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Stabilisation of Expansive Subgrade Soils with Slag and Cement for Road Construction

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DECLARATION

To the best of my knowledge and belief this thesis contains no material previously published by any other person except where due acknowledgment has been made.

This thesis contains no material which has been accepted for the award of any other degree or diploma in any university.

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ABSTRACT

This research evaluates the suitability of using slag and cement as stabilisers to improve the performance of expansive soil as a subgrade for road pavement. Several laboratory tests were conducted to determine the characteristics of the expansive soil used and associated behaviour. The tests included the particle size distribution, standard proctor compaction, soil particle density, Atterberg limits, free swelling, acidity and basicity measurement and permeability. The performance tests included the California bearing ratio (CBR) test, unconfined compressive strength (UCS) test and repeated load triaxial (RLT) test. The use of slag as well as slag accompanied with cement as stabilisers followed three proportion schemes. The selection of a specific stabiliser proportion was determined based on obtaining the UCS test results that satisfied the required standard as a subgrade. Based on the results of this study the recommended stabiliser proportion specific to the studied soil was 13.5% slag + 1.5% cement at 28 days curing time. This mixture resulted in a remarkable increase in UCS strength of eight times magnitude higher than the strength of the non-stabilised soil. The corresponding CBR values were more than four times higher than the minimum required for designing road pavement. The resilient modulus of the soil stabilised with this mix (determined from the RLT test) was found to be dependent on the deviator stress. The model that was found to correlate the best with the deviator stress was the hyperbolic correlation model, judged based on the highest coefficient of determination value of R^2 = 0.96. The results presented herein confirm that exploitation of the by-product material of slag can indeed be useful, both in improving the strength of subgrade soils and sparing the environment a significant potential pollutant.

PUBLICATION

The following journal paper was submitted out of this research:

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1.1. Background of the Study

Road pavements commonly consist of several layers of various materials and thickness, as shown in Figure 1.1. Each layer assists systematically in supporting traffic load and distributing it safely to the foundation soil, which is known as a subgrade. The subgrade may be either a native soil or an imported material. When the native soil is deemed to be unsuitable as a subgrade, it is normally treated (stabilised) appropriately and used to avoid the high cost that may be incurred for imported material.

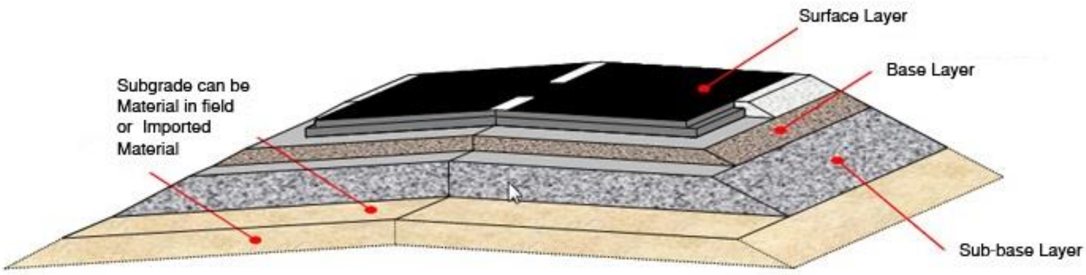

Figure 1.1: Road pavement layers

To determine the type of subgrade material that can be used as well as the appropriate type of treatment, a series of soil investigation has to be undertaken. Stability of the subgrade is normally expressed in terms of bearing capacity, which is related to certain geotechnical properties of the soil.

One type of foundation soil that is deemed problematic is called "reactive" or "expansive" soil, which is comprised primarily of a certain type of clay. This soil swells if there is an increase in its water content, and shrinks away if there is a reduction in water content. The fluctuation of moisture content usually occurs due to seasonal condition. The change in the soil volume associated with expansion or shrinkage results in deformation of the ground, either vertically or horizontally. In the case of pavement, this deformation can lead to substantial distortion of the road surface. The expansive soils commonly exist in most areas of Australia and can result in enormous damage to buildings and roads (Fredlund, 2006; Karunarathne, 2013).

Swelling and shrinkage of expansive soils are influenced by the following factors: the type and amount of clay minerals and cautions, water content, dry density, soil structure, and loading conditions. Several methods are used to reduce the volume change of expansive soils, and the method commonly used is the chemical soil stabilisation. In this method, a certain amount of a chemical compound is added to the expansive soil. The addition of lime, cement, fly ash, and other chemical compounds as additives in the soil stabilisation process has been successfully adopted for years (Neeraja and Rao, 2010).

To minimize the cost of subgrade stabilisation as well as reducing adverse environmental impact, road planners tend to reuse industrial waste for soil subgrade stabilisation. One of the iron making industrial wastes is ground granulated blast furnace slag (GGBS), which has been lately utilised as a stabiliser material for general ground improvement purposes. GGBS is commonly mixed with other stabilising materials such as cement or lime in stabilised road pavement layers.

In designing a highway pavement, engineers commonly rely on the results of what is called California Bearing Ratio test (CBR) of the subgrade to assess both the strength of the native soil and adequacy of the stabilised material. The CBR test is carried out both in-situ and laboratory, and the subgrade may be treated to achieve a certain CBR value. However, Satyanarayana and Rama (2005) found that the CBR testing method is limited due to its empirical nature. Furthermore, other test methods such as Group Index, Mc Leod and AASHTO do not take into consideration the risk of shear failure in the subgrade.

Cheung (1994) suggested that the resilient elastic modulus can be used to assess the stiffness of a subgrade material. This resilience modulus depends on confining stress, axial stress and matrix suction (pore water pressure) of the materials. The resilient modulus can be determined in the laboratory using the repeated load triaxial (RLT) test. This test is essentially a cyclic version of the conventional monotonic triaxial compression test; the cyclic or repeated load application is thought to more accurately simulate actual traffic loading, which is usually imposed on road pavement layers.

Some resilient modulus mathematical correlation models have been developed in the literature based on laboratory tests in order to determine resilient modulus value of certain material. These models correlate the resilient modulus with certain geotechnical properties and performances. In the current study, a specific resilient modulus mathematical correlation model was developed based on the repeated load triaxial (RLT) test.

1.2. Research Objectives

This research assesses the performance of an expansive soil stabilised with slag and cement mixed in certain proportions for a variety of geotechnical properties of the stabilised soil. Based on the background above, the objectives of this research are:

1. To determine the optimum proportion of stabiliser used in stabilised expansive soil that meets the allowable standard for subgrades of road pavements;

2. To evaluate the performance of a selected stabilised soil in terms of strength, bearing capacity and resilient modulus; and

3. To develop a correlation model between the resilient modulus and stresses applied to selected stabilised soil.

1.3. Scope of Work

The following points have been considered during this research:

- The work was undertaken on expansive soil retrieved from Western Australia.
- Preliminary tests were performed to the soil in order to verify that the soil can be classified as an expansive soil.
- The stabilisers used were limited to slag and cement.
- The standard laboratory compaction test was used to determine the maximum dry unit weight of both natural soil and stabilised soil.

- Strength of the treated soil was assessed by the unconfined compressive strength (UCS) test for curing time of 7, 14 and 28 days.
- Evaluation of the bearing capacity from the CBR test and resilient modulus from the RLT tests were made based on the result of the UCS tests after a specified curing time.

1.4. Thesis Outline

This thesis is organised into five chapters with the outline of each chapter as follows. Chapter 1 introduces the background of the study, research objectives, scope of works and outline of the thesis. Chapter 2 presents a review on expansive soils, soil stabilisations, slag as a soil stabiliser, design of stabilised soil subgrade, and resilient modulus. Chapter 3 presents the purpose of the experimental study, type of material used in the experiment, sample preparation methods and description of the research experiments. Chapter 4 presents and discusses the experimental results. Finally, Chapter 5 summaries and concludes the research findings and provides some recommendations for further studies.

Chapter 2. Literature Review

2.1 Introduction

This chapter presents some theories and previous research regarding stabilisation of expansive soils as well as utilisation of iron slag as a stabiliser. The literature review is divided into several sections, including review of expansive soils, review of soil stabilisation, slag as a soil stabiliser, design of stabilised soil subgrade and resilient modulus.

The review of expansive soils explains how expansive soils are identified in the field, where they exist, what problems and damage they can cause, and what kind of treatment they can receive to improve them. The review also includes coverage of the application methods of stabilisation treatment.

This chapter discusses slag as being one of the additive materials used in soil stabilisation. It starts with identifying the slag, followed by describing some chemical and physical properties of the slag. Some previous studies about a slag utilisation in soil stabilisation are also reviewed herein.

This chapter also discusses one of the important aspects in designing soil subgrade in terms of soil response to the load applied by traffic. To this end, the resilient elastic modulus of the subgrade material is explained in terms of its definition, calculation, correlation with other soil properties and soil state. Finally, the resulting correlations are compared with various correlation trends developed by previous studies.

2.2 Review of Expansive Soils

2.2.1. Identification of Expansive Soils

Expansive soils experience a significant change in volume due to changes in water content. The change in volume can be exhibited in the form of swelling (when the water content increases) or shrinkage (when the water content decreases). This behaviour is usually found in soils containing clay minerals of the smectite group, which includes montmorillonite and bentonite. The presence of expansive soil in the field can be easily recognised in the dry season by the appearance of deep cracks on the ground surface. The cracks usually form in roughly polygonal patterns as shown in Figure 2.1

Figure 2.1: Polygonal pattern of some soil surface cracks (Jones and Jefferson, 2012)

2.2.2. Locations of Expansive Soils

Expansive soils are found in many places of the world, mainly in arid and semi-arid areas. Global distribution of arid and semi-arid areas are shown in Figure 2.2.

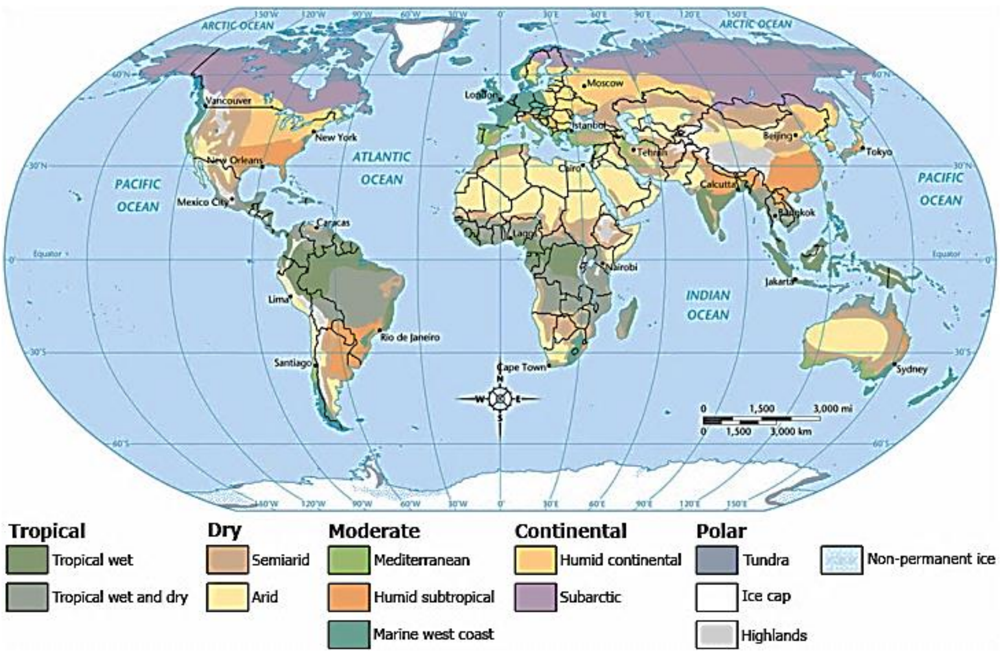

Figure 2.2: Worldwide climate classification map (Retrieved September 9, 2013, from http://en.wikipedia.org)

It can be seen from this figure that almost 95% of the Australian land is classified as arid and semi-arid. According to Davenport (2007), the expansive clays in Western Australia have been identified in several regions including Dalwallinu, Ongerup, Ravensthorpe, Newman, Kalgoorlie, Boulder, Perth metropolitan area (Kalamunda, Midland, Guilford, Gooseberry Hill, Swanview, Maylands, Kenwick, Armadale, Maddington, Viveash), Moora, Geraldton, Lake King, Coolgardie, Katanning, Mundijong, Jerramungup, Kununurra, Collie and Bunbury.

2.2.3. Problems of Expansive Soils

The swelling and shrinkage behaviour of expansive soils has been a worldwide problem imposing significant hazards to engineering structures. Structures most susceptible to damage are usually lightweight construction such as one floor houses and road pavements. Zheng et al. (2009) stated that the excessive volume changes of expansive soils can cause severe damage to buildings and other structures located on this kind of soil. The volume change may occur repeatedly and over a long time span.

Expansion of reactive soils causes foundation and road pavement damages by the uplift movement as the soil swell with moisture increases. Figure 2.3 and Figure 2.4 show damages of some structures due to expansive soil deformation.

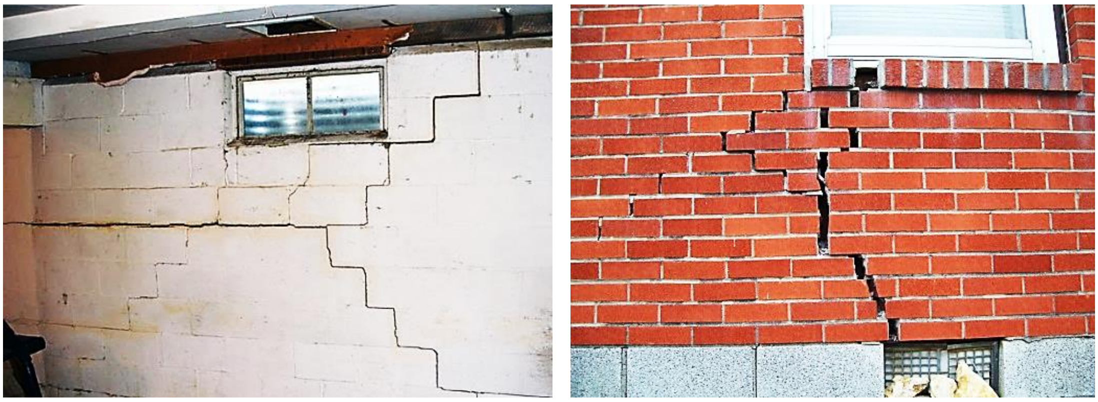

Figure 2.3: Wall building damages due to expansive soil deformation (Retrieved September 9, 2013 from http://www.basementsystems.ca, http://images.reachsite.com/)

Figure 2.4: Various road pavement damages due to expansive soil deformation (Retrieved September 9, 2013 from http://www.geoengineer.org, http://geosurvey.state.co.us and http://www.fhwa.dot.gov/)

Fluctuation of moisture content that triggers volumetric changes in expansive soils is not due to seasonal variation solely; there are several other causes such as, improper drainage system, local transpiration characteristics, local surface heat and poor irrigation infrastructures.

2.2.4. Degree of Expansion of Expansive Soils

Thorough site investigation supported with laboratory testing is required to determine the degree of expansion of expansive soils. Once determined, the degree of expansion can be used to decide on the type of treatment required for the soil. The appropriate soil treatment can minimise the degree of soil deformation and supported structures to within acceptable values.

Some laboratory tests have been developed to measure the swelling potential of expansive soils under changing moisture conditions. These tests include free swell test, expansion index, consolidation-swelling, California Bearing Ratio (CBR), potential volume change (PVC), and coefficient of linear extensibility (COLE). However, scientists tend to identify the shrink-swell behaviour by analysing a combination of physical, chemical, and mineralogical soil properties (Chen et al., 2011).

Muntohar (2006) concluded that three main soil properties influence soil swelling potential as follows: plasticity index, liquid limit and clay fraction. These properties were used to predict the degree of swelling of expansive soil. Skempton (1953) made a specific correlation between the plasticity index and clay content. The clay content can be presented as a percentage of soil particles less than 2 μm in size. Skempton's correlation was defined as a single parameter called clay activity, and was used to classified clays as shown in Table 2.1.

Table 2.1: Classes of clays according to activity (Skempton, 1953)

Clay activity value (A)	Activity classification
< 0.75	Inactive
0.75 – 1.25	Normal
> 1.25	Active

The activity value of clay can be obtained through the following formula:

$$\text{Activity (A)} = \frac{\text{Plasticity Index}}{\text{Clay Content}} \qquad (2.1)$$

Some other criteria regarding classification of expansive soils were used by Chen (1975), Holtz and Gibbs (1954) and Bureau of Indian Standards 1498 (1970), then by Sridharan and Prakash (2000). These criteria are summarised in Table 2.2 (using liquid limit as a defining parameter) and Table 2.3 (using plasticity index as a defining parameter). Another guide was suggested by the Austroads to identify and make a qualitative classification of expansive soils, as shown in Table 2.4.

Table 2.2: Soil degree of expansion by liquid limit

Degree of expansion	Liquid limit (%)	
	Chen (1975)	Indian Standards 1498 (1970)
Low	< 30	20–35
Medium	30–40	35–50
High	40–60	50–70
Very High	> 60	70–90

Table 2.3: Soil degree of expansion by plasticity index

Degree of expansion	Plasticity index (%)		
	Holtz and Gibbs (1954)	Chen (1975)	Indian Standards 1498 (1970)
Low	< 20	0–15	< 12
Medium	12–34	10–35	12–23
High	23–45	20–55	23–32
Very High	> 32	> 35	> 32

Table 2.4: Soil degree of expansion by other measures

Degree of expansion	Shrinkage limit (%)	Shrinkage index (%)	Free swell index (%)	% expansion in oedometer* (Holtz and Gibbs, 1954)	% expansion in oedometer+ (Seed et al., 1963)
Low	> 13	< 15	< 50	< 10	0.0–1.5
Medium	8–18	15–30	50–100	10–20	1.5–5.0
High	6–12	30–60	100–200	20–30	5–25
Very High	< 10	> 60	> 200	> 30	> 25

*From dry to saturated condition under a surcharge of 7 kPa.
+From compacted, saturated condition under a surcharge of 7 kPa.
Note: Shrinkage index = plastic limit – shrinkage limit.

2.2.5. Classification of Expansive Soils

The Unified Soil Classification System (USCS) designates a two letter symbol and a group name for each soil. A visual-manual procedure can also be used to identify soils easily in the field; however, all classifications provided in this research are based on the laboratory testing-based procedure. The Austroads (2010) Guide to Pavement Technology, Part 2: Pavement Structural Design provides a guide to the identification and qualitative classification of expansive soils, as shown in Table 2.5.

Table 2.5: Classification of expansive soils by Austroads (2010)

Expansive nature	Liquid limit (%)	Plasticity index	PI × (% < 0.425 mm)	Potential swell (%)*
Very high	> 70	> 45	> 3200	> 5.0
High	> 70	> 45	2200–3200	2.5–5.0
Moderate	50–70	25–45	1200–2200	0.5–2.5
Low	< 50	< 25	< 1200	< 0.5

*Swell at OMC and 98% MDD using standard compactive effort; 4 days soak based on 4.5 kg surcharge.

2.2.6. Treatments of Expansive Soils

According to Edil (2002), the unsuitable soil should be replaced by rocks that have better ability to support loads. However, due to the fact that the replacement cost of unsuitable soil, such as, the expansive soil may be very expensive, improving the geotechnical characteristics of this soil by a certain appropriate stabilisation method could be a viable option. Stabilisation of expansive soils should be guided through a series of geotechnical investigations starting with careful site investigations.

After preliminary field investigations and evaluation of soil properties in the laboratory, of the most suitable treatment could be determined. Nelson (1992) reported some treatments of expansive soils; those are chemical additives, pre-wetting, soil replacement with compaction control, moisture control, surcharge loading and thermal loading.

