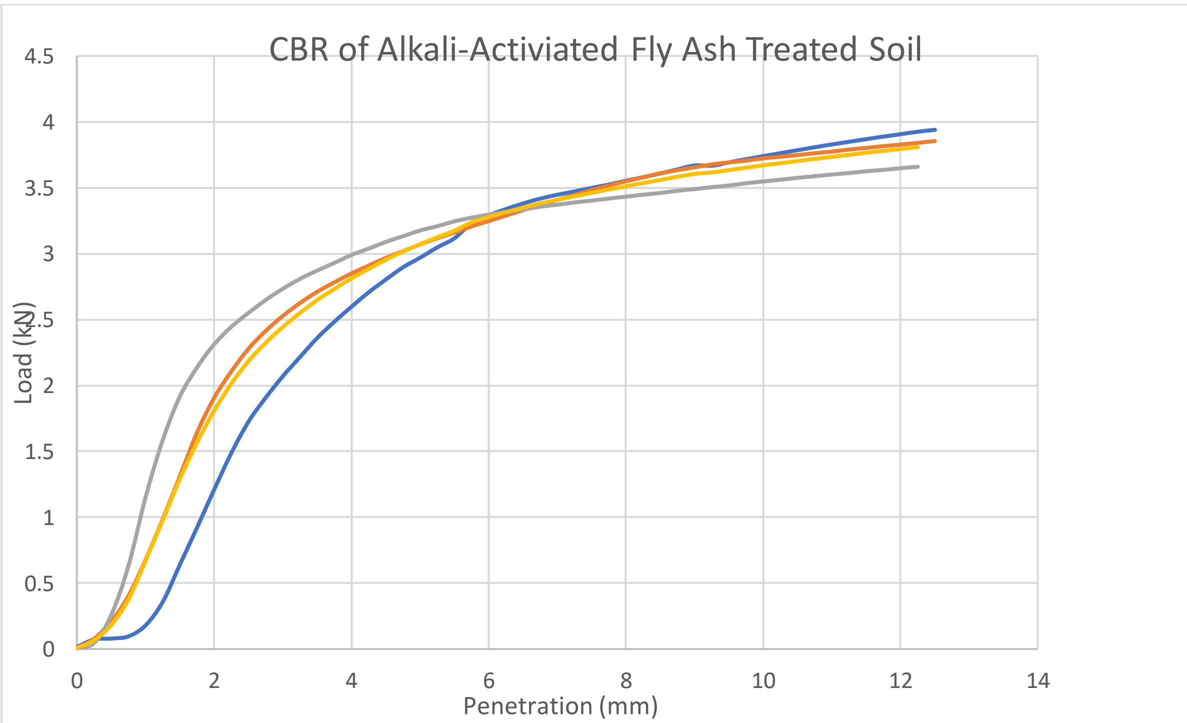

Figure 29- CBR of fly ash treated samples (Author 2019)

4.9 Uniaxial Compressive Strength Testing

UCS testing was conducted IAW AS 1289.6.4.1. The treated samples were tested in the Load Trac 2 test machine in the USQ soil laboratory. Three untreated samples were prepared, although one sample was destroyed before testing as it could not be removed from the mould without breaking. The remaining two samples were tested and both tests resulted in a maximum load of approximately 530N. This corresponded to values of 190kPa. The failure modes for both the samples was due to plastic deformation. The samples bulged at either end then failed. This was the expected failure mode for highly plastic clay samples.

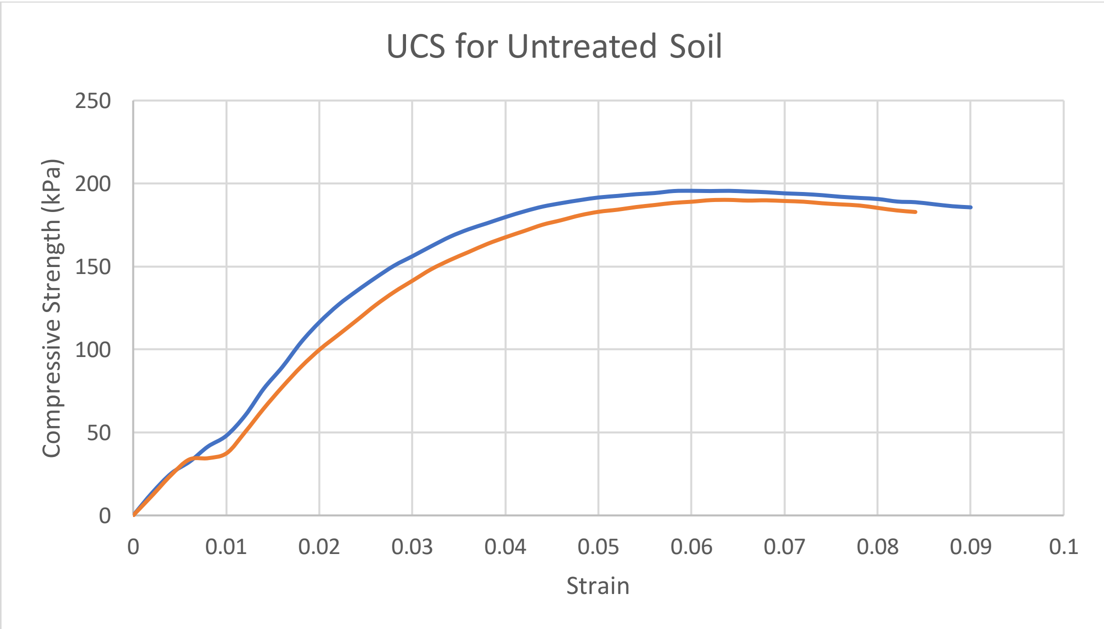

Figure 30- UCS chart for untreated soil (Author 2019)

Figure 31- Plastic failure of untreated sample (Author 2019)

There were three treated samples prepared in the same manner as the untreated samples. These samples were removed from their moulds after preparation and wrapped in cling film to preserve their moisture content. They were then sealed in an airtight container for 28 days to cure. The cured samples were tested using the same test apparatus. Two of the samples achieved similar maximum strengths, with 0.947kN and 1.04kN. These two values corresponded to a value of 485kPa and 412kPa. The third sample tested lower, with a maximum strength of 0.5kN. It was noted before testing that this sample had a slightly convex surface when it mated the loading piston. This convex surface coupled with the failure cracking radiating from this surface indicate that this irregularity may have resulted in the premature failure of the specimen.

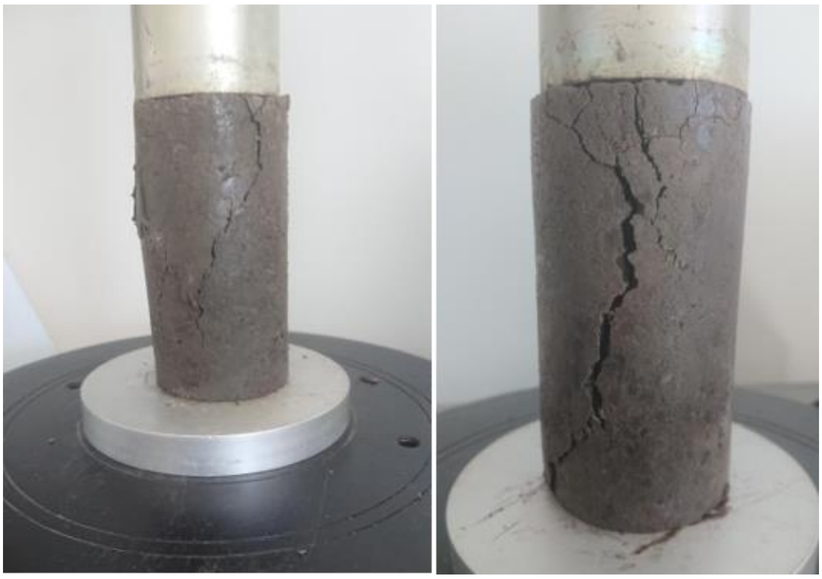

Figure 32-Brittle failure of fly ash treated samples (Author 2019)

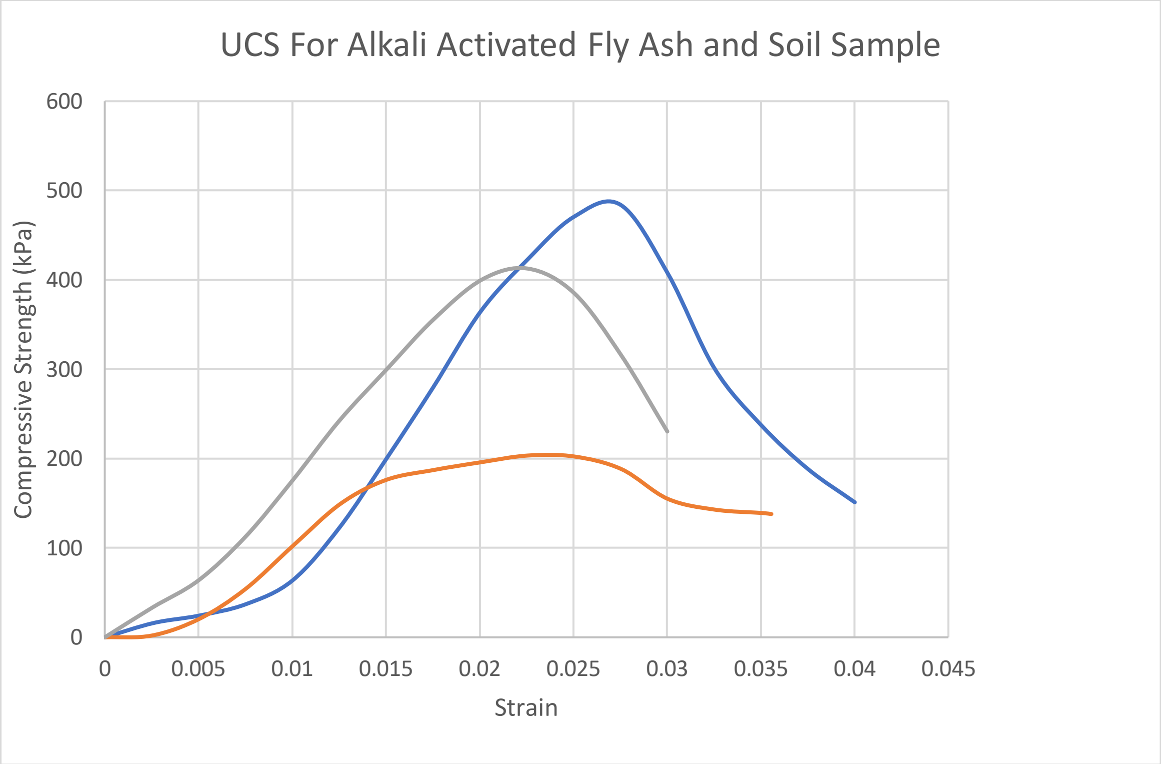

Figure 33-UCS of treated sample

Figure 34- Brittle failure of all three fly ash treated samples (Author 2019)

4.10 Free Swell Index Testing

After the samples were checked, the free swell index of each was measured with the following formula.

$$FSI\,(\%) \;=\; \left[\frac{V_d - V_k}{V_k}\right]\;100$$

Where V_d is the volume of the sample in distilled water and and V_k is the volume of the sample in kerosene. The free swell index is given in a percentage. The untreated sample had a final volume of 16.5cm^3 and the treated sample had a volume of 15cm^3. The kerosene soaked samples for the untreated and the treated samples were 9cm^3 and 10cm^3 respectively. This gave a free swell index of 50% for the treated sample, and a free swell index of 75% for the untreated samples. This reduction in free swell shows a positive improvement in the sample quality after treatment.

Chapter 5

Pavement Design

5.1 Introduction

In order to test the viability of the stabilisation method, pavements for various situations needed to be designed. The pavements were designed as if the native soil had the same properties as the soil used in the experiments. The design guides used were the Queensland Department of Transport and Main Roads regulations and Austroads guidelines for this task.

- An unpaved road
- A lightly trafficked pavement
- A heavy duty pavement

In Australia, the Austroads publications provide guidance on all aspects of road design. The standards define a subbase as the trimmed or prepared portion of the natural formation on which the pavement is constructed. The Austroads Guide to pavement Technology Part 2, Chapter 5 describes the difficulties of constructing pavements over expansive subgrades. Table 5.3 gives a guide to classifying expansive soils. It also states that the swell test is preferred to the plasticity index if facilities are available. Considering the sample tested exhibited a swell of 5.0% it was on the very border between High and Very high for expansive nature. Austroads provides guidance on stabilised materials in part 4D of their pavement design guides, with table 2.4 showing that the only option suitable for a clay soil with a plasticity index greater than 20 is Lime.

Part four notes that for soils stabilised with lime, the rate of strength gain is considerably less than materials stabilised with cement or cementitious binders. It is for that reason that curing times must be allowed. The standard also notes that if the aim of stabilisation is to reduce plasticity, without achieving high strength gains then lower binder contents are sufficient. Part 4.8 of the supplement provides guidance on determining lime content required to achieve stabilisation, method B uses CBR, which is what was used for these designs.

The empirical (chart) methods were used for the both the lightly trafficked roads and the unsealed roads. For the heavy duty pavement, a mechanistic design method was adopted to determine an appropriate solution. This was accomplished using the evaluation version of the Circly™ 6.0 pavement design software.