The pre-wetting treatment is aimed to increase the water content in the expansive foundation soils so that most of the expansion occurs before the construction begins. The high water content condition is then maintained in order to reduce the change of soil volume by shrinkage, thereby preventing damage to structures. A lower effort of soil compaction, equivalent to Standard Proctor Compaction test, on low density soil and at the above optimum moisture content can generate a smaller swelling potential of soil (Holtz, 1959). This soil compaction treatment can be applied on highway construction or light building. Surcharge loading treatment by placing a heavy load on the top of the soil surface is generally applied to expansive soil with low to moderate swelling pressures. This treatment is aimed to counteract pressure exerted by soil swelling.

Zha et al. (2006) studied the effect of fly ash on stabilising expansive soils without any other stabiliser. They found that there was no significant improvement of the UCS early strength; however, there was a striking increase of UCS values after 7 days of curing. Another study conducted by Solanki and Zaman (2010) confirmed the same trend of increase in UCS value of their samples. This result was observed when they evaluated the performance of two subgrade soils (*CL* and *CH*) stabilised with three different stabilisers (hydrated lime or lime), class C fly ash (*CFA*), and cement kiln dust (*CKD*).

Based on some type of soil treatments reported by considering the expansive soils classified as fine-grained soil, the recommended treatment for expansive soils is chemical stabilisation.

2.3 Review of Soil Stabilisation

2.3.1. Definition of Soil Stabilisation

As defined by McGraw-Hill Dictionary of Architecture and Construction (2003), soil stabilisation is a chemical or mechanical treatment designed to increase or maintain the stability of a soil mass or otherwise to improve its engineering properties by increasing its shear strength, reducing its compressibility, or decreasing its tendency to absorb water.

Another description provided by Joint Departments of the Army and Air Force, USA (1994) defines soil stabilisation as the process of blending and mixing materials with a soil to improve certain properties. The process may include blending of soils to achieve a desired gradation or mixing of commercially available additives that may alter the gradation, texture or plasticity, or act as a binder to cement the soil. The additive can be a manufactured commercial product or a made by-product of an industrial process. This additive when added to the soil in a proper quantity may improve some engineering characteristics of the soil such as strength, texture, workability, and plasticity.

In the first description mentioned above, there are two methods of soil stabilisation, namely mechanical stabilisation and chemical stabilisation. The mechanical stabilisation is carried out by the native soil with another of different gradation so that the final mixture has a gradation in accordance with the design requirements. The mixing process can be performed directly in the field or performed at a different location before being brought back to the job site, then the mixture is spread in the field and compacted to achieve the required density. The chemical stabilisation is executed by adding other additive materials at a certain portion of the treated soil. The type of stabiliser depends on the treated soil. Eventually, improvement occurs due to the chemical reaction between the additive and soil as well as strength and stiffness, the additive may also improve gradation, workability and plasticity of subgrade soil (USA Corps of Engineers, 2004)

2.3.2. Chemical Stabilisation and Additive Materials

Chemical stabilisation for subgrades in road pavement construction aims to modify the soil into a stable subgrade. The stable subgrade can be achieved by increasing the soil particle size through a reduction in the plasticity index, reducing the shrink/swell potential and cementation. The additive used in this method is considered as ingredients of pavements with cemented base. Petry and Little (2002) listed cemented additive materials used to stabilise soils. They divided the additives into three stabiliser types: (1) traditional stabilisers, such as hydrated lime, Portland cement, and fly-ash; (2) by-product stabilisers, such as cement kiln dust, lime kiln dust and slag; and (3) non-traditional stabilisers, such as sulfonated oils, potassium compounds, ammonium chloride, enzymes and polymers.

Texas Department of Transportation (2005) proposed several factors that can be used to select a suitable stabiliser, namely, soil mineralogy and content (sulfates, organics), soil classification (gradation and plasticity), goals of treatment, mechanisms of additives, desired engineering and material properties (strength, modulus), design life, environmental conditions (drainage, water table), engineering economics (cost savings versus benefit).

Based on the above criteria, Texas Department of Transportation (2005) proposed additives to be used with each category as listed in Table 2.6. However, the information in Table 2.6 is expected to be used as guidelines only, and some laboratory tests should be carried out to prove that the selected additive can achieve the degree of stabilisation required by the standard.

Table 2.6: Suggested initial additive(s) material (Texas Department of Transportation, 2005)

No	Soil subgrade (particle size ≥ 25% passing sieve No. 200)	Suggested initial additive(s) material
1	PI < 15	Cement, Asphalt (PI > 6), Lime-Fly Ash
2	15 ≤ PI < 35	Lime, Lime-Cement, Lime-Fly Ash, Fly Ash, Cement
3	PI ≥ 35	Lime, Lime-Cement, Lime-Fly Ash

In choosing the appropriate stabiliser, Veith (2000) suggested that environmental consideration has to be regarded. According to this, utilisation of slag or GGBS (as a by-product and waste material) in soil stabilisation works has become a popular choice for engineers and compared with lime and cement which may generate large amounts of carbon dioxide during their production. Carbon dioxide is a greenhouse gas that may warm the Earth's surface by reducing outward radiation. Moreover, GGBS is much cheaper than many other cementing agents, rendering feasibility of highway projects possible.

2.4 Slag as Soil Stabiliser

2.4.1. Slag Identification

There are two groups of slag, namely iron and steel slags and nonferrous slags. A slag derived from producing iron in a blast furnace is called blast furnace slag. Blast furnace slag can be produced in three forms: air-cooled, granulated, and expanded forms. Air-cooled slag is commonly used in concrete, asphalt and road bases, and as a fill material. Granulated slag is used as slag-cement. Expanded slag is used as an aggregate; it consists mainly of calcium, iron, un-slaked lime, and magnesium. Steel slag usually contains sufficient amounts (on the order of 30–50%) of lime, which can be mixed with fly ash to provide lime for pozzolanic reactions (Al-Rawas et al., 2002). This pozzolanic reaction further enhances the stabilisation process over time.

The GGBS on its own has only mild cementitious properties and is generally used in combination with Portland cement or hydrated lime (calcium hydroxide). In most practical purposes, GGBS needs to be activated and accelerated by alkali (Higgins, 2005).

2.4.2. Chemical Composition of GGBS

Predominant chemical constituents in GGBS such as CaO, SiO2, Al2O3, and MgO encourages replacing Portland cement with. For example, the Ecocem Ireland Ltd, Dublin (a green cement company) produces GGBS, which contains less than 1% silica and less than 1 ppm water soluble chromium IV, as shown in Table 2.7. For comparison, this table displays the four main chemical constituent proportions for both Portland cement and GGBS. The GGBS used in the current study was supplied by BGC Cement, Canning Vale, Western Australia; its specifications as presented in Table 2.8

Table 2.7: Chemical constituent proportions of Portland cement and GGBS

Chemical constituents	Portland cement	GGBS
CaO	65%	40%
SiO_2	20%	35%
Al_2O_3	5%	10%
MgO	2%	8%

Table 2.8: GGBS specification provided by BGC Cement

Ingredient	Formula	Content
Calcium Oxide	CaO	30–50%
Silica, Amorphous	SiO_2	35–40%
Aluminium Oxide	Al_2O_3	5–15%
Sulphur	S	< 5%

2.4.3. Physical Properties of GGBS

The physical properties of GGBS produced by Ecocem, include colour, bulk density (in its loose state), bulk density (in its vibrated state), relative density and surface area. These physical properties are listed in Table 2.9.

Table 2.9: Physical properties of GGBS provided by Ecocem GGBS

Colour	Off-white powder
Bulk density (loose)	1.0–1.1 tonnes/m^3
Bulk density (vibrated)	1.2–1.3 tonnes/m^3
Relative density	2.85–2.95
Surface area	400–600 m^2/kg Blaine

Source: The Ecocem Ireland Ltd.

Having similar main chemical constituents with Portland cement, GGBS also reacts with water but at a slower rate than Portland cement. To trigger this condition, Portland cement is usually used to activate GGBS (Ecocem, 2012). Another work also indicated that GGBS can be used as a cement replacement; in fact up to 85% of the Portland cement is replaced with the GGBS (Neeraja and Rao, 2010; Department for International Development, 2000).

2.4.4. Utilisation of GGBS in Soil Stabilisation

Utilisation of GGBS in soil stabilisation has been investigated by many researchers at a laboratory scale. Ouf (2001) used expansive clay from Egypt as a main material and used GGBS and lime as a stabiliser material in his study. The study included either mixing the soil with GGBS alone or with GGBS plus. In the case of GGBS only, several mixes were tried varying the GGBS by up to 10% by dry weight of the soil, whereas up to 30% replacement was tried for the case with hydrated lime. Adding GGBS only to the expansive soil resulted in a small increase in the optimum moisture content (OMC) and small decreases in the maximum dry density (MDD).

Ouf (2001) also noted that when using GGBS only (as a stabiliser) to this soil, the pozzolanic reaction between the soil and GGBS took some time to occur. This advantageous slow rate of the pozzolanic reaction gives enough time for finalising any stabilisation engineering work in the field, for example, providing appropriate time for mixing and compaction process in subgrade works.

Ouf (2001) found that the UCS value increased by the addition of slag under certain conditions. He divided the UCS samples into several curing periods; 7 days, 28 days and 3 months. It was observed that 6% of slag alone can increase the strength for 7 and 28 days curing periods; however, for 3 months curing period, 4% of slag alone was the optimum slag content. The increasing of UCS value also occurred with increasing of slag plus hydrated lime. It was found that 6% of slag with up to 30% of soil dry weight replacement of hydrated lime showed remarkable increases in the mixture strength.

Two reactions occurred in the mixture of clay stabilised with GGBS and lime, namely hydration of GGBS activated by lime and clay-lime reaction. Wild et al. (1996) claimed that the reaction rate of hydration of slag activated by lime was faster than that of the clay-lime mixture; this means that, increasing the ratio of lime to slag makes a complicated mixture characteristic regarding its pozzolanic behaviour.

Wild et al. (1998) observed that the optimum ratio of lime to slag depends on several factors including the soil type, clay content, curing conditions and curing periods. Ouf's experiments showed that the curing period depends on the combination of additive ratio and strength of the mixture. He found that the mixture of 28 days curing has higher strength than that of the mixture of 7 days curing. Furthermore, he found the increases of ratio of lime to GGBS leads to increase in the UCS, since GGBS needs more lime for its activation.

Higgins (2005) indicated that a commonly used stabilising blend in Australia comprises 85% slag and 15% hydrated lime. Higgins noted that the Australian practice differs from that of the United Kingdom (UK) and South Africa, where in the former the slag and lime are pre-blended rather than being added to the soil in two separate operations. The South African specifications suggest a slag to hydrated-lime ratio of 4 to 1 as being the optimum proportions. This corresponds to a slag to quicklime ratio of 5.2 to 1; since slag is not used for sulfate-resistance in South Africa, the optimum proportions would presumably be optimised for strength. In the

UK, quicklime is normally used as in an initial application of at least 1.5% of quicklime by weight of dry soil.

Higgins (2005) claimed that it is necessary to provide sufficient alkalinity to activate the slag, and hence modify the clay. Where the slag is being used for enhanced resistance to sulphate expansion, the proportion of slag should sensibly be at least equal to that of the quicklime, and typically for high resistance to sulphate expansion, a ratio of 3:1 slag to quicklime might be appropriate. The higher ratios up to 6:1 (slag to lime) are possible and can give even greater sulphate resistance. Currently there are insufficient data to recommend ratios greater than 6:1. Based on Higgins' summary above, it can be assumed that the slag to lime ratio required to achieve optimum strength of the stabilised soil may not be defined accurately, since it depends on the type of soil. Higgins (2005) also concluded that stabilisation with lime plus slag, effectively combats the expansion associated with the presence of sulphate in soil and equally combats expansion associated with sulfides such as pyrites.

The lime plus slag stabilisation offers three other advantages for soil stabilisation: (1) a slower early-rate of strength development, which provides considerably more time for construction operations; (2) an extra ability to self-heal in the case of early-life damage caused by overloading; and (3) in the long-term, there is increased strength due to enhanced pozzolanic reaction, which will ultimately improve the overall performance of the subgrade.

A stabiliser made of a combination of slag and cement to treat expansive soil was also investigated by Cokca et al. (2008). They utilised granulated blast furnace slag (GBFS) and GBFS-cement (GBFSC) to decrease the expansion of expansive soils. Slag and cement were added to a soil sample in proportions of 5% and 25% by weight, respectively. The results of the study indicated that the treatment with slag and cement altered the grain size distribution of the expansive soils. Specifically, the clay fraction decreased whilst the silt fraction increased. The plasticity index decreased and the specific gravity increased for all mixes of slag and cement. The slag and cement mixes also decreased both the swelling magnitude and time to reach 50% of the total swelling (t_{50}) values of specimens.

Cokca et al. (2008) found that the mixture of 75% expansive soil + 25% GBFSC reduced the swelling potential to only 6%, which also almost satisfies the irrigation water standards. Two separate mixtures of 80% expansive soil + 20% GBFS and 85% expansive soil + 15% GBFSC, reduced the swell percent from 29.4% to 10.9% and 3.1%, respectively, after 7 days of curing. Increasing the proportion of slag and cement to this expansive soil with longer curing times did not give additional effect to the reduction of the swell potential. As mentioned before, from an environmental point of view, using slag rather than cement seems to be a better alternative. Therefore, using 15% slag may be a better choice if environmental factors and swell percentage are considered together.

In process of chemical stabilisation, any hazardous waste can be converted into a form that is less soluble or less toxic. Many toxic metals found in sewage sludge have a low solubility at higher pH levels. The combination of slag and cement can raise the pH level of waste material. The presence of ferrous iron and sulfur compounds in the slag-cement mixture makes this mixture as a reducing agent to convert toxic metals into less toxic forms (Slag Cement, 2005).

Another study was performed by Kavak et al. (2011) on a low-plastic clay soil. They stabilised the soil with lime and ground granulated blast furnace slag (GGBFS) and used seawater for the hydration process (i.e. curing). The unconfined compression test and California Bearing Ratio (CBR) tests were conducted on soil samples treated with 5% lime and 3.33% slag and cured for 28 days. Kavak et al. (2011) concluded that the optimum results were obtained when the lime to slag ratio was 1.5:1 by weight. Using this ratio, the strength of treated soil was increased to more than eight times the initial of untreated samples strength (i.e. reference strength), reaching 2500 kPa (with seawater). Soaked CBR values also increased to more than ten times the reference value. From this study, it can be concluded that lime and slag can be used in combination as an additive to treat expansive clay soils. They also claimed that a certain amount of lime was needed to activate the slag; it should be noted; however, that the lime portion of 5% of this stabilisation work was the one that produced the best result rather than being the minimum ratio required for activating the slag

The study of Veith (2000) showed that the use of slag in clay stabilisation reduced the soil swelling potential from more than 28% down to only 4%. The reduction of the swelling potential is attributed to formation of cementitious gels when the slag is activated with a small percentage of lime. The cementitious gels bind the soil particles together, which suppress the swelling pressure of the expansive particles when exposed to water.

For soils stabilised with cement, the strength originates mainly from the pozzolanic reaction of the cement. Lu et al. (2004) mixed 10% and 15% cement with clay before adding granulated blast furnace slag (GBFS) in various proportions. They noted that the UCS values of the cement-stabilised soil increased by the addition of 5%, 10%, 15% and 20% slag. This increase was noted for curing times of 7, 14 and 28 days. The contribution of slag to the degree of improved may be explained by the fact that the chemical composition of the slag is almost identical to that of cement, which contributes to the cementitious, pozzolanic and hydration reactions.

Yadu and Tripathi (2013) studied the effect of Granulated Blast Furnace Slag (GBFS) on stabilising soft soil. The soft soil was classified as *Clays with silt of intermediate compressibility* (CI-MI) based on the Indian Standard Classification System (ISCS). They mixed the soil with proportions of 3%, 6%, 9% and 12% slag. Some laboratory tests were performed to examine the performance of the mixture. They found that there was a decreasing trend of both the liquid limit and plastic limit when the slag was added; for example, the plasticity index decreased from 17% without slag to 13% with 9% added slag.

Based on the standard compaction tests, both the maximum dry density (MDD) and optimum moisture content (OMC) increased with increasing percentage of slag. Yadu and Tripathi (2013) also reported a reduction of swelling pressure from about 42 kPa (no GBFS) to about 34 kPa for the 9% GBFS. Also with the addition of 6% GBFS, there was an increase of the unconfined compressive strength (UCS) value from about 118 kPa to about 155 kPa. However, the strength increased to only 145 kPa with the addition of 12% GBFS. The same trend was observed for CBR tests. The CBR value was 20% for 6% GBFS of unsoaked sample and about 10% on 9% GBFS of 4 days soaked sample. It appears from these results that while the addition of GBFS improves the overall soil performance, there is an optimum amount of GBFS that can achieve the best results.

2.5 Design of Stabilised Subgrade Soil

2.5.1. Definition of Subgrade

According to Illinois Department of Transportation, Bureau of Materials and Physical Research (2000), the subgrade for road pavement is a surface layer of soil that is formed or constructed in accordance with the trajectory of the road, it has a side boundary in accordance with a specified cross-sectional shape and is able to support the sub-base, base and surface layers of the road.

On fill areas, subgrade embankment is usually shaped with a fairly stable side slopes, while in areas of excavation, the subgrade soil is dug up or cut with a transverse width. Figure 2.5 illustrates a typical cross section of cut and fill of subgrade.

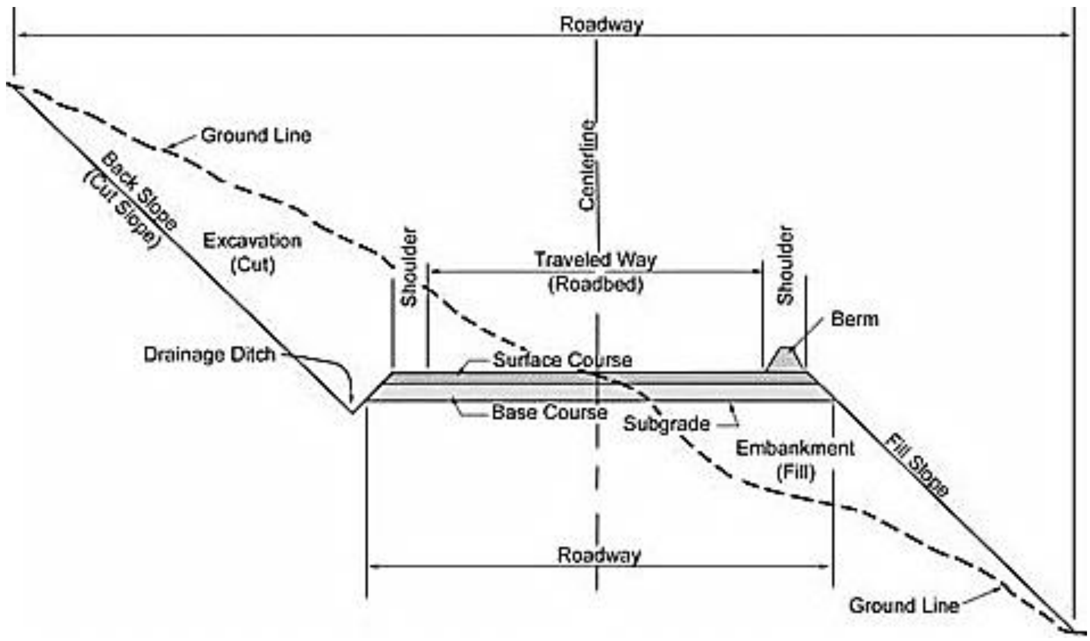

Figure 2.5: Typical cross section of cut and fill of subgrade (Retrieved November 19, 2013, from http://www.fs.fed.us)

The subgrade, which is located at the lowest level of road pavement layer structure, has a very important role, because its main function is to withstand the burdens of the overlying layers. After going through the process of cut and fill, subgrade surface is formed in such a way that it matches the desired shape and elevation and then the sustainability of subgrade is improved after going through the process of stabilisation, compaction and reinforcement (Nazzal and Mohammad, 2010).

2.5.2. Subgrade Performance

In order to attain a maximum level of service as a subgrade for road pavement, it should have an appropriate specific performance. This performance is normally tied to several main characteristics that have interrelated effects; these are: moisture content, load bearing capacity and shrink-swell potential.

Studies proved that increasing the moisture content in a soil mass reduces soil cohesion and friction angle (Kong et al., 2000). These conditions lead to decrease in the internal friction and soil may fail by its self-weight or external load. The water content within the subgrade may result from the groundwater level, drainage, or infiltration

The bearing capacity of a subgrade is a measure of its capability to support loads transmitted from the overlying pavement layers plus external loads from traffic. The bearing capacity of the subgrade depends on the soil type, moisture content and degree of compaction (if any). To get an idea as to how much compaction should be performed and what methods are used in the field, the compaction test has to be performed first in the laboratory. Based on the results of this laboratory compaction test, the optimum moisture content and maximum density of the soil can be obtained. The subgrade can meet the design requirements if it does not experience excessive deformation under certain loads.

The shrinkage-swelling potential is a soil unique characteristic especially for clay, which depends on the fluctuation of the moisture content. Changes in the moisture content can cause expansive soil to deform excessively, which may damage supported structures.

The in-situ soil should have the appropriate subgrade stability. In this regard, the subgrade can be assumed to be satisfactory if:

- It can properly support overlying pavement layers as well as loads from compaction;
- It can resist pavement rebound deflection due to load pressure; and

- It can prevent excessive rutting and shoving during construction and able to withstand the operational loads without excessive deformation.

If the criteria above cannot be fulfilled, there should be a special treatment to improve the subgrade performance (Illinois, 2000).

2.5.3. Subgrade Design

Satisfactory performance of the subgrade over the life span of the pavement cannot be understated during design. The achievement of this performance also should consider the cost of design and construction. There are three basic characteristics of subgrade performances that have to be considered and addressed: moisture content, load bearing capacity (strength) and shrink-swell potential.

In the United States, commonly three basic subgrade stiffness/strength characterizations are used: California Bearing Ratio (CBR), modulus of subgrade reaction, and elastic (resilient) modulus. Several fields or laboratory tests should be performed to identify certain engineering properties of the subgrade material, such as CBR tests, soil classification and repeated load tests. Table 2.10 presents relative CBR values for subbase and subgrade soils, and Table 2.11 shows various description of subgrade material properties for pavement design.

Table 2.10: Relative CBR values for subbase and subgrade soils

CBR (%)	Material	Rating
> 80	Subbase	Excellent
50 to 80	Subbase	Very Good
30 to 50	Subbase	Good
20 to 30	Subgrade	Very good
10 to 20	Subgrade	Fair-good
5 to 10	Subgrade	Poor-fair
< 5	Subgrade	Very poor

Source: American Concrete Pavement Association; Asphalt Paving Association; State of Ohio; State of Iowa; Rollings & Rollings (1996).

Table 2.11: Subgrade material properties for pavement design

Sub grade soils for design	Unified Soil Classifications	Load support and drainage characteristics	Modulus of subgrade reaction (k), psi/inch	Resilient modulus (M_R), psi	CBR range
Crushed Stone	GW, GP, and GU	Excellent support and drainage characteristics with no frost potential	220 to 250	Greater than 5,700	30 to 80
Gravel	GW, GP, and GU	Excellent support and drainage characteristics with very slight frost potential	200 to 220	4,500 to 5,700	30 to 80
Silty gravel	GW-GM, GP-GM, and GM	Good support and fair drainage, characteristics with moderate frost potential	150 to 200	4,000 to 5,700	20 to 60
Sand	SW, SP, GP-GM, and GM	Good support and excellent drainage characteristics with very slight frost potential	150 to 200	4,000 to 5,700	10 to 40
Silty sand	SM, non-plastic (NP), and > 35% silt	Poor support and poor drainage with very high frost potential	100 to 150	2,700 to 4,000	5 to 30
Silty sand	SM, Plasticity Index (*PI*) <10, and <35 % silt	Poor support and fair to poor drainage with moderate to high frost potential	100 to 150	2,700 to 4,000	5 to 20
Silt	ML, > 50% silt, liquid limit < 40, and *PI* <10	Poor support and impervious drainage with very high frost value	50 to 100	1,000 to 2,700	1 to 15
Clay	CL, liquid limit > 40 and *PI* >10	Very poor support and impervious drainage with high frost potential	50 to 100	1,000 to 2,700	1 to 15

Source: American Concrete Pavement Association; Asphalt Paving Association; State of Ohio; State of Iowa; Rollings and Rollings (1996).

Seco et al. (2011) stated that the use of CBR tests on subgrade soil as an approach in designing road pavement may not be satisfactory. The road design methods should rather be developed through road pavement performance tests in the field and laboratory. The road design method is more focussed on soil stiffness and the soil's ability to prevent excessive deformation as initiated by the UK Highways Agency (Hossain, 2010).

Another test that is used to determine the minimum strength of every layer of pavements is the Unconfined Compressive Strength (UCS) test. For subgrade layers, Joint Departments of the Army and Air Force, USA (1994) suggests minimum UCS values presented in Table 2.12.

Table 2.12: Minimum unconfined compressive strength test value

Stabilised soil layer	Minimum unconfined compressive strength*	
	Flexible pavement	Rigid pavement
Base course	750 psi (5.171 MPa)	500 (3.447 MPa)
Subbase course, selected material or subgrade	250 psi (1.724 MPa)	200 (1.379 MPa)

* Unconfined compressive strength for cement, lime, lime-cement, and lime-cement-fly ash stabilised soils determined at 7 days for cement stabilisation and 28 days for lime. Lime fly ash or lime-cement-fly ash stabilisation. (1 psi = 6.894757 kPa).

To simulate traffic loading mechanism subjected to subgrade layer, recently, a repeated loading test is applied to the non-stabilised and stabilised soils. In the repeated loading test, the soil behaviour during loading and the strength of soil can be identified thoroughly; those two parameters can also indicate the stability of the soil. Therefore, soil strength and behaviour during the repeated loading test can be used as a basis for treating the soil during the construction process of road. The construction process will affect the long-term performance of the constructed pavement.

2.6 Review of Resilient Modulus

2.6.1. Definition of Resilient Modulus

Caunce (2010) suggested that stiffness (resilient elastic modulus) is a function of confining stress, axial stress and matrix suction (pore water pressure) of the materials. A material's resilient modulus is actually an estimate of its modulus of elasticity (E), which is calculated by dividing the stress by the corresponding strain for a slowly applied load. The subgrade resilient modulus (M_R) is a measure of the stiffness of subgrades, which is calculated similarly but for rapidly applied loads or recoverable axial strain experienced by pavements under traffic load. Figure 2.6 illustrates how the resilient modulus is derived, which can be defined as follows:

$$M_R = \frac{\sigma_1 - \sigma_3}{\varepsilon_r} \tag{2.2}$$

where: M_R = resilient modulus;

$\sigma_1 - \sigma_3$ = maximum repeated axial deviator stress; and

ε_r = maximum resilient axial strain (recoverable axial strain).

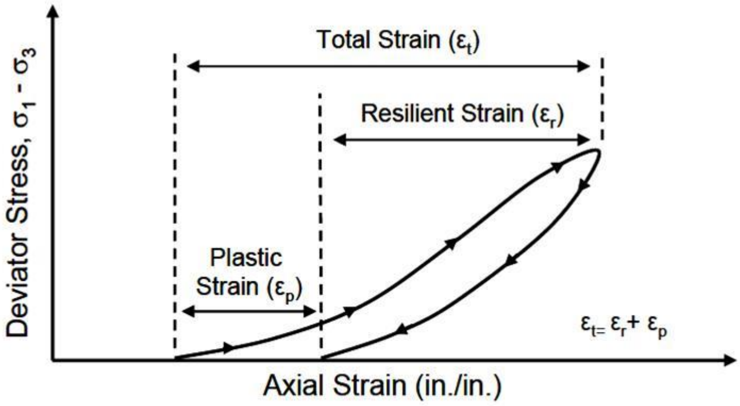

Figure 2.6: Resilient modulus definition

Some engineering firms still rely on the use of empirical correlation to determine the resilient modulus from tests such as R-value, Soil Support Value (SSV) and California Bearing Ratio (CBR) or Limerock Bearing Ratio (LBR). For example, Bandara and Rowe (2002) presented the following correlation of the resilient modulus (in *psi*) with CBR values for roadway constructions in Florida.

$$M_R = 1500 \times \text{CBR} \tag{2.3}$$

This correlation was claimed to be more suitable for fine-grained soils, with the range of the constant to vary from 750 to 3000. However, this correlation may not be suitable for all types of soils. For example, Thompson and Robnett (1979) study could not relate the resilient modulus to CBR using those equations for Illinois soils. Additionally, Rada and Witczak (1981), who used granular material, reported that the CBR-relationship cannot be used to identify the stress dependence of resilient modulus.

The subgrade, which is subjected to repeated or cyclic loading, experiences repeated plastic strain (i.e. irrecoverable upon unloading) during the loading period. For design purposes, this cumulative plastic strain occurs below the threshold stress of the soil. The threshold stress of the subgrade soil is the maximum stress that can be applied to the sample without causing the cumulative strain to exceed 10 percent after 1000 cycles (Putri et al., 2012). However, in terms of cohesive soils subjected to load untypical of traffic loading condition (> 80 kN), the excessive permanent strain can still be observed during the repeated loading test. It should be highlighted that the resilient modulus of subgrade soil does not indicate its strength but stiffness. Therefore, for the mechanistic-empirical design/analysis of multi-layered flexible pavement system, the value of the material resilient modulus of the soil subgrade represents actual material properties as a response to traffic load on that soil.

2.6.2. Resilient Modulus Determination

On a laboratory scale, the resilient modulus is determined using the repeated load triaxial (RLT) test. This test applies a repeated axial cyclic stress of fixed magnitude, load duration and number of cycles to a cylindrical specimen. While the specimen is subjected to this cyclic stress (σ_1-σ_3), it is also subjected to a static confining stress (σ_3) provided by the triaxial pressure chamber, as shown in Figure 2.7. The RLT test is essentially a cyclic version of the traditional monotonic triaxial compression test; the cyclic load application is thought to more accurately simulate the actual traffic loading.

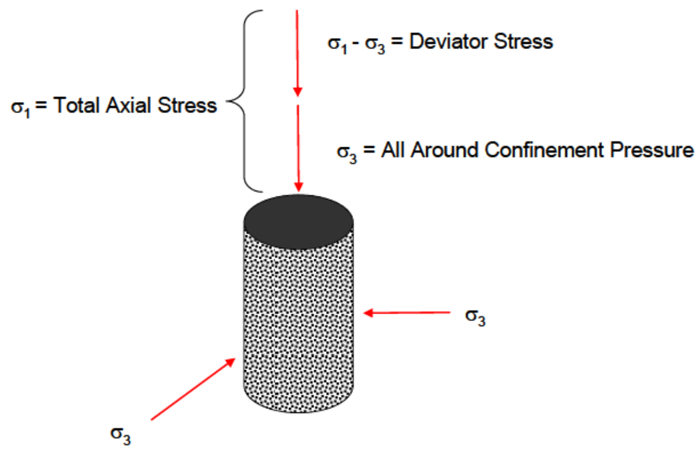

Figure 2.7: Stresses applied in the repeated load triaxial test

To use the Repeated Load Triaxial test in pavement design, it was standardised by the American Association of State Highway and Transportation Officials (AASHTO) to be a standard method of test as specified in the AASHTO “T307-99, Determining the Resilient Modulus of Soils and Aggregate Materials”. The current study uses this standard method in performing the repeated load triaxial test to obtain the stabilised soil resilient modulus of subgrade samples.

2.6.3. Resilient Modulus Model Correlations

A number of resilient modulus correlations with other stress, load and some soil properties have been developed by many researchers. Some models are suitable for granular soils and some others for fine-grained soils. For clay soils, which exhibit plasticity, the resilient modulus value normally depends on the moisture content and deviator stress. Therefore, one or two of these two factors may be included as variables in a model correlation. Some studies showed that, in Repeated Load Triaxial (RLT) test, the influence of confining pressure (σ_3) on plastic subgrade soil is fairly small (Li and Selig, 1994; Rahim, 2005), therefore, in some correlation models for fine-grained soils it was not possible to rely on the confining pressure as one of its variables.

The resilient modulus of fine-grained soils is unpredictable and its value can have a wide range of 14 MPa to 140 MPa (Li and Selig, 1994). The same type of soil may exhibit a remarkably different response depending on the stress state and water content. Li and Selig (1994) indicated that models generated for the resilient modulus of fine-grained soils should consider soil physical state, stress state, and soil type and its structure. The soil physical state is closely related to dry density and moisture content. Lee et al. (1997) found that the resilient modulus decreases with the increase of moisture content. The stress state is related to the magnitude of deviator stress, confining pressure, number of repetitive loading and their sequence. The soil type and its structure depend on the compaction method. Therefore, Li and Selig (1994) suggested generate of the resilient modulus of fine-grained soil correlation model with reference to the applied deviator stress at the optimum moisture content condition.

Austroads (2012) summarised some models for the nonlinear resilient modulus of unbound granular materials and subgrade, as shown in Table 2.13. In addition, the National Cooperative Highway Research Program (NCHRP), in their final report of

Laboratory Determination of Resilient Modulus for Flexible Pavement Design (1998) and Li and Selig (1994) summarised the cohesive subgrade soil resilient modulus in some following models as shown in Table 2.14. It can be seen, based on most model correlations, that the stress state significantly influences the resilient modulus of cohesive subgrade soil.

Table 2.13: Some resilient modulus nonlinear material models for unbound granular materials and subgrade (Austroads, 2012)

Model	Reference	Stress variables	Model parameters
$M_R = k\ (\sigma_3)^n$	Dunlap and Texas (1963) and Monismith et al. (1967)	σ_3	k_1, k_2
$M_R = k\ (\theta)^n$	Seed et al. (1967), Brown and Pell (1967) and Hicks and Monismith (1971)	θ	k_1, k_2, k_3
$M_R = k_1 p_a [\frac{\theta}{p_a}]^{k_2} [\frac{\tau}{p_a}]^{k_3}$	Uzan (1985)	θ, τ	k_1, k_2
$M_R = \frac{\theta}{q}(A + Bq)$	Nataatmadja and Parkin (1989) and Nataatmadja (1992)	θ, q	A, B
$M_R = k_1\ (\frac{J_2}{\tau_{oct}})^{k_2}$	Johnson et al. (1986)	J_2, τ	k_1, k_2
$M_R = k_1 p_a [\frac{\theta}{p_a} + 1]^{k_2} [\frac{\tau}{p_a} + 1]^{k_2}$	Austroads (2007)	θ, τ	k_1, k_2, k_3

p_a = atmospheric pressure (101.4 kPa)

Masada et al. (2006) used a fine-grained subgrade soil available in Wayne Country, Ohio to evaluate five resilient modulus models. After comparing the overall average values of the coefficient determination (R^2) of each model with several soil samples, they found that the hyperbolic model was the most appropriate with average R^2 = 0.982. The second appropriate model was the Octahedral model with average R^2 = 0.764, followed by the bilinear model with average R^2 = 0.736. The lowest R^2 was determined for the Semi log model. They concluded that the resilient modulus is only correlated with the deviator stress (σ_d).

Table 2.14: Various resilient modulus correlation model with stress dependent of cohesive soils

Model	Reference	Variable	Remarks
Bilinear Model $M_R = K_1 + K_2\sigma_d$, when $\sigma_d < \sigma_{di}$ $M_R = K_3 + K_4\sigma_d$, when $\sigma_d > \sigma_{di}$	Thompson and Robnett (1976)	M_R = Resilient Modulus σ_d = Deviator Stress σ_{di} = Deviator Stress at which the slope of M_R versus σ_d changes. K_1, K_2, K_3 and K_4 = model parameters dependent upon soil type and its physical state (K_2 and K_4 are usually negative)	
Power Model (1) $M_R = k\,(\sigma_d)^n$	(Moossazadeh and Witczak, 1981)	M_R = Resilient Modulus σ_d = Deviator Stress. *k* and *n* = model parameters dependent upon soil type and its physical state (*n* is usually negative)	Moossazadeh and Witczak (1981) obtained good agreement with the test results on three fine-grained soils from San Diego, Illinois, and Maryland with the determination of *k* = 0 to 200 and *n* =- 1.0 to 0 for resilient modulus (ksi) and deviator stress (psi). Pezo et al. (1991) obtained a range of *k* = 6,000 to 55,000 and *n* = -0.34 to -0.04 for Austin soil (A-7-6) for resilient modulus and deviator stress in units of psi.
Power Model (2) $M_R = k\,(\frac{\sigma_d}{\sigma'_3})^n$	Brown et al. (1975) and Brown (1979)	M_R = Resilient Modulus σ_d = Deviator Stress σ'_3 = Confining Stress	Consideration of effective confining stress (σ'_3) for saturated overconsolidated soils
Semilog Model $M_R = 10^{(k-n\sigma_d)}$ or $\log(M_R) = k - n\sigma_d$	Fredlund et al. (1977)	M_R = Resilient Modulus σ_d = Deviator Stress. *k* and *n* = model parameters dependent upon soil type and its physical state.	Fredlund et al. (1977) obtained the range of parameters *k* = 3.6 to 4.3 and *n* = 0.005 to 0.09 for resilient modulus and deviator stress in units of kPa. This model was applied for a moraine glacial till.
Hyperbolic Model $M_R = \frac{k + n\,\sigma_d}{\sigma_d}$	Drumm et al. (1990)	M_R = Resilient Modulus σ_d = Deviator Stress. *k* and *n* = model parameters dependent upon soil type and its physical state.	Drumm et al. (1990) proposed this model for fine-grained soils. For the Tennessee soils tested, the range of parameters *k* = 2-70 and from *n* = 2-12. The unit for resilient modulus is ksi and deviator stress is psi.
Octahedral Model $M_R = k\frac{\sigma_{oct}^n}{\tau_{oct}^m}$	Shackel (1973)	M_R = Resilient Modulus σ_{oct} = Octahedral normal Stress. τ_{oct} = Shear Stress.	Shakel (1973) derived this model, however, it is more difficult to apply

2.6.4. Resilient Modulus Correlation Trends

Some researchers formulated various models as a function of the influencing factors, such as soil water content, dry density, method of compaction, deviator stress, confining pressure, thixotropy, and freeze-thaw cycles (Kim and Kim, 2007).

The first trend correlating the resilient is modulus inversely proportional with the deviator stress. Kim and Kim (2007) proved this trend for sandy-silty-clay soils at optimum moisture content. Drumm et al. (1990) applied the Hyperbolic model to their Tennessee soil specimen to correlate the resilient modulus and deviator stress. They concluded that the resilient modulus decreased with increasing deviator stress. This trend was also verified by Lee et al. (1995) on cohesive soils. Pezo and Hudson (1994) found the same trend for some non-granular soil samples extracted from a road across the state of Texas.

Edil et al. (2006) proved the trend above for non-stabilised clays: red silty clay, red clay and brown silt. However, when they mixed the soil with fly-ash, they found conflicting results. Figure 2.8 shows: (a) typical curves showing resilient modulus versus deviator stress for soil compacted at optimum water content; and (b) soil–fly ash mixture prepared 7% wet of optimum water content (Edil et al., 2006). The phenomenon of this reversal resilient modulus curve also occurred in the study by Trzebiatowski et al. (2004).