Table 7- Binder selection chart (Austroads 2019)

Particle size	**More than 25% passing 75 µm sieve**			**Less than 25% passing 75 µm sieve**		
Plasticity index (PI)	PI ≤ 10	10 < PI < 20	PI ≥ 20	PI ≤ 6 & PI x %passing 75 µm ≤ 60	PI ≤ 10	PI > 10
Binder type						
Cement and cementitious blends(1,3)	Usually suitable	Doubtful	Usually not suitable	Usually suitable	Usually suitable	Usually suitable
Lime	Doubtful	Usually suitable	Usually suitable	Usually not suitable	Doubtful	Usually suitable
Bitumen	Doubtful	Doubtful	Usually not suitable	Usually suitable	Usually suitable	Usually not suitable
Bitumen/ lime blends	Usually suitable	Doubtful	Usually not suitable	Usually suitable	Usually suitable	Doubtful
Granular	Usually suitable	Usually not suitable	Usually not suitable	Usually suitable	Usually suitable	Doubtful
Dry powder polymers	Usually suitable	Usually suitable	Usually unsuitable	Usually suitable	Usually suitable	Usually not suitable
Other proprietary chemical products(2)	Usually not suitable	Usually suitable	Usually suitable	Usually not suitable	Doubtful	Usually suitable

5.2 Lightly Trafficked Roads

Austroads Guide to Pavement Technology Part 2, chapter 12 gives guidance on the design of lightly-trafficked roads. The guide defines these as flexible pavements with a design traffic in the range of 10^3-10^5. The standard also states that 'Environmental conditions can have a more significant impact on the development of distress in lightly-trafficked roads than moderate-to-heavily trafficked roads. Designers need to consider the following

- the potential of moisture ingress to cause weakening of subgrades and to cause volume changes in expansive soils
- the potential of moisture ingress to cause weakening of unbound material

Keeping these points in mind during the design of the road, it is evident that stabilisation of expansive soils is important for even lightly-trafficked roads. Table 12.2 in the Design Guide gives an estimate of heavy axle group volumes for lightly-trafficked streets. The trial road designed was determined to be a local access street with no buses and it had a design life of 20 years. This gives a design traffic of $4x10^4$ ESA (Equivilent Standard Axles).

Guidance on the selection of pavement types based on traffic volume can be found in table 2.2.1 in the TMR supplement for Austroads part 2. The road was determined to be an urban road, with the table recommending four options:

- Lightly bound granular base with sprayed seal or asphalt surfacing
- Unbound granular pavement with sprayed seal surfacing
- Unbound granular pavement with thin asphalt surfacing
- Asphalt over foamed bitumen stabilised base pavement

The Austroads design guide states that lightly bound bases are typically used for rehabilitation works, rather than new roads, so a design using that method was not considered. An unbound granular pavement with either a sprayed seal or thin asphalt surfacing is recommended for lightly trafficked roads. Thin asphalt coatings are susceptible to fatigue cracking, although when used in a low traffic area the risk of fatigue cracking is low (Austroads 2017). The standard also goes on to state that thin asphalt surfaces are more resilient to minor traffic damage and provide a smoother and more durable surface than a sprayed seal. It is for these reasons that a thin asphalt surface over an unbound granular base was chosen.

The subgrade for the road was the same soil as used in the laboratory, an expansive clay with a soaked CBR of 2.4%. Three designs were considered.

- non stabilised subgrade
- lime stabilised subgrade
- fly ash geopolymer stabilised subgrade

The design for the lime stabilised subgrade and the geopolymer subgrade was identical, given that the design guide states. ‘In using figure 12.2, selected subgrade and lime-stabilised materials normally have a maximum design CBR of 15%, irrespective of the measured CBR results.’ Given that the CBR results for fly ash stabilisation gave a 28 day CBR of 15% the design using the Austroads chart method will be identical irrespective of which stabilisation method is used. A cost comparison was conducted for the two methods to determine which was cheaper. All pavement designs will be considered ‘at grade’. The material locally available for all designs will be CBR 80, CBR 45 and CBR 15. These material choices are all commonly available from quarries and wholesale suppliers.

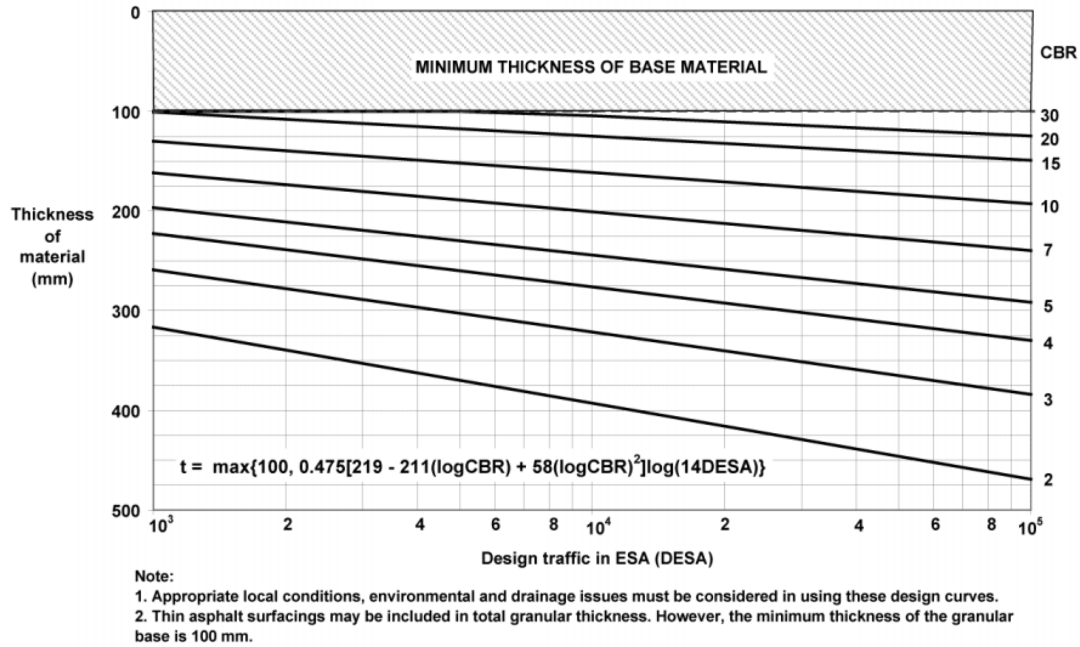

Figure 35- Design chart for lightly trafficked roads (Austroads 2018)

5.2.1 Non-Stabilised subgrade

The Pavement design and Supplement from The Queensland Transport and Main Roads Department (QTMR) states that for flexible pavements over subgrade material with a high or very high expansive nature, the minimum cover should be determined by figure 5.3.5 of that guide. The guide also states that the thicknesses stated in figure 5.3.5 are intended to mitigate the risk based on the importance of the road, so it may not be economic to provide these cover thicknesses, especially for low traffic areas, so this table was not applied, especially since the chart did not extend to the lower design traffic required for this road.

Austroads 2017 states that a low permeability lower subbase of select fill capping should be provided above the expansive soil and that this capping should be encased in a geosynthetic liner. The guide states that the capping layer should extend at least 1.5m past the pavement surface and sealed shoulders should be provided in order to mitigate moisture level changes in the subgrade surrounding the pavement.

Figure 12.2 of Austroads (2017) was used to obtain the total pavement thickness required. Using equation given in the figure, a thickness of 402mm was obtained. This was rounded up to 410mm of total material over the subgrade.

IAW 5.3.5 of Austroads 2017, a capping layer of 150mm was applied, consisting of densely graded gravel with a CBR of 15 and a low permeability. This also meets the requirements of TMR (2018), table 5.9. This capping layer was wrapped in a geotextile liner and extended a minimum of 500mm past the edge of the pavement, as directed in 5.3.5 of Austroads (2017). The drainage for the pavement should also be incorporated into this capping layer, instead of in the subgrade, as is the case with other subgrade soils.

Above this capping layer was placed a 120mm CBR15 subbase, then a 100mm CBR80 base, which supported a 40mm densely graded asphalt wearing surface. In order to improve the water resistance unbound materials below the asphalt, a bitumen seal layer should be placed directly to the base surface before the application of the asphalt base. This densely graded asphalt is recommended on lower trafficked roads as they do not see enough traffic to close up cracking in the asphalt as the bitumen oxidises over time, Austroads (2017).

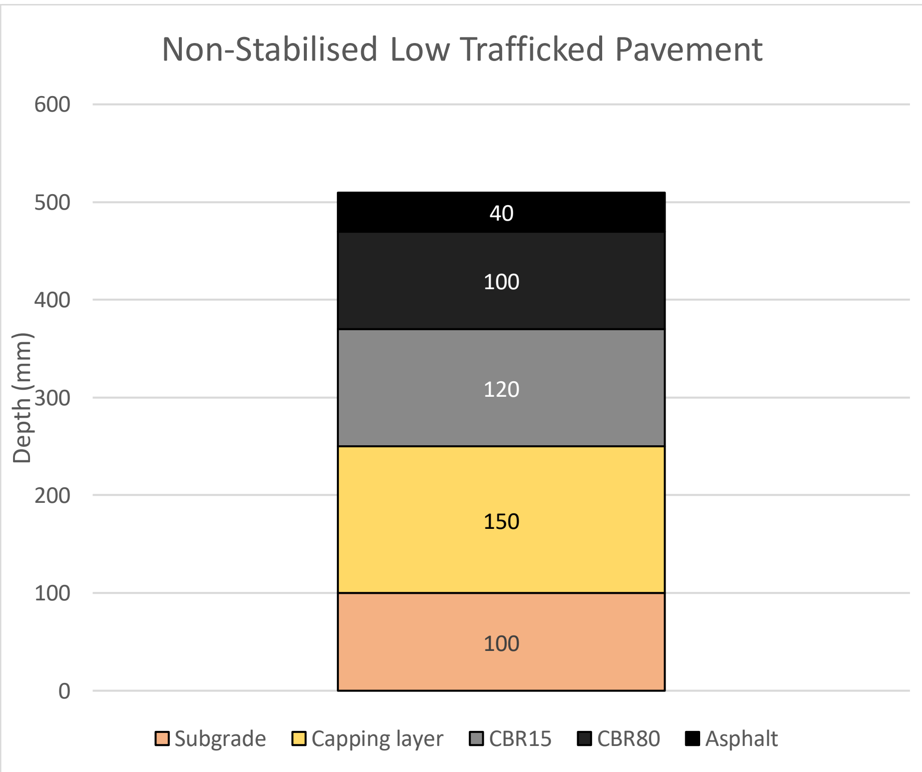

Figure 36- Pavement design for non-stabilised low traffic road (Author 2019)

This pavement design meets the thickness standards as set out in Austroads and TMR guidelines, with thought given to controlling moisture ingress into the expansive subgrade. Given the low traffic volume of the road, it would not be practical or financially feasible to apply all mitigation methods.

5.2.2 Geopolymer stabilised subgrade

A fly ash geopolymer can be used in the same way as lime stabilisation, reducing plasticity and volume changes due to moisture ingress. For the purposes of design, the same design guidelines will be used as for lime stabilisation.

An improved layer of 200mm was adopted first, meeting the requirements of table 5.9 of TMR (2018). The design CBR of the stabilised subgrade was determined to be the minimum of, (1) 15%, (2) The results of the CBR tests(15%), or (3) the value determined from the support provided by the underlying material (in situ material), as follows.

$$CBR_{stabilised\ subgrade} = CBR_{underlying\ material} \times 2\left(\frac{thickness\ of\ stabilised\ subgrade}{150}\right)$$

$$= 2.4 \times 2\left(\frac{200}{150}\right) = 6.4\%$$

This gives a design subgrade of 6%, which according to table 12.2 of Austroads (2017) requires 245mm of cover. The thin asphalt layer of 40mm was supported by a 100mm CBR 80 granular base. This is supported by 120mm subbase of CBR15 material. This gives a total cover of 260mm over the improved subgrade, and a cover of 460mm over the in situ subgrade. This improved subgrade should be constructed at least 500mm past the edge of the road as with the previous example. A sprayed seal should also be applied directly over the base surface to improve the water resistance of the pavement. The design for a lime stabilised subgrade would be identical, given the limiting factor in the improved subgrade strength was its thickness over the in situ subgrade.