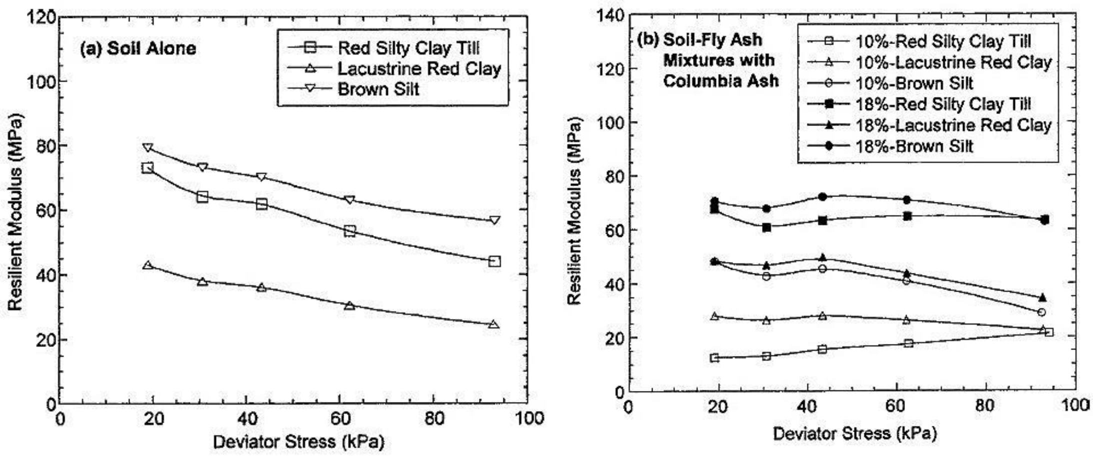

Figure 2.8: Various resilient modulus correlation trends (Edil et al., 2006)

Chapter 3. Experimental Study

3.1. Introduction

This chapter describes the experimental program implemented in this study. It is divided into several sections, including the purpose of the experiments, material selection, material preparation, specimen preparation and testing. Specifications of the stabilisers (slag and cement) and equipment are presented in this chapter. The experimental work was performed at the Geomechanics and Pavement Laboratory of the Department of Civil Engineering, Curtin University, Western Australia.

The experimental work was arranged in three stages. The experimental study flow chart, presented in Figure 3.1, illustrates the position of every stage of the experimental study and the relationship among them. The first stage is the selection of a suitable expansive soil based on its geotechnical properties. To determine the soil's geotechnical properties, some laboratory tests were performed, including the particle size distribution, Atterberg limits, soil particle density and free swelling. The second stage is the determination of the ideal proportions of stabilised soils based on achieving the minimum required strength, in terms of the Unconfined Compressive Strength at the optimum moisture content of the soil-additive mixture. The third stage is an assessment of the performance of the stabilised soil based on the California Bearing Ratio test and Repeated Load Triaxial test. Presentation and discussion of the experimental results are presented in Chapter 4.

3.2. Purposes of Experimental Study

The purposes of the experimental program can be summarised as follows:

1. To select the appropriate expansive soil.
2. To find the appropriate proportions of the additives in terms of the required percentage of slag and cement that can achieve an acceptable level of improvement.
3. To determine the geotechnical properties of non-stabilised soil and stabilised soil.

4. To analyse the performance of the stabilised expansive soil in terms of strength, bearing capacity and resilient modulus.
5. To derive an empirical model that can correlate the resilient modulus with other soil geotechnical properties.

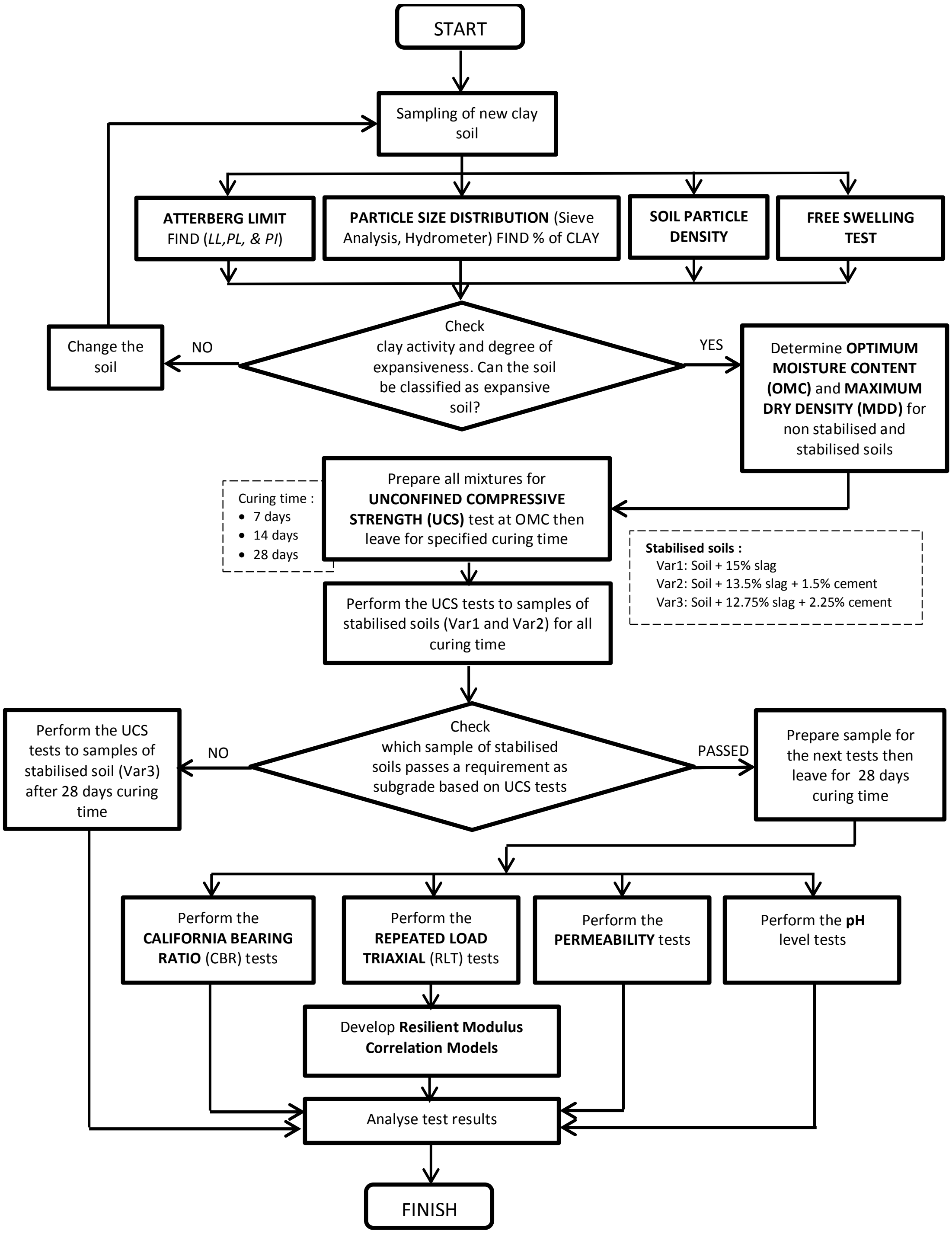

Figure 3.1: Flow chart of experimental study

3.3. Materials

3.3.1. Soil

Online references were used to trace the exact locations of expansive soils in Western Australia. Four different locations were visited including Mundijong, Karnup, Guilford and Baldivis (see Figure 3.2). Availability of expansive soils in Guildford and Baldivis was based on information received from the Council of the Town of Bassendean and the City of Rockingham, respectively.

Figure 3.2: Locations of expansive soils in Western Australia used for the current study

The procedure of sampling and preparation of soils were conducted according to the Australian Standard AS 1289.1.2.1 (1998). This standard describes various considerations in sampling soil for engineering purposes such as earthworks and pavements. The standard also regulates, in detail, the procedure of classification of packing and transporting samples to the testing location. The appropriate amount of clay or silty clay soil samples from those four locations were taken to the laboratory as disturbed samples. These disturbed samples were stored in plastic bags and were then used for preliminary tests.

The expansive soil layer in Baldivis area was located beneath a sandy soil layer and was found only after excavation of the surface layer. This excavation was performed as part of some construction works for a housing development in that area. Figure 3.3 shows the first stage of the process of taking the soil samples for preliminary tests after the excavation of surface layers.

Figure 3.3: First stage of soil samples taken for preliminary tests

The preliminary tests were conducted to ensure that the soils used can be categorised as an expansive soil. These tests include the particle size distribution, Atterberg limits, and free swell index tests. Based on the results of these tests, the calculated soil activity level was used as a reference in comparison with the standard value. It was decided that the soil from Baldivis can be used as the main material in this research as a typical expansive soil. The location of soil used can be seen in the map shown in Figure 3.4.

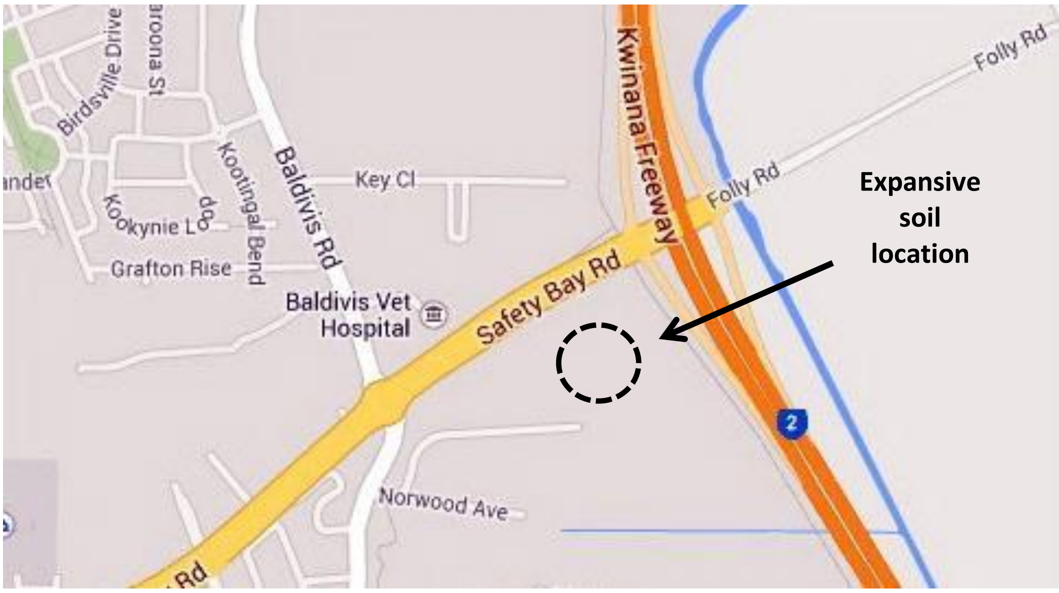

Figure 3.4: Expansive soil location in Baldivis area

The second stage of the sampling process aimed at taking adequate quantities of soil to perform all planned experiments. This sampling process was supported by a mini excavator to dig the hard clay soil. To make sure that the soil taken was not contaminated with any other soil or organic material, the soil in the excavator bucket was checked regularly and approved before it was placed in a suitable container. Figure 3.5 shows the second stage of the sampling process.

Figure 3.5: Second stage of soil samples taken for sample preparation

3.3.2. Cement

The first additive used in this study is Portland cement. No additional treatment applied to this additive. Table 3.1 presents the general specifications of the Portland cement provided by Cockburn Cement Limited in Western Australia.

Table 3.1: General specifications of Portland cement provided by Cockburn Cement Limited

Parameter	Method	Units	Typical	Range	AS3972- 1997 Limits
Chemical Analysis					
SiO_2	XRF	%	21.1	20.4 – 21.8	
Al_2O_3	XRF	%	4.7	4.3 – 5.1	
Fe_2O_3	XRF	%	2.7	2.5 – 2.9	
CaO	XRF	%	63.6	62.6 – 64.6	
MgO	XRF	%	2.6	2.4 – 2.8	
SO_3	XRF	%	2.5	2.2 – 2.8	3.5% max
LOI	AS2350.2	%	2	1.0 – 3.0	
Chloride	ASTM C114	%	0.01	0.01 – 0.03	
Na_2O equiv.	ASTM C114	%	0.5	0.40 – 0.60	
Fineness Index	AS2350.8	m^2/kg	400	370 – 430	
Normal Consistency	AS2350.3	%	28.5	27.5 – 29.5	
Setting Times					
- Initial	AS2350.4	mins	120	90 – 150	45 mins min
- Final		mins	195	165 – 225	10 hours max
Soundness	AS2350.5	mm	1	0 – 2	5 mm max
***Compressive Strengths**	AS2350.11				
	3 days	MPa	38	35 – 42	
	7 days	MPa	47	44 – 51	25 MPa min
	28 days	MPa	60	56 – 64	40 MPa min

**Normen Standard sand used*

3.3.3. Ground Granulated Blast Furnace Slag (GGBS)

The second additive is Ground Granulated Blast Furnace Slag (GGBS), which can be obtained from the Portland cement suppliers in Western Australia. It is available as an off-white powder in 20 kilogram paper bags, as shown in Figure 3.6. The GGBS specifications provided by BGC Cement, Canning Vale, Western Australia, were previously presented in Table 2.8

Figure 3.6: Ground Granulated Blast Furnace Slag (GGBS)

3.4. Preparation of Material and Specimens

A reinforced plastic container was used to store the disturbed soil samples (Figure 3.7). Some soil sample preparation stages were carried out to suit the required experimental activities such as the drying, crushing and sieving. The drying process was performed in two stages. In the first stage, the soil was dried in open air (Figure 3.8). In the second stage, the soil chunks (maximum 50 mm size) were dried in electric oven at 106°C for an overnight (Figure 3.9). Two hammers were used to break the soil chunks into smaller size (Figure 3.10). The use of mechanical crushing to break the soil chunks were performed using the Los Angeles Abrasion machine. Using this machine in crushing the soil chunks was only allowed after the particle size distribution testing. All soil particles should pass the 2.36 mm sieve size.

Figure 3.7: Soil sample container

Figure 3.8: Open air drying

Figure 3.9: Electric oven

Figure 3.10: Various hammers to make smaller soil size

3.5. Experimental Work

3.5.1. Particle Size Distribution

Sieve analysis was used to separate the fine and coarse grained particle using the 75 µm sieve. This was carried out according to the Australian Standard AS 1289.3.6.1 (2009), as shown in Figure 3.11. For particle smaller than 75 µm, the sedimentation method using the hydrometer was conducted according to the Australian Standard AS 1289.3.6.3 (2003), as shown in Figure 3.12.

Figure 3.11: Sieve analysis

Figure 3.12: Hydrometer test

3.5.2. Soil Particle Density

The soil particle density was determined in accordance with the Australian Standard AS 1289.3.5.1 (2006), using the density bottle method. About 100 grams of dry soil retained on the 2.36 mm sieve were placed in water filled the bottle, before being weighed (Figure 3.13).

Figure 3.13: Soil particle density measurement

The particle soil density test quantifies the density of the individual particles (grains) that make up the soil mass. The value of this test was used in the sedimentation (hydrometer) test calculations for particle size determination.

3.5.3. Atterberg Limits

The Australian Standard AS 1289.3.2.1 (2009) was used to determine the plastic limit of the tested soil (Figure 3.14). The liquid limit was determined using the Casagrande apparatus in accordance with the Australian Standard AS 1289.3.1.2 (2009), as shown in Figure 3.15.

Figure 3.14: Plastic limit measurement

Figure 3.15: Liquid limit measurement

The Australian Standard AS 1289.3.3.1 (2009) sets out a method to calculate the plasticity index of a soil as derived from the liquid limit and plastic limit (i.e. Plasticity index = Liquid limit – Plastic limit). The degree of plasticity of soil can be determined from the guidelines given in Table 3.2.

Table 3.2: Description of fine soil based on plasticity index value

Plasticity Index (PI)	Soil Description
0	Non-plastic
1–5	Slightly plastic
5–10	Low plasticity
10–20	Medium plasticity
20–40	High plasticity
> 40	Very high plasticity

Based on the American Society for Testing of Materials, ASTM D2487 (2011), the particle size distribution and Atterberg limits can be used to classify the soil. The soil moisture content determination was performed in accordance with the Australian Standard AS 1289.2.1.1 (2005); using the oven drying method.

3.5.4. Free Swelling Tests

To ensure the soil is expansive in nature, the free swelling test and shrinkage tests were performed on the natural soils considered in the current study. The tests followed the procedures described by the Australian Standard AS 1289.7.1.1 (2003). The Free Swelling Test procedure proposed by the Indian Standard IS 2720-40 (1977) was used to determine the Free Swell Index of soil.

In this test, two graduated cylinders were used, as shown in Figure 3.16. Each cylinder containing a dry soil specimen was filled with distilled water and kerosene. Both specimens were then left for 24 hours for absorption into the soil. The final volume of soil was measured in each cylinder and was used to calculate the soil Free Swell Index. Some researchers suggest the swelling potential prediction shown in Table 3.3.

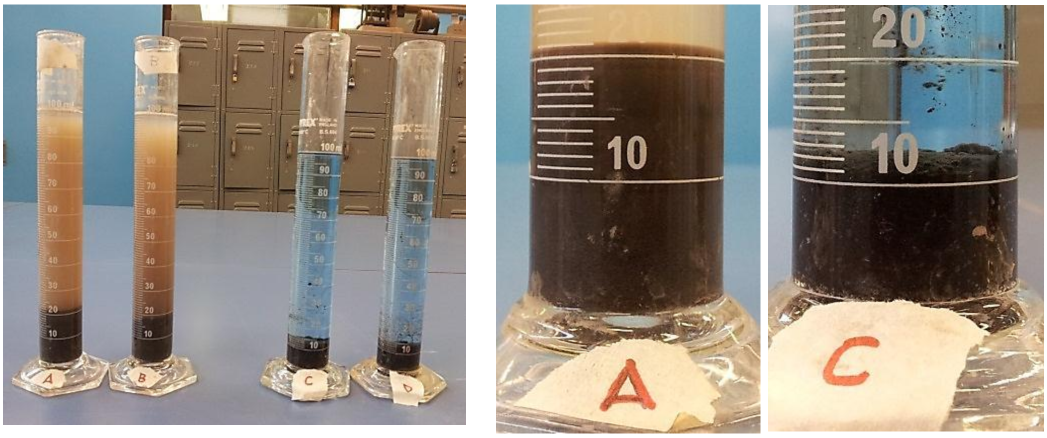

Figure 3.16: Free swelling test of expansive soil

Table 3.3: Swelling potential prediction in soils

Parameter	References	Degree of expansion			
		Low	Medium	High	Very high
LL (%)	Chen (1975)	< 30	30–40	40–60	> 60
PI (%)	Chen (1975)	0–15	10–35	20–55	> 55
PI (%)	Holtz and Gibbs (1956)	< 20	12–34	23–45	> 45
Clay Content (%)	Holtz and Gibbs (1956)	< 17	12–27	18–37	> 27
Clay Content (%)	Holtz (1959)	–	13–23	20–31	> 28
Swell Percent (%)	Thomas et al. (2000)	< 3.0	3.0–6.0	6.0–9.0	> 9.0
Swell Pressure (kPa)	Thomas et al. (2000)	< 81	81–153	153–225	> 225
Activity	Skempton (1953)	< 0.75	0.75–1.25	> 1.25	

3.5.5. Optimum Moisture Content and Maximum Dry Density

To determine the relationship between the moisture content and dry density of the tested soils, the procedure described by the Australian Standard AS 1289.5.1.1 (2003) were followed. The soil was compacted using standard compactive effort of 596 kJ/m^3 (see Figure 3.17) over a range of moisture content to generate the curve of the maximum mass of dry soil as a function of the water content. This curve is then used to determine the optimum moisture content and corresponding maximum dry density.

Figure 3.17: Standard compaction test for optimum moisture content

3.5.6. Stabilisation Process

The stabilisation process was started after the expansive soil was ready for mixing. Several preparation stages including removing the initial water content by oven drying overnight, soil crushing and sieving. For the mixing process, the sample of dry soil should pass the 2.36 mm sieve size.

When the soil was taken from the storage, it usually contained a small amount of water. To determine this initial soil water content immediately, the standard microwave unit and one set of desiccator were used. The value of the initial water content was taken as a reference to which water is added to cover the range of moisture content intended for the test (which also embraces the optimum value). The measurement of the initial water content of soil is shown in Figure 3.18.

Figure 3.18: Measurement of soil initial water content before mixing

Based on previous studies, the use of 15% stabiliser were adequate to be added to 100% of dry soil (Figure 3.19). This stabiliser proportion was divided into three combinations of slag plus cement at different slag-cement ratios. The ratios of slag to cement in a 15% stabiliser (to 100 % soil) are: 100–0, 90–10 and 85–15. These ratios are illustrated in Figure 3.20, and the stabiliser compositions are listed in Table 3.4. The shape and size of the specimens were made according to the laboratory standard manual of each test.

Figure 3.19: Material proportion on stabilised soils

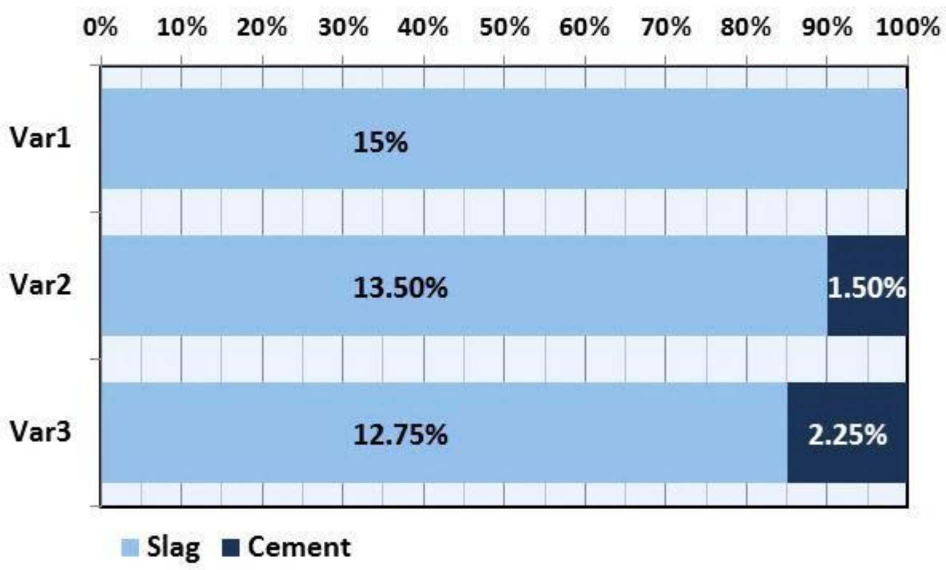

Figure 3.20: Proportion of stabiliser type in 15% stabiliser used

Table 3.4: Stabiliser composition in stabilised soils

Sample Code	Stabiliser composition
Var0	100% soil + 0% stabiliser
Var1	100% soil + 15.00% slag
Var2	100% soil + 13.50% slag + 1.50% cement
Var3	100% soil + 12.75% slag + 2.25% cement

The mixing process was carried out using an electrical soil mixer equipped with a stirrer and a timer (Figure 3.21). All materials were mixed for about 1-2 minutes until the mixture looks homogeneous, then a certain amount of water was added slowly while mixing. The amount of water added was based on the calculation of the percentage of water at optimum moisture content for each mixture composition. Each stabilised soil composition has its own optimum moisture content value that was determined from the compaction tests. The mixing process was completed when the water had already been absorbed evenly throughout the mixture.

The mixture compaction process is necessary to be performed within the first 45 minutes after the first water was added to the mixture to avoid the crystallisation process of cement (the critical time) which ends of cement crystals interlock

(Montgomery, 1998). The stabilised soils were compacted using the standard compaction test. The compacted stabilised soils were removed from the compaction mould using a mechanical hydraulic jack (Figure 3.22); the unconfined specimens were then wrapped with plastic bags to prevent any other physical disturbance during the curing period (Figure 3.23).

Figure 3.21: Soil mixer machine

Figure 3.22: Hydraulic jack

Figure 3.23: Unconfined stabilised soil specimens after compaction

3.5.7. Unconfined Compressive Strength (UCS) test

The Main Road Laboratory Standard test method, WA 143.1 (2012) was followed for the UCS test. In order to obtain the best mixture to be used as a subgrade for road pavement, all compositions were compacted in steel mould with 115 mm in height and 105 mm in diameter. The standard compaction test was used to generate the specimen maximum dry densities. Three combinations of soil and stabilisers were used (see illustration in Figure 3.24). These specimens were cured over three different periods of time, with at least 4 identical specimens for each period as seen in Table 3.5. The results presented in this study are the average of the three adjacent values of the three specimens.

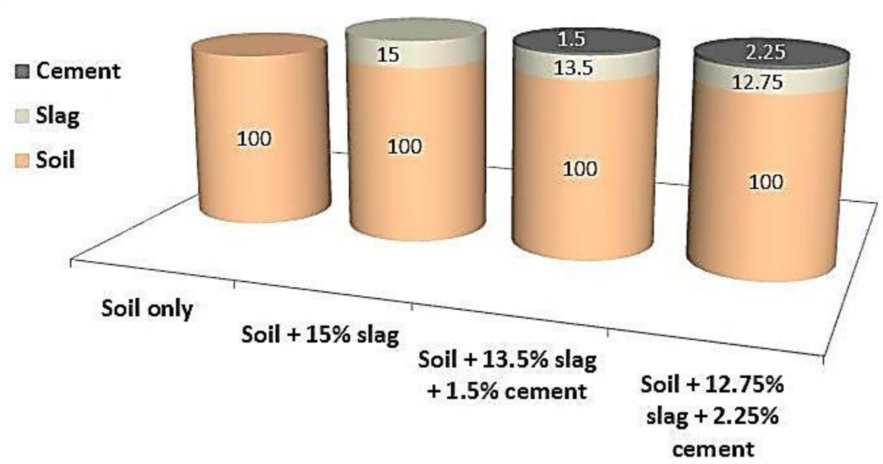

Figure 3.24: Mixture proportions of UCS test specimens

Table 3.5: Specimen number and various curing time for UCS tests

Specimen types	No curing time	7 days curing time	14 days curing time	28 days curing time
100% soil (Non-stabilised soil)	4	-	-	-
100% soil + 15.00% slag	-	4	4	4
100% soil + 13.50% slag + 1.50% cement	-	4	4	4
100% soil + 12.75% slag + 2.25% cement	-	-	-	4
Total number of specimens	4	8	8	12

The UCS test were performed using GCTS STX-300 (Stress-Path Soil Triaxial System) computerised triaxial testing machine, as shown in Figure 3.25. This machine is equipped with all necessary softwares to automatically perform all triaxial stages. The user can easily control the test via the Graphical User Interface to control the machine and generate a report.

Figure 3.25: Unconfined Compressive Strength (UCS) test

To allow the chemical reactions to take place between the soil and its stabiliser, the samples were cured over periods of 7, 14 and 28 days. These strength values so obtained from the average of three test results were compared to the minimum strength required for the subgrade layer, as previously shown in Table 2.12.

The next level of performance test was pursued on samples that passed the minimum strength of UCS for subgrade layer. A specific mixture of soil and stabiliser was used along with a certain curing time (28 days).

3.5.8. California Bearing Ratio (CBR) test

To evaluate the mechanical strength of the selected stabilised soil mixtures, the California Bearing Ratio (CBR) test was performed in accordance with the Australian Standard AS 1289.6.1.1 (1998). The mixture of the expansive soil that has 13.5% slag plus 1.5% cement was compacted using the standard compactive effort (596 kJ/m^3) in a standard cylindrical metal mould. Two metal surcharges each weighing 2.25 kg, were placed on top of the CBR specimen after 28 days of curing. The CBR specimens were then soaked in water for 4 days at a constant temperature before being tested.

Loading the CBR samples was performed using the Universal Testing Machine (IPC Global UTM-25), which can record both load and displacement automatically. The CBR test results were compared to the minimum CBR value as suggested by Austroads for subgrade (CBR_{min} = 5%). It should be noted, however, that in the case of designing stabilised subgrade, it is recommended to have a CBR value greater than 15% (Austroads, 2010). Figure 3.26 shows the procedure of the CBR test.

Figure 3.26: California Bearing Ratio (CBR) test

3.5.9. Repeated Load Triaxial (RLT) Test

Based on the UCS tests, a selected mixture (soil + stabilisers) that has a strength value exceed 1.724 MPa (see Table 2.12) was compacted at 100% optimum moisture content using the standard compactive effort. The resilient modulus of this selected mixture was obtained through the repeated load triaxial (RLT) test in accordance with the American Association of State Highway and Transportation Officials, standard method of test, AASHTO T.307-99 (2007). In this standard, the subgrade specimen was subjected to a series of loading and unloading sequence. The RLT test was performed using the UTM-14P (14 kN Pneumatic - Universal Testing Machine). This machine is equipped with a digital controller and a specific testing software module. Some programmable test standards are installed, including AASHTO.T.307-99 (see Figure 3.27).

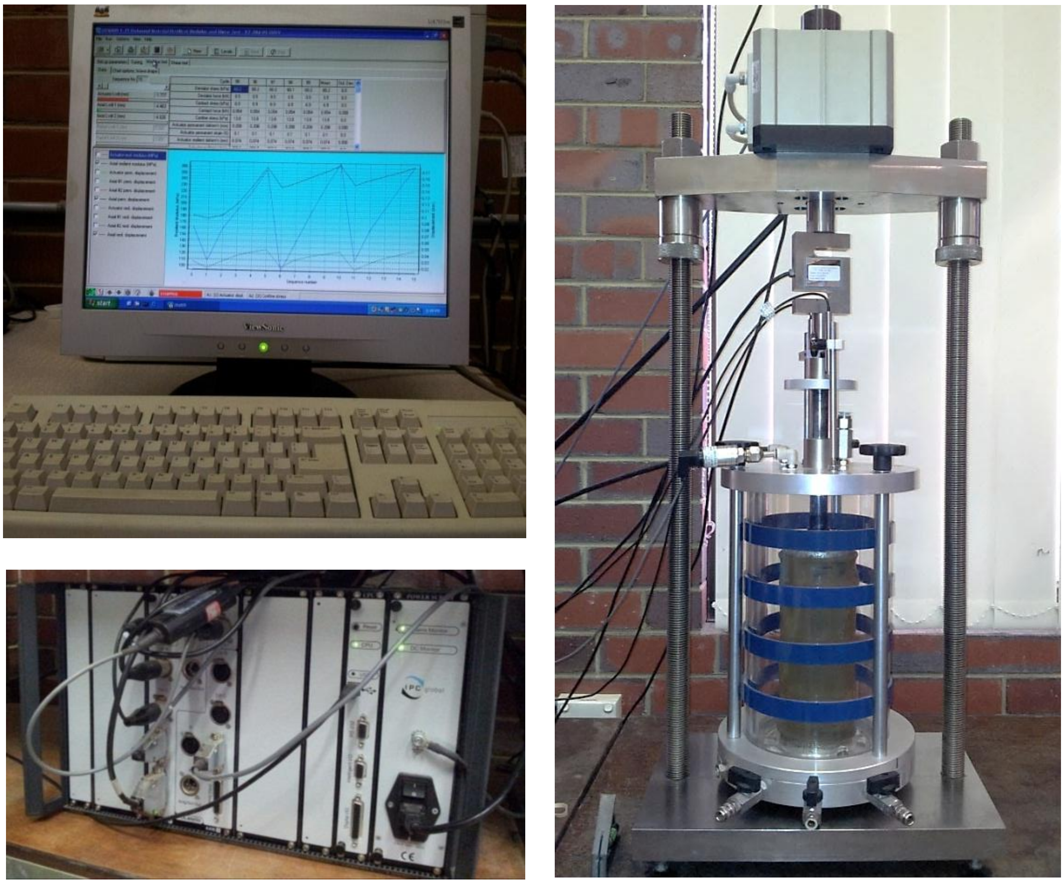

Figure 3.27: Repeated Load Triaxial test equipment

There were 15 loading sequences applied in this test which consist of three stages of different static confining pressure (σ_3). The three different confining pressures applied were 41.4 kPa, 27.6 kPa and 13.8 kPa. Details of stress combinations applied are shown in Table 3.6. In every sequence, each sample was set to receive one combination of deviator stress and confining pressure, as shown in Figure 3.28.

Table 3.6: Stress combinations in RLT test (AASHTO T.307-99, 2007)

Sequence No	Confining Pressure (σ_3), kPa	Max. Axial Stress (σ_{max}), kPa	Cyclic Stress (σ_{cyclic}), kPa	Constant Stress (0.1 σ_{max}), kPa	No. of (Load), cycles
0	41.4	27.6	24.8	2.8	500–1000
1	41.4	13.8	12.4	1.4	100
2	41.4	27.6	24.8	2.8	100
3	41.4	41.4	37.3	4.1	100
4	41.4	55.2	49.7	5.5	100
5	41.4	68.9	62.0	6.9	100
6	27.6	13.8	12.4	1.4	100
7	27.6	27.6	24.8	2.8	100
8	27.6	41.4	37.3	4.1	100
9	27.6	55.2	49.7	5.5	100
10	27.6	68.9	62.0	6.9	100
11	13.8	13.8	12.4	1.4	100
12	13.8	27.6	24.8	2.8	100
13	13.8	41.4	37.3	4.1	100
14	13.8	55.2	49.7	5.5	100
15	13.8	68.9	62.0	6.9	100

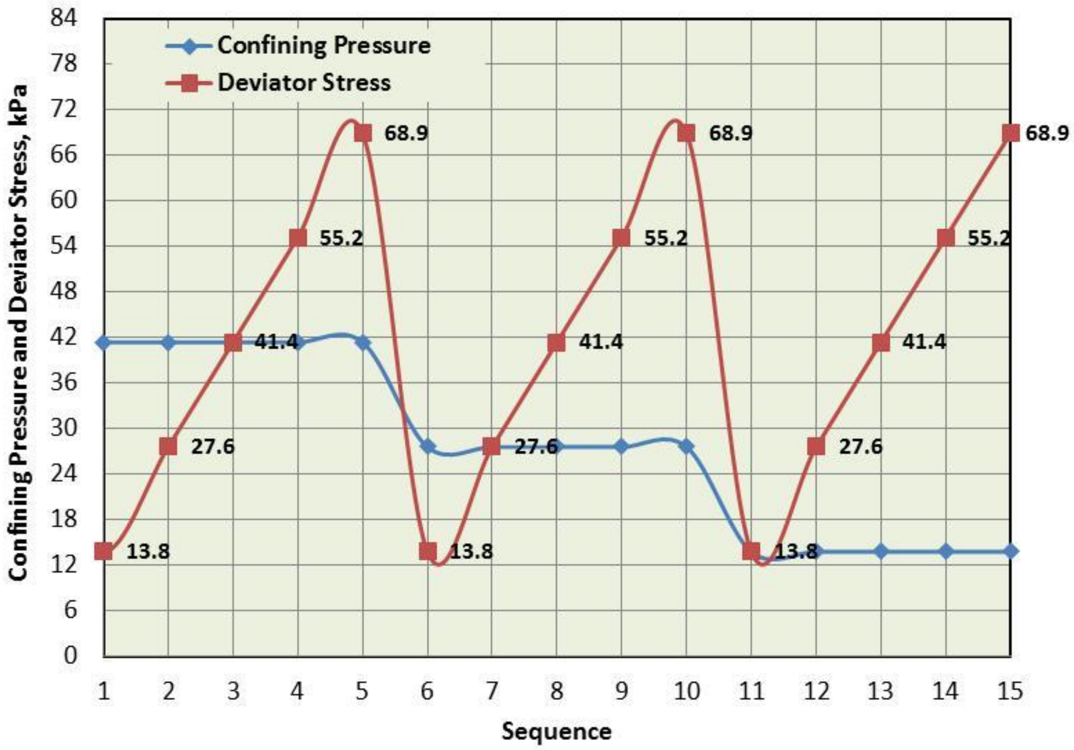

Figure 3.28: Confining pressure and deviator stress applied to each specimen of RLT test (AASHTO.T.307-99, 2007)

3.5.10. Acidity and Basicity Measurements

The measurements of the acidity and basicity of the stabilised soils were conducted according to the Australian Standard AS 1289.4.3.1 (1997). A portable digital reading that displays the pH meter equipped with a rod measuring probe was used for this purpose. The rod probe was dipped into a soaked specimen in order to obtain its pH value, as shown in Figure 3.29. This measurement was applied to both the natural soils and stabilised soils.

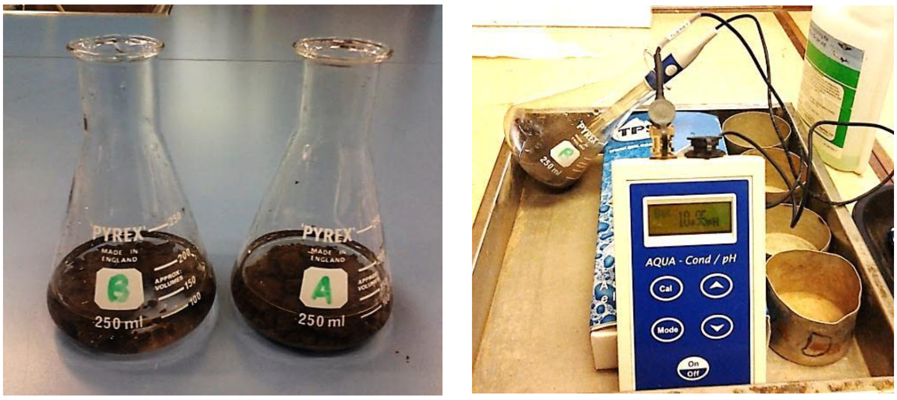

Figure 3.29: Soil pH measurement

3.5.11. Permeability Test

Permeability test on the stabilised soils aims to analyse how quickly water at a certain temperature will flow through the mixture. This was conducted according to the Australian Standard AS 1289.6.7.2 (2001), which is known as the Falling Head test method. It was measured by allowing water to flow through a remoulded specimen. This method is suitable for soil with a coefficient of permeability of about 10^{-7} to 10^{-9} metres per second. Several preparation stages of the soil specimens were performed, including mixing using mechanical mixer, compaction to achieve a maximum dry density of the mixture and curing for 28 days. To avoid wall seepage, a certain amount of glue was used as a smear to close the top edge void of the permeameter cylinder, as shown in Figure 3.30. This technique allows the water to flow only through the body of the specimen. Reading of water height in the standpipe was conducted for three days.

Figure 3.30: Permeability test

Chapter 4. Discussion of Results

4.1. Introduction

This chapter presents results, analysis and discussion of the laboratory tests. The chapter is divided into three sections. The first section presents the results of the preliminary soil testing program. The second section presents the performance results of the stabilised soils. The last section presents some empirical correlations (models) of the resilient modulus.

The preliminary tests aimed at characterising the expansive clay before being stabilised. The tests include the particle size distribution (sieve analysis and hydrometer), soil particle density, Atterberg limits and free swell index. These preliminary tests were performed to choose the clay material that fulfils the requirements of being expansive in nature.

The second section presents the results of some tests, including OMC-MDD using the standard compaction test, unconfined compressive strength, California bearing ratio, repeated load triaxial test, pH measurement, and permeability test.

The third section presents some resilient modulus empirical correlation models which had been released in previous studies. Those models have their own specific dependent variable relying upon their each physical properties and stress state. A selection of the best resilient modulus correlation model was designed based on the highest coefficient determination value (R^2) of each model to the selected stabilised soil.