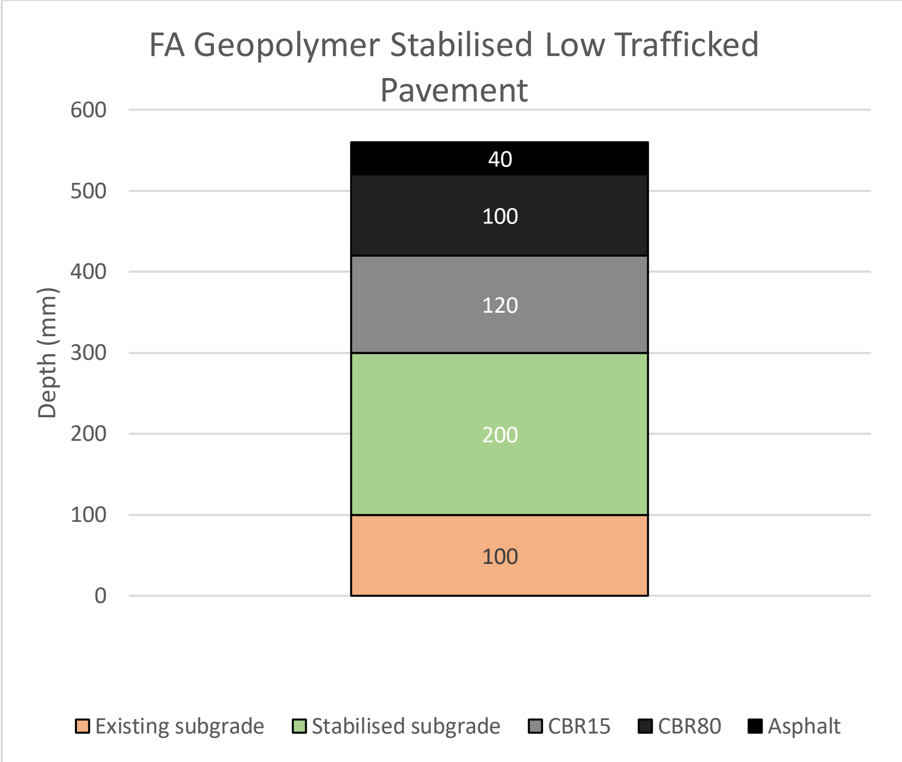

Figure 37- Fly ash geopolymer stabilised low traffic road (Author 2019)

5.3 Unsealed Pavement

Guidance for the design of unsealed pavements is contained in Austroads Guide to Pavement Technology Part 6 : unsealed roads (2009). The major difference in an unsealed road is that instead of using an asphalt or bituminous seal, the surface layer is referred to as the 'wearing course' or ' sheeting layer'. This layer is usually made of locally sourced naturally occurring gravel Austroads (2009). The wearing course has to provide good wearing resistance, to ensure a low level of lost material, as well as a low permeability, to reduce the chance of potholes and rutting, Austroads (2009). Given that a lot of unsealed roads are in remote areas, the use of local materials is important in keeping the cost to a minimum, by reducing haulage costs. Austroads (2009) details procedures for winning local materials for use in these pavements. Due to most materials being won locally, there is limited scope for the use of high quality materials, such as those that would be found when constructing sealed pavements. For the purpose of design, the materials presented were limited to:

- a wearing course of CBR40 natural gravel
- a CBR30 base natural gravel

- a CBR15 natural gravel
- a CBR7 natural gravel

The guide breaks down roads into classifications based on their design traffic and intended use. Table 2.1 in Austroads (2009) details the classifications of these roads based on typical traffic numbers and configuration. The modelled road will be a class 'U2' road, with a design ESA of 1 $x10^5$. Table 2.1 of the guide describes a class U2 road as:

- Mostly all-weather former pavement with some drainage. Two pavement layers over subgrade.
- With granular or modified materials in the wearing course.

Table 2.2 in the standard states that a typical application of a U2 road would be a main link between communities, national parks, recreational areas or a haul road. It will also be capable of sustaining traffic at speeds up to 100 km/h and have two lanes plus a shoulder.

5.3.1 Lime Stabilised Unsealed Pavement Design

As the existing subgrade has a CBR <3, there is a requirement for the subgrade to be stabilised to a depth of 100-150mm, IAW Figure 4.3 of Austroads (2009). This stabilisation is usually done with lime but could also be done with a fly ash geopolymer. For this design the author used a 150mm lime stabilised layer to bring the subgrade design CBR up to 3% so figure 4.3 could be used to determine pavement thickness. The stabilised layer should be extended past the shoulders of the road to the drainage area to ensure moisture changes do not affect the road surface.

With a design traffic of $1x10^5$, figure 4.3 shows that a minimum thickness of 340mm is required. Given the additional cost when constructing pavements with more layers, the design was only made with two pavement layers, not including the wearing coarse, this was in line with table 2.1 of Austroads 2.1. The subbase chosen was a 140mm CBR7 natural gravel, over which a 200mm CBR 30 base was applied.

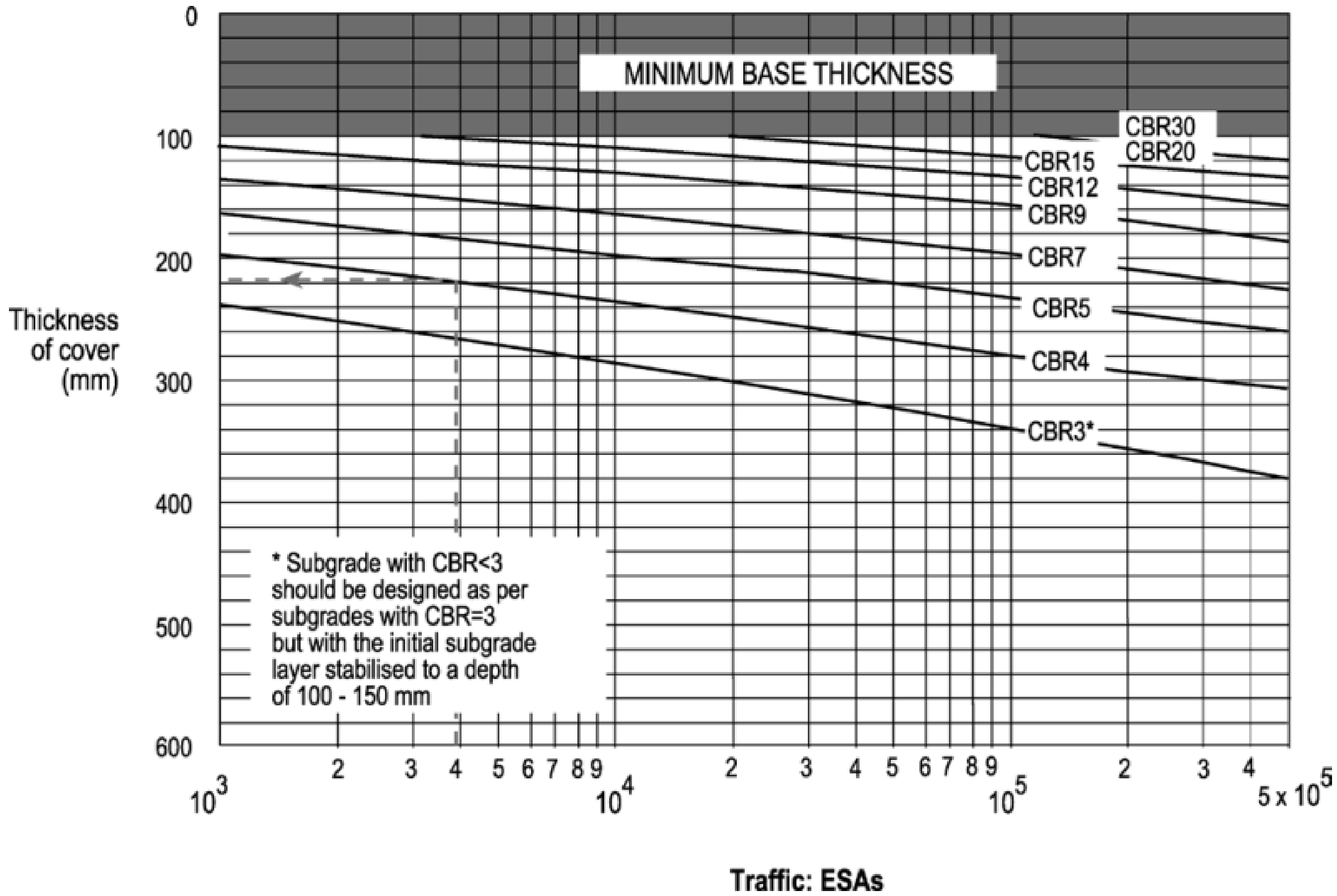

Figure 38- Unsealed road design chart (Austroads 2019)

The wearing coarse was a natural gravel with a 4 day soaked CBR of at least 40%. This was in line with table 3.5 of Austroads (2009). The thickness of the wearing course was 100mm thick. Although this additional thickness is not needed initially, unsealed pavements lose a significant amount of gravel over time, through traffic and patrol grading. Part 8.3 of the guide states that up to 150mm of wearing material can be lost over an 8-12 year period, although 100mm is typical. The design assumed a typical gravel loss of 100mm before resheeting. This additional pavement depth also allows extra depth to reduce the chance of the expansive subgrade swelling due to moisture ingress.

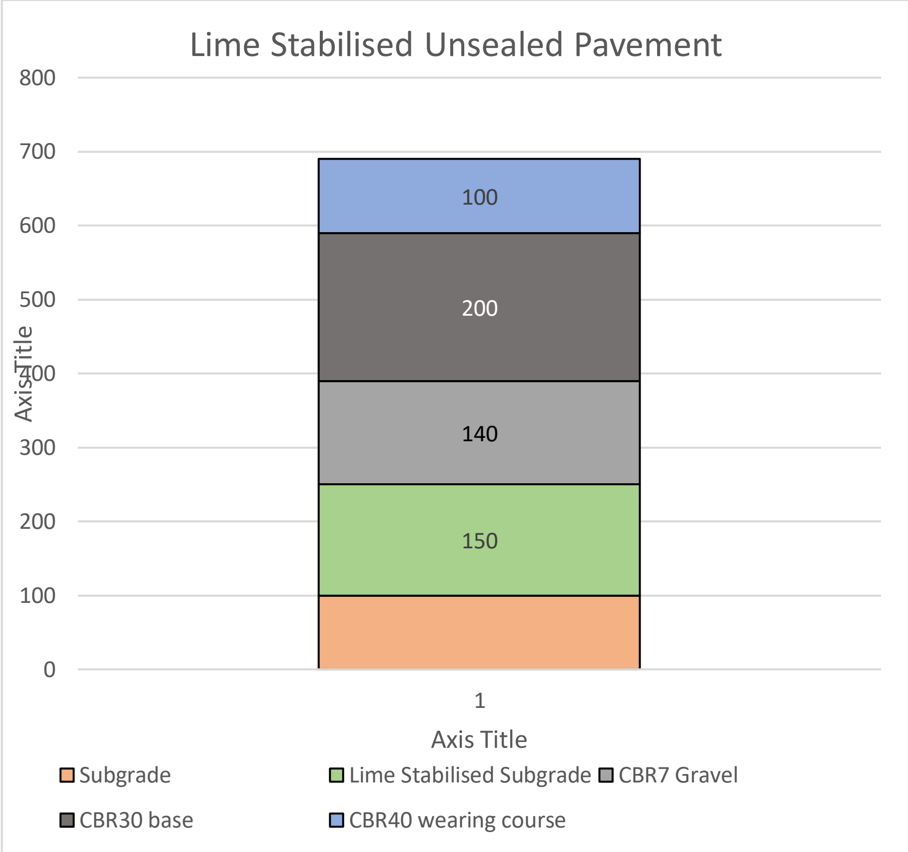

Figure 39- Lime stabilised unsealed road design (Author 2019)

5.3.2 Geopolymer Stabilised Unsealed Pavement

The geopolymer stabilised road started with a fly ash stabilised subgrade with a thickness of 400mm. This thicker layer was chosen to ensure that less subbase and base materials had to be sought. Although the 400mm stabilised layer needs to be compacted in two layers it means that more of the available material can be used.

$$CBR_{improved\ subgrade} = CBR_{underlying\ material} \times 2\left(\frac{thickness\ of\ selected\ subgrade}{150}\right)$$

$$= 2.4 \times 2\left(\frac{400}{150}\right) = 12.8\%$$

Using the equation to determine effective subgrade CBR as with previous designs, this comes to an effective subgrade CBR of 12. Using figure 4.3 of Austroads Pt 6 (2009) the required thickness comes to 140mm. A 140mm CBR30 base was laid over this geopolymer stabilised subgrade layer. The wearing course will consist of 100mm CBR40 natural gravel, to account for wearing surface loss from traffic and patrol grading, as with the previous design.