4.2. Soil Properties

4.2.1. Particle Size Distribution

Since the expansive soil is categorised as fine-grained soil, all soil tests were focused on silt or clay soil. The Unified Soil Classification System, USCS, indicates that these types of soil should form at least 50% of the material passing the 0.075 mm sieve size. The particle-size distribution test was performed using the sieve analysis followed by the hydrometer test, and the result can be seen in Figure 4.1

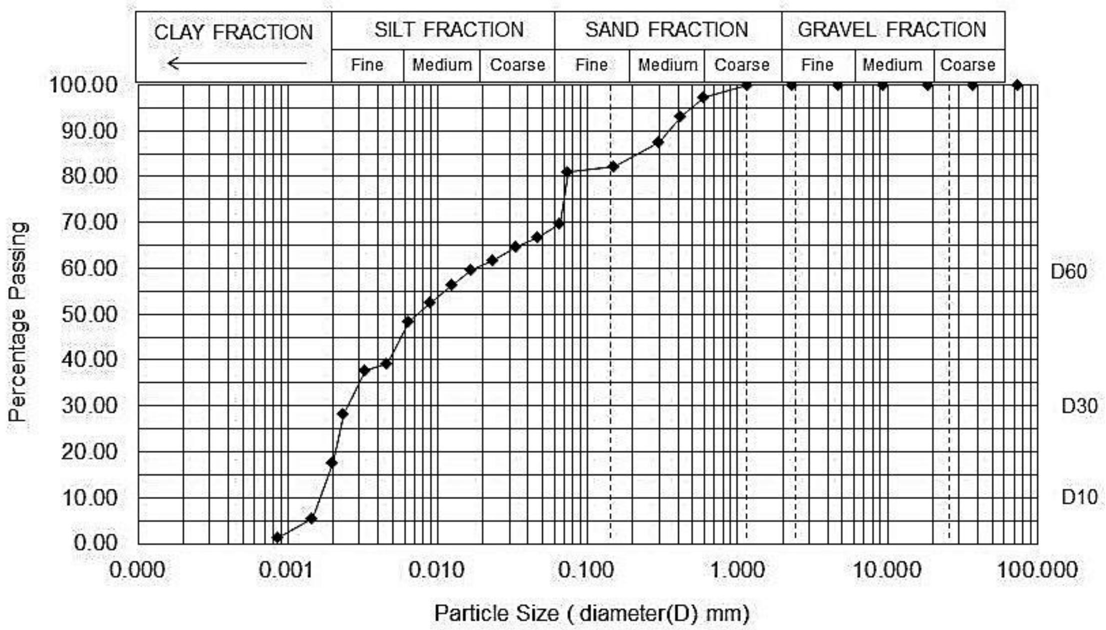

Figure 4.1: Particle-size distribution of selected soil

Based on the particle size distribution, the selected soil comprises a Sand fraction of 19.12%, a Silt fraction of 52.45% and a Clay fraction of 28.43%. The proportion of the particle size is illustrated in the Figure 4.2. Based on hydrometer test, the percentage of particle size less than 425 µm is 93.17.

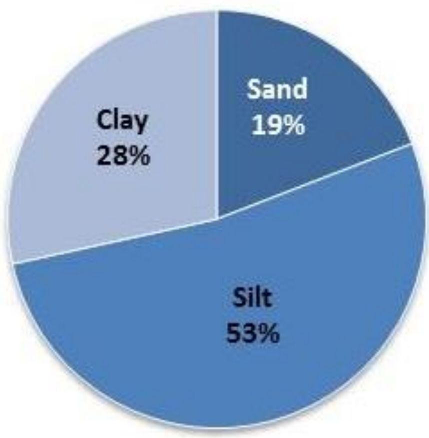

Figure 4.2: Proportion of particle size distribution of soil used

4.2.2. Atterberg Limits

The Plasticity Index (*PI*) of the soil used was found to be 26.48, which was calculated from the measured values of Liquid Limit (*LL*) of 68.00 and Plastic Limit (*PL*) of 41.52. According to Table 2.5 provided by the Austroads Guide to Pavement Technology, if (*PI*) × (% < 425 μm) = 26.48 × 93.17 = 2467.14; the selected soil can be classified as *moderate* to *high* in terms of its degree of expansiveness. In addition, based on the average of the two hydrometer test results, the soil activity can be calculated using Equation 2.1 given in Chapter 2, as shown in the Table 4.1. It can be concluded that the selected soil is active.

Table 4.1: Clay Activity calculation

Soil Sample	Plasticity Index (*PI*)	% of clay (particle size finer than 2μm)	Activity (*PI* ÷ % clay)
1	26.48	15.65	1.69
2	26.48	19.73	1.34
Average			1.52 (active)

4.2.3. Free Swelling Test

Measurement of the final height of soil specimens tested for the Free Swelling Index was made over a period of 24 hours. The change in height of both specimens after at least 24 hours was read as a volume change. As explained in the previous chapter, cylinder A and B were filled with distilled water while cylinder C and D were filled with kerosene, as shown in Figure 4.3.

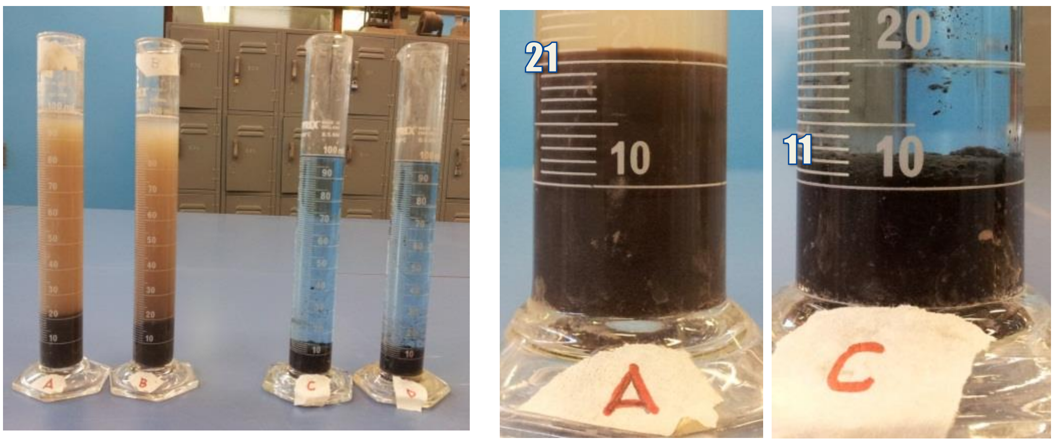

Figure 4.3: Free Swelling Test of expansive soil

The soil specimens in distilled water (A and B) experienced significant swelling, whereas the soil specimens in kerosene (C and D) did not experience change in volume (compare Tube A and C in Figure 4.3). Based on the expansion measurement, the Free swell index can be calculated as follows:

$$\text{Free swell index} = \frac{(21\text{-}11)}{11} \times 100\% = 90.\ 91\%$$

The result of the free swell index calculation indicates that the degree of expansion of this soil is *Medium*, as categorised by Table 2.4 given previously in Chapter 2.

4.2.4. Optimum Moisture Content and Maximum Dry Density

The standard proctor tests produced values of optimum moisture content (OMC) and maximum dry density (MDD) that varied with each mixture of the soil/stabiliser, as shown in Table 4.2. Based on the results in this table, it can be seen that the values of the OMC and MDD for all mixes are somehow close to each other. It can also be seen that the mixes have MDD values higher than that of the non-stabilised soil while they have OMC values lower than that of the non-stabilised soil. As explained by Yadu and Tripathi (2013), the higher MDD values of the mixes compared to that of the non-stabilised soil is attributed to the replacement of soil by the slag and cement additives in the mixtures which have relatively higher specific gravity than the soil. On the other hand, the lower OMC values of the mixes compared to that of the non-stabilised soil is attributed to the decreased quantity of free soil, hence, smaller surface area required less water.

Figure 4.4 shows the compaction curves for the tested mixes in terms of dry unit weight versus water content for a standard compaction effort. It can be seen that the soil mixture with 13.5% slag + 1.5% cement has different trend compared to other mixtures and non-stabilised soil, and it has the highest maximum dry unit weight (13.94 kN/m^3) and the lowest optimum moisture content (28.58%). The replacement of 0.75% slag to cement on the third mixture (Var3) has led to the need for additional water (0.85%) in the mixture to achieve its maximum dry density. The values of optimum moisture content were used in preparation of the specimens of the RTL test.

Table 4.2: Value of OMC and maximum dry unit weight of non-stabilised soil and stabilised soil

No	Non-stabilised and stabilised soil	Sample variation code	Optimum moisture content (%)	Maximum dry unit weight (kN/m^3)
1	100% Soil (Non-stabilised soil)	Var0	29.50	13.53
2	100% Soil + 15% Slag	Var1	29.04	13.89
3	100% Soil + 13.5% Slag + 1.5% Cement	Var2	28.58	13.94
4	100% Soil + 12.75% Slag + 2.25% Cement	Var3	29.43	13.60

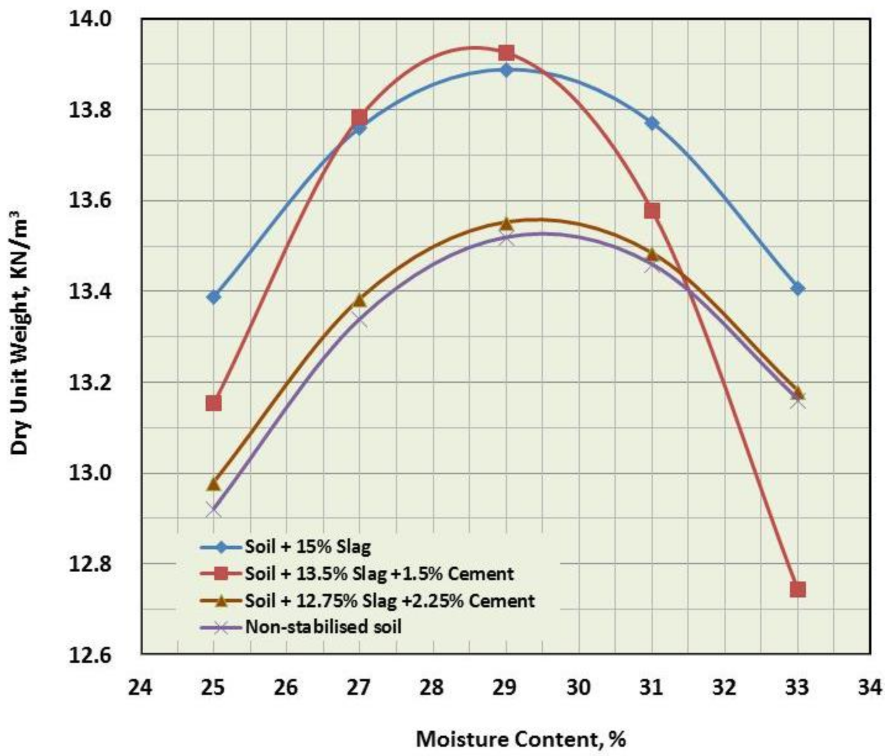

Figure 4.4: Correlation between dry unit weight and moisture content of non-stabilised and stabilised soils

4.2.5. Unconfined Compressive Strength (UCS)

The Unconfined Compressive Strength (UCS) tests of non-stabilised and stabilised soils were performed on specimens after 7 days, 14 days and 28 days curing time. The intention is to investigate the influence of time on the UCS test results, and the type and number of specimens are as shown previously in Table 3.4 and Table 3.5, respectively.

Three types of specimens were tested on the 7th day of curing. The stress-strain curves of these tests are presented in Figure 4.5. It can be seen that the non-stabilised soils have strain about 3% at its maximum stress values (about 0.25 MPa); on the other hand, all those stabilised soils have strain about 1.5% at its maximum stress values (between 1 MPa and 1.2 MPa), indicating a significant influence of the stabilisation on the material stiffness. The same trend was observed for samples cured for 14 days and 28 days, as shown in Figure 4.6 and Figure 4.7.

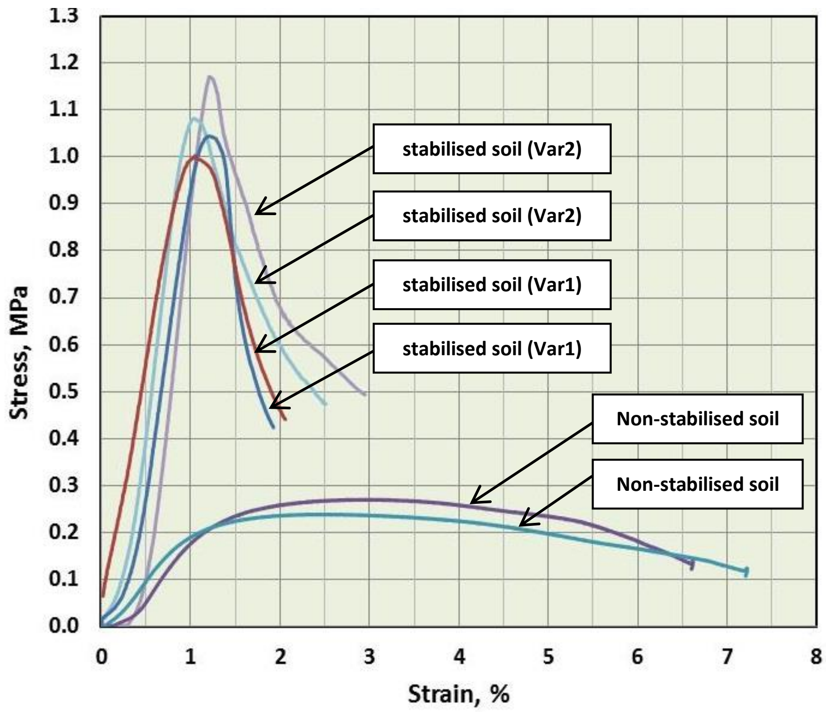

Figure 4.5: Stress-strain curves of UCS tests of non-stabilised and stabilised soils at 7th day curing time

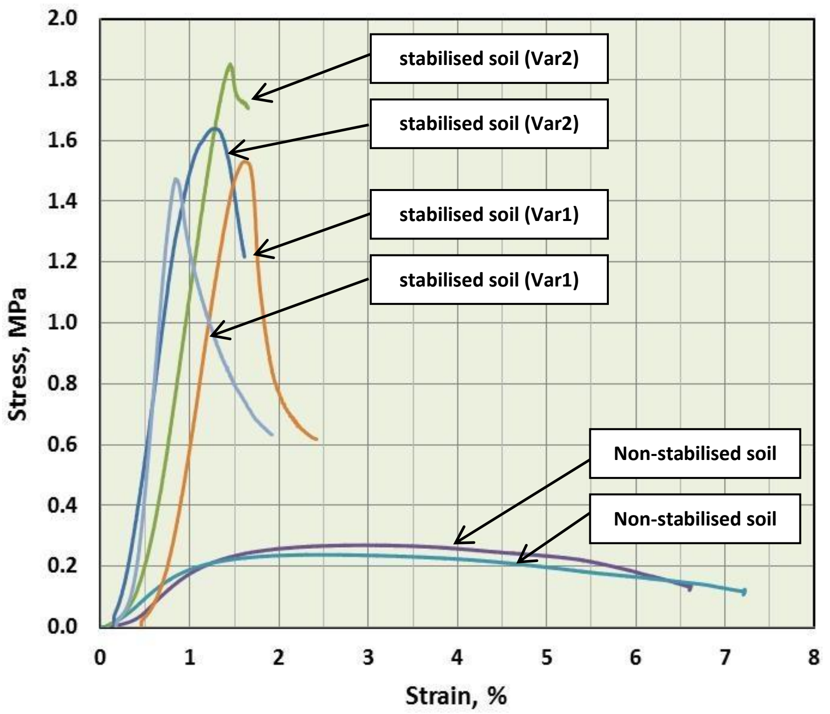

Figure 4.6: Stress-strain curves of UCS tests of non-stabilised and stabilised soils at 14th day curing time

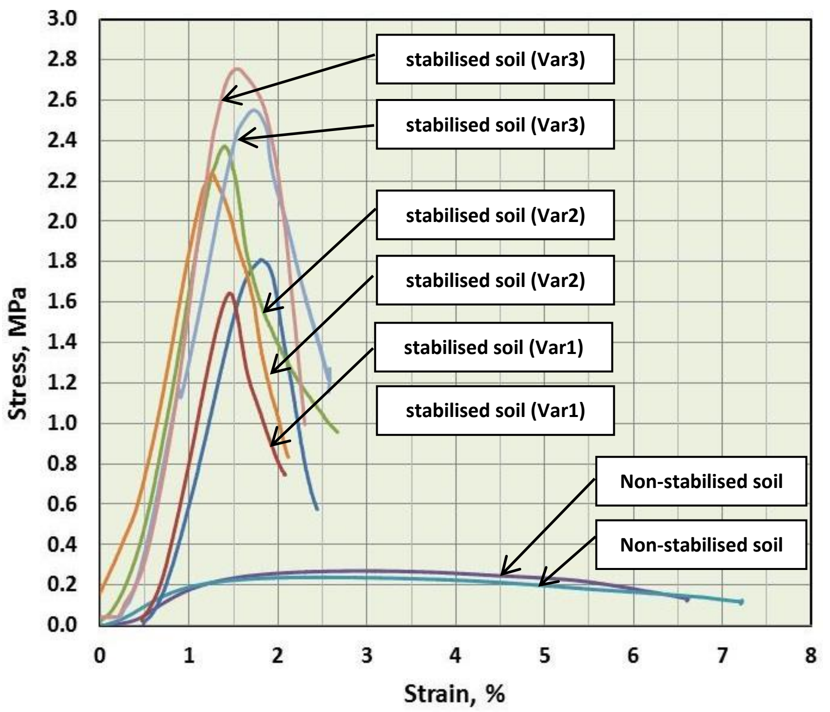

Figure 4.7: Stress-strain curves of UCS tests of non-stabilised and stabilised soils at 28th day curing time

It can be seen from Figure 4.8 that after 7 days curing time, the UCS value of the non-stabilised soil was only 0.195 MPa, compared with 1.021 MPa for the sample treated with 15% slag. Replacing 1.5% of the slag with cement increased the strength from 1.021 MPa to 1.178 MPa. It is noted, however, that the highest UCS value of 1.178 MPa is still below the minimum UCS value of 1.724 MPa required for the subgrade.

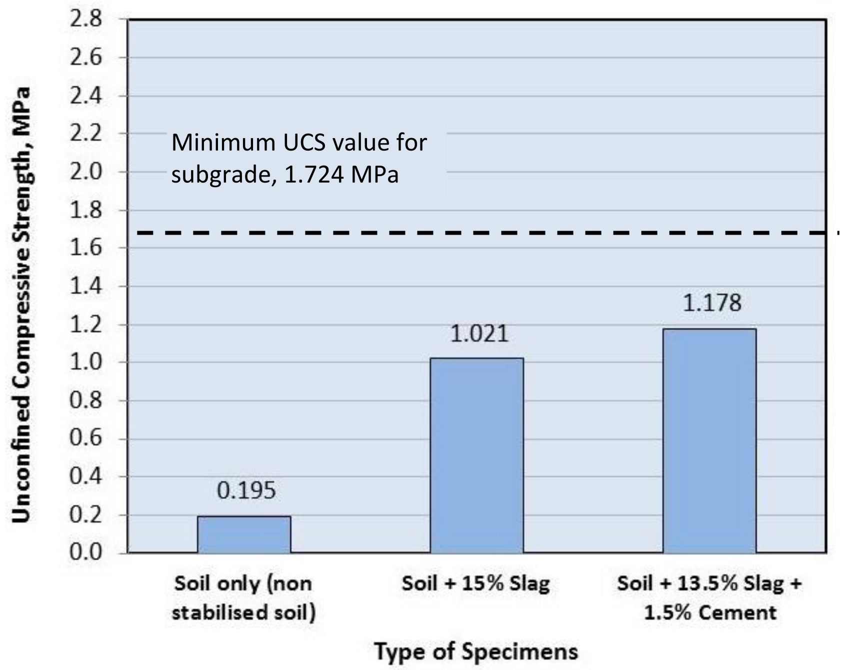

Figure 4.8: The UCS value of different mixes at 7 days curing time

Figure 4.9 shows the UCS results after 14 days curing time. The results of the 14 day curing are higher than those for the 7 day curing time, reflecting the increase of strength with time. This indicates that the pozzolanic reaction still occurs on stabilised soil after 7 days curing time, whereas the small increase on non-stabilised soil is the effect of the reduction of water content on the specimen. The mix that contains soil + 13.5% slag + 1.5% cement satisfied the minimum UCS value required for the subgrade (i.e. UCS_{min} = 1.724 MPa).