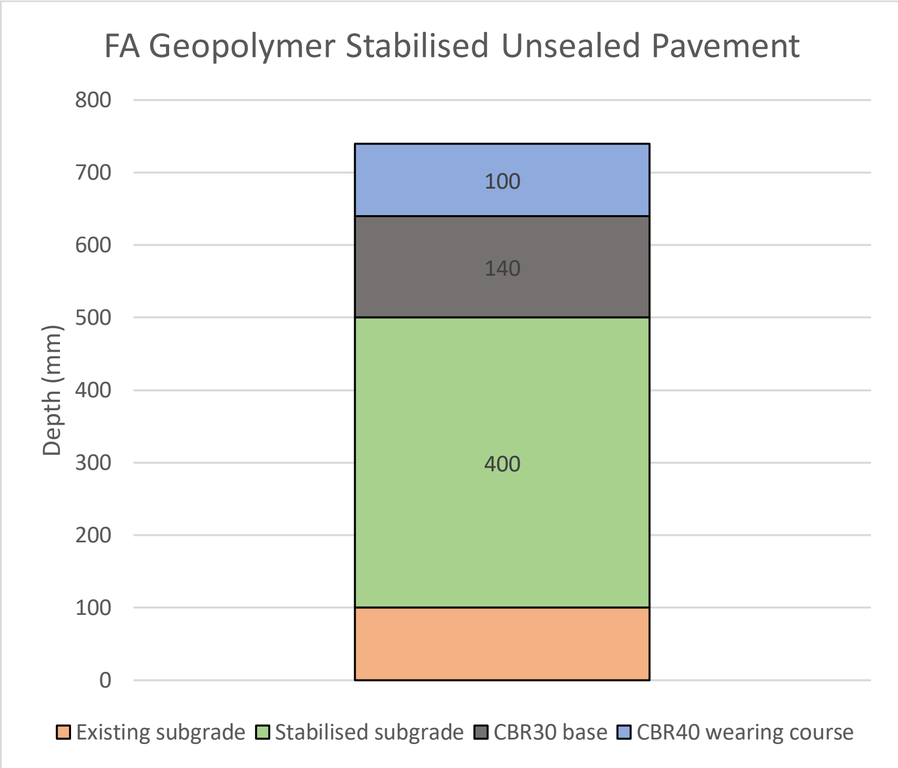

Figure 40- Fly ash geopolymer stabilised unsealed pavement design (Author 2019)

5.4 Heavy Duty Pavements

Heavy duty pavements are typically classified as pavements with a design traffic of at least 10^7 ESA. These heavy duty pavements are usually freeways and other major routes. Subsequently the reliability factor for these pavements are much higher than other pavements. Austroads Pt 2 assigns a reliability of 97.5% be used in the design of heavy duty pavements, which was adopted in these designs. Table 2.2.1 of the TMR supplement to Austroads Pt 2 recommends a number of pavement types as suitable for heavy duty pavements:

- Full depth asphalt
- Thick asphalt over cemented subbase
- Sprayed seal over granular base
- Thick asphalt over lean-mix asphalt

A combination pavement was chosen for this design, with a thick asphalt over a granular base, over a cemented subbase. The mechanistic design method was used when designing this pavement, with the use of an evaluation version of Circly™ version 6.0. Given that only an evaluation version of Circly™ was available, the pavement design may not be entirely optimised, although will meet the requirements of Austroads (2017). A number of assumptions were made during the pavement design process:

- Design traffic of 5×10^7 ESA
- Reliability of 97.5%
- Material available is identical to previous designs.
- Minimum cover over expansive subgrades determined by TMR Sup fig 5.3.5
- Minimum asphalt thickness of 175mm
- Cemented subbase thickness must be 150-200mm

The guide for minimum cover over expansive soils is contained in TMR (2018). As noted earlier, the in-situ subgrade has been determined to be 'very high', using table 5.3.5 in TMR (2018). The standard offers significant guidance for the treatment options available for expansive soils and notes that not all of these options are financially viable for all projects. The guide goes on to state that 'these thicknesses are intended to mitigate the risk based on the importance of the road (for example, low risk for heavily trafficked pavements, and higher risk for lower trafficked pavements). However, it may not always be economic to provide these cover thicknesses, particularly for pavements with low traffic and where suitable fill materials are not readily available. In such circumstances, a design solution that accepts the potential impacts and addresses these through appropriate maintenance may be necessary' TMR (2018). Given that the reliability of this pavement is 97.5%, the risk of subgrade expansion due to moisture ingress should be mitigated as much as possible, so the minimum thicknesses given in figure 5.3.5 were used. For a very highly expansive subgrade the minimum cover over the expansive subgrade was determined to be 1200mm. The minimum asphalt thickness of 175mm and the 150-200mm subbase thickness were obtained from TMR (2018) table 2.2.8(a).

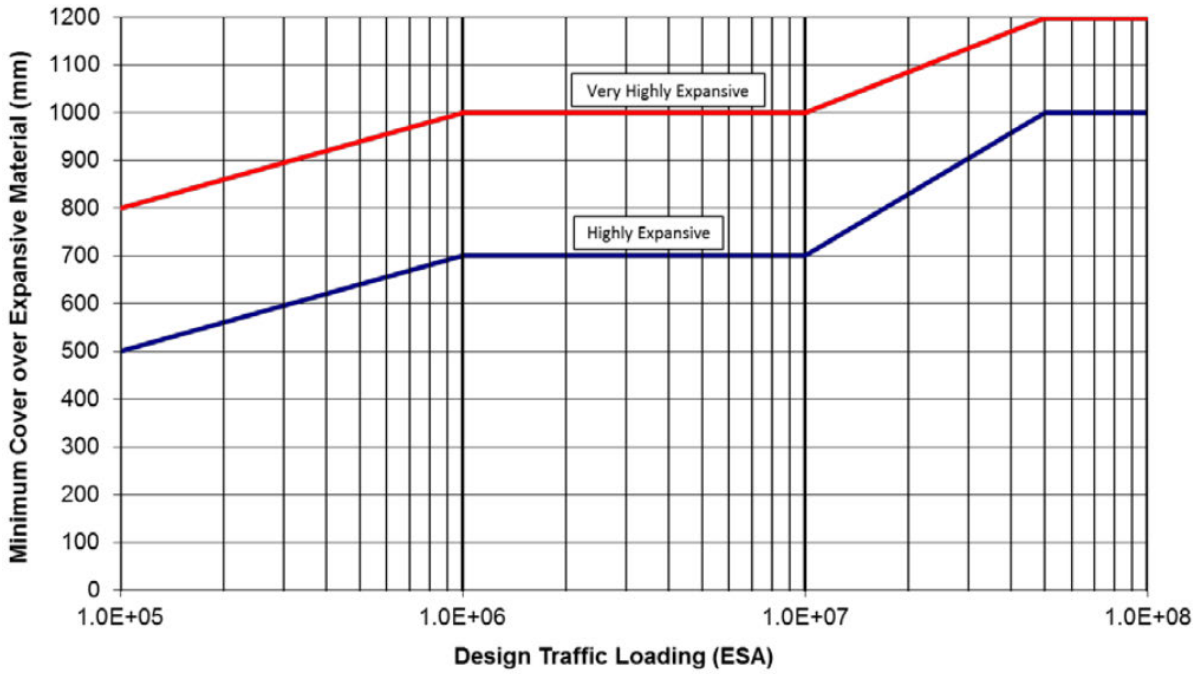

Figure 41- Minimum cover for expansive soils (TMR 2018)

Given the low risk needed for this pavement, other mitigation factors should be employed in addition to the minimum cover requirements.

5.4.1 Unstabilised Subgrade

When designing pavements with a subgrade design CBR of less than 3%, TMR (2018) notes that a soft subgrade treatment that results in a presumptive CBR of at least 3% is typically provided. Using table 5.9 of TMR (2018), it can be seen that a 200mm coarse granular fill is required for this subgrade. The granular fill will be of type 2.4 material and be wrapped in a geotextile material.

Table 8-Minimum cover for adoptive CBR (TMR 2018)

Subgrade CBR (%) (At Design Density and Moisture Conditions)	Minimum Thickness (mm) of Coarse Granular or Rock Fill Required for the Adoption of a Presumptive Design CBR of 3%
1.0	400
1.5	300
2.0	200
2.5	150
3.0	0

Given the minimum required cover is 1200mm, a significant amount of selected fill will be required to make up the additional thickness below the pavement subbase. A final pavement thickness of 525mm was adopted, so the total selected fill thickness came to 475mm. The quality of the selected fill can be lower, which reduces the cost of the fill. This cost reduction would be particularly important considering the large thickness required. A selected fill with a CBR of 6% was chosen to keep material costs down.

This selected fill was placed over the geotextile wrapped coarse granular fill and compacted in two layers. The design CBR of the selected subgrade was determined to be the minimum of, (1) 15%, (2) The results of the CBR tests(6%), or (3) the value determined from the support provided by the underlying material (in situ material), as follows.

$$CBR_{selected\ subgrade} = CBR_{underlying\ material} \times 2\left(\frac{thickness\ of\ selected\ subgrade}{150}\right)$$

$$= 2.4 \times 2\left(\frac{675}{150}\right) = 21.6\%$$

The limiting factor for the design CBR of the selected subgrade was determined to be the CBR test result of 6%. The pavement was then designed over the top of this subgrade using Circly 6.0.Given that cemented materials and asphalts were more expensive than unbound granular materials, the design process sought to minimise the thicknesses of these layers while ensuring that the Cumulative Damage Factor (CDF) was less than 1. The pavement returned a favourable result in Circly, with the figure indicating the asphalts cumulative damage factor to be at 0.854 and the cemented layer to be at 0.337.

Results:

Layer No.	Thickness	Material ID	Load ID		Critical Strain	CDF
1	175.00	Asph3000	ESA750-Full		-9.55E-05	8.54E-01
2	200.00	Gran_450		n/a		n/a
3	150.00	Cement3500	ESA750-Full		-5.60E-05	3.37E-01
4	0.00	Sub_CBR6	ESA750-Full		1.63E-04	2.84E-05

Figure 42-Circly design output (Author 2019)

A satisfactory design was obtained using the software which minimised both the asphalt thickness and the cemented layer. The design chosen incorporated a 150mm cemented subbase, overlain with a 200mm layer of CBR45 unbound crushed rock, with 175mm of asphalt. The cemented subbase had a modulus of 3500, and the asphalt was 3000MPa.

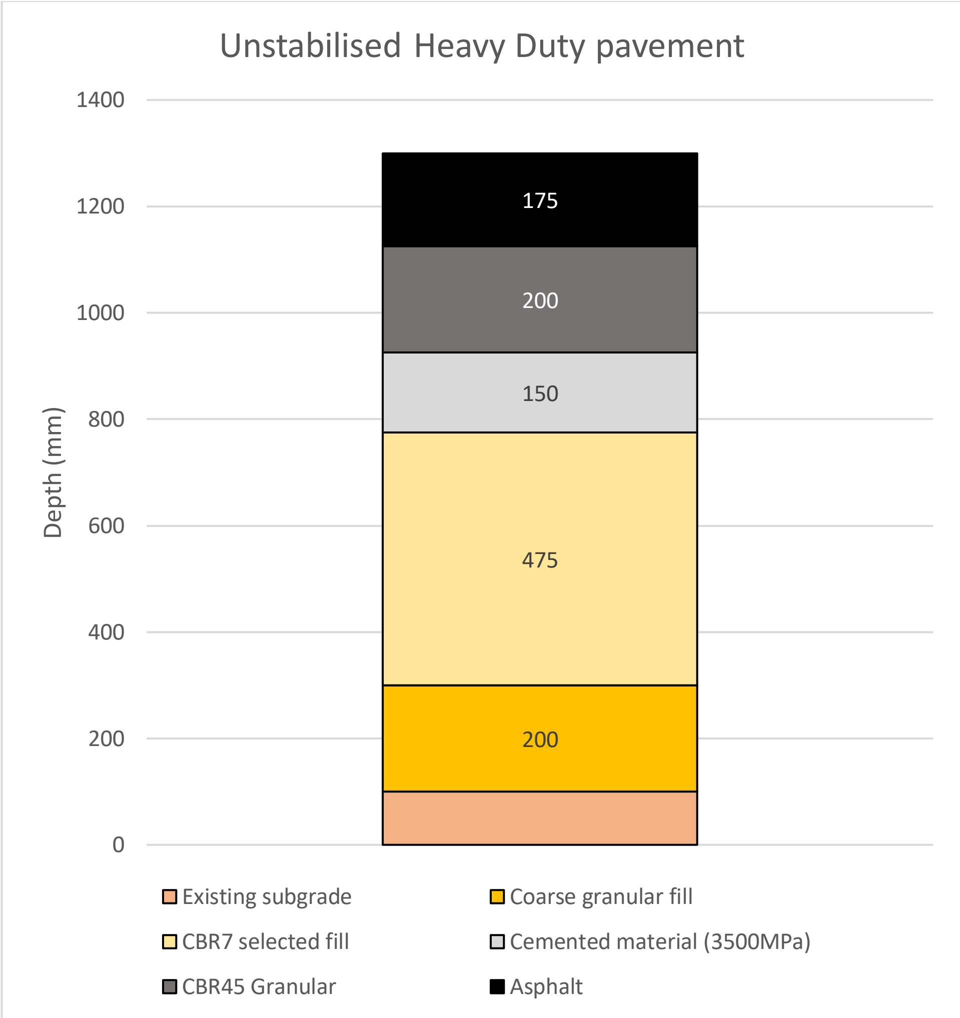

Figure 43- Unstabilised heavy duty pavement (Author 2019)

5.4.2 Fly Ash Geopolymer Stabilised Heavy Duty pavement

For the design of the fly ash stabilised geopolymer subgrade design, the same design assumptions were used as for the previous example. With the previous design, there was a need to remove 675mm of subgrade and replace with selected fill material. This decision meant that a lot of material would have to be taken off site and disposed of and a large amount of extra fill hauled to the site. This would add a significant amount of cost and time to construction. To minimise the amount of new material needed, this design stabilises the in-situ subgrade using the geopolymer mix. Given the required thickness of the improved subgrade, the material would still have to be excavated, and then treated with the alkali activated geopolymer mix and compacted in three layers, given the standard 250mm maximum layer thickness required for compaction.

The design CBR for the improved subgrade was determined using the same method as previously, either the minimum of, (1) 15%, (2) The results of the CBR tests(15%), or (3) the value determined from the support provided by the underlying material (in situ material), as follows.

$$CBR_{improved\ subgrade} = CBR_{underlying\ material} \times 2^{\left(\frac{thickness\ of\ selected\ subgrade}{150}\right)} =$$

$$2.4 \times 2^{\left(\frac{675}{150}\right)} = 21.6\%$$

Given that the soaked CBR testing of the geopolymer stabilised material was 15%, the design CBR for the subgrade was determined to be 15%. As with the previous design, the goal was to minimise the thicknesses of more expensive materials, such as the asphalt and the cemented subbase. The chosen pavement design was a 150mm cemented granular subbase with, with a 200mm CBR45 granular layer, and a 175mm asphalt layer. The asphalt had a modulus of 3000MPa, and the cemented subbase had a modulus of 3500MPa. A waterproofing bituminous seal was applied directly beneath the asphalt layer to waterproof the unbound granular layer.

Results:

Layer No.	Thickness	Material ID	Load ID	Critical Strain	CDF
1	175.00	Asph3000	ESA750-Full	-9.64E-05	8.95E-01
2	200.00	Gran_450	n/a		n/a
3	150.00	Cement3500	ESA750-Full	-4.16E-05	9.48E-03
4	0.00	Sub_CBR15	ESA750-Full	1.14E-04	2.23E-06

Figure 44- Circly output (Author 2019)

The pavement returned a favourable result in Circly, with the figure indicating the asphalts cumulative damage factor to be at 0.895 and the cemented layer to be at 0.00948. Although the cemented layer could have taken a much higher cumulative stress factor, the thickness of the layer could not be reduced due to the constraints given in the TMR and Austroads guides.

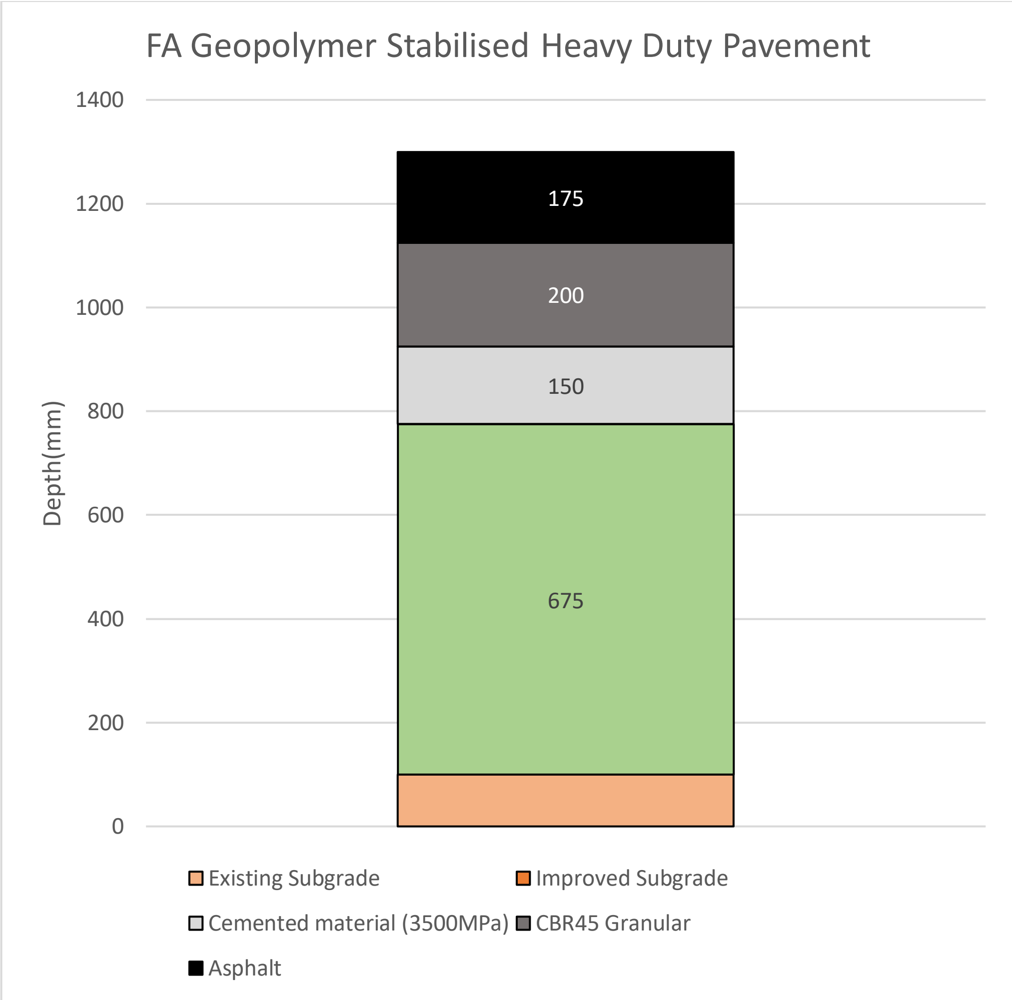

Figure 45- Fly ash geopolymer stabilised heavy duty pavement design (Author 2019)

CHAPTER 6

DISCUSSION

6.1 Atterberg Limits

As discussed in the Literature review, the Atterberg limits are an important measure of the soil's potential behaviour. These limits are frequently used by engineers for all soil tests. The plastic limit of the natural soil was 44%, which decreased to 36% after the activated fly ash treatment. The LL of 44% is within the range expected for an expansive clay, even if it is on the low side. The reduction to 36% is significant as it shows an improvement in the soil's cohesion at higher moisture contents. Nelson and Millar (1992) note that generally soil with a liquid limit of less than 30% have a low potential for swelling. As with all attributes for expansive soil, when viewed in isolation they aren't 100% indicative of the soil's swelling behaviour. When combined with other attributes these individual improvements can provide a more accurate picture.

The plastic limit of the untreated soil was 20% and improved to 25% after treatment. Although the plastic limit is rarely used in isolation as a reliable indicator of soil behaviour and swell potential, its use in the calculation of the plasticity index is what makes improvements in the figure important. The plasticity index, which is generally agreed upon to be one of the best indicators of shrink-swell, improved from 24, to 9. This was a huge reduction in PI and represents a positive change to the soil's characteristics. A high plasticity is indicative of low strength, high swell and unreliability. During testing of the treated soil, the author noted the consistency of the soil had changed dramatically. Initially, the soil was difficult to add water to as it became sticky, and a lot of work was required to ensure the water permeated evenly through it. After treatment the soil accepted water far easier and was not as sticky or difficult to work with. There was a 'sandy' texture to the soil that had not been present before treatment. This sandy texture was attributed to the polymerisation of the soil during treatment forming larger particles. These larger particles meant that the total surface area to volume ratio of the soil will have dramatically decreased. If that had happened it would help explain the improvement in the plasticity index and workability of the soil. By reducing the surface area to volume ratio, the amount of water that can be adsorbed by the clay will have reduced, meaning that a reduction in shrink-swell is likely.

This reduction in shrink swell was observed during the linear shrinkage test. Prior to treatment the linear shrinkage across three samples was 21%. Nelson and Miller (1992) define this level of shrinkage as critical. After treatment the linear shrinkage reduced to an average of 11%. This can still be considered moderate, although it is a god improvement. The reduction in linear shrinkage confirms what was speculated above, that the reduction in plasticity index and the sandy texture of the treated soil indicated a reduction in the shrink swell behaviour of the soil.

6.2 California Bearing ratio

The results have indicated that the treatment of expansive soil with alkali activated fly ash geopolymers can improve the bearing capacity of a soil. The untreated soil achieved a very low CBR test result, with an average strength of 2.4%. and had an average vertical swell of 5%. After treatment the CBR improved dramatically to 15%, and the vertical swell reduced to 0.85%. The initial result of 2.4% is very low, and for pavement applications it requires considerable extra design considerations to be usable. The QMTR Pavement Design Supplement (2018) outlines the procedures for designing and constructing pavements in Queensland and has a number of addition requirements if the subgrade CBR is less than 3%. Some of these additional requirements include:

- Covering the soft subgrade with >200mm of strong granular fill
- Adding an improved layer of bound (cemented) fill
- Geotextile wrapped granular material (unbound granular or recycled)

These additional requirements raise the cost of road projects due to the additional material and work. The design supplement states that for subgrades with a design CBR of 15% and greater, there are no additional requirements. This shows that the improvement of CBR after treatment can be effective enough that expansive soils can safely used as subgrades without further treatment or design considerations.

The QMTR design supplement also has similar requirements for soils that exhibit vertical swell during CBR testing. According to the regulations a CBR swell of 5.0% shows a very high expansive nature. The treatment options for very expansive soils include;

- Additional drainage installation
- Impermeable moisture barriers over expansive soil

- Additional unbound fill
- Excavation, removal and replacement of expansive soil

Table 9- Expansive soil classifications (TMR 2019)

Expansive Nature	Weighted Plasticity Index (WPI) (PI x % < 0.425 mm)	CBR Swell (%)[1]
Extreme	> 4200	> 10.0
Very high	3200 – 4200	5.0 –10.0
High	2200 – 3200	2.5 – 5.0
Moderate	1200 – 2200	0.5 – 2.5
Low	< 1200	< 0.5

Notes:

1. Swell at OMC, 95% to 98% MDD (standard compactive effort), four-day soaked, and using 4.5 kg surcharge.

The regulations show that soils with a CBR swell of 0.5-2.5% are classed as moderately expansive. These moderate soils do not require any of the additional treatment options described above. This means that according to current design regulations in Queensland, the treatment option tested can provide improvements to bearing capacity and shrink swell behaviour good enough that no further remediation is necessary.

6.3 Unconfined Compressive Strength

The UCS testing of the untreated samples resulted in a 190 kPa failure point for both samples. These samples failed in a plastic manner, with bulging at both ends of the sample. After testing the average of results was 450 kPa. After treatment the failure mode also changed from plastic to shear failure. This failure mode could be the result of the formation of polymer networks during the treatment reaction and the expulsion of water during the reaction. The 450 kPa result shows a 237% improvement in shear strength. This is not as drastic as the improvements in CBR but is still a positive result. Anusha (2017) achieved a UCS of 360 kPa using a similar ratio of geopolymers. This similar result was encouraging although the improvements were not as substantial as Murmu (2018). Murmu achieved results as high as 2400 kPa using a 20% fly ash content. Murmu also notes that there has been a vast difference in UCS results using all stabilisation methods. They go on to note that due to the widely different properties of expansive soils, treatment options and results may vary from soil to soil. It is for this reason that they stress each stabilisation treatment be tailored to the individual soil. The also noted that there was a direct correlation between the increase in shear test and fly ash content. They hypothesise that this is due to the availability of more alumina and silica, which leads to the formation of denser geopolymer matrices.

Although the improvements were not as large as the CBR tests, a 237% improvement is still significant and shows that geopolymer stabilisation can be a viable treatment to improve the shear strength of expansive soils.

6.4 Pavement Design Comparison

6.4.1 Lightly Trafficked Pavement

These designs are similar, although the design with the capping layer would require more material to be excavated and disposed of than the stabilised design. The cost that this would add to the construction are variable given the availability of capping material and the distance from the source of the imported material. The geotextile wrapped capping layer does have the advantage of being impermeable, with drainage being included to ensure that the natural subgrade does not absorb moisture. The permeability of the improved subgrade has not been tested and there is the possibility that the 200mm layer may not be sufficient to prevent water ingress. This is important given the guide acknowledges that lightly trafficked roads are more susceptible to environmental damage from moisture ingress than other more trafficked pavements. This could be mitigated to an extent by including drainage in the stabilised subgrade, and geotextile wrapping. Until the permeability of the geopolymer stabilised soil is known, it cannot be accurately predicted whether it will provide adequate moisture protection for the natural subgrade.

6.4.2 Unsealed Pavement

The advantage of the geopolymer stabilised design is the pavement thickness over the in-situ subgrade is substantially deeper. This means that moisture changes in the subgrade are far less likely to affect the pavement. The use of a thick layer of improved subgrade also means there is far less material that needs to be hauled to the site or won locally. The disadvantage of this is that more of the subgrade needs to be initially excavated. Both designs require four layers of compaction, so would require similar time and machinery costs.

6.4.3 Heavy Duty Pavement

The two pavement designs are very similar, and both performing similarly under load. The differences in cost and construction time would depend on a number of factors. The unstabilised design requires a large volume of imported material, to make up the 675mm layer of capping and selected fill. This means that there will be increased material cost as well as large disposal costs. The final cost of this would depend on how far the selected fill material had to travel and the distance from a disposal site for the removed subgrade. The geopolymer stabilised design removes the need for this large amount of imported material as it uses the subgrade available. With the addition of 20% fly ash, there would still be a small amount of material that would have to be disposed of, although mush less than using the other method. The cost of stabilisation chemicals would likely be offset by this reduction and could prove cost effective on heavy duty pavement projects over expansive subgrades.

6.5 Cost Benefit Analysis

There was a limited amount of research available for the cost benefit of geopolymer concretes over Portland cement. There are a number of variables the make it difficult to provide an accurate assessment of the costs, both from a financial and emissions standpoint. These variables can include

- Distance from production source
- Distance from bulk storage to site
- Production cost
- Greenhouse Gas Emissions during production
- Type of transport
- Market price
- Electricity usage

As can be seen, many of the factors have a large geographical component, with transportation cost and emissions being a substantial part of the total cost. The research my McLellan et al (2011) was one of the only studies conducted from an Australian perspective. They noted that the costs, particularly the financial costs were highly dependent on the distance from the source material production point or port, and the job site. McLellan et al (2011) also noted that the market for Portland cement is far more competitive and prices were less varied with distance. This competitive market also meant that market prices tended to vary much less than with come components required for geopolymers, such as sodium hydroxide. The research concluded that from an Australian perspective, the financial cost of geopolymer cement can be up to twice as high as ordinary Portland cement. McLellan et al (2011) primarily attributes this to the variability of feedlot distances for source materials. Interestingly, McLellan noted that if a carbon tax was introduced, which is increasingly more likely given the global push for sustainability, the cost benefit would likely reach parity. This parity was obtained with a carbon cost of $20 per tonne, which McLellan notes is possible.