The group cured for 28 days had one additional mix of stabilised soil where the slag ratio was reduced further from 13.5 to 12.75 in favour of the cement (i.e. the ratio is 12.75% slag + 2.25% cement). Based on the results of the 28 days curing time in Figure 4.10, the soil stabilised with 13.5% slag + 1.5% cement exhibited UCS strength that is about eight times higher than that of non-stabilised soil. On the other hand, the UCS value of the fourth mixes was equal to 2.632 MPa, which is ten times higher than that of the non-stabilised soil, as shown in Figure 4.11. This means that replacement of 0.75% slag with cement increased the strength by about 22%.

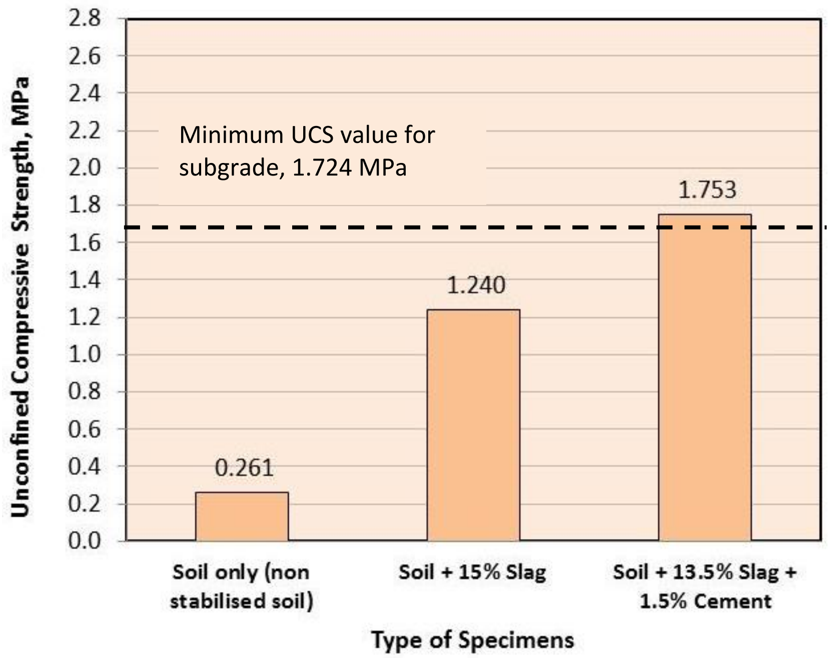

Figure 4.9: The UCS value of different mixes at 14 days curing time

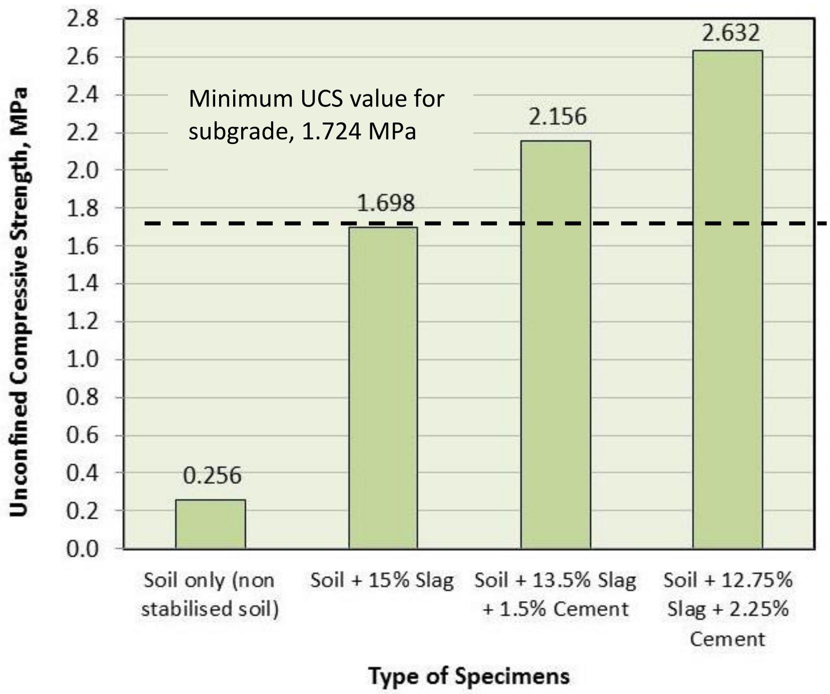

Figure 4.10: The UCS value of different mixes at 28 days curing time.

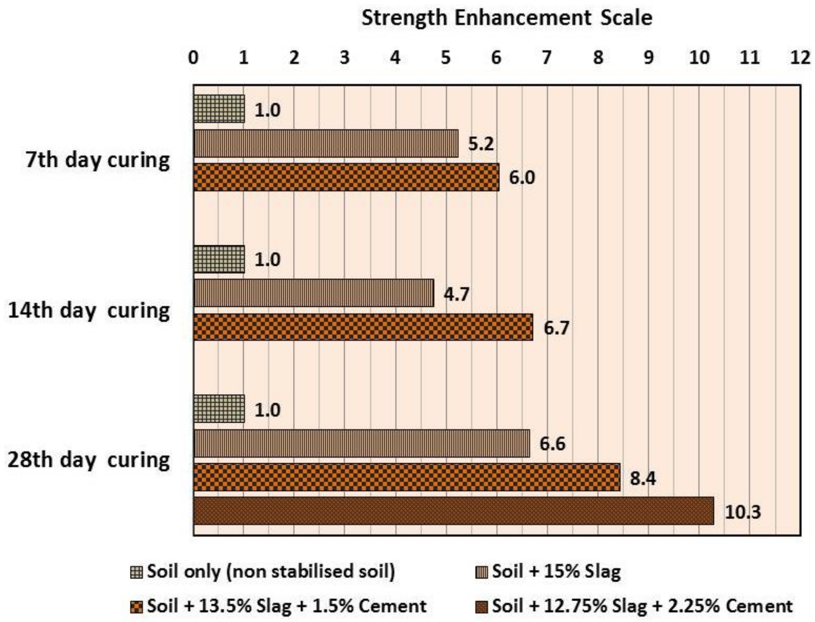

Figure 4.11: The strength enhancement scale of the UCS value of different mixes and curing time

Since the stabilised soil with the proportion of soil + 13.5% slag + 1.5% cement passed the minimum UCS value required for subgrade, this proportion is deemed more efficient than the one with 12.75% slag + 2.25% cement from an economic point of view.

4.2.6. California Bearing Ratio (CBR)

The mix of soil + 13.5% slag + 1.5% cement was used for the CBR testing. Figure 4.12 shows the penetration level of the three during load increments in the CBR test. The average value of the three CBR test results after four days soaking is 61.70%, as shown in Table 4.3. It means that the CBR value of the stabilised soil is four times higher than the minimum CBR value required for subgrade, as required by Austroads for pavement design (CBR_{min} = 15%).

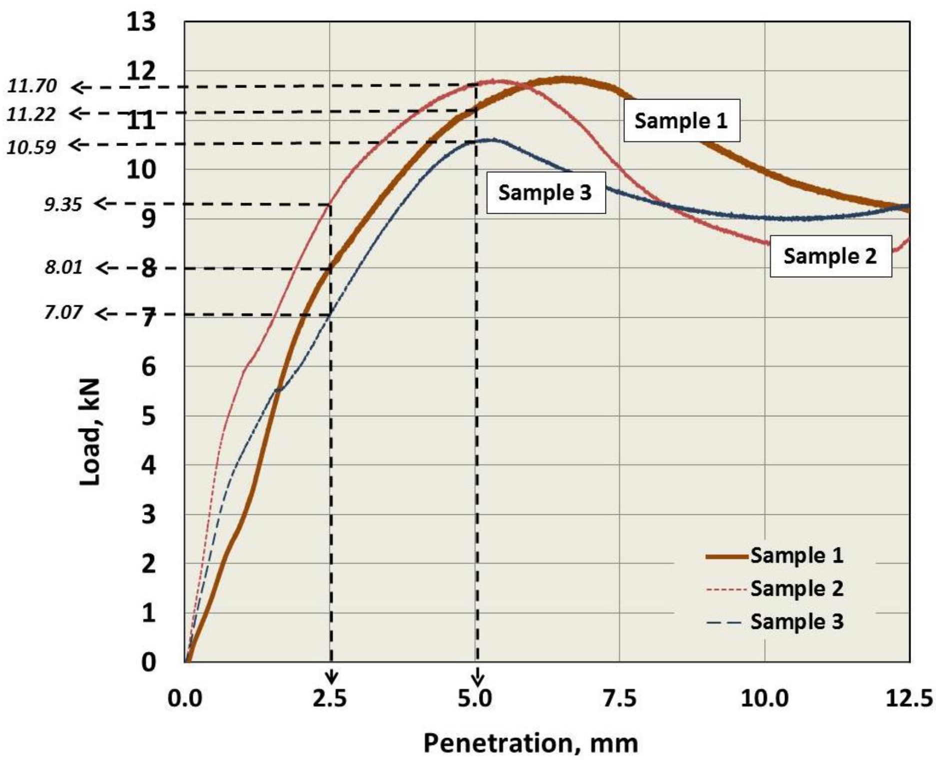

Figure 4.12 : Penetration level of the three samples of stabilised soil used for the CBR tests

Table 4.3: California Bearing Ratio calculations

Stabilised soil, 4 days soaked	Sample 1		Sample 2		Sample 2	
Load - CBR	Load (kN)	CBR (%)	Load (kN)	CBR (%)	Load (kN)	CBR (%)
CBR @ 2.5 mm = Load x 100/13.2	8.01	60.69	9.35	70.86	7.07	53.55
CBR @ 5.0 mm = Load x 100/19.8	11.22	56.67	11.70	59.07	10.59	53.47
Higher CBR Value (%)		60.69		70.86		53.55
Average CBR Value of three samples (%)	61.70					
Minimum CBR for subgrade (%)	15					
Ratio (minimum standard: test result)	**1: 4.11**					

4.2.7. Acidity and Basicity Measurements (pH test)

The acidity and basicity measurements were performed on both non-stabilised and stabilised soils contained 13.5% slag + 1.5% cement. The measurements were performed through pH test tools. Figure 4.13 shows the results of the pH measurements, which indicates that the value changed from that within the neutral area (6.11) to the basicity area (11.6). The increase in the pH value is due to the effect of the pozzolanic reaction between the soil and the stabiliser materials (GGBS and cement). The higher pH level of this mixture is expected and can change the toxic metals into less toxic forms of hazardous waste material.

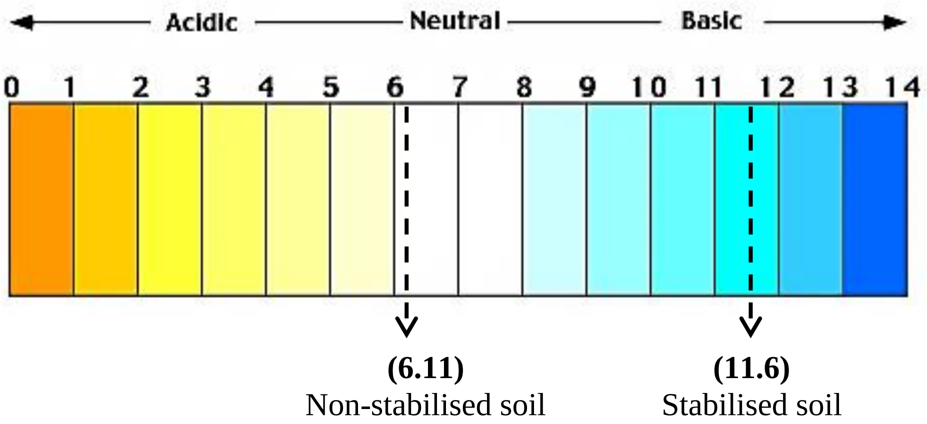

Figure 4.13: The pH measurement results of non-stabilised and stabilised soils

4.2.8. Permeability Test

The permeability test was performed on the stabilised soil over a period of three days. Over this period, no significant change of the water height in the standpipe was occurred and no water flow through the outlet permeameter cylinder was observed. To examine this, the permeameter baseplate was removed for inspection and was found that the bottom side of the specimen was still dry, as shown in Figure 4.14.

It is assumed that more time is needed to have water at the bottom side of the specimen, otherwise, a new modification should be made to the standpipe in order to have high water pressure. It was then concluded that the stabilised soil has low coefficient of permeability. In terms of the subgrade material, this condition means that it is unlikely that water will pass from the ground through to the upper layer of the road pavement and vice versa. This condition is expected to prevent water from leaching cement paste, binders, and fine materials.

Figure 4.14: Bottom side of permeability test specimen

4.2.9. Repeated Load Triaxial Test (RLTT)

The resilient modulus of the selected stabilised soil was determined on specimens at 28 days curing through Repeated Load Triaxial Test as referenced by AASHTO T.307-99 (2007). Three stages of confining pressures: 41 kPa, 28 kPa and 14 kPa, were applied in a total of 15 sequences to several identical stabilised soil triaxial specimens. In every sequence, each sample was set to receive one pair of deviator stress and confining pressure (see Figure 3.28). The result of the RLT test was taken from the average of three adjacent values of three specimens, as shown in Table 4.4

Figure 4.15 shows correlation charts between the deviator stress (σ_d) and resilient modulus (M_R) of three tested specimens. In terms of the effect of stress state, all curves show the same trend, where the M_R value increases with increasing σ_d. Another trend is the reduction of confining pressure starting from 41 kPa, 28 kPa and 14 kPa along repeated loading did not give significant effect to the resilient modulus value.

Table 4.4: Resilient modulus of three adjacent values from the RLT tests

Sample	Sequence	Deviator stress (σ_d), kPa	Resilient modulus (M_R), MPa
1	2	21.67	115.76
	3	35.84	152.10
	4	48.48	195.35
	5	60.90	234.20
	7	20.97	108.08
	8	35.30	159.00
	9	47.68	205.00
	10	59.70	248.57
	12	20.68	121.06
	13	34.30	169.19
	14	46.63	212.77
	15	58.45	249.29
2	2	19.86	139.61
	3	33.26	168.80
	4	46.00	208.53
	5	58.59	238.72
	7	20.04	151.23
	8	34.72	182.46
	9	47.97	217.76
	10	61.04	250.71
	12	21.47	143.79
	13	35.39	182.74
	14	47.89	217.93
	15	60.07	246.85
3	2	20.17	176.75
	3	34.09	198.40
	4	46.96	232.75
	5	59.52	260.26
	7	19.93	168.45
	8	33.76	196.83
	9	46.30	236.69
	10	58.28	278.44
	12	20.52	181.36
	13	34.66	215.82
	14	47.93	249.28
	15	60.56	277.44

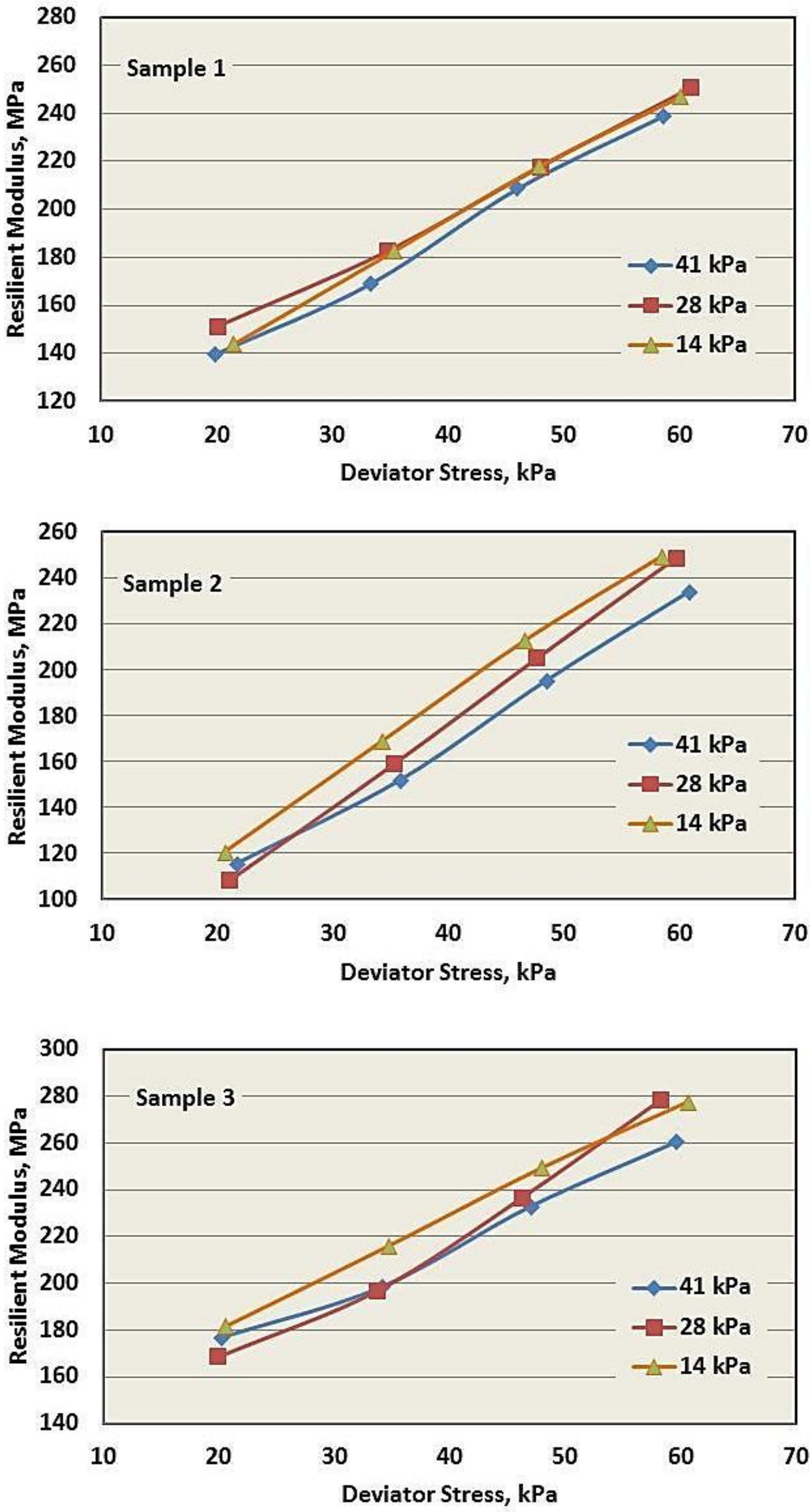

Figure 4.15: Correlation between resilient modulus and deviator stress in three different confining pressures of three tested specimens

Since the change of confining pressure has negligible effect on the resilient modulus, several resilient modulus correlation models were suggested based only on the deviator stress (σ_d). To this end, the results are plotted into one curve as shown in Figure 4.16 in terms of M_R versus σ_d. Three recommended models were chosen (see Table 4.5) to express variation of M_R with σ_d. In order to compare those models, the bilinear model was simplified into one linear model; when $\sigma_d > \sigma_{di.}$ Therefore, the resilient modulus values in sequence 1, 6 and 11 were omitted.