From an environmental perspective, the main reduction in greenhouse gas emissions stems from the fact that fly ash is a waste product from the production of electricity, so is virtually emission free. The alkaline activator is primarily produced through the electrolysis of brine and consumes a significant amount of energy. The long transportation distances for certain materials as noted by McLellan et al (2011) also contribute to the emission cost of geopolymers. The emissions cost of Portland cement is estimated by Anusha (2017) to be approximately 1.0 tonnes of Co2 per tonne, which was confirmed by McLellan (2011). Both Anusha and McLellan found that the greenhouse gas emissions of fly ash geopolymer cement to be much lower than OPC. McLellan estimated that the reduction for Australian to be in the range of 44-64%, depending on location. Anusha reported a reduction of 60-80%. This difference is likely due to the fact that Anusha's study looked at global figures, while McLellan's was focused on Australia.

Overall, the financial expense of geopolymers may impact their adoption in soil stabilisation, although it is foreseeable that this could be negated by the introduction of a carbon tax. Some of this financial detriment is overcome by the fact that the greenhouse emissions of geopolymer cement is so much lower than that of Portland cement. Coupling this with the fact that fly ash is a waste product, the argument could be made that the disposal and carbon benefits outweigh the financial cost. The environmental and ethical costs of engineering are becoming more important to people and this is likely to become more important as time goes on.

6.5 Project Specification

There is one point that has not been met for the project specification. 'Determine whether geopolymer stabilisation could be useful in stabilizing foundations of existing structures in

South East Queensland' After review of the literature it became apparent that the majority of the research available for geopolymer stabilisation pertains to its use in stabilising soil before construction of foundations and pavements. The use of geopolymers for a remediation measure for existing structures is very limited in scope, and its other uses were more widely applicable. It was for this reason that its use for other applications in Queensland was explored more thoroughly.

CHAPTER 6

CONCLUSION

6.1 Conclusion

Expansive soils pose significant issues for governments, councils, residents and engineers. The soil's unpredictable levels of shrink and swell can damage pavements and structures built over them. The project set out to investigate the possibility of using these geopolymers to stabilise expansive clays in South East Queensland and to investigate whether they are suitable for stabilising foundations of existing structures.

The first objectives of the project were met after considerable research was undertaken into the use of geopolymer stabilisation and reactive soils. The research uncovered the many problems faced when working with expansive soils such as; low bearing capacity, unpredictable shrink-swell, and difficulty with workability. Using local knowledge, the author identified a location that was suitable for testing and took samples from the soil. Preliminary testing was conducted on the soil and it was confirmed that the author's observations were correct, the soil was an expansive clay suitable for the project. Research was conducted to determine the optimal geopolymer mix, accounting for performance increases, cost, safety and workability. It was determined that a lower concentration of alkaline activator solution would perform as well as a more concentrated solution. This reduction in alkaline concentration resulted in many benefits. The sodium hydroxide component of the geopolymer was found to be the most expensive component and also contributed to the highest percentage of greenhouse emissions. Along with the environmental and financial benefits, this lower concentration also reduced the safety risks associated with handling highly caustic materials.

Testing procedures for identifying expansive soils were researched, and a testing regime was decided on that provided a good indication of the soils properties and allowed the effects of the treatment to be measured accurately. Laboratory testing was organised within USQ Toowoomba and the testing was conducted using the appropriate procedures and standards. The testing was completed successfully, although there were a few tests that had to be repeated as samples were improperly prepared. This was a learning experience for the author as it was their first time conducting many of these tests.

The results from the tests were analysed and compared to current stabilisation methods. It was concluded that geopolymer stabilisation could provide sufficient performance increases to be a viable alternative to traditional stabilisation methods such as cement or lime. Although the performance increases were comparable, there were disadvantages to geopolymers, the main one being the cost compared to traditional methods. The research found that for Australia, the cost of geopolymer stabilisation could be as high as double that of ordinary Portland cement. An interesting fact found during the cost analysis was that the cost would reach parity if a carbon tax was introduced. A carbon tax is a possibility in the future and if one is introduced, geopolymers will become a far more attractive option for stabilisation. Although the costs are higher than traditional methods, the carbon cost of geopolymers is substantially lower and finds a use for waste fly ash which diverts it from land fill and provides another source of income for power companies.

The design of pavements with geopolymer stabilised subgrades was undertaken and assessed for adequacy using the Austroads guides. Technically speaking the pavements met the design standards and in certain situations it would be favourable to used fly ash geopolymer stabilisation instead of other methods of subgrade stabilisation. There are cost problems associated with fly ash geopolymers, mainly given the cost of sodium hydroxide in Australia, but should a carbon tax be introduced, geopolymers would likely be cheaper than lime. A set of standards should be introduced by Austroads or a relevant state body to outline the procedure for stabilising expansive soils with fly ash based geopolymers.

The final objective, which was to determine whether geopolymer stabilisation could be used to stabilise existing structures in Queensland was not reached, as the author decided that stabilisation of existing structures was a very limited scope and not of great importance, given the relatively small amount of material required for stabilisation of existing foundations. Instead the author focused on determining if geopolymers could be used in Queensland to replace existing stabilisation methods for large projects such as pavements and new structures. As discussed in the above paragraph, it could provide a sustainable, low carbon alternative to traditional methods , provided the cost decreases to make it economical.

6.2 Further Work

There are a few ideas for further work on the subject that the author suggests. Firstly, during the research it was noted that traditional lime and cement stabilisation techniques can fail to provide adequate improvements for some soils. These soils usually contain high levels organic materials or sulphides. These soils are not uncommon in Queensland and are generally dealt with using other methods, such as removal and replacement or the addition of extra improved layers. Investigation should be made into whether alkali activated geopolymers encounter the same difficulties as cement and lime in these soils. Given that geopolymers gain their strength from a completely different chemical reaction than cement, it is possible that they will be able to stabilise these soils more effectively than traditional methods. Researchers wishing to investigate this possibility should find a variety of these difficult soils and test them using the methods outlined in this paper.

Another option that needs to be explored is the practical application of alkali activated geopolymers in Queensland. A suitable road project could be identified and geopolymer stabilisation techniques could be applied to determine an effective method of treatment. This could then be monitored to see if it provides a long-term improvement in performance.

References

- Amiralian, S, Chegenizadeh, A, & Nikraz, H 2012 ‘ A review on the lime and fly ash application in soil stabilization’ *International Journal of Biological, Ecological and Environmental Sciences* (IJBEES) Vol. 1, No. 3, 2012

- Anusha, T. and Ramakrishna, M, 2017 ‘Soil stabilization by alkali activated fly ash’ *Asian Journal of Science and Technology* Vol. 08, Issue, 01, pp.4239-4243, January 2017

- Australian Standard AS 1289.1.1-2001 Sampling and Preparation of soils Preparation of disturbed soil samples for testing

- Australian Standards AS 1289.2.1.1-2005 ‘Methods of testing soils for engineering purposes, determination of soil moisture contents’

- Australian Standards AS 1289.3.1.1-2009 ‘Soil classification tests- Determination of the liquid limit of a soil- Four point Casagrande method’

- Australian Standards AS1289.3.2.1-2009 ‘Determination of the plastic limit of a soil- Standard method’

- Australian Standard AS1289.3.3.1-2009 ‘Calculation of the plasticity index of a soil

- Australian Standard AS1289.3.4.1-2008 ‘Determination of the linear shrinkage of a soil- standard method

- Australian Standard AS1289.3.6.1-2009 ‘Determination of the particle size distribution of a soil- standard method of analysis by sieving

- Australian Standard AS1289.5.1.1-2017 ‘Soil compaction and density tests- Determination of the dry density/moisture content relation of a soil using standard compactive effort’

- Australian Standard AS1289.6.1.1-2014 ‘Soil strength and consolidation tests- Determination of the California bearing ratio of a soil-Standard laboratory method for a remoulded specimen

- Guide to Pavement Technology Part 1 – Austroads 2009, amended August 2018 accessed 2019 www.austroads.com.au

- Guide to Pavement Technology Part 2 – Pavement structural design, Austroads 2018, Published October 2018, accessed 2019 www.austroads.com.au

- Guide to Pavement Technology Part 4D– Stabilised materials Austroads 2019, edition 2.1 published April 2019 www.austroads.com.au

- Guide to Pavement Technology Part 6- Unsealed Pavements, edition 1.1 published August 2018 www.austroads.com.au

- AustStab Technical Note, 'Cement Stabilisation Practice', No 5 March 2012

- AustStab Technical Note, 'Lime Stabilisation Practice', No 1F January 2008

- Bade, R, Khan, T 2018, 'Behaviour of expansive black cotton soil treated with sodium silicate' *International Journal of Advances in Science Engineering and Technology*,Volume-6, Issue-2, Apr.-2018

- Bentz, D, 2014 'Activation Energies of High Volume Fly Ash Ternary Blends: Hydration and Setting'- Cement and Concrete Composites, Volume 53 Pages 214-223, Published October 2014
- Black, J 2012, 'Mix design process for alkaline activated class F fly ash geopolymer concrete' *University of New South Wales, Australian Defence Force Academy*

- Caunce, C 2010, 'Effective Road Pavement Design for Expansive Soils in Ipswich' University of Southern Queensland

- Geoengineer.org ,www.geoengineer.org/education/pavement-geotechnics/soil-and-pavement-cracks-in-swelling-ground viewed 10 July 2019

- Hurley, C, 1965, 'Sodium silicate stabilization of soils: A review of the literature' University of Illinios 1965

- Ikeagwuani, C, Nwuonu, D 2019, 'Emerging trends in soil stabilization: A review, *'Journal of rock mechanics and geotechnical engineering'* Issue 11, pp423-440 2019

- Kabir S, Alengaram U, Jumaat M, Sharmin A and Islam A ,2014, 'Influence of molarity and chemical composition on the development of compressive strength in POFA based geopolymer mortar' *Advances in Materials Science and Engineering, volume 2015.*

- Krishna, B 2017, 'Soil Stabilisation with fly ash*' International Journal of research sciences and advanced engineering* Volume 2, Issue 19 Jul-Sep 2017

- Nadgouda K, Hegde R, 'The effect of Lime Stabilisation on Properties of Black Cotton Soil', *Indian Geotechnical Conference 2010',* December 16-18 2010 Bombay, India

- McLellan B, Williams R, Lay J, van Reissen A, Corder G, 2011 '*Costs and carbon emissions for geopolymer pastes in comparison to ordinary Portland cement'* Journal of cleaner production, volume 19 2011 , pages 1080-1090

- Morsy, M, Alsayed, S, Al-Salloum, Y, 2014, 'Effect on Sodium Silicate to Sodium Hydroxide Ratios on Strength and Microstructure of Fly Ash Geopolymer' *Arabic journal of science and engineering,* Published April 2014

- Murmu, A, Dhole, N, Patel, A 2018, 'Stabilisation of black cotton soil for subgrade application using fly ash geopolymer' *Road materials and pavement design* April 2018

- Omo, Y, & Kalita, A, 2017, ' California Bearing Strength and unconfined compressive strength of Soil-Fly ash glass fiber mixture' *International Journal of Engineering Technology , Management and Applied Sciences.* Volume 5 Issue 6 June 2017

- Palomo, A & Fernandez-Jimenez, A 2011, 'Alkaline activation , procedure for transforming fly ash into new materials. Part 1: Applications', *'World of Coal ash conference May 9-12 2011, Denver Colorado USA*

- Pourakbar S, Asadi A, Huat B, Fasihnikoutalab M, 2015, Journal of *Environmental Geotechnics* September 2015

- Queensland Department of Transport and Main Roads, 'Supplement to part 2: Pavement Structural Design of the Austroads Guide to Pavement Technology. July 2018

- Nelson, J &Miller, J, E 1992 'Expansive Soils: Problems and Practice in Foundation and Pavement Engineering. John Wiley and Sons Inc 1992

- Das, B & Sobhan, K, 'Principles of Geotechnical Engineering' 8th Edition Cengage Learning USA, 2014

- Kwan, T, 2018, 'Management of Soil Problems', 1st Edition, Springer International Publishing, USA 2018.
- USDA Soil classification www.nrcs.usda.gov/wps/portal/nrcs/detail/soils/survey/?cid=nrcs142p2_054167, visited 12 July 2019

- White G, 2010, '*Stabilisation of unsealed roads'* IPWEA NSW division annual conference 2010

- Yun-Ming L, Cheng-Yong H, AAl Bakri M 2016, *Structure and properties of clay based geopolymer cements: A review* Progress in Materials Science volume 83

Appendix A -Project Specification

ENG4111/4112 Research Project Specification

For: Dylan Daley

Title: Use of geopolymer for stabilizing expansive soils

Major: Civil Engineering

Supervisor: Andreas Nataatmadja

Enrolment: ENG4111-EXT S1 2019
ENG4112-EXT S2 2019

Project Aim: 'To investigate the possibility of using geopolymers containing sodium silicate and sodium hydroxide to reduce the shrink-swell variation in expansive clay soils in the South East Queensland region and its use to stabilise the foundations of existing structures.

Programme: Version 2- 12th April 2019

1. Research information relating to the use of geopolymers for geotechnical stabilisation, shrink-swell characteristics of reactive clays and appropriate testing procedures.

2. Identify 1-2 locations likely to be suitable for testing. If soil tests are available use those, if not conduct preliminary soil tests to identify suitability for testing. Preliminary tests most likely to consist of Plasticity index test and identification of Atterberg limits, most usefully Liquid Limit. (10 May)

3. Identify best geopolymer ratio and solution for testing, using past research results. (17 May)

4. Decide on appropriate testing procedures for samples using Australian Standards and past research. Most likely tests to be conducted will be California Bearing Ratio, Clay Content, Plasticity index and Atterburg Limits. (17 May)

5. Organise laboratory access and equipment for testing. Ensure risk assessments are completed prior to commencement of tests. (31st May)

6. Conduct testing for samples IAW Australian Standards. At this stage testing likely to consist of CBR, Atterburg Limits, clay content, Plasticity index. (Completed by 31 July)

7. Analyse test results and compare with current stabilisation methods for expansive soils. (31 Aug)

8. Determine on whether geopolymer stabilisation could be useful in stabilizing foundations of existing structures in South East Queensland

9. Write up final project report (30 Sep)

Appendix B – Risk Assessment

University of Southern Queensland

USQ Safety Risk Management System

Print View

Version 2.0

Safety Risk Management Plan

Risk Management Plan ID: RMP_2019_3472	*Status:* *Approve*	Current User: i:0#.w|usq\w0070200	Author: i:0#.w|usq\w0070200	Supervisor: i:0#.w|usq\u1065097	Approver: i:0#.w|usq\u1065097

Assessment Title:	geopolymer stabilisation of expansive soils	Assessment Date:	4/05/2019
Workplace (Division/Faculty/Section):	Z11	Review Date:	4/05/2019 (5 years maximum)

Approver: *Andreas Nataatmadja*	*Supervisor: (for notification of Risk Assessment only)* *Andreas Nataatmadja*

Context

DESCRIPTION:

What is the task/event/purchase/project/procedure?	Conducting preliminary testing (atterburg limits, clay content and OMC) of expansive clay soils and then preparing samples with a fly ash/sodium silicate solution. After a curing time conduct CBR and shrink swell testing
Why is it being conducted?	To investigate the effects of geopolymers on stabilising expansive soils
Where is it being conducted?	in laboratory Z11
Course code (if applicable)	ENG4111/4112
Chemical Name (if applicable)	Sodium silicate /sodium hydroxide

WHAT ARE THE NOMINAL CONDITIONS?

Personnel involved	
Equipment	CBR testing machine, cassagrande bowl testing machine, drying oven, compression machine,
Environment	
Other	
Briefly explain the procedure/process	Preliminary testing of samples to find the OMC, Atterburg limits, shrink-swell limits and CBR test results. Samples will then be prepared with a sodium silicate and fly ash mix to stabilise the sample. Sodium hydroxide will be used to accelerate the reaction. After curing the samples will testing again using the CBR test and shrink-swell tests. The results will then be analysed for effectiveness of geopolymerisation

Assessment Team - who is conducting the assessment?

Assessor(s):	Dylan Daley
Others consulted: (eg elected health and safety representative, other personnel exposed to risks)	

Risk Matrix

Probability	Insignificant No Injury 0-$5K	Minor First Aid $5K-$50K	Moderate Med Treatment $50K-$100K	Major Serious Injury $100K-$250K	Catastrophic Death More than $250K
	Consequence				
Almost Certain 1 in 2	M	H	E	E	E
Likely 1 in 100	M	H	H	E	E
Possible 1 in 1,000	L	M	H	H	H
Unlikely 1 in 10,000	L	L	M	M	M
Rare 1 in 1,000,000	L	L	L	L	L

Recommended Action Guide

Extreme:	E= Extreme Risk – Task *MUST NOT* proceed
High:	H = High Risk – Special Procedures Required (Contact USQSafe) Approval by VC only
Medium:	M= Medium Risk - A Risk Management Plan/Safe Work Method Statement is required
Low:	L= Low Risk - Manage by routine procedures.

Risk Register and Analysis

	Step 1	Step 2	Step 2a	Step 2b	Step 3			Step 4				
	Hazards: From step 1 or more if identified	**The Risk:** What can happen if exposed to the hazard without existing controls in place?	**Consequence:** What is the harm that can be caused by the hazard without existing controls in place?	**Existing Controls:** What are the existing controls that are already in place?	**Risk Assessment:** Consequence x Probability = Risk Level			**Additional Controls:** Enter additional controls if required to reduce the risk level	**Risk assessment with additional controls:** Has the consequence or probability changed?			
					Probability	Risk Level	ALARP		Consequence	Probability	Risk Level	ALARP
	Example											
	Working in temperatures over 35^0 C	*Heat stress/heat stroke/exhaustion leading to serious personal injury/death*	*catastrophic*	*Regular breaks, chilled water available, loose clothing, fatigue management policy.*	*possible*	*high*	*No*	*temporary shade shelters, essential tasks only, close supervision, buddy system*	*catastrophic*	*unlikely*	*mod*	*Yes*
1	Sodium Silicate	Causes eye irritation	Minor	wear eye protection, gloves and all PPE in accordance with SDS. Eye wash station available.	Rare	Low	☐					☐
2	CBR and shri...	Drop injury to feet from heavy sample mould	Minor	Wear steel capped boots, prepare sample in accordance with standards	Rare	Low	☐					☐
3	sodium hydr...	Cause eye and skin burns	Moderate	Wear eye protection, gloves and full body coverings as recommended by SDS.	Rare	Low	☐					☐
4	Preparing sa...	Getting fingers of other body parts jammed in compression tool	Minor	Be careful when conducting experiment	Unlikely	Low	☐					☐

Step 5 - Action Plan (for controls not already in place)

	Additional Controls:	*Exclude from Action Plan: (repeated control)*	*Resources:*	*Persons Responsible:*	*Proposed Implementation Date:*

Supporting Attachments

No file attached

Step 6 – Request Approval

Drafters Name: *Dylan Daley* *Draft Date:* *4/05/2019*

Drafters Comments: *RMP will be reviewed during course of experiments and be reviewed if necessary*

Assessment Approval: **There are risks not marked as ALARP** 1

Maximum Residual Risk Level: **Low - Manager/Supervisor Approval Required** 1

Document Status: *Approve*

Step 6 – Approval

Approvers Name: *Andreas Nataatmadja* *Approvers Position Title:*

Approvers Comments:

I am satisfied that the risks are as low as reasonably practicable and that the resources required will be provided.

Approval Decision: Approve *Approve / Reject Date:* *7/05/2019* *Document Status:* *Approve*

Appendix C- Laboratory Worksheets

Liquid Limit Testing

Four Point Casagrande Bowl Method- AS 1289.3.1.1

Project	FA Geopolymers
Date of Testing	Jun 19
Technician	D. Oxley
Location	USQ Toowoomba

Sample Description	Clay, Mosborg, untreated

	Number of Blows			
	41	32	23	16
Container Weight (g)	110.9	114.7	110.3	108.3
Wet Soil+ Container (g)	133.0	136.8	140.5	143.2
Dry Soil + Container (g)	123.6	127.3	127.2	127.5
Moisture Content (%)	42.5	43.2	44.1	45

Liquid Limit (25 blows) %	44%

Liquid Limit Testing

Four Point Casagrande Bowl Method- AS 1289.3.1.1

Project	FA Geopolymer testing
Date of Testing	6/8/19
Technician	D.Daly
Location	USQ Toowoomba

Sample Description	Black Clay, 20% FA, 5MOL NaOH+NaSiO3, 28 day cure

	Number of Blows			
	41	32	20	15
Container Weight (g)	109.4	108.3	109.1	110.4
Wet Soil+ Container (g)	133.3	129.3	129.0	135.4
Dry Soil + Container (g)	124.8	121.8	121.8	126.15
Moisture Content (%)	35.5	35.9	36.2	37

Liquid Limit (25 blows) %	36%

California Bearing Ratio

Conducted IAW AS 1289.6.1.1

Project	FA Geopolymer stab
Date of Testing	28/6/19
Technician	D.Dailey
Location	USQ Toowoomba

Sample Description	Sample 1, Untreated, 4 Day soak

Maximum Dry Density	t/m3	1.5596 t/m3
Optimum Moisture Content	%	20
Field Moisture Content	%	17.5

Dry Density (before soaking)	t/m3	1.537 t/m³
Density ratio (before soaking)		98.6
Moisture Content (Before Soaking)	%	20
Moisture Ration (Before Soaking)	%	100
Days Soaked		4
Surcharge Weight	kg	4.5
Swell (After Soaking)	%	6.58
Moisture Content (After Soaking)	%	36.5 - hmm?

K = 0.2325 kN/Div

Piston Penetration (mm)	Division	Load kN
0.5	0	0
1	0	0
1.5	0	0
2	0	0
2.5	0	0
3	0	0
4	0	0
5	~~0.01~~ 1	0.23
7.5	~~0.01~~ 1	0.23
10	~~0.01~~ 1	0.23
12.5	~~0.01~~ 1	0.23

Sample bad, fell apart after soaking

CBR at 2.5mm or 5mm (%)	0

California Bearing Ratio

Conducted IAW AS 1289.6.1.1

Project	FA Geopolymer stab
Date of Testing	28/6/19
Technician	D. Dailey
Location	USQ Toowoomba

Sample Description	Sample 2, Untreated, 4 Day soak

Maximum Dry Density	t/m3	1.5596 t/m3
Optimum Moisture Content	%	20
Field Moisture Content	%	17.5

Dry Density (before soaking)	t/m3	1.549
Density ratio (before soaking)		99.3
Moisture Content (Before Soaking)	%	20
Moisture ~~Ration~~ Ratio (Before Soaking)	%	100.5
Days Soaked		4
Surcharge Weight	kg	4.5
Swell (After Soaking)	%	5.95
Moisture Content (After Soaking)	%	28.4

Piston Penetration (mm)	Division	Load
0.5	1	0.23
1	1	0.23
1.5	1.2	0.277
2	1.3	0.3
2.5	1.4	0.33
3	1.6	0.37
4	2	0.46
5	2	0.46
7.5	2	0.46
10	2.1	0.482
12.5	2.1	0.488

CBR at 2.5mm or ~~5mm~~ (%)	2.46

California Bearing Ratio

Conducted IAW AS 1289.6.1.1

Project	FA Geopolymer stab
Date of Testing	28/6/19
Technician	D. Daley
Location	USQ Toowoomba

Sample Description	Sample 3, Untreated, 4 Day soak.

Maximum Dry Density	t/m3	1.5596 t/m3
Optimum Moisture Content	%	20
Field Moisture Content	%	17.5

Dry Density (before soaking)	t/m3	1.5518
Density ratio (before soaking)		99.5
Moisture Content (Before Soaking)	%	20.2
Moisture Ration (Before Soaking)	%	101
Days Soaked		4
Surcharge Weight	kg	4.5
Swell (After Soaking)	%	4.1
Moisture Content (After Soaking)	%	27.3

Piston Penetration (mm)	Division	Load
0.5	1	0.23
1	1.1	0.226
1.5	1.3	0.30
2	1.5	0.348
2.5	1.5	0.348
3	1.6	0.37
4	1.7	0.395
5	2	0.46
7.5	2	0.46
10	2.1	0.49
12.5	2.1	0.49

CBR at 2.5mm or ~~5mm~~ (%)	2.64

Appendix D -Raw LoadTrac 2 Data

Fly Ash Geopolymer Treated CBR test results

Sample 1

Time msec	Load kN	Disp mm
0	0.015572	-40.789
14994	0.073069	-40.537
28744	0.079657	-40.285
41494	0.097025	-40.033
54494	0.18028	-39.787
69244	0.35935	-39.536
83994	0.64624	-39.283
96994	0.92354	-39.036
110245	1.2134	-38.785
123995	1.4907	-38.535
138495	1.7297	-38.286
151245	1.9106	-38.032
164245	2.0711	-37.786
176995	2.2166	-37.536
190745	2.3622	-37.283
203495	2.4861	-37.032
216245	2.5993	-36.783
228995	2.7071	-36.536
242245	2.8036	-36.287
255495	2.8964	-36.037
267745	2.9719	-35.779
280745	3.0479	-35.536
293745	3.1168	-35.287
306245	3.1785	-35.034
318745	3.233	-34.788
332496	3.2911	-34.536
345746	3.3384	-34.288
359496	3.3803	-34.038
372496	3.4175	-33.788
386246	3.4468	-33.534
398746	3.4726	-33.288
412246	3.4989	-33.036
424496	3.5247	-32.788
437746	3.5534	-32.53
450247	3.581	-32.288
462996	3.6085	-32.036
476996	3.6397	-31.786
489246	3.6672	-31.534
502496	3.693	-31.286
514746	3.7169	-31.036
526996	3.7397	-30.788
539746	3.7624	-30.538
552997	3.7852	-30.286
565747	3.808	-30.032
577747	3.8289	-29.785
590997	3.8487	-29.538
604247	3.869	-29.286
616997	3.8882	-29.038
630248	3.9062	-28.785
643497	3.9253	-28.536
656747	3.9397	-28.285