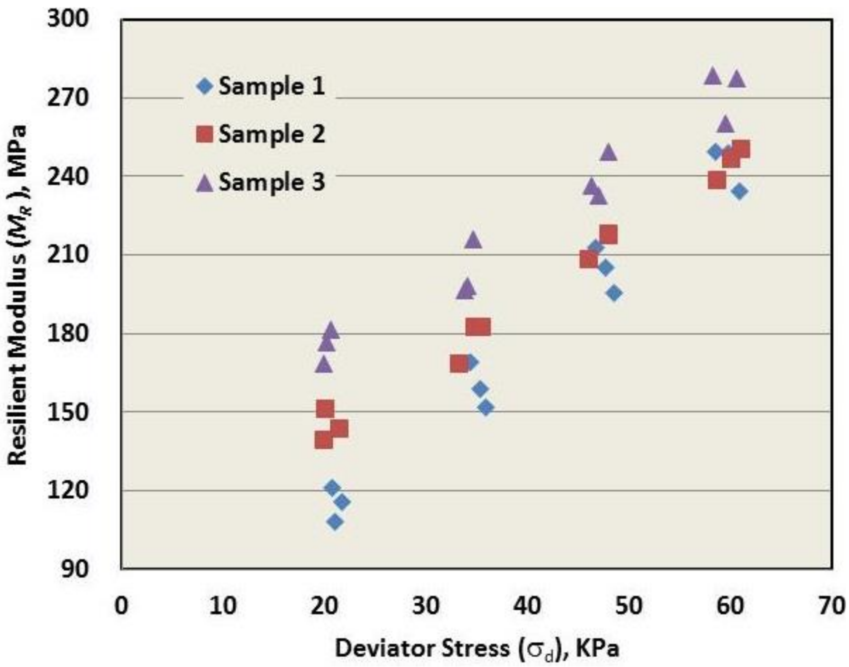

Figure 4.16: The plotting of three adjacent RLT test results

Table 4.5: Recommended resilient modulus correlation models

No	Model names	Model equations
1	Bilinear Model	$M_R = K_1 + K_2\sigma_d$, When $\sigma_d < \sigma_{di}$ $M_R = K_3 + K_4\sigma_d$, When $\sigma_d > \sigma_{di}$
2	Power Model 1	$M_R = k\,(\sigma_d)^n$
3	Hyperbolic Model	$M_R = \dfrac{k+n\,\sigma_d}{\sigma_d}$

Three recommended models were plotted in three separated graphs. The linear model is presented in Figure 4.17, the power model as shown in Figure 4.18 and the hyperbolic model is displayed in Figure 4.19. The best correlation model was selected based on the highest coefficient determination value (R^2) of each model.

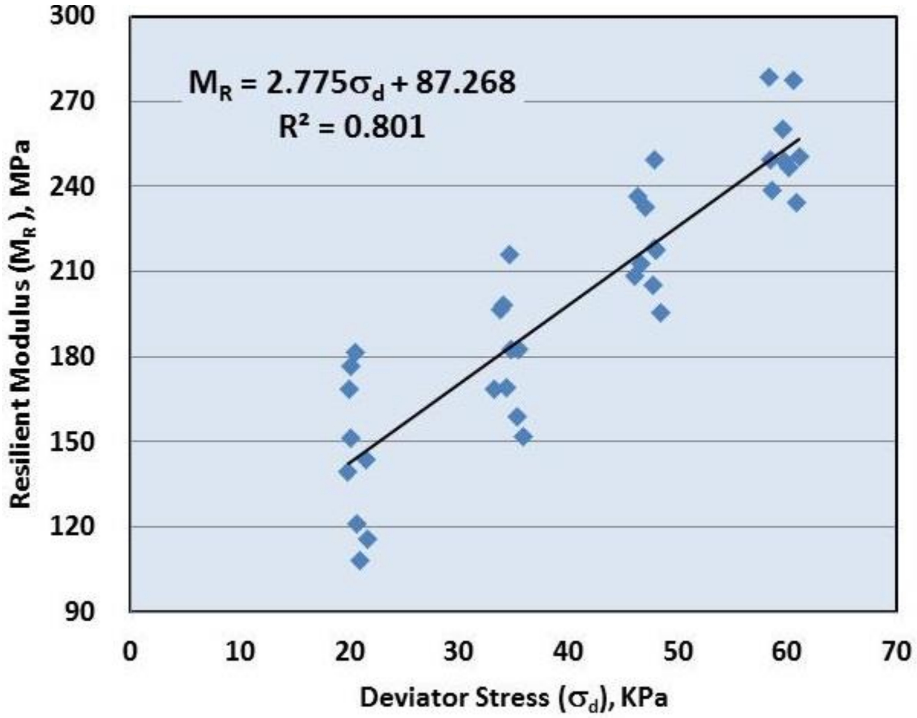

Figure 4.17: The linear model graph plot of M_R versus σ_d

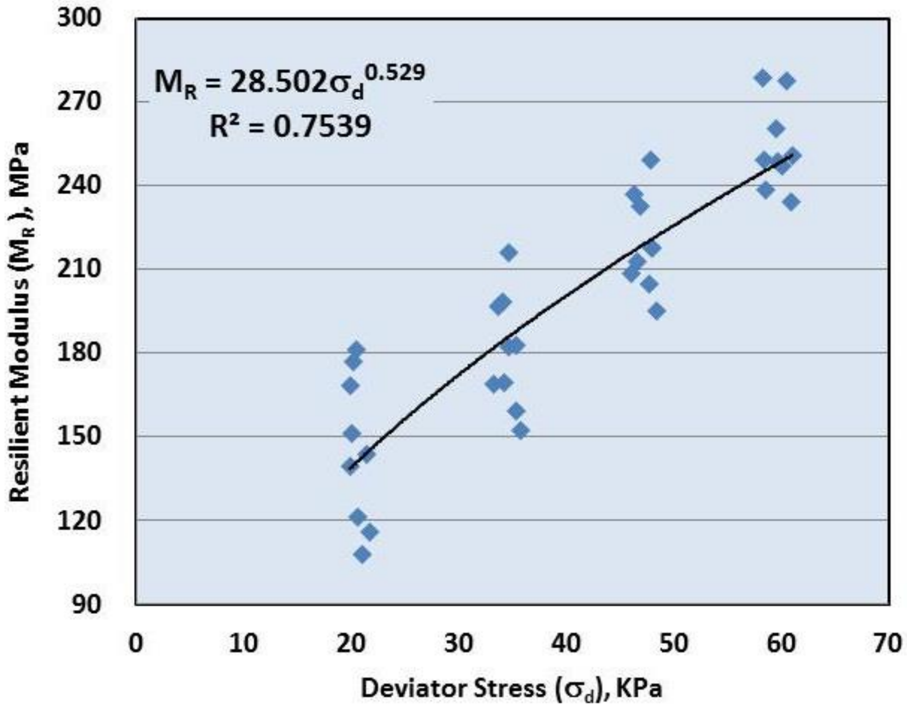

Figure 4.18: The power model graph plot of M_R versus σ_d

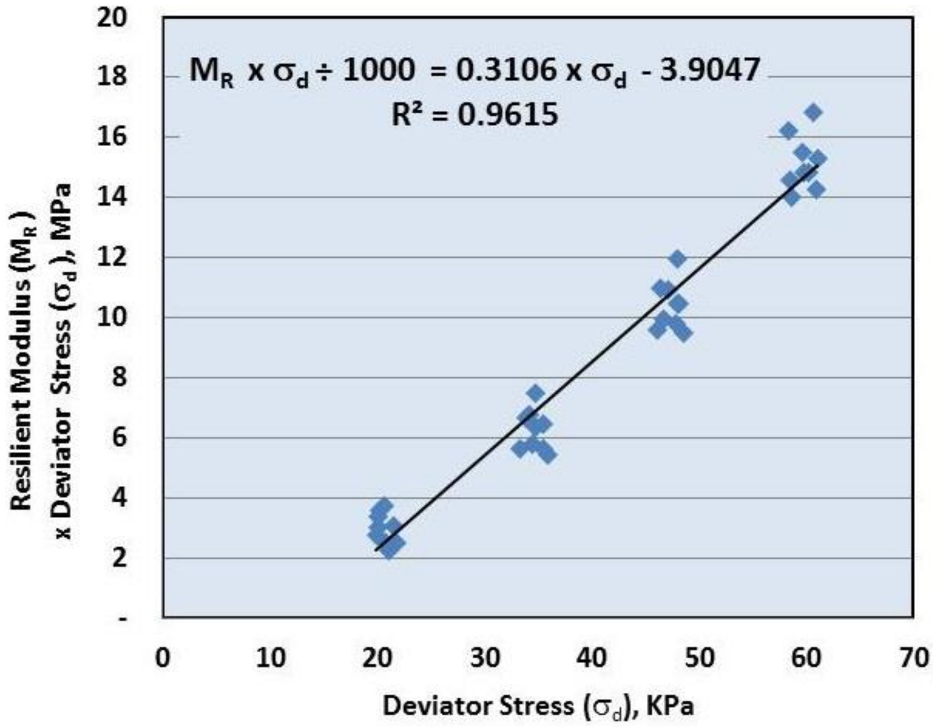

Figure 4.19: The hyperbolic model graph plot of M_R versus σ_d

Based on those three figures, it can be seen that the hyperbolic model generated the highest R^2. The generated correlation models are summarised in Table 4.6.

Table 4.6: Generated resilient modulus correlation models

No	Model names	Generated model correlations	R^2
1	Bilinear Model	$M_R = 2.775\sigma_d + 87.268$	0.801
2	Power Model 1	$M_R = 28.502\ (\sigma_d)^{0.529}$	0.7539
3	Hyperbolic Model	$M_R = \dfrac{310.6\ \sigma_d - 3904.7}{\sigma_d}$	0.9615

Chapter 5. Conclusions and Recommendations

5.1. Summary

Stabilisation of expansive soils with Ground Granulated Blast Furnace Slag (GGBS) or slag mixed with cement can provide significant improvement to performance of subgrades supporting road pavements. Following is a summary of the work presented and discussed in this thesis.

In Chapter 2, some theories and previous studies of soil stabilisation have been discussed. A review of several identifications of expansive soils has been presented. The initial soil identification can be made by rough detection of the presence of deep cracks within the ground surface of clay soils in dry season. This could be ascertained by thorough soil investigation supported by a series of laboratory test to determine certain soil properties along with specific measurement of the degree of soil expansion and activity.

Chapter 2 also reviewed some treatment methods of expansive soil with a focus on soil stabilisation. Stabilisation using GGBS as an additive was also presented. It was shown that using of GGBS in clay stabilisation results in a small increase in the optimum moisture content and small decrease in the maximum dry density in compaction tests. Another feature of using GGBS is its slow rate of pozzolanic reaction with soils, which is advantageous when sufficient time is needed for finishing subgrade works (Ouf 2001). A decreasing trend of both liquid limit and plastic limit by the addition of GBFS was found in previous studies (Yadu and Tripathi, 2013).

Chapter 2 revealed that combining GGBS with lime or cement as a compound stabilised material can assist in initiating the GGBS reaction. The ratio of slag to cement or lime depends on the soil type, clay content, curing conditions and curing periods. The use of GGBS accompanied with cement could modify grain size distribution and decrease the swell percentage of the treated soil (Cokca et al., 2008).

The bearing capacity and strength of stabilised clay increase significantly after being stabilised with GGBS accompanied with lime; however, there was a certain amount of lime that can be added up for optimum stabilisation. It was assumed that the contribution of hardening reaction in soil stabilised with slag would continue along its curing period due to the similar chemical composition between slag and cement; therefore, the longer the curing time, the better the performance of the stabilised soil. Subgrade performance is affected by several interrelated characteristics, including moisture content, load bearing capacity and shrink-swell potential. Hence, appropriate treatment methods should consider these three characteristics to achieve the required subgrade stabilities. Reliability of the subgrade stability should be maintained during its service life in responding to any fluctuated traffic load. This fluctuated load can be simulated in laboratory scale via repeated load triaxial testing.

The repeated load triaxial test was designed and standardised by AASHTO as a Standard Method of Test as specified in the AASHTO “T307-99 Determining the Resilient Modulus of Soils and Aggregate Materials”. This test uses both deviator stress and confining pressure as variables to determine the resilient modulus (M_R). The resilient modulus (which represents material stiffness) is affected by soil state and stress state. Some resilient modulus correlation mathematical models were developed to simplify interrelationship between M_R and both soil state and stress state.

Previous studies concluded that for fine-grained soils, the M_R correlation models normally depend upon the moisture content and deviator stress. As the moisture contents of all samples were marinated equal to the optimum moisture content obtained from the compaction test, the deviator stress may be a sole variable used in this model. In this chapter, several M_R correlation models were tried to determine the one that suits the stabilised soil used in this thesis.

Chapter 3 discussed the purpose of the experiments, material selection, material preparation, specimen preparation and experimental work. The expansive soil as main material was selected based on its properties and behaviour. The degree of expansion and soil activity, were the criteria used in selecting the suitable soil for the experimental work. These criteria were generated based on soil properties such as particle size distribution, soil particle distribution and Atterberg limits. The GGBS and cement as stabiliser materials were described herein. To examine the reliability of the stabilised soils, a series laboratory tests and measurements were designed based on subgrade requirements. The laboratory tests were conducted in accordance with the Australian Standards, Indian Standards, Main Road Laboratory Standard Procedures (WA) and AASHTO. This chapter explained which standard should be used.

Chapter 4 presented the test results described in Chapter 3 in the form of tables and figures. The following list is a summary of the laboratory test activities:

- Based on the particle size distribution tests, it was concluded that the soil is categorised as fine-grained with a clay content of about 28%. Atterberg limits measurement on this soil produced plasticity index (*PI*) of 26.48%. This *PI* value was divided by the percentage of soil particle less than 2 μm in size to obtain clay Activity value. Based on calculation in Table 4.1, the clay Activity is 1.52, and the soil is categorised as active soil.

- The free swell index of this soil is 90.91%, which categorise the soil as *medium* in terms of the degree of expansion (Table 2.4). While, in terms of expansiveness, *PI* value and the result of *PI* multiplied by the percentage of soil particle less than 0.425mm, the Austroads classified the soil as between *moderate* and *high* (Table 2.5).

- In order to obtain the maximum dry density of non-stabilised and stabilised soils, the determination of optimum moisture content (OMC) were conducted on all samples. An amount of 15% stabiliser was added to 100% of dry soil that divided into three different percentage proportions of slag to cement; 15–0, 13.5–1.5 and 12.75–2.25. Every mixture has its own specific OMC value. The mixture with mix of soil + 13.5% slag + 1.5% cement has the highest dry unit weight (13.94 kN/m^3), and the lowest OMC value (28.58%). The optimum moisture content values listed in Table 4.2 were used in preparation sample specimens.

- The UCS tests were performed on all samples representing non-stabilised and stabilised soils at three curing times of 7 days, 14 days and 28 days. Based on the stress-strain curves of all UCS test specimens, the stiffness of stabilised soil was found to be higher than that of the non-stabilised soil. Furthermore, there was strength increase of stabilised soils during these curing times. The stabilised soils that contain cement have higher UCS values than those of the soil stabilised without cement. Based on this test, it was decided that the recommended stabiliser proportion that fulfilled the standard minimum as a subgrade material is soil + 13.5% slag + 1.5% cement. This stabiliser proportion gave a remarkable strength of about eight times more than the strength of the non-stabilised soil. This mixture was selected to be used in other next performance tests such as CBR and RTL tests.

- The permeability test showed that the stabilised soil mixture can be categorised as a low permeable material. It appears that the addition of stabilisers densifies the soil, hence, reduces its permeability, as evidenced from the maximum dry density the mixtures which was found to be higher than that of the non-stabilised soil.

- The pH measurement on both non-stabilised and stabilised soils showed that the initially neutral soil becomes basic after stabilisation (pH = 11.6). This high pH level is expected and can transform the toxic metals of hazardous waste material into less toxic forms.

- The bearing capacity measurement through the CBR test on the stabilised soil concluded that, after 28 days curing and 4 days soaking, there was a remarkable increase in the CBR value. Based on the average of three CBR tests, the stabilisation gavelled to CBR value was more than four times the minimum required for a subgrade need to satisfy the minimum requirement by Austroad (CBR_{min} = 15%).

- The influence of increasing the confining pressures from 14 kPa to 41 kPa in the RLT test on the resilient modulus was negligible. Three resilient modulus correlation models were examined against the results of this study; these are Bilinear Model, Power Model and Hyperbolic Model, all depending solely on the deviator stress. The coefficient of determination value (R^2) generated by those three models indicated that the Hyperbolic Model correlates the best with the deviator stress with R^2 = 0.9615, as listed in the Table 4.6. This model should be assessed to see if it is applicable to others, because in most cases, the models were developed for certain soils.

5.2. Conclusions

Thorough geotechnical investigation supported by laboratory tests were carried out to determine the soil classification and the degree of expansion of the clay soil. The degree of expansion is useful in predicting the deformation that may occur in the soil due to moisture variation. Deformations may affect to the stability of road pavement if they exceed allowable limits. This condition can be an important reference in deciding the type of treatment required for a stable soil subgrade supporting road pavement.

Soil stabilised for subgrades may combine chemical and mechanical processes. Three stabiliser proportions have been designed and applied to stabilise the expansive soil; some laboratory tests were performed on various mixes. Based on the laboratory test results, the recommended additive proportion to stabilise the Baldivis expansive soil is *13.5% slag + 1.5% cement*. This mixture was proven to be effective in satisfying the allowable standard as subgrade of road pavements.

The Unconfined Compressive Strength of stabilised samples at 28 days curing resulted in satisfactory UCS values. Moreover, bearing capacity of this stabilised soil shows a remarkable improvement of about 400% higher than the minimum standard of CBR value for designed subgrade projects. This stabilised soil may be categorised as less permeable material subgrade which prevents any ground water to flow through the adjacent pavement layers. The enhancement of pH value of stabilised soil compared with non-stabilised soil was assumed as a result of the pozzolanic reaction between the soil and stabilisers that has influence in forming good environment. Furthermore, this basicity behaviour is believed to provide contribution to strength development in the hydration of Portland cement.

The hyperbolic correlation model was found to best represent the resilient modulus as a function of the deviator stress.

5.3. Recommendations for Furture Research

All mixtures of the stabilised soils used in this study were tested at the optimum moisture content (OMC) during performance testing. However, in real stabilisation works, this OMC condition is difficult to achieve in the field due to seasonal changes and fluctuation of field temperature. In addition, it is very hard to ascertain the homogeneousness of moisture content of the mixture on every road part. Therefore, in the next study, it is recommended to conduct the performance tests at various moisture content conditions, such as 80% OMC, 90% OMC and 110% OMC. Such study can examine whether the mixtures still satisfy the required standard of the mixture or not. This is true especially in Australia where there are ground moisture movements during the summer and winter seasons. This condition may lead to pavement deteriorations on the long run. The subgrade that underlies all pavement layers with a direct contact with the ground may contribute to this deterioration. It is recommended therefore to evaluate the effect of loss of stabilisation effectiveness on pavement performance, especially to the stabilised subgrade with slag and cement.

The low cost of GGBS compared to cement may encourage using GGBS as a soil stabilizer in high proportion. However, in using lime as a soil stabilizer, previous study proved that there is an optimum amount of lime that can be used in the mixture. The performance of stabilised soil (with lime) will tend to drop if an excessive amount of lime is used. This condition may happen with the soil stabilised with GGBS. Therefore, a thorough study may need to be carried out to determine how much of GGBS can be used in soil stabilisation without causing adverse effects that may negate its benefit as shown in the present research.

The amount of water set in the mixture with slag and cement was based on the optimum moisture content value. On the other side, the performance of cement may depend on the amount of cement which fits with water-cement ratio (w/c). It is recommended to find how much water should be used to achieve the best performance of cement reaction.

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