Sample 2

Time msec	Load kN	Disp mm
0	0.0053903	-41.129
14250	0.078459	-40.878
26500	0.21142	-40.624
40250	0.40487	-40.376
54250	0.68457	-40.122
67750	0.98583	-39.878
81500	1.3146	-39.626
95500	1.6464	-39.375
109500	1.91	-39.124
122750	2.1064	-38.878
137250	2.2807	-38.625
150751	2.4143	-38.376
164501	2.5287	-38.125
178001	2.6269	-37.873
191501	2.7107	-37.627
203751	2.7826	-37.377
217001	2.8497	-37.127
229751	2.9096	-36.877
243001	2.9677	-36.628
256001	3.0192	-36.374
269501	3.0719	-36.119
281001	3.118	-35.873
293501	3.1605	-35.627
307001	3.2066	-35.378
319251	3.248	-35.124
332001	3.2887	-34.878
344751	3.3306	-34.625
357252	3.3695	-34.375
370252	3.4085	-34.127
384002	3.4456	-33.878
397002	3.4827	-33.625
410002	3.5169	-33.378
423002	3.5498	-33.125
436752	3.5798	-32.874
449752	3.6091	-32.628
462752	3.6337	-32.378
475502	3.6558	-32.121
489502	3.6774	-31.874
501752	3.6924	-31.628
514002	3.7061	-31.377
528252	3.7235	-31.123
541252	3.7355	-30.874
553002	3.7481	-30.627
566002	3.7624	-30.375
578253	3.7744	-30.125
591253	3.79	-29.877
603753	3.8026	-29.619
616503	3.8157	-29.374
630003	3.8277	-29.125
643253	3.8409	-28.877
655253	3.8541	-28.628

Sample 3

Time msec	Load kN	Disp mm
0	0.0059892	-38.591
13246	0.046716	-38.339
28496	0.26173	-38.087
41246	0.63366	-37.838
55996	1.1607	-37.585
69746	1.5925	-37.335
83497	1.9172	-37.089
96247	2.14	-36.835
108997	2.313	-36.588
121497	2.4466	-36.337
133747	2.5526	-36.085
146497	2.6496	-35.837
159747	2.7335	-35.589
173497	2.8095	-35.337
185497	2.8724	-35.087
197747	2.9299	-34.838
211497	2.9904	-34.589
223747	3.0389	-34.335
237497	3.0904	-34.081
250747	3.133	-33.834
264747	3.1755	-33.589
277747	3.209	-33.335
291497	3.245	-33.089
304748	3.2731	-32.839
318248	3.2965	-32.587
329998	3.318	-32.333
342248	3.3354	-32.087
356248	3.3564	-31.837
368998	3.3719	-31.584
381998	3.3887	-31.339
394498	3.4025	-31.087
407248	3.4192	-30.83
420498	3.4324	-30.588
432748	3.4474	-30.339
445498	3.4606	-30.09
458248	3.4773	-29.837
469998	3.4899	-29.59
483748	3.5049	-29.333
496998	3.5169	-29.09
510498	3.5342	-28.838
523499	3.548	-28.584
536999	3.5612	-28.34
549249	3.575	-28.09
562249	3.5881	-27.837
575999	3.6007	-27.586
588999	3.6121	-27.333
601749	3.6253	-27.086
614499	3.6355	-26.837
627499	3.6486	-26.586
640749	3.6594	-26.338

UCS Testing for Untreated Soil

Sample 1

Time msec	Load N	Disp mm
0	106.88	-30.457
7487	134.94	-30.252
13487	158.82	-30.048
19237	173.75	-29.847
25737	192.26	-29.655
31237	205.99	-29.456
37487	231.67	-29.252
43487	265.1	-29.054
48987	292.57	-28.854
55487	324.21	-28.651
61487	349.89	-28.453
67487	371.98	-28.252
72987	389.89	-28.056
78987	407.21	-27.853
84987	423.33	-27.651
90238	435.87	-27.455
95987	449	-27.257
102237	462.14	-27.05
107987	472.29	-26.852
113487	480.65	-26.656
119488	489.6	-26.452
125988	497.96	-26.243
131738	504.53	-26.056
137738	509.9	-25.854
143738	514.68	-25.651
150488	518.86	-25.455
156238	521.84	-25.248
161988	524.83	-25.054
168238	527.22	-24.856
174738	530.8	-24.648
180488	531.99	-24.455
186488	532.59	-24.253
192238	533.79	-24.053
198488	533.79	-23.855
204238	533.79	-23.653
210488	533.19	-23.452
216488	533.19	-23.248
222238	532.59	-23.054
227988	531.4	-22.855
233988	530.8	-22.646
239738	530.2	-22.449
245488	527.81	-22.245
250988	527.81	-22.051
256488	526.02	-21.854
262738	524.23	-21.651
268738	523.64	-21.457

Sample 2

Time msec	Load N	Disp mm
0	113.44	-19.187
6997	139.12	-18.974
12497	163	-18.782
18497	182.71	-18.584
24747	184.5	-18.38
30497	191.06	-18.18
36247	217.34	-17.984
42247	246.59	-17.786
48497	274.06	-17.585
54747	300.33	-17.378
60497	322.42	-17.177
65747	340.93	-16.98
71497	360.04	-16.779
77497	379.14	-16.583
83747	396.46	-16.382
89247	411.39	-16.184
95497	426.31	-15.987
101247	438.85	-15.783
106997	449.6	-15.586
112997	460.94	-15.38
118997	470.5	-15.177
125247	478.85	-14.987
131497	487.81	-14.786
137247	494.38	-14.587
143747	501.54	-14.384
149247	506.92	-14.183
155247	510.5	-13.98
161497	514.68	-13.783
167497	518.26	-13.58
172997	521.84	-13.383
178497	524.23	-13.186
184747	527.22	-12.981
190748	528.41	-12.783
196248	528.41	-12.584
201998	529.61	-12.385
208248	529.61	-12.181
214498	529.61	-11.984
220248	528.41	-11.784
225248	527.81	-11.587
232248	527.22	-11.383
237748	524.83	-11.181
243998	522.44	-10.98
249748	521.25	-10.781

UCS Testing for Fly Ash Geopolymer Treated Soil

Sample 1

Time msec	Load kN	Disp mm
0	0.029946	-27.467
13994	0.062887	-27.205
26744	0.079657	-26.966
40744	0.10601	-26.716
53744	0.16111	-26.466
67244	0.28389	-26.215
80494	0.4468	-25.961
92744	0.6109	-25.716
107494	0.79058	-25.461
119494	0.91336	-25.212
132494	1.0158	-24.965
146494	1.0469	-24.716
158244	0.88641	-24.462
170494	0.666	-24.215
182743	0.53304	-23.966
195493	0.42943	-23.715
208493	0.35157	-23.466

Sample 2

Time msec	Load kN	Disp mm
0	0.077261	-28.083
13493	0.081453	-27.83
27493	0.11978	-27.578
40493	0.18866	-27.332
53743	0.28748	-27.082
66993	0.38631	-26.826
79993	0.4426	-26.582
93493	0.46656	-26.329
105493	0.48573	-26.076
118493	0.5025	-25.826
132243	0.5013	-25.579
145243	0.47255	-25.326
158242	0.40367	-25.08
170492	0.37912	-24.83
183492	0.37193	-24.578
186258	0.36954	-24.527

Sample 3

Time msec	Load kN	Disp mm
0	0.084448	-26.708
13493	0.15273	-26.454
26993	0.21561	-26.207
40493	0.31623	-25.957
54493	0.4474	-25.705
67243	0.58575	-25.457
79493	0.70793	-25.201
92743	0.82292	-24.957
106743	0.91935	-24.695
118742	0.9469	-24.451
130992	0.8912	-24.203
143242	0.74626	-23.951
154992	0.56958	-23.707

Appendix D- Circly Output Files

Heavy Duty Unstabilised Pavement

```
CIRCLY  - Version 6.0 (30 January 2015)

Job Title: demo1

Damage Factor Calculation

Assumed number of damage pulses per movement:
   Combined pulse for gear (i.e. ignore NROWS)

Traffic Spectrum Details:

   Load   Load                   Movements
   No.    ID
    1     ESA750-Full            5.00E+07

Details of Load Groups:

   Load   Load                  Load                  Load             Radius    Pressure/    Exponent
   No.    ID                    Category              Type                       Ref. stress
    1     ESA750-Full           ESA750-Full           Vertical Force      92.1     0.75          0.00

   Load Locations:
   Location    Load                 Gear          X           Y      Scaling      Theta
   No.         ID                   No.                              Factor
    1          ESA750-Full           1         -165.0        0.0    1.00E+00       0.00
    2          ESA750-Full           1          165.0        0.0    1.00E+00       0.00
    3          ESA750-Full           1         1635.0        0.0    1.00E+00       0.00
    4          ESA750-Full           1         1965.0        0.0    1.00E+00       0.00

Layout of result points on horizontal plane:
   Xmin:  0    Xmax:  165    Xdel:  165
   Y:     0

Details of Layered System:

   ID: Aust2004-2 Title: Austroads 2004 - Example 2 - Asphalt Pavement containing Cemented Layer

   Layer  Lower   Material               Isotropy   Modulus    P.Ratio
   No.    i/face  ID                                (or Ev)    (or vvh)  F           Eh          vh
    1     rough   Asph3000               Iso.       3.00E+03   0.40
    2     rough   Gran_450               Aniso.     4.50E+02   0.35      3.33E+02    2.25E+02    0.35
    3     rough   Cement3500             Iso.       3.50E+03   0.20
    4     rough   Sub_CBR6               Aniso.     6.00E+01   0.45      4.14E+01    3.00E+01    0.45

   Performance Relationships:
   Layer  Location Material              Component  Perform.   Perform.  Traffic
   No.             ID                               Constant   Exponent  Multiplier
    1     bottom   Asph3000              ETH         0.004067    5.000      1.600
    3     bottom   Cement3500            ETH         0.000350   12.000     12.000
    4     top      Sub_CBR6              EZZ         0.009300    7.000      1.100

   Reliability Factors:
   Project Reliability: Austroads 97.5%
   Layer  Reliability  Material
    No.   Factor        Type
    1      0.67        Asphalt
    3      0.50        Cement Stabilised
    4      1.00        Subgrade (Austroads 2004)

   Details of Layers to be sublayered:
   Layer no.  2:  Austroads (2004) sublayering

Results:

   Layer  Thickness  Material              Load                     Critical     CDF
   No.               ID                    ID                       Strain
    1      175.00    Asph3000              ESA750-Full              -9.55E-05    8.54E-01
    2      200.00    Gran_450                             n/a                    n/a
    3      150.00    Cement3500            ESA750-Full              -5.60E-05    3.37E-01
    4        0.00    Sub_CBR6              ESA750-Full               1.63E-04    2.84E-05
```

Fly Ash Geopolymer Stabilised Heavy-Duty Pavement

```
CIRCLY  - Version 6.0 (30 January 2015)

Job Title: stab

Damage Factor Calculation

Assumed number of damage pulses per movement:
   Combined pulse for gear (i.e. ignore NROWS)

Traffic Spectrum Details:

   Load   Load                 Movements
   No.    ID
    1     ESA750-Full          5.00E+07

Details of Load Groups:

   Load   Load                 Load                 Load             Radius    Pressure/    Exponent
   No.    ID                   Category             Type                       Ref. stress
    1     ESA750-Full          ESA750-Full          Vertical Force     92.1     0.75          0.00

   Load Locations:
   Location   Load                 Gear        X           Y      Scaling     Theta
   No.        ID                   No.                            Factor
    1         ESA750-Full           1       -165.0        0.0    1.00E+00      0.00
    2         ESA750-Full           1        165.0        0.0    1.00E+00      0.00
    3         ESA750-Full           1       1635.0        0.0    1.00E+00      0.00
    4         ESA750-Full           1       1965.0        0.0    1.00E+00      0.00

Layout of result points on horizontal plane:
   Xmin:  0    Xmax:  165    Xdel:  165
   Y:     0

Details of Layered System:

   ID: Aust2004-2 Title: Austroads 2004 - Example 2 - Asphalt Pavement containing Cemented Layer

   Layer  Lower    Material               Isotropy   Modulus    P.Ratio
   No.    i/face   ID                                (or Ev)    (or vvh)  F          Eh         vh
    1     rough    Asph3000               Iso.       3.00E+03   0.40
    2     rough    Gran_450               Aniso.     4.50E+02   0.35      3.33E+02   2.25E+02   0.35
    3     rough    Cement3500             Iso.       3.50E+03   0.20
    4     rough    Sub_CBR15              Aniso.     1.50E+02   0.45      1.03E+02   7.50E+01   0.45

   Performance Relationships:
   Layer  Location Material               Component  Perform.   Perform.  Traffic
   No.             ID                                Constant   Exponent  Multiplier
    1     bottom   Asph3000               ETH         0.004067    5.000     1.600
    3     bottom   Cement3500             ETH         0.000350   12.000    12.000
    4     top      Sub_CBR15              EZZ         0.009300    7.000     1.100

   Reliability Factors:
   Project Reliability: Austroads 97.5%
   Layer  Reliability  Material
    No.   Factor        Type
    1      0.67        Asphalt
    3      0.50        Cement Stabilised
    4      1.00        Subgrade (Austroads 2004)

   Details of Layers to be sublayered:
   Layer no.  2:  Austroads (2004) sublayering

Results:

   Layer  Thickness  Material               Load                        Critical     CDF
   No.               ID                     ID                          Strain
    1       175.00   Asph3000               ESA750-Full                 -9.64E-05    8.95E-01
    2       200.00   Gran_450                               n/a                      n/a
    3       150.00   Cement3500             ESA750-Full                 -4.16E-05    9.48E-03
    4         0.00   Sub_CBR15              ESA750-Full                  1.14E-04    2.23E-06
